The construction materials industry is a major user of the world’s resources. While enormous progress has been made towards sustainability, the scope and opportunities for improvements are significant.

To further the effort for sustainable development, a conference on Sustainable Construction Materials and Technologies was held at Coventry University, Coventry, U.K., from June 11th - 13th, 2007, to highlight case studies and research on new and innovative ways of achieving sustainability of construction materials and technologies. This book presents selected, important contributions made at the conference. Over 190 papers from over 45 countries were accepted for presentation at the conference, of which approximately 100 selected papers are published in this book. The rest of the papers are published in two supplementary books.

Topics covered in this book include: sustainable alternatives to natural sand, stone, and portland cement in concrete; sustainable use of recyclable resources such as fly ash, ground municipal waste slag, pozzolan, rice-husk ash, silica fume, gypsum plasterboard (drywall), and lime in construction; sustainable mortar, concrete, bricks, blocks, and backfill; the economics and environmental impact of sustainable materials and structures; use of construction and demolition wastes, and organic materials (straw bale, hemp, etc.) in construction; sustainable use of soil, timber, and wood products; and related sustainable construction and rehabilitation technologies.
Sustainable Construction Materials and Technologies

Editors

Yoon-moon Chun
Research Associate, UWM Center for By-Products Utilization, University of Wisconsin – Milwaukee, Milwaukee, Wisconsin, USA

Peter Claisse
Professor of Construction Materials, Faculty of Engineering and Computing, Coventry University, Coventry, UK

Tarun R. Naik
Research Professor and Academic Program Director, UWM Center for By-Products Utilization, University of Wisconsin – Milwaukee, Milwaukee, Wisconsin, USA

Eshmaiel Ganjian
Senior Lecturer in Civil Engineering, Faculty of Engineering and Computing, Coventry University, Coventry, UK
# Table of contents

Preface XI

Contributions of the concrete industry toward sustainable development 1

*K. Sakai*

Site trials of concrete with a very low carbon footprint 11

*P.A. Claisse, E. Ganjian & H. Sadeghi-Pouya*

Sustainability of the cement and concrete industries 19

*T.R. Naik*

Closed Cycle Construction – A process for the separation and reuse of the total C&D waste stream 27

*E. Mulder, L. Feenstra & T.P.R. de Jong*

Effect of GRP industrial waste on durability of cement mortars 35

*F. Tittarelli & G. Moriconi*

Engineering properties of oil palm shell (ops) hybrid concrete for lightweight precast floor slab 41

*C.H. Ng, Z. Ideris, S.P. Narayanan, M.A. Mannan & V.J. Kurian*

Durability of concrete mixed with fine recycled aggregates 45

*S. Levy & P. Helene*

An experimental investigation on the viability of using fine concrete recycled aggregate in concrete production 53

*S.R. Sarhat*

Effect of different recycled aggregates on mortar performance 59

*V. Corinaldesi, S. Monosi & G. Moriconi*

Researching recycled aggregate structural concrete for buildings 63

*N. Morohashi & T. Sakurada*

Recycling of concrete in precast concrete production 69

*V. Corinaldesi & G. Moriconi*

Concrete paving products made with recycled demolition aggregates 77

*K. Tang, M.N. Soutsos & S.G. Millard*

Using tires rubber waste for modification of concrete properties 85

*G. Skripkiuñas, A. Grinys & M. Daukšys*

Employment of waste sand to compose fibre reinforced cement composites 91

*J. Katzer*

Portland cement concrete using high levels of microfines 101

*PN. Quiroga & D.W. Fowler*

Utilization of organo-modified reservoir sludge as fine aggregates in cement mortars 107

*W.Y. Kuo, J.S. Huang, T.E. Tan & C.Y. Chou*

Study on fly ash-saturation in cementitious materials 115

*L. Guangcheng, L. Yunhua & X. Youjun*
Ultra fine fly ash concrete
B.K.T. Kandie & E.A. Byars

Achieving sustainable construction through use of fly ash in concrete – An Indian experience
M. Mokal

“Eco-pad” in-situ mixed concrete pavement with a 93–100% total recycled content
B.W. Ramme, T. Jansen, A. Tawil & D.B. Anderson

Properties of shotcrete with highly functional fly ash used as dust-reducing agent
K. Sasaki, M. Ishii, Y. Butou & K. Yuno

The use of fly ash for enhancing durability and sustainability
L.K.A. Sear

Cracking behavior of concrete containing fly ash due to drying shrinkage
T. Seo, Y. Ohno & T. Nakagawa

Potential uses for coal combustion by-products for sustainable construction materials
T.F. Vandivort & P.F. Ziemkiewicz

Properties of calcium enriched fly ash and its utilization in concrete
X. Ma, Y. Gong & X. Gong

Production of gypsum products from waste battery acid
G.M. Cann, P.A. Claisse & J.P. Lorimer

Use of waste gypsum as construction low strength materials
S. Karami, P.A. Claisse, E. Ganjian, H.S. Pouya & R. Greaves

Development of novel cementitious binders using plasterboard waste and pozzolanic materials for road bases
H.S. Pouya, E. Ganjian, P.A. Claisse & S. Karami

Investigation into the use of cement stabilised gypsum waste as a backfill material
W. Rahman, G. Ghataora & D.N. Chapman

The importance of mixture proportioning in sustainable construction
N.G. Maldonado & P. Helene

Research and development of new building materials on the basis of industrial waste materials in the field of rehabilitation of concrete structures
V. Petranek, P. Matulova & G. Michalcova

Development of an optimal high-performance concrete mixture for Tennessee bridge decks
R.M. Salem, E.G. Burdette & J.H. Deatherage

Establishing optimum mixture proportions for concrete durability using recyclable by-products
D.M. Vruno

Effects of cement and mineral admixture on migration of chloride ions and generation of hydration heat in concrete
J. Yoo, S. Park & B. Oh

Hemp as a filler of cement-bonded particleboards
J. Bydžovský & F. Khestl

Chemical and physical characterization of coarse bauxite residue (red sand) for concrete making
M.G. Davoodi, H. Nikraz & E. Jamieson

Effect of ground perlite incorporation on the performance of blended cements
T.K. Erdem, Ç. Meral, M. Tokyay & T.Y. Erdoğan

Use of Magnesium Oxide-cement binders for the production of blocks with lightweight aggregates
N. Vlasopoulos & C.R. Cheeseman
Leaching behavior and environmental impact of concrete manufactured with biomass ashes
G. Fava, M.L. Ruello & D. Sani

The application of paper sludge ash to extremely stiff consistency concrete product
H. Fujiwara, M. Maruoka, K. Koibuchi & K. Fujita

Alkali-activation of natural pozzolan for geopolymer cement production
D. Bondar, C.J. Lynsdale, A.A. Ramezanianpour & N.B. Milestone

Effect of spent catalyst, obtained from the catalytic cracking of petroleum, on the compressive strength of concrete
K.O. Ampadu, K. Torri & J.J. Vordoagu

Sulfate resistance of mortars with and without silica fume and natural pozzolan
F. Canpolat, M.A. Yurdusev, S. Targan & K. Yilmaz

Strength of mortar and concrete using fine powder of molten slag made from municipal waste as a cementitious material
K. Fukuzawa, D. Kwak, O. Abe, H. Ojima & H. Yoshida

Use of supplementary cementing materials for cost-effective repair of marine corrosion damage
D.V. Reddy, J.C. Bolivar, V. Gutierrez & Z.A. Saleh

Change of concrete properties due to impressed current of cathodic protection
T. Suginoue & H. Seki

Advanced calcareous ceramics via novel green processing and supercritical carbonation
E. Farahi, P. Purnell & N.R. Short

Supercritical carbonation of Casamic
S.J. Shaw, C.L. Page, A.R. Brough, J.P. Forth, M.M. Page, T.R. Jones & M. Carsana

Investigations on the efficiency of Enhanced Porosity Concrete in Containing vehicular oil spills
B.V. Bhayani, T.M. Holsen & N. Neithalath

CO2 uptake by concrete hardened in a simulated flue gas
Y. Shao & X. Zhou

Early hydration of calcium sulfoaluminate-based cements for structural applications
M. Marroccoli, M. Nobili, A. Telesca & G.L. Valenti

Reactive magnesium oxide cements: properties and applications
L.J. Vandeperre, M. Liska & A. Al-Tabbaa

Bacterial carbonate precipitation reduces the permeability of cementitious materials
W.D. Maynck, K. Cox, N.D. Belie & W. Verstraete

Sustainable, ‘green’ solutions for concrete pavement rehabilitation. A feasibility (pilot) study
J.N. Karadelis, K. Koutselas, A. Khan, Z. Jiebin, K. Moscicka & A. Plachta

Effect of loading level, cooling regime, polypropylene fibers, and coating type on the behavior of high strength concrete columns in fire
H.I. El Sayad, M.M. Abdel-Razek & H.S. Haddad

Effect of simulated desert climate and sustained moderate temperature on some properties of concrete with and without polypropylene fibers
H.I. El Sayad

Rheological and engineering properties of SCLC
A.A. Maghsoudi & S.M. Pour

Coefficient of moisture contraction – A new concrete material parameter
Thaumasite sulphate attack on self-compacting concrete with limestone filler
B. Persson 461

Effect of polypropylene fiber addition on restrained plastic shrinkage cracking of SCC
A.A. Maghsoudi, R. Rahgozar & M. Motavaselian 471

High performance grout connection for pre-cast concrete structures
M.A.A. Kharazi 479

Sustainable concrete arch bridges
J. Radic, Z. Savor & J. Bleiziffer 487

Determination of the amount of hydrated cement in hardened concrete and mortar by gravimetric thermal analysis on macro and semi-micro scales
F. Gomà 493

Assessment of the fracture of three-point bending concrete specimens
A. Ziliukas & A. Augonis 503

The economics of recycling in the US construction industry
C. Meyer 509

An analysis of environmental and fiscal impacts of recycling during Kern Center construction
M.U. Christiansen 515

Impact of regulations on fly ash marketing in the United States
G.C. Plunk & D.C. Goss 521

Quantifying VOCs in products off-gassed during the construction of MSOE's Kern Center and proposed strategies for reducing employee exposure
S. Botic & C. Diggelman 527

Relative thermal performance of three test buildings – thermal mass v insulation
K.A. Heathcote & G. Moor 541

LEED-NC version 2.2 rating system applications of common structural materials
J.E. Maher & K.W. Kramer 549

A life cycle perspective on recycling construction materials (The most sustainable materials may be the ones we already have)
T.R. Napier, D.T. McKay & N.D. Mowry 563

Demonstrating how plasterboard can be collected more efficiently on construction and demolition sites in the UK
S.B. Emery, D.N. Smith, S. Johansson & J. Cope 575

Extending experimental data to investigate phosphogypsum use in light brick by Artificial Neural Networks
M.A. Yurdusev, A.A. Kumanlioglu, Y. Abali, M.S. Zeybek & F. Canpolat 589

Opportunities for wealth generation through small scale sustainable building materials production
S.N. Mclean, A. Williams & D.R. Moore 593

Electrokinetic treatment for freezing and thawing damage mitigation within limestone
H.E. Cardenas, P. Patrui & P. Dubasi 603

Carbonation and hydration of mortars with calcium hydroxide and calcium silicate binders

Biostore: towards sustainable composite waste reuse
B.K.C. Chan & A.W.L. Dudeney 623

Review of composting and anaerobic digestion of municipal solid waste and a methodological proposal for a mid-size city
M.R.Q. Silva & T.R. Naik 631
<table>
<thead>
<tr>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Innovative use of clay backfill at the new Wembley Stadium, UK</td>
<td>645</td>
</tr>
<tr>
<td>S.W. Carley, A.S. O’Brien, F.A. Loveridge &amp; Y.S. Hsu</td>
<td></td>
</tr>
<tr>
<td>A rationale for the production of Devon Cob to ensure performance</td>
<td>657</td>
</tr>
<tr>
<td>K.A. Coventry &amp; J. Griffiths</td>
<td></td>
</tr>
<tr>
<td>Mixture proportioning and strength prediction technique for soil mortar made using soil generated at construction sites</td>
<td>665</td>
</tr>
<tr>
<td>T. Yoshida &amp; Y. Kitamoto</td>
<td></td>
</tr>
<tr>
<td>Utilization of olive oil liquid waste as an additive to roadway construction</td>
<td>675</td>
</tr>
<tr>
<td>O.A. Abaza</td>
<td></td>
</tr>
<tr>
<td>Investigation into the use of cement stabilised sand and silty sand in road pavement construction in Bangladesh</td>
<td>685</td>
</tr>
<tr>
<td>W. Rahman, R. Freer-Hewish &amp; G. Ghataora</td>
<td></td>
</tr>
<tr>
<td>Monitoring of strawbale and non-food-crop based walling systems</td>
<td>697</td>
</tr>
<tr>
<td>S. Goodhew, R. Griffiths, P. de Wilde &amp; T. Simmons</td>
<td></td>
</tr>
<tr>
<td>Development of sustainable forms of construction</td>
<td>705</td>
</tr>
<tr>
<td>Sustain the chapel building – a demonstration of sustainable materials renovation</td>
<td>717</td>
</tr>
<tr>
<td>Environmental science in building construction: new course development for undergraduate students</td>
<td>725</td>
</tr>
<tr>
<td>C. Diggelman</td>
<td></td>
</tr>
<tr>
<td>Cardboard: An innovative construction material</td>
<td>731</td>
</tr>
<tr>
<td>J. Schönwälder &amp; J.G. Rots</td>
<td></td>
</tr>
<tr>
<td>Improvement mechanism of bondability in uf-bonded reed board by pf</td>
<td>741</td>
</tr>
<tr>
<td>J. Torkaman</td>
<td></td>
</tr>
<tr>
<td>Sustainable construction in Yellowstone Park: A case study</td>
<td>745</td>
</tr>
<tr>
<td>A.S. Denzer &amp; K.E. Hedges</td>
<td></td>
</tr>
<tr>
<td>Ultrasound assisted crystallisation of synthetic gypsum from used battery acid</td>
<td>753</td>
</tr>
<tr>
<td>H. Abdul-Jabbar, H. Al-Daffaee, P.A. Claisse &amp; J.P. Lorimer</td>
<td></td>
</tr>
<tr>
<td>Manufactured aggregate from waste materials</td>
<td>763</td>
</tr>
<tr>
<td>R. Lupo, M. Tyrer, C.R. Cheeseman &amp; S. Donatello</td>
<td></td>
</tr>
<tr>
<td>Evaluation and minimization of life cycle environmental risk of concrete structures</td>
<td>769</td>
</tr>
<tr>
<td>T. Noguchi &amp; S. Fujimoto</td>
<td></td>
</tr>
<tr>
<td>Comparison between mechanical properties of SCC containing Rice-Husk Ash and normal concrete</td>
<td>779</td>
</tr>
<tr>
<td>A.S. Momtazi, M.A. Mehrdad, K.A. Nia &amp; A. Azhari</td>
<td></td>
</tr>
<tr>
<td>Comparative study between properties of SCC containing lime-stone powder and NC</td>
<td>785</td>
</tr>
<tr>
<td>A.S. Momtazi, A.H. Modaraei, A. Azhari &amp; K.A. Nia</td>
<td></td>
</tr>
<tr>
<td>Compressive response of plastered straw bale wall panels</td>
<td>789</td>
</tr>
<tr>
<td>S. Vardy &amp; C. MacDougall</td>
<td></td>
</tr>
<tr>
<td>Author index</td>
<td>801</td>
</tr>
</tbody>
</table>
Preface

The construction materials industry is a major user of the world’s resources. While enormous progress has been made towards sustainability, the scope and opportunities for further improvements are significant. This conference was held at Coventry University, Coventry, U.K., from June 11th – 13th, 2007, to highlight case studies and research that show new and innovative ways of achieving sustainability of construction materials and technologies.

In 2006, papers were invited on all the different materials that are used in construction including cementitious materials (fly ash, wood ash, slag, silica fume, natural pozzolans, and other similar materials), aggregates, concrete, timber, masonry, metals, plastics, glass, bitumen, lime, and gypsum. Papers on paints, adhesives, preservatives, and preservation processes were also invited.

Over 190 papers from 45 countries were accepted for publication, following review. The papers covered a wide range of topics including sustainable alternatives for aggregates and cementitious materials in concrete; making concrete sustainable; the economics and environmental impact of sustainable materials; sustainable use of gypsum, lime, soil, natural fibers, wood, and other organic materials; and sustainable construction methods and technologies. The accepted papers are published in three volumes: Main, Special Sessions, and Supplementary. Wherever possible, papers were grouped by their subject matter for the convenience of readers.

Conference organizers would like to thank Committee members and others who worked hard to prepare and distribute call for papers and program, to review manuscripts, and to publish proceedings. We greatly appreciate the cooperation of the authors in considering and incorporating review comments for revision of their manuscripts.

It is our sincere hope that the proceedings will be useful to civil engineers, architects, construction managers, researchers, and all other professions related to the construction industry, for achieving sustainable development.

We are considering holding the next conference in 2010 at a venue yet to be decided.

Yoon-moon Chun  
Peter Claisse  
Tarun R. Naik  
Eshmaiel Ganjian

Editors, International Conference on Sustainable Construction Materials and Technologies
Sponsors
Contributions of the concrete industry toward sustainable development

K. Sakai
Department of the Safety Systems Construction Engineering, Kagawa University, Takamatsu, Japan

ABSTRACT: Sustainable development has become more and more important in the 21st century. The human race is steadily transitioning its socioeconomic system in an effort to solve its resource, energy and environmental problems. The concrete industry is certainly no exception. This paper discusses the latest trends in environmental aspects of concrete and related environmental technologies, as well as presents the “JSCE Recommendation of Environmental Performance Verification for Concrete Structures (Draft).” It also proposes a method for the assessment of environmental benefits of constructing concrete structures as a means for the concrete industry to contribute toward sustainable development.

1 INTRODUCTION

Only a mere 200 some years had passed since the end of the Industrial Revolution. The human race found itself faced with two extremely serious problems: resource/energy depletion and global warming. These problems arose as a result of the human race’s enjoyment of convenient and comfortable lifestyles.

A sense of crisis arising from these circumstances has led to various movements over the past several decades. The first warning came in the form of a report to the Club of Rome entitled “Limits of Growth” [Meadows D.H., Meadows D.I., Randers & Behren III 1971] Despite its devastating content, the report was not taken very seriously at the time of its publication. The “Declaration of the Human Environment” was adopted at the UN Conference on the Human Environment held in Stockholm in 1972, presenting the following principle concerning the environment-related rights and responsibilities of the human race:

“Man has the fundamental right to freedom, equality and adequate conditions of life, in an environment of a quality that permits a life of dignity and well-being, and he bears solemn responsibility to protect and improve the environment for present and future generations.”

In other words, the Declaration of the Human Environment was the first clear statement of the human race’s responsibility to protect and improve the environment, in addition to its fundamental right, and the Declaration greatly influenced subsequent international trends in environmental conservation. As the implementing agency of this Declaration, the UN Environment Programme (UNEP) was established. In 1987, the UN World Commission on Environment and Development published the Brundtland Report [The World Commission on Environment and Development 1987], which presented the following well-known concept of sustainable development:

“Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs.”

In 1992, 20 years after the Stockholm Conference, the Rio Declaration was adopted at the UN Conference on Environment and Development (Earth Summit) held in Rio de Janeiro. At this conference, the importance of sustainable development to maintain balance between environmental conservation and economic growth was once again clarified, and participating nations signed the United Nations Framework Convention on Climate Change, which went into effect in 1994. The Kyoto Protocol for reduction in greenhouse gases was adopted at the Third Conference of the Parties to the United Nations Framework Convention on Climate Change (COP3) held in Kyoto in 1997, and entered into force in February 2005. The Protocol defines greenhouse gas emission reduction targets for 2008/2012 for individual advanced countries, with the levels of 1990 set as the standard. The so-called “Kyoto Mechanism,” including the clean development mechanism, emissions trading and other mechanisms for promoting the reduction of greenhouse gas emissions in developing countries and regions, was also introduced under the Protocol.

Although the US government withdrew from the Kyoto Protocol, a new path was set for reduction of
greenhouse gases at the COP11 held in Montreal in 2005, where an agreement was reached on the commencement of post-COP3 discussions including the United States. While the rise of global-scale resource, energy and warming issues has been recognized as important by the concrete industry and various research activities have been promoted, such activities are still in their infancy and environmental aspects have not actually been included in the design of concrete structures. As mentioned above, however, it is obvious that the consideration of resource, energy and warming issues is essential to all future socioeconomic activities, and the concrete industry is certainly no exception. Fortunately, international concrete conventions focused on “sustainability” have been held in recent years. This proves the recognition of the importance of the environment in the field of concrete technologies.

This paper discusses the latest trends in environmental aspects of concrete and related environmental technologies, as well as presents the “Recommendation of Environmental Performance Verification for Concrete Structures (Draft),” which was recently published by JSCE. It also proposes a method for the assessment of environmental benefits of constructing concrete structures as a means for the concrete industry to contribute toward sustainable development.

2 ENVIRONMENTAL ASPECTS OF CONCRETE AND ENVIRONMENTAL TECHNOLOGIES

2.1 Aggregate

It is said that, of the approximately 26 billion tons of annual material flow throughout the world, aggregate used as a construction material accounts for some 20 billion tons [Brown 2001]. In other words, natural sand, gravel and crushed stone exist most abundantly in the natural world, thus these materials are most frequently used for infrastructure development. Although the aggregate situation varies by country and region, the replacement of natural aggregate with crushed stone aggregate has progressed in recent years due to environmental restrictions. It is believed that this has also led to changes in the quality of aggregate, which greatly affects the quality and durability of concrete. Crushed stone aggregate usually has sharp edges, thereby increasing the water content of concrete, and the energy used for the production of crushed stone has an adverse impact on the environment. Moreover, the development of quarries is also causing environmental problems.

In recent years it has been said that infrastructure development has matured in Japan. While the number of national highways and bridges 50 years or older managed by nation and local governments in Japan was 8,191 in 2005, it is expected to increase to 63,494 by 2025 [Ministry of Land, Infrastructure and Transport 2005]. Reconstruction of structures is also increasing in urban areas. According to an estimate by the Development Bank of Japan [Development Bank of Japan 2002], the total amount of concrete masses to be generated in 2025 is estimated to be 210 million tons, which is a twofold increase of the 112 million tons generated in 2005. These amounts are extremely high, considering that the total concrete production for Japan in 2005 was approximately 285 million tons. While most concrete masses have been used as base course materials, their demand is expected to decrease dramatically. Since the use of these as recycled aggregate is crucial considering the pressure on waste disposal sites and the preservation of aggregate resources, the development of technologies for recycled aggregate production is being promoted.

Technologies for recycled aggregate production in Japan include the heating and rubbing [Tateyashiki, Okamoto, Nishimura & Kuroda 2000], eccentric-shaft rotor [Yanagibashi, Yonezawa, Kamiyama & Inoue 1999] and mechanical grinding [Yoda, Shintani, Takahashi & Yanagase 2004] methods. In the heating and rubbing method, concrete masses are heated at 300°C and the cement paste content is weakened to remove mortar and cement paste from the aggregate. Figure 1 shows an overview of a recycled aggregate production system using this method. Figure 2 illustrates the recycled coarse and fine aggregate produced by the system. While the production of recycled aggregate generated a large amount of fine powder, it also indicated the possibility of using fine powder like this as a substitute solidification material for the deep mixing stabilization method (soil cement walls) [Uchiyama, Kuroda & Hashida 2003]. In the eccentric-shaft rotor method, crushed concrete lumps are passed downward between an outer cylinder and an inner cylinder that eccentrically rotates at a high speed to separate it into coarse aggregate and mortar through a grinding effect. Figure 3 presents an overview of a recycled aggregate production system using this method. Mechanical grinding is a method used to produce coarse and fine aggregate by separating a drum into small sections with partitions, loading the drum with iron balls for grinding and rotating the partitions. Figure 4 shows an overview of the recycled aggregate production system using this method. The coarse aggregate produced by these methods has been used for actual construction projects.

In Japan, JIS A 5021 [Japanese Standards Association 2005] was established in 2005 as a standard for high-quality recycled aggregate H for concrete, which is produced through advanced processing, including crushing, grinding and classifying, of concrete masses generated in the demolition of structures. Recycled aggregate H must have physical properties satisfying
the requirements listed in Table 1. There are also upper limits for the amounts of deleterious substances contained in recycled aggregate H, as shown in Table 2. JIS A 5023 has also been established as a standard for recycled concrete using low-quality recycled aggregate L. This type of concrete includes backfilling, filling and leveling concrete, and the use of Type B blended cement and admixture is required as a measure against alkali-silica reactivity.

Production of recycled aggregate usually involves greater energy use compared with that of virgin aggregate, thus leading to greater environmental impacts. While finding ways to deal with these environmental impacts as an external cost is a major issue, it would be ideal to absorb this cost in the entire production/use system of recycled aggregate. As shown in Figure 5, Kuroda et al. [2004] established an on-site concrete resource recycling system from this standpoint, and proved that CO₂ emissions from the system were lower than those in which it was used as a conventional base course material. This was achieved by reducing the amount of transported material and the use of fine powder as a substitute solidification material for ground improvement. Yanagibashi [2004] studied CO₂ emissions in the cases of use of recycled aggregate for base course material made of concrete masses, use of recycled coarse aggregate for concrete and recycled sand for base course and backfill materials, and use of
recycled coarse aggregate for concrete and the remaining amount for cement clinker. The paper concluded that the use of recycled aggregate was an environmentally friendly method that could contribute to the preservation of natural aggregate since CO₂ emissions were nearly identical.

Iron and steel slag is also used as aggregate. In 2005, 25,747,000 tons of blast-furnace slag and 14,897,000 tons of steelmaking (converter and electric-furnace) slag were generated in Japan [Nippon Slag Association 2005]. Of these, blast-furnace slag was primarily used as aggregate for concrete, and its amount was 3,158,000 tons.

While fly ash has been used as a substitute admixture for cement, the idea of using this as fine aggregate has emerged in recent years [Shikoku Chapter of Japan Society of Civil Engineers 2003]. Considering the worldwide increase in fly ash production expected in the future, it is desirable to design a mix that integrates substitutes for cement and fine aggregate.

In recent years, melting treatment of waste and sewage sludge has become common in Japan due to pressures on waste disposal site and problems related to dioxin and heavy metals. Molten slag is generated as a residue after melting treatment. In 2004, 144 waste and 18 sewage sludge melting treatment facilities were in operation throughout Japan, producing 480,000 and 44,000 tons of molten slag, respectively.

Table 1. Physical properties requirements for recycled aggregate H.

<table>
<thead>
<tr>
<th>Items</th>
<th>Coarse aggregate</th>
<th>Fine aggregate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oven-dry density, g/cm³</td>
<td>not less than 2.5</td>
<td>not less than 2.5</td>
</tr>
<tr>
<td>Water absorption, %</td>
<td>not more than 3.0</td>
<td>not more than 3.0</td>
</tr>
<tr>
<td>Abrasion, %</td>
<td>not more than 35</td>
<td>NA</td>
</tr>
<tr>
<td>Solid volume percentage for shape determination, %</td>
<td>not less than 55</td>
<td>not less than 53</td>
</tr>
<tr>
<td>Amount of material passing test sieve 75 μm, %</td>
<td>not more than 1.0</td>
<td>not more than 7.0</td>
</tr>
<tr>
<td>Chloride ion content</td>
<td>not more than 0.04</td>
<td></td>
</tr>
</tbody>
</table>

Table 2. Limits of amount of deleterious substances for recycled aggregate.

<table>
<thead>
<tr>
<th>Category</th>
<th>Deleterious substances</th>
<th>Limits (mass %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Tile, Brick, Ceramics, Asphalt</td>
<td>2.0</td>
</tr>
<tr>
<td>B</td>
<td>Glass</td>
<td>0.5</td>
</tr>
<tr>
<td>C</td>
<td>Plaster</td>
<td>0.1</td>
</tr>
<tr>
<td>D</td>
<td>Inorganic substances other than plaster</td>
<td>0.5</td>
</tr>
<tr>
<td>E</td>
<td>Plastics</td>
<td>0.5</td>
</tr>
<tr>
<td>F</td>
<td>Wood, Paper, Asphalt</td>
<td>0.1</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>3.0</td>
</tr>
</tbody>
</table>

recycled coarse aggregate for concrete and the remaining amount for cement clinker. The paper concluded that the use of recycled aggregate was an environmentally friendly method that could contribute to the preservation of natural aggregate since CO₂ emissions were nearly identical.

Iron and steel slag is also used as aggregate. In 2005, 25,747,000 tons of blast-furnace slag and 14,897,000 tons of steelmaking (converter and electric-furnace) slag were generated in Japan [Nippon Slag Association 2005]. Of these, blast-furnace slag was primarily used as aggregate for concrete, and its amount was 3,158,000 tons.

While fly ash has been used as a substitute admixture for cement, the idea of using this as fine aggregate has emerged in recent years [Shikoku Chapter of Japan Society of Civil Engineers 2003]. Considering the worldwide increase in fly ash production expected in the future, it is desirable to design a mix that integrates substitutes for cement and fine aggregate.

In recent years, melting treatment of waste and sewage sludge has become common in Japan due to pressures on waste disposal site and problems related to dioxin and heavy metals. Molten slag is generated as a residue after melting treatment. In 2004, 144 waste and 18 sewage sludge melting treatment facilities were in operation throughout Japan, producing 480,000 and 44,000 tons of molten slag, respectively.

Waste molten slag production is estimated to reach as high as 2.7 million tons in the future. Melting treatment was also commenced in 2005 for 600,000 tons of illegally dumped industrial waste in Teshima, Kagawa Prefecture [Takatsuki 2003]. Approximately 300,000 tons of molten slag is to be produced for 10 years, and the prefecture government has decided to use the entire amount as aggregate for concrete. Basic studies on the application of municipal solid waste and Teshima molten slag to concrete have been conducted [Matsuka, Sakai, Nakamura & Kusanagi 2006a], [Matsuka, Sakai, Nishigori, Yokoyama, Nishimoto & Onodera 2006b]. Molten slag is characterized by low water-retentivity since it is vitreous. When it is used in large amounts, bleeding increases and leads to a decrease of the strength of concrete and, in the case of reinforced concrete, a decrease in the bond of reinforcing bars to concrete. The permissible replacement ratio of molten slag for fine aggregate thus varies according to the performance required for concrete. It is also necessary to pay attention to aluminum, which is a metal contained in molten slag, since it may cause expansion and deterioration of concrete as it reacts with the alkali of cement and generates hydrogen. In Japan, JIS A 5031 [Japanese Standards Association 2006] was established for molten slag aggregate for concrete. Table 3 lists the physical properties of molten slag required by this standard.

2.2 Cement

Cement is the most important basic material for infrastructure development. There are two aspects of
cement production and its relation to the environment—CO₂ emissions and the use of waste as a raw material and fuel. Japan's cement production, including blended cement and export, has amounted to more than 800 million tons in the last 10 years. The world's cement production is thought to be approximately 2 billion tons at present. Figure 6 shows the cement demand predicted by Humphreys et al. [2002]. According to this prediction, production is expected to reach approximately 4 billion tons 30 years from now. Figure 7 displays the CO₂ emission unit of cement production in each country/region. Based on the mean value 0.87(kg-CO₂/kg-Cement) of these, CO₂ emissions from cement production totaling 4 billion tons is estimated to be approximately 3.5 billion tons, which is 2.7 times as high as Japan's current total CO₂ emissions (approx. 1.3 billion tons). Also, based on the lowest CO₂ emission unit of Japan (0.73), the world's CO₂ emissions are expected to reach approximately 2.9 billion tons 30 years from now. This means that the world's cement-originated CO₂ emissions can be reduced by approximately 8% through the use of Japan's cement production technology.

Nearly 30 million tons of waste and byproducts are currently used as raw materials and fuels for cement production in Japan. Such raw materials include blast-furnace slag, fly ash and sewage sludge, and fuels include waste tires and plastic. Figure 8 illustrates the changes in cement production and use of waste and byproducts in Japan. It can be seen that the use of waste and byproducts is steadily increasing while cement production is decreasing. The cement industry is setting a target value of 400 kg/t-cement for 2010. Figure 9 shows an international comparison of energy consumption per ton of cement clinker, in which Japan displays incomparably high energy efficiency. It can be seen from this that Japan has exceptional cement production technology. This means that CO₂ emissions associated

<table>
<thead>
<tr>
<th>Items</th>
<th>Molten slag</th>
</tr>
</thead>
<tbody>
<tr>
<td>Items</td>
<td>Coarse aggregate</td>
</tr>
<tr>
<td>Oven-dry density, g/cm³</td>
<td>not less than 2.5</td>
</tr>
<tr>
<td>Water absorption, %</td>
<td>not more than 3.0</td>
</tr>
<tr>
<td>Soundness, %</td>
<td>not less than 12</td>
</tr>
<tr>
<td>Solid volume percentage for shape determination, %</td>
<td>not less than 55</td>
</tr>
<tr>
<td>Amount of material passing test sieve 75µm, %</td>
<td>not more than 1.0</td>
</tr>
</tbody>
</table>

Table 3. Physical properties requirements for molten slag aggregates.
with the construction of concrete structures is being reduced as a consequence.

2.3 Admixtures

It can be said that basic technologies have already been established for the use of admixtures in concrete [Japan Society of Civil Engineers 1996] [Japan Society of Civil Engineers 1999]. Admixtures have been used for concrete for the effective use of industrial byproducts and improvement in concrete properties through their utilization. In recent years, however, their importance in the reduction of environmental impacts has also attracted attention.

As shown in Figure 10, approximately 67% of blast-furnace slag was used for blast-furnace cement in Japan in 2005. This means that the performance of blast-furnace slag is used in the most rational manner. Figure 11 presents the breakdown of the use of coal ash (9,792,000 tons) in Japan in 2004 [Japan Coal Energy Center 2006]. Of the 70% used in the field of cement, the majority was used as a raw material for cement production. Although the amount of fly ash that can be used for concrete is approximately 20% of the total production of coal ash, its use as an admixture for concrete is only about 1%. This situation is not appropriate considering the excellent performance of fly ash as an admixture. Approximately 10% of the coal ash produced in Japan is currently disposed of in landfills.

Figure 12 shows the estimated volumes of admixture production in the world in 2002 and 2020 [Jahren 2003]. In particular, the production volumes of fly ash and blast-furnace slag are expected to increase 1.7 and 3 times, respectively. By effectively using these increased volumes of admixtures, it will become possible to reduce environmental impacts caused by the future increase in concrete demand.
3 JSCE RECOMMENDATION OF ENVIRONMENTAL PERFORMANCE VERIFICATION FOR CONCRETE STRUCTURES

The Japan Society of Civil Engineers (JSCE) published the Recommendation of Environmental Performance Verification for Concrete Structures (draft) [Japan Society of Civil Engineers 2006] in 2005. This Recommendation (draft) provides general principles of consideration concerning environmentality when conducting design, construction, use, maintenance/management, dismantling, disposal and reuse after dismantling of a concrete structure. Its purpose is to extend the application of performance verification concept on safety, serviceability and durability of concrete structures to the “environment.” The Recommendation (draft) consists of chapters on general rules, environmentality, evaluation and verification of environmental performance, inspection and records. The followings are listed as items that must be taken into account when considering the environmental aspects of concrete structures:

(1) Greenhouse gas, air contaminants, resources/energy, waste
(2) Water and soil contaminants
(3) Noise/vibration
(4) Others

The followings are also provided as methods to be used for evaluation of environmental performance:

(1) LCA method for evaluation of the emission of greenhouse gases and air contaminants, consumption of resources/energy, waste generation, etc.
(2) Appropriate testing and measurement methods for identification and quantitative evaluation of the substances causing water and soil contamination
(3) Direct measurements, reliable prediction methods or a combination of both for evaluation of noise, vibration, etc.
(4) Others

The Recommendation (draft) also gives examples of basic environmental impact units and integration factors necessary for evaluation of environmental performance related to design, construction and other aspects of concrete structures.

The verification of environmental performance is the act of confirming that the value of performance retained by a structure (R) is larger (or smaller) than the set value (S) based on the performance requirements for the structure concerning the environmental aspect subject to the verification. For example, verification of CO₂ emissions can be passed if the calculated value of CO₂ associated with an actual structure is smaller than the absolute value of CO₂ emissions set as the performance requirement (or the value based on the reduction rate from the standard value). In the case of the use of a byproduct, the minimum amount that must be used is set as the performance requirement, which is satisfied if the amount used for a structure is larger than this value. While environmental performance requirements may include legal regulations, demands of the owners and intentions of the designer, they are referred to as the regulation value, limit value and target value, respectively, in the Recommendation (draft).

When considering the environmental performance by comparative design, the designer is not necessarily required to set a target value, and “verification” will become unnecessary if the decision-maker selects an appropriate one from the environmental performance evaluation results of several designs.

As for environment-related international standards, there is the ISO 14000 series. In ISO 14040, the principle and framework of environmental management (life cycle assessment) are provided. This standard is for general products, and its life cycle assessment is composed of the purpose, definition of the scope, inventory analysis, impact assessment and interpretation. A standard for buildings and civil engineering structures is ISO 15686-6 [2004]. This standard is for the consideration of environmental impacts in service life planning of structures, and provides general procedures for environmental impact assessment of design options. Also, ISO/TC59/SC17/WG3 is currently examining the draft of environmental product declaration standards for buildings and civil engineering structures [ISO/DIS21930 2006]. The purpose of this draft is to make environmental product declarations (EPD) on environmental impacts, resource use, discharge of waste and other matters by looking at buildings and civil engineering structures as “products.” Although the objects of these two ISO standards (drafts) are identical, their ideas and methods are different.

In the design of concrete structures, safety, serviceability and durability have been verified. In the future, the consideration of environmentality will become necessary in addition to these. It is, however, obvious that environmentality is essentially different from safety etc. This means that, in the case of design for safety, a structure is required not to cause direct damage to people or society by its destruction. As for environmentality, no systems have been established for the control of greenhouse gases, air contaminants, resources/energy and waste, although direct contamination and noise have been taken into account under legal regulations. This is because the final damage of global environment issues has yet to be clarified. Although it is easy to imagine that various substances generated as a result of human activities will eventually cause damage to humans, society and ecosystems, the problem is that it is difficult to evaluate these damages quantitatively. It is, however, necessary to prevent further aggravation of the situation.
by not taking any measures merely because of the lack of clarity regarding these damages.

The Recommendation of Environmental Performance Verification for Concrete Structures (draft) presented here was developed based on the idea that systems to enable reduction of environmental impacts should be established in a variety of concrete-related activities. While this Recommendation (draft) is based on the performance verification system of the JSCE Standard Specifications for Concrete structures, it can also be used for comparative design related to environmental performance or environmental labels/declarations of ISO. This Recommendation (draft) can therefore be considered as the concrete version of ISO environmental standards.

4 ENVIRONMENTAL BENEFITS FROM THE CONSTRUCTION OF CONCRETE STRUCTURES

Civil engineering structures are the foundation of socioeconomic activities, and have contributed greatly to the establishment of a pleasant and efficient living environment. While there is an unfortunate aspect where construction of civil engineering structures is seen as a symbol of environmental destruction, this is thought to be due to insufficient evaluation of environmental impacts caused by construction of civil engineering structures. In other words, although it is extremely important to evaluate how construction of civil engineering structures contributes to social and environmental benefits as a means of fulfilling social accountability, few comprehensive studies have been conducted, especially concerning the latter.

With these circumstances in the background, the author studied the direct environmental impact due to the construction of a reinforced concrete underpass at an intersection with chronic traffic congestion and the indirect reduction effect by the clearance of the traffic congestion, for comprehensive evaluation of the environmental benefits of the construction. The one-lane underpass used for this study was actually constructed at an intersection of six-lane national road, where traffic flows between the suburbs and center of a local city are concentrated. Figures 13 and 14 show the plan view of the intersection and cross section of the underpass, respectively. Approximately 150,000 automobiles pass through this intersection daily.

To evaluate the environmental benefits of the construction of this underpass, its environmental impacts by the construction itself were calculated. Figure 15 shows the CO₂, SOx and NOx emissions from materials manufacture, transport and construction. While CO₂ emissions from materials manufacture were high, the percentage of construction and transport increased for SOx and NOx. Next, the environmental impacts due to automobile traffic were calculated based on the traveling speed of automobiles before and after construction of the underpass. Following the construction, daily CO₂ emissions and oil use were reduced by 1.434 and 0.557 tons, respectively. The reduction of
daily SOx and NOx emissions was 9 and 1 kg, respectively. The relationship between the impact generated by the construction of the underpass and the reduction of impact by improvements in traffic can be studied for individual emission substances. Table 4 shows the calculated result of years until the counterbalance of construction impact and traffic impact reduction. However, since nonmetals, iron and wood was also used as resource consumption for the construction of the underpass, integration by the method of LIME [Itsubo & Inaba 2005] was conducted to evaluate these comprehensively. Figure 16 shows the environmental-benefit evaluation by the construction of underpass. The integrated impact on the vertical axis represents the integrated index calculated by considering weight to the respective impacts of greenhouse gases, acidification substances, air contaminants and resource consumption. This result shows that the environmental impacts generated at the time of construction of the underpass will be offset by the reduction of environmental impacts of automobile traffic in approximately 9 years. In general, the time it takes until such an offset becomes shorter and the significance of the construction of an underpass becomes higher when the traffic volume is larger and the degree of congestion is higher.

While this paper discussed an evaluation method for environmental benefits by construction of a concrete structure as an example, the construction of structures has many aspects, such as economic benefits and social benefits as a lifeline. It will eventually become necessary to evaluate the benefits of construction by considering all these factors. In any case, it is obvious from the above example that the environment is an element that must be taken into account in infrastructure development.

5 CONCLUDING REMARKS

For the concrete industry to contribute to the sustainable development of mankind, it is necessary to promote technical development for further reduction of environmental impacts. To promote this, it will be necessary to introduce environmental design systems based on environmental performance, develop environmental performance evaluation tools and construct systems for their actual application.

Since environmental consideration could lead to an increase in cost and the reduction of environmental impacts was not a design requirement, environmental impacts have been thought of as negative externality. The comprehensive evaluation bidding system is currently attracting attention in Japan as a system for enabling internalization of negative externality. The bidding system enable absorption of the cost increases resulting from environmental consideration. It is desirable to actively use this system in the future.

Since the industrial revolution, the system of human society has been formed with a focus on the economy. In other words, the cost of “things” is the most important factor in mass production and mass consumption, and environmental, cultural and social aspects have been controlled by the viewpoints of economic aspects. Fortunately, however, the human race has finally begun to realize that policies that put the economy ahead of all else may even jeopardize their survival. In the future, the human race must balance these economic, environmental, cultural and social aspects. One major problem is the regional economic disparity that currently exists in the world. It is necessary to eliminate this regional economic disparity by taking the above aspects into account. But if this cannot be done successfully, a new negative externality may emerge in the world. One of the keys is the CDM established by the Kyoto Protocol. An enormous amount of concrete will be consumed throughout the world in the future as an

<table>
<thead>
<tr>
<th>Items</th>
<th>Impact due to construction (kg)</th>
<th>Traffic impact reduction per day (kg/day)</th>
<th>Years until counterbalance</th>
</tr>
</thead>
<tbody>
<tr>
<td>CO2</td>
<td>4862629</td>
<td>1434.3</td>
<td>9.3</td>
</tr>
<tr>
<td>NOx</td>
<td>23503</td>
<td>8.5</td>
<td>7.6</td>
</tr>
<tr>
<td>SOx</td>
<td>3660</td>
<td>0.8</td>
<td>12.2</td>
</tr>
<tr>
<td>Oil</td>
<td>745898</td>
<td>556.7</td>
<td>3.7</td>
</tr>
</tbody>
</table>

Table 4. Years until counterbalance of construction impact and traffic impact reduction.

Figure 16. Environmental-benefit evaluation by the construction of underpass.
important material for infrastructure development. And since the sustainable development of the world is directly connected to the survival of the human race, it is the duty of advanced countries to develop technologies and systems for environmental impact reduction related to concrete, enlighten other advanced countries that are not environmentally conscious and transfer technologies to developing countries.

REFERENCES

Jahren, P. 2003. What are the options? The CO2 case. STF22A03610 Report, SINTEF.
Site trials of concrete with a very low carbon footprint

P.A. Claisse, E. Ganjian & H. Sadeghi-Pouya
Faculty of Engineering and Computing, Coventry University, Priory Street, Coventry, UK

ABSTRACT: This paper reports on six site trials of sustainable concrete mixtures. The first three were concrete barriers for landfill, the fourth was a trench fill and the fifth and sixth were roads and car parks. The methods and mixtures evolved from moderately conventional designs containing ashes and slags and some cement which were blended at the plant to a super-sulphated pre-blended powder containing waste plasterboard and no cement. The strengths were not as high as a typical structural concrete but were fully adequate for each application. The paper concludes that while the technical viability of each mix was demonstrated, problems with insurance, capital funding, and environmental regulators have prevented their wider application.

1 INTRODUCTION

1.1 The six trials

This paper reports on six site trials of sustainable concrete mixtures carried out by the Construction Materials Applied Research Group at Coventry University. The objectives of the paper are:

• To illustrate how a wide and evolving range of sustainable concrete mixtures have performed well under site conditions.
• To investigate the design and exploitation of the site trials.
• To discuss how trials of this type may be optimised for their primary purpose which is to persuade industrial enterprises to exploit the results of the underlying research.
• To demonstrate a wide range of applications for concrete with relatively low strength.

The first three trials were intended to demonstrate the viability of concrete barriers as part of a composite waste containment system for landfills. They were constructed in 1999–2000 with a total of 70 m³ of concrete. Each cell contained two different concrete mixtures.

The fourth trial was a trench trial of a single mix intended for use as mine or trench backfill. It was constructed in 2004 with 7 m³ of concrete.

The fifth trial was a slab in a car park built in 2006 with 16 m³ of semi-dry concrete.

The sixth trial was an access road which was constructed in 2006 by stabilising 72 m³ of soil with a sustainable “cement” and then placing 6 m³ of a semi-dry paste (grout) as a road base.

Extensive monitoring of the first three trials was carried out until 2006 when they were demolished. Monitoring of the fourth is planned and is in progress on the fifth and sixth.

1.2 The “novel blended cement”

For the first four trials the cementitious components were delivered to the batching plant and mixed on site (in the plant for mixes trials 1 to 3 and in the truck-mixer for trial 4). This was found to be inaccurate and difficult and also unlikely to lead to commercial exploitation. Therefore for trials 5 and 6 a pre-blended powder was made which could be treated exactly like cement at a batching plant.

The components of the “novel blend” are:

• Basic oxygen slag from steel manufacture (80%)
• Waste plasterboard (15%)
• Kiln by-pass dust from cement manufacture (5%)

These components were chosen partially because the client was interested in finding uses for waste gypsum. Once the gypsum had been included to make a “super-sulphated” mix any cement could have caused problems with expansion. The components were inter-ground to form a grey powder which looks exactly like cement.

2 TRIALS 1–3

2.1 Introduction

Three cells were constructed on a licensed landfill operation site at Risley, Cheshire UK with different cementitious composite mineral waste materials

### 2.2 Layout and construction methods of the cells

A typical test cell is shown schematically in Figure 1. The barrier is made up of two layers of concrete with a layer of clay between them. These inverted pyramid shape cells measured 8 metres wide and contained waste to a maximum depth of 1.1 metres. The slopes of the cell sides were 30° and the cells contained 5.4 m³ of waste. The excavation was carried out with a hydraulic excavator which was also used to place the concrete and waste mixes designed for the different cells. The compaction of concrete layers was carried out by two poker vibrators and the compaction of clay layer was carried out using the outside surface of the excavator's bucket.

### 2.3 Mixture designs

The primary containment method for the barriers is “chemical” containment. In this system any leachate entering the barrier is buffered to high pH which causes harmful species to precipitate and become bound in the barrier. The mix designs were therefore developed to give a good buffering capacity. This was measured by monitoring the “through pH” of the eluent from the permeability test cells. The mix designs and their properties are shown in tables 1 and 2. Waste sodium sulphate solution from lead-acid battery recycling was used as mix water in two mixtures.

### 2.4 Observations from the construction

During the construction of cell numbers 2 and 3 the mix proportions actually used were different to what was designed in laboratory due to some practical problems encountered in the batching plant and placement of some of materials (inaccurate weightings of different materials and partial hydration of CKD while stored at the plant). The mixes actually made were tested and showed higher permeabilities than the mixes designed initially in the laboratory.

### 2.5 Emplacement of waste and leachate

Due to size and shape constraints of the cells shredded waste was used. It was placed and compacted up to the top level of the test cells. A leachate which provided the most aggressive solution representing the leachates found in the landfill was obtained from the leachate treatment plant for the site and the cells were filled 100 mm below the top giving a 1 m head at the deepest point. The cells were covered with a tarpaulin rain cover to prevent rainwater ingress and contain odour.

### 2.6 Instrumentation and sampling

Two types of sampling lines were used between the layers of the cell liners using 3 mm plastic tubes in both. In one type the end of the 3 mm plastic tubes were glued inside porous stone discs of 60 mm diameter. In the other type the layer was drilled and the 3 mm plastic tubes ends were sealed in place in the set concrete with sponge around the end of the line. The sampling lines were placed as an array in the various liner materials.

---

**Figure 1. Typical trail test cell layout.**

<table>
<thead>
<tr>
<th>Barrier (lower concrete) width</th>
<th>7.87m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Waste depth</td>
<td>1.1m</td>
</tr>
<tr>
<td>Tarpaulin supported on timbers (buried in trench at edges)</td>
<td></td>
</tr>
</tbody>
</table>
Table 1. Composition of mixes used in the three trial cells at Risley.

<table>
<thead>
<tr>
<th>Composition of top layer concrete for cell No.1:</th>
<th>Proportions used</th>
<th>kg/m³</th>
<th>% By mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spent Borax from silver refining</td>
<td>450</td>
<td>18.5</td>
<td></td>
</tr>
<tr>
<td>Ferrosilicate sand</td>
<td>895</td>
<td>36.8</td>
<td></td>
</tr>
<tr>
<td>20 mm Limestone</td>
<td>1085</td>
<td>44.7</td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>210</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Composition of top layer mortar for cell No.2:</th>
<th>Proportions used</th>
<th>kg/m³</th>
<th>% By mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ferrosilicate slag sand (&lt;5 mm)</td>
<td>1575</td>
<td>65.9</td>
<td></td>
</tr>
<tr>
<td>Cement Kiln Dust (CKD) – 60%</td>
<td>490</td>
<td>20.5</td>
<td></td>
</tr>
<tr>
<td>Lagoon Ash from coal burning – 40%</td>
<td>325</td>
<td>13.6</td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>200</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Composition of top layer concrete for cell No.3:</th>
<th>Proportions used</th>
<th>kg/m³</th>
<th>% By mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ferrosilicate slag (&lt;20 mm)</td>
<td>715</td>
<td>29.8</td>
<td></td>
</tr>
<tr>
<td>Limestone (≤20 mm)</td>
<td>1105</td>
<td>46</td>
<td></td>
</tr>
<tr>
<td>Ferrosilicate slag sand (&lt;5 mm)</td>
<td>340</td>
<td>14.2</td>
<td></td>
</tr>
<tr>
<td>Lagoon Ash from coal burning – 40%</td>
<td>240</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>220</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Composition of lower layer concrete for cell No.1:</th>
<th>Proportions used</th>
<th>kg/m³</th>
<th>% By mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground Granulated Blastfurnace Slag (GGBS)</td>
<td>180</td>
<td>7.6</td>
<td></td>
</tr>
<tr>
<td>Ordinary Portland Cement (OPC)</td>
<td>20</td>
<td>0.9</td>
<td></td>
</tr>
<tr>
<td>Chrome Alumina Slag (40 mm)</td>
<td>1515</td>
<td>64.2</td>
<td></td>
</tr>
<tr>
<td>Green Sand (ex-casting)</td>
<td>645</td>
<td>27.3</td>
<td></td>
</tr>
<tr>
<td>Na₂SO₄ solution from battery recycling</td>
<td>295</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Composition of lower layer concrete for cell No.2:</th>
<th>Proportions used</th>
<th>kg/m³</th>
<th>% By mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chrome Alumina Slag (&lt;40 mm)</td>
<td>1175</td>
<td>49.6</td>
<td></td>
</tr>
<tr>
<td>Chrome Alumina Slag (&lt;5 mm)</td>
<td>720</td>
<td>30.4</td>
<td></td>
</tr>
<tr>
<td>Green sand from casting moulds</td>
<td>100</td>
<td>4.2</td>
<td></td>
</tr>
<tr>
<td>Cement Kiln Dust – 60%</td>
<td>165</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>Pulverised Fuel Ash – 40%</td>
<td>210</td>
<td>8.8</td>
<td></td>
</tr>
<tr>
<td>T1Sodium sulphate Solution (lt)</td>
<td>165</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Composition of lower layer concrete for cell No.3:</th>
<th>Proportions used</th>
<th>kg/m³</th>
<th>% By mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chrome Alumina Slag (&lt;40 mm)</td>
<td>1175</td>
<td>50.3</td>
<td></td>
</tr>
<tr>
<td>Chrome Alumina Slag (&lt;5 mm)</td>
<td>720</td>
<td>30.8</td>
<td></td>
</tr>
<tr>
<td>Green sand</td>
<td>110</td>
<td>4.7</td>
<td></td>
</tr>
<tr>
<td>Ordinary Portland Cement – 5.2%</td>
<td>185</td>
<td>7.9</td>
<td></td>
</tr>
<tr>
<td>Cement Kiln Dust – 69.8%</td>
<td>120</td>
<td>5.2</td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>240</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 2. Properties of concrete used in cells 1,2 and 3.

<table>
<thead>
<tr>
<th></th>
<th>7 days strength (MPa)</th>
<th>28 days strength (MPa)</th>
<th>Intrinsic permeability to water @ 28 days (m/s)</th>
<th>Intrinsic permeability to leachate @ 28 days (m/s)</th>
<th>Thro’ pH water#</th>
<th>Through pH leachate#</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cell 1 top</td>
<td>5</td>
<td>4.5</td>
<td>$1.5 \times 10^{-8}$</td>
<td>$4.0 \times 10^{-8}$</td>
<td>10</td>
<td>–</td>
</tr>
<tr>
<td>Cell 1 base</td>
<td>11</td>
<td>13</td>
<td>No flow</td>
<td>$2 \times 10^{-12}$</td>
<td>–</td>
<td>8.5</td>
</tr>
<tr>
<td>Cell 2 top</td>
<td>1.1</td>
<td>1.7</td>
<td>$4.5 \times 10^{-9}$</td>
<td>$5 \times 10^{-9}$</td>
<td>11.8</td>
<td>12.3</td>
</tr>
<tr>
<td>Cell 2 base</td>
<td>4.4</td>
<td>6.9</td>
<td>$2.3 \times 10^{-9}$</td>
<td>$4.5 \times 10^{-9}$</td>
<td>10.1</td>
<td>9.9</td>
</tr>
<tr>
<td>Cell 3 top</td>
<td>0.9</td>
<td>1.3</td>
<td>$1.2 \times 10^{-8}$</td>
<td>$7.5 \times 10^{-9}$</td>
<td>12.2</td>
<td>12.1</td>
</tr>
<tr>
<td>Cell 3 base</td>
<td>2.8</td>
<td>6</td>
<td>$1.2 \times 10^{-8}$</td>
<td>$6.2 \times 10^{-9}$</td>
<td>8.5</td>
<td>7.6</td>
</tr>
</tbody>
</table>

* Initial pH of leachate: 5.1–5.4.
# The “through pH” is the pH of the outflow from the permeability test.
and levels. Liquid samples were obtained by applying a vacuum to the lines.

2.7 Initial observations

Cell 3 needed to be refilled after 12 months. The reason for this was inadequate compaction of the clay layer leading to an increased permeability. The effective indicated permeability was calculated and found to correspond to nearly the same permeability as a Bentonite Enhanced Sand liner and indicates satisfactory performance even when very poor construction practice was evident but it did affect the modelling considerably. The permeability calculated from these site observations was therefore used for the clay layer.

2.8 Comparison between model and observations

The capacity factor, and diffusion coefficient values obtained from laboratory diffusion tests on the top and bottom mixes used in the site trial cells together with the initial concentrations of different elements in site leachate and the mixes used in the cell (from pore pressed solutions) were used to model the transport in the barriers. The model simulates pressure driven flow and diffusion using a finite step code written to run as a macro in Microsoft Excel. The modelled concentration is plotted against measured collected sample concentration values in figures 2 to 4 for Ca, Na and K for cell 2. On these graphs error bars are shown between the 10th and 90th percentiles from the probability calculations at ages of two and four years. The observed concentrations which are shown on the graph are based on the average from up to four different samples taken in different parts of the cells. For some of these a considerable spread of results was recorded.

3 TRIAL 4

3.1 Introduction

This trial was a trench fill. The total quantities of materials used were:
Red Gypsum: 5.3 Tonne
BOS weathered slag: 8 Tonne
Water: 2400 litres
Calculated yield: 7.4 m³

This mix was selected to give a strength which was adequate for trench filling but not too high for subsequent excavation if this became necessary.
Mix procedure.

Both the slag and the gypsum were supplied in sling bags containing approximately one tonne each. These were discharged onto a conveyor and loaded directly into a truck-mixer. The trench was approximately 5 meters long by 1 meter wide by 2 meters deep. The mix filled the trench to within 0.5 meters from the top.

3.2 Observations

3.2.1 From loading mixer
• Some difficulties were experienced with the conveyor and the entire loading process took approximately two hours.
• The anticipated problems with handling the red gypsum did not occur.
• The gypsum had not consolidated in the sling bags and flowed freely out of them.
3.2.2 From samples taken
- The flow of the first sample was measured to be 545 mm (very close to 560 mm target).
- Some unmixed lumps of red gypsum were observed. These were extracted using a 5 mm sieve. The quantity was observed to decrease from the first to the second sample but increased again for the third sample.
- Initial set was 1.5 hours from discharge and final set approximately 7 hours.

3.2.3 From discharge
- The material was observed to flow freely in the trench.
- A small number of agglomerated lumps up to 150 mm in size were observed.
- The material was self-compacting and air could be seen escaping (venting to surface) indicating that vibration was not necessary.
- The trench into which the mix was poured had partly filled up with rain water since it was pumped out earlier in the morning. The first 0.5 m³ discharge into the trench initially appeared to mix with the water in the trench, but very rapidly “clear” water separated out on top of the mix. The mix did not appear to be adversely affected by being poured into standing water. It is suggested that this degree of cohesion is superior to the performance of foamed concrete where some grout loss might be observed in these circumstances.
- The surface of the pour remained reasonably flat along the trench even though it was only poured from one end. It was difficult to see the exact slope due to the free water on the surface but the final slope was minimal.

3.3 Testing
The 28 days result of the average of 3 cubes (50 mm) which were taken from top, middle and bottom of the truck mixer load was 1.8 MP

4 TRIAL 5

4.1 Introduction
A 6 m by 17 m area of car park was used for a sub-base trial. The area was stripped of the existing hardcore to expose the sub grade, which was hard clay. This design was adopted to be similar to the existing layers of constructed car park area to provide comparable data between conventional sub-bases and the RCC layer.

The mixture design used the “novel blend” and is shown in table 3.

4.2 Placing concrete and compaction
To give a thickness of 100 mm after compaction a layer of 160 mm of concrete was placed and levelled manually. As the concrete was delivered in three truck loads; placing and compaction of RCC layer was carried out in three segments of the allocated area. As a result, a slightly different moisture content and compaction level was expected for sections. The workability of the mixes on site was, however, observed to be very consistent. The hot weather on the day was compensated for with accurate batching of water content.

The placed concrete layer was then compacted using a 3-Tonne vibrating roller.

4.3 Testing
The test results are shown in table 4.

5 TRIAL 6

5.1 Introduction
A 22 m length of the site access road (4 m wide) was constructed using stabilised soil and a paste (grout) sub-base both of which used the “novel blend” in place of cement.

Table 5 shows the proportions of materials used for soil stabilisation and semi-dry paste. The volume of stabilised soil needed for the site trial was estimated at about 72 m³ in a loose condition including wastage. For the semi-dry paste, the volume of material needed was estimated to be about 6 m³. Half of the area was constructed using a conventional base course with same thickness as the semi-dry compacted paste. Finally, the surface of the whole road was paved using 50 mm bituminous of wearing surface.

The blended powder was spread over the area using a volumetric method. In order to achieve the required
amount of binder per cubic metre of the compacted soil, 150 mm of the blended powder was spread over the trail area. The mixture of soil and powder was blended using a rotavating blending machine. The powerful and heavy blade of the machine provided a homogeneous blend of soil and binder along the road.

As the natural moisture of the soil used was not enough to provide the optimum compaction of stabilised soil, extra water was added to the mixture using a mobile sprinkler. Then the mixture was blended again using rotavating blender and levelled using the JCB. The control of the water content visually was quite challenging at that stage because of the hot weather and quick evaporation. However, despite of this, the moisture content and compaction of the stabilised soil was satisfactory.

### 5.3 Construction method – base course

A layer of 100 mm semi-dry compacted paste was designed to be laid on top of the stabilised soil. As the water content of the paste was limited to 13%, a volumetric mixer was used to mix the blended binder with water. The mixer contained a vessel to accommodate the binder and a 1600 litre tank of water. The binder passed to a screw by means of a belt conveyor where the water was added and mixed in the extending arm. The only challenge was measuring the amount of water because the mixer was not equipped by any means to measure the water added to the mixture. Therefore, the required amount water was adjusted based on visual inspection and past experience.

### 5.4 Test results

The test results are shown in tables 6 and 7.

### 6 DISCUSSION

#### 6.1 Strengths

The strengths of the mixtures from all of the trials (except the soil stabilisation) are summarised in table 8. It may be seen that the material for trials 5 and 6 are the highest despite having no ordinary cement in it. This was helped by the very low water contents in the mixtures but it still demonstrates that viable mixtures can be made without cement. The “novel blend” was not designed for use as a partial replacement of cement and is not recommended for use with any cement.

#### 6.2 Interpreting the trials

Transport processes in landfills are associated with a high degree of uncertainty. The processes which were modelled in trials 1–3 used real landfill leachate which was undergoing biological reactions throughout the experiment and took place in a site environment with all of the associated uncertainty. The combination of these factors with the uncertainty associated with the stated assumptions in the model has given rise to some unexpected events. Nevertheless this is a useful exercise to indicate likely trends in a real environment. Long term results which will involve transport processes far closer to the steady state are actually likely to be more accurate.

In all of the trials the batching was less accurate than it was in the laboratory work but in every case the mix proved tolerant enough to perform adequately. The conditions of a site trial are likely to be worse than full-scale production because of problems with plant. For example in trial 4 the mixing in the truckmixer

<table>
<thead>
<tr>
<th>Mix Code</th>
<th>Soil %</th>
<th>Binder %</th>
<th>Moisture content %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stabilised soil (soil50/Binder50)</td>
<td>50</td>
<td>50</td>
<td>14</td>
</tr>
<tr>
<td>Semi-dry paste (PG15/BPDS80-.13)</td>
<td>–</td>
<td>100</td>
<td>13</td>
</tr>
</tbody>
</table>
was visibly incomplete but the mix still performed well. In each case it was concluded that the mix would perform adequately in full scale use. The road in trial 6 has been used by heavy vehicles and plant for a considerable time and is performing very well with no signs of distress.

6.3 Future use of the research

The experimental programme for trials 1–3 cost over 0.5 million pounds and the barriers were scheduled to be tested in a full-scale waste containment cell (approximately one hectare) but the site operator’s insurers would not carry the risk, despite further containment being designed to go below the experimental liner.

A major mine backfill project has been planned with the mix from trial 4. This would involve placing 100,000 m$^3$ per year for the foreseeable future (the void to be filled is 10 million m$^3$). The material would be poured down bore-holes into a partially flooded mine. This project has received full regulatory approval but awaits funding. This project would require a significant capital investment and the company has recently been taken over so there may be a long delay.

A further site trial was planned with the “novel blend” used in trials 5 and 6. This would have involved filling a dis-used pedestrian subway in Coventry city centre. Despite being fully contained by the existing structure this pour did not receive approval from the Environment Agency and will not now go ahead. The environmental concerns were all addressed but

---

### Table 6. Compressive strength of stabilised sub-grade together with in-situ density and moisture.

<table>
<thead>
<tr>
<th>Mix Code</th>
<th>Laboratory tests</th>
<th>Insitu tests</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strength at days (MPa)</td>
<td>Density kg/m$^3$</td>
</tr>
<tr>
<td>Unstabilised Soil-B (Lab.)</td>
<td>0.08 0.21 0.13 13.40</td>
<td>1835.1</td>
</tr>
<tr>
<td>Soil-B 50/Binder 50 (Lab.)</td>
<td>0.94 1.43 5.98 13.90</td>
<td>2030.15</td>
</tr>
<tr>
<td>Soil 50/Binder 50 (Site-Mid)</td>
<td>0.78 1.11 5.80 8.02</td>
<td>2033.06</td>
</tr>
<tr>
<td>Soil 50/Binder 50 (Site-End)</td>
<td>0.96 1.15 5.10 7.90</td>
<td>2027.55</td>
</tr>
</tbody>
</table>

### Table 7. Compressive strength of laboratory and site semi-dry paste layer.

<table>
<thead>
<tr>
<th>Mix Code</th>
<th>Laboratory tests</th>
<th>Insitu tests</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strength at days (MPa)</td>
<td>Density kg/m$^3$</td>
</tr>
<tr>
<td>Semi dry paste (Lab.)</td>
<td>5.10 12.80 30.55 13</td>
<td>2540</td>
</tr>
<tr>
<td>Semi dry paste (Site-Begin)</td>
<td>3.20 6.64 19.90 8.55</td>
<td>2487</td>
</tr>
<tr>
<td>Semi dry paste (Site-End)</td>
<td>2.95 6.20 22.40 7.76</td>
<td>2492</td>
</tr>
<tr>
<td>Core (Mid)</td>
<td>- - -</td>
<td>-</td>
</tr>
<tr>
<td>Core (End)</td>
<td>- - -</td>
<td>-</td>
</tr>
</tbody>
</table>

### Table 8. Summary of strengths.

<table>
<thead>
<tr>
<th>Trial</th>
<th>Pour</th>
<th>Cementitious component</th>
<th>Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Cell 1 top</td>
<td>Spent borax 100%</td>
<td>4.5</td>
</tr>
<tr>
<td>2</td>
<td>Cell 2 top</td>
<td>CKD 60%, Lagoon ash 40%</td>
<td>1.7</td>
</tr>
<tr>
<td>3</td>
<td>Cell 3 top</td>
<td>CKD 60%, Lagoon ash 40%</td>
<td>1.3</td>
</tr>
<tr>
<td>1</td>
<td>Cell 1 base</td>
<td>GGBS 90%, OPC 10%, Sodium sulphate</td>
<td>13</td>
</tr>
<tr>
<td>2</td>
<td>Cell 2 base</td>
<td>CKD 60%, PFA 40%, Sodium sulphate</td>
<td>6.9</td>
</tr>
<tr>
<td>3</td>
<td>Cell 3 base</td>
<td>OPC 5%, CKD, 70%, Lagoon ash 25%</td>
<td>6</td>
</tr>
<tr>
<td>4</td>
<td>Trench fill</td>
<td>BOS 60%, Red Gypsum 40%</td>
<td>1.8</td>
</tr>
<tr>
<td>5</td>
<td>Sub-base</td>
<td>BOS 80%, PB 15%, BPD 5%</td>
<td>10.8</td>
</tr>
<tr>
<td>6</td>
<td>Base course</td>
<td>BOS 80%, PB 15%, BPD 5%</td>
<td>30.55</td>
</tr>
</tbody>
</table>
the relevant committee twice delayed the decision to subsequent monthly meetings and did not give approval in time. The matter was complicated because the committee mistakenly associated the trial with complications at the local cement works including tyre burning trials and a release of kiln dust which spread across a large part of the town.

Nevertheless the pressure is growing to reduce the environmental impact from cement production and site trials represent the only route to take solutions from the laboratory into practice.

7 CONCLUSIONS

Viable mixtures which contain little or no Portland cement can be made for a wide variety of applications. Site trials represent the best route to develop these mixtures for commercial use.

Pre-blended mixtures are the best way to use powder which contains several mineral wastes.

While it is relatively easy to demonstrate the viability of cementitious mixtures which are sustainable there are many difficulties which may prevent their industrial use. These include:

- Insurance problems
- Lack of capital investment
- Environmental concerns which may or may not have any scientific basis.

ACKNOWLEDGEMENTS

The authors acknowledge the considerable support provided to enable us to carry out the work reported in this paper. Supporting organisations have included ENTRUST, MIRO, Lafarge PLC, Huntsman Tioxide, WRAP, EPSRC and NERC and our major partners have been Imperial College and Birmingham University.

REFERENCES


Sustainability of the cement and concrete industries

T.R. Naik

UWM Center for By-Products Utilization, University of Wisconsin-Milwaukee, Milwaukee, WI, USA

ABSTRACT: Sustainability is important to the well-being of our planet, continued growth of a society, and human development. Concrete is one of the most widely used construction materials in the world. However, the production of portland cement, an essential constituent of concrete, leads to the release of significant amount of CO₂, a greenhouse gas (GHG); production of one ton of portland cement produces about one ton of CO₂ and other GHGs. The environmental issues associated with GHGs, in addition to natural resources issues, will play a leading role in the sustainable development of the cement and concrete industry during this century. For example, as the supply of good-quality limestone to produce cement decreases it will become more difficult to produce adequate amounts of portland cement for construction. There is a possibility that once there is no more limestone, say in a geographical region, and thus no portland cement, all of the employment associated with the concrete industry as well as new construction projects will be terminated. Due to limited natural resources, concern over GHGs, or, both, there are regions in the world where cement production is being curtailed, or, at least, cannot be increased to keep up with the population increase. Therefore, it is necessary to look for sustainable solutions for future concrete construction. A sustainable concrete structure is constructed in order to ensure that the total environmental impact during its life cycle, including its use, will be minimal. Sustainable concrete should have a very low inherent energy requirement, be produced with little waste, be made from some of the most plentiful resources on earth, produce durable structures, have a very high thermal mass, and be made with recycled materials. Sustainable constructions have a small impact on the environment. They use “green” materials, which have low energy costs, high durability, low maintenance requirements, and contain a large proportion of recycled or recyclable materials. Green materials also use less energy and resources, and can lead to high-performance cements and concrete. Concrete must keep evolving to satisfy the increasing demands of all of its users. Designing for sustainability means accounting for the short-term and long-term environmental consequences in the design.

1 INTRODUCTION

According to the World Commission on Environment and Development of the UN, sustainability means “meeting the needs of the present without compromising the ability of the future generations to meet their own needs” [UNFCCC COP9 Report 2004]. The sustainability of the cement and concrete industries is imperative to the well-being of our planet and to human development. However, the production of portland cement, an essential constituent of concrete, leads to the release of a significant amount of CO₂ and other greenhouse gases (GHGs) [Malhotra 2004]. The environmental issues associated with CO₂ will play a leading role in the sustainable development of the cement and concrete industry during this century. One of the biggest threats to the sustainability of the cement industry is the dwindling amount of limestone in some geographical regions. Limestone is essential to the production of portland cement. As limestone becomes a limited resource, employment and construction associated with the concrete industry will decline. Therefore, those involved with these industries must develop new techniques for creating concrete with a minimal use of limestone. Concrete production is not only a valuable source of societal development, but also a significant source of employment. Concrete is the world’s most consumed man-made material. It is no wonder that in the U.S.A. alone, concrete construction accounted for two million jobs in 2002 [United States House Resolution 394 2004]. About 2.7 billion m³ of concrete was produced in 2002 worldwide. This equals to more than 0.4 m³ of concrete produced per person annually worldwide. Therefore, to create not only sustainable societal development, but also to sustain employment, such as batch plant operators, truck drivers, ironworkers, laborers, carpenters, finishers, equipment
operators, and testing technicians, as well as professional engineers, architects, surveyors, and inspectors, the concrete industry must continue to evolve with the changing needs and expectations of the world.

2 WHAT IS SUSTAINABILITY?

Limestone is used to manufacture portland cement (Fig. 1). Currently, portland cement is the most commonly used material in producing concrete.

Entire geographical regions are running out of limestone resources to produce cement. And major metropolitan areas are running out of materials to use as aggregates for making concrete. Sustainability requires those in the construction industry to take the entire life-cycle, including construction, maintenance, demolition, and recycling of buildings into consideration [McDonough et al. 1992, Worrell & Galtisky 2004].

A sustainable concrete structure is one that is constructed such that the total societal impact during its entire life-cycle is minimal. Designing with sustainability in mind includes accounting for the short-term and long-term consequences of the structure. In order to decrease the long-term impact of structures, the creation of durable structures is paramount.

Building in a sustainable manner and scheduling appropriate building maintenance are significant in the “new construction ideology” of this new century. In particular, to build in a sustainable manner means to focus attention on the effects on human health, energy conservation, and physical, environmental, and technological resources for new and existing buildings. It is also important to take into account the impact of construction technologies and methods when creating sustainable structures [McDonough et al. 1992]. An integrated sustainable design process can reduce project costs and operating costs of the building or the infrastructure construction.

There are many challenges associated with portland cement production. Of these, energy and resource conservation, the cost of producing portland cement, and GHG emissions are the most significant. Therefore, supplementary cementing materials such as fly ash and slag should replace larger amounts of portland cement in concrete. However, before any construction occurs, all aspects of the building materials to be used should be evaluated. In order to build structures and infrastructures that are cost efficient, environmentally friendly, and durable, the impact of the building materials on local and worldwide air conditions must be examined [McDonough et al. 1992]. CO₂ emissions are expected to rise by about 50% by 2020 from the current levels (Table 1) due to portland cement production [Malhotra 2004].

For each ton of portland cement clinker, 1.5 to 10 kg of NOₓ is released into the atmosphere. In 2000, worldwide cement clinker production was approximately 1.5 billion tons. It means that in 2000 between 23 and 136 billion kg of NOₓ was produced to make portland cement clinker [Malhotra 2004].

If the challenges associated with reducing CO₂, NOₓ, and other GHGs are to be met, then the concrete industry must develop other materials to replace portland cement. Use of blended cements and organic chemical admixtures must be significantly increased for sustainability of the cement and concrete industries.

3 CONCRETE

For over 200 years, concrete has been accepted for its long-lasting and dependable nature. In addition to durability and dependability, concrete also has superior energy performance, is flexible in design, affordable, and is relatively environmentally friendly [Cement

Table 1. CO₂ Emissions in 2002.

<table>
<thead>
<tr>
<th>Country</th>
<th>Percent CO₂</th>
</tr>
</thead>
<tbody>
<tr>
<td>U.S.A.</td>
<td>25</td>
</tr>
<tr>
<td>E.U.</td>
<td>20</td>
</tr>
<tr>
<td>Russia</td>
<td>17</td>
</tr>
<tr>
<td>Japan</td>
<td>8</td>
</tr>
<tr>
<td>China</td>
<td>&gt;15</td>
</tr>
<tr>
<td>India</td>
<td>&gt;10</td>
</tr>
</tbody>
</table>

Total CO₂ emissions worldwide: 21 billion tons [Malhotra 2004].

Figure 1. Limestone for cement and concrete [modified from Worrell and Galtisky 2004].
Association of Canada 2004]. It can be expected that concrete will be needed to both increase industrialization and urbanization while protecting the environment. To do this, the concrete industry should consider recycling industrial by-products such as fly ash safely and economically. When industrial by-products replace cement, even up to 70%, in concrete, the environmental impact improves along with the energy efficiency, and durability of concrete [Naik et al. 2003].

Concrete is a building material that is not only strong and durable, but can also be produced in ways that are environmentally friendly and architecturally moldable in esthetically pleasing forms [National Building Museum 2004]. With sustainable concrete structures and infrastructure, the concrete industry can develop a sustainable future for generations to come. Furthermore, buildings that are constructed to be both durable and environmentally friendly often lead to higher productivity because the buildings generally lead to better air quality and therefore, higher productivity [Coppola et al. 2004]. One example of the advantage of sustainable concrete is buildings constructed with concrete that have reduced maintenance and energy costs. Another is concrete highways, which reduce the fuel needed for heavily loaded trucks. A third example of the benefits of sustainable concrete construction is illustrated in insulating concrete homes that have energy reductions of up to 40% [Cement Association of Canada 2004].

The cement and concrete industries can make substantial contributions to sustainable developments by creating and adopting technologies that can reduce the emissions of greenhouse gases. The cement and concrete industries could contribute to meeting the goals and objectives of the 1997 Kyoto Protocol [UNFCCC COP9 Report 2004]. Among other things, Kyoto Protocol requires to meet a target of reduction in GHGs to 1990 level. Those involved with the manufacture of portland cement would have a huge impact on the sustainable development of the concrete industry as a whole because currently cement production contributes slightly over 7% of worldwide GHGs (primarily CO2); or, about 1.6 billion tons of GHGs. Any future taxes on GHG emission would have noticeable and noteworthy economic impact on the price of cement.

There are a number of characteristics that apply to innovative concrete products. First, they are produced with precast or cast-in-place reinforced concrete elements that are made with portland cement and pozzolanic materials that include renewable and/or recycled components. Second, innovative concrete products are constructed to enhance the performance of concrete elements, which may also contain recycled concrete as aggregates. High-performance materials are intended to reduce cross-sections and the volume of concrete produced. They are also intended to increase the durability of concrete structures to minimize the maintenance needs of the concrete construction, and limit the amount of non-renewable special repair materials that need to be used in the maintenance of the concrete [Coppola et al. 2004].

Concrete producers are creating sustainable solutions for many market sectors including agriculture and construction. In agriculture, integrated waste management solutions have been developed that convert manure into biogas, nutrient rich fertilizer, and reusable water. In construction, industrial, commercial, and institutional buildings are being constructed so that they are more energy efficient, have better air quality, and necessitate less maintenance [McDonough et al. 1992].

U.S. foundries generate over 7 million tons (8 million tons) of by-products. Wisconsin alone produces nearly 1.1 million tons (1.25 million tons) of foundry by-products, including foundry sand and slag. Most of these by-products are landfilled. Landfilling is not a desirable option because it not only causes a huge financial burden to foundries, but also liability for future environmental costs, and restrictions associated with landfilling. One of the innovative solutions appears to be high-volume uses of foundry by-products in concrete and other construction materials [Naik & Kraus 1999].

In addition, in 1996 the USA produced 136 million tons of construction and demolition (C & D) debris, about 1.5 kg per person per day. About 25 to 40% of landfill space is C & D debris [McKay 2004]. If this trend continues, the cost of landfilling will continuously increase, as will the potential health and environmental risks of landfill materials. Furthermore, the cost of landfilling is escalating due to shrinking landfill space and stricter environmental regulations. Such C & D debris (e. g., concrete as well as gypsum wallboards from C & D debris) can be recyclables to make new concrete [UWM-CBU 2007].

A study was reported in 1999 whose aim was to evaluate the environmental impact of controlled low-strength materials (CLSM) incorporating cement and industrial by-products such as coal fly ash and used foundry sand [Naik and Kraus 1999]. The results demonstrated that excavatable flowable slurry incorporating fly ash and foundry sand as a replacement for up to 85% of fly ash could be produced. In general, inclusion of both clean and used foundry sand caused a reduction in the concentration of certain contaminants. The use of foundry sand in flowable CLSM slurry, therefore, provided a favorable environmental performance.

4 PORTLAND CEMENT

Portland cement is not an environmentally friendly material because its manufacture creates greenhouse gas emissions; and, it also reduces the supply of good-quality limestone and clay. As good engineers, we
must reduce the use of portland cement in concrete. We must use more blended cements in order to reduce the need of portland cement clinker per ton of blended cement produced by blending with the clinker other pozzolanic materials, such as coal or wood fly ash, slag, silica fume, and/or other pozzolanic materials. As a cement production feed material, instead of clay, industrial by-products such as used foundry sand and/or coal combustion products (CCPs) such as fly ash should be used in optimum possible quantity.

The most energy intensive stage of the portland cement production is during clinker production. It accounts for all but about 10% of the energy use and nearly all of the GHGs produced by cement production. Kiln systems evaporate inherent water from the raw meal and calcine the carbonate constituents during clinker preprocessing [Worrell & Galtisky 2004].

Sources of CO₂ and GHG emissions in the manufacturing of portland cement [Malhotra 2004] are:
- from calcinations of limestone = ±50–55%;
- from fuel combustion = ±40–50%; and,
- from use of electric power = ±0–10%.

5 INNOVATIVE CEMENT PRODUCTS

While the embodied energy linked to concrete production is low, pozzolanic materials (PM), especially coal fly ash, have been used by the concrete industry for over 70 years. Their use can contribute to a further reduction of concrete’s embodied energy. When used wisely and judiciously, PM can improve the long-term properties of concrete. Fly ash can, and does, regularly replace portland cement in concrete [Malhotra 2004, Naik et al. 2003, Mehta 2002].

One process that is even more environmentally friendly and productive is the use of blended cements. Blended cements have been used for many decades. Blended cements and are made when various amounts of clinker are blended and/or interground with one or more additives including fly ash, natural pozzolans, slag, silica fume, and other PM. Blended cements allow for a reduction in the energy used and also reduce GHG emissions [Malhotra 2004, Mehta 2002].

Most innovative concrete mixtures make use of PMs to partially replace cement. The advantages of blended cements include increased production capacity, reduced GHG emissions, reduced fuel consumption in the final cement production, and recycling of PMs [Worrell 2004, Cement Association of Canada 2004].

The manufacture of portland cement is the third most energy intensive process, after aluminum and steel. In fact, for each ton of portland cement, about six million BTU of energy is needed [Naik & Kraus 1999].

Although cement production is energy inefficient, there have been major initiatives that have reduced energy consumption [Worrell 2004]. Of these, the most significant has been the replacement of wet production facilities with dry processing plants. In addition, the cement industry has also moved away from petroleum-based fuel use.

Despite these advances, there are still some shortcomings when energy use is evaluated for the concrete industry. Dry process cement plants use pre-heaters, which increase the alkali content of cement [Worrell 2004]. Thus, cement producers need to continue to develop ways to control the alkali content without increasing the energy consumption levels of the cement [Coppola et al. 2004]. Furthermore, current innovations and energy savings are linked to the amount of energy consumption by converting wet-process to dry-process cement production and the number of pre-heaters needed to complete the process [Worrell 2004].

For each million tons of capacity, a new portland cement plant costs over 200 million dollars. The cost associated with the production of portland cement, along with the CO₂ emissions and energy issues, makes it unlikely that developing countries will be able to employ modern technology to reduce GHGs. Also, government regulations of GHGs will likely force the cement industry to create blended cements and use supplementary materials for blended cements in order to meet the societal development needs [Malhotra 2004, Worrell 2004].

To produce one ton of portland cement, 1.6 tons of raw materials are needed. These materials include good quality limestone and clay. Therefore, to manufacture the current production of 1.6 billion tons of cement annually, at least 2.5 billion tons of raw materials are needed [Wu 2000].

As good engineers, we must employ environmentally friendly materials to reduce the use of portland cement by replacing a major part of portland cement by PMs for use in concrete. In the USA, such materials – primarily fly ash, slag, silica fume, natural pozzolans, rice-husk ash, wood ash, and agricultural products ash – are available, for up to 70% replacement [Naik et al. 2003]. All these materials can be used to supplement the use of cement in concrete mixtures while improving product durability.

One of the important benefits of the increased use of other types of cementitious materials (such as PMs) is the reduction of GHG emissions. With a replacement of cement with other recyclable resources, worldwide CO₂ emissions would be reduced. A replacement of 50% of cement worldwide by other cementitious materials would reduce CO₂ emissions by 800 million tons. This is equivalent to removing approximately 1/4 of all automobiles in the world [Malhotra 2004].
Fly ash availability in the USA in 2005 was estimated at 120 million tons by the American Coal Ash Association [ACAA 2006]; by 2010 it is estimated to be 160 million tons. Portland cement availability in 2002 was estimated at 80 million tons; by 2010 it is estimated to be 100 million tons. The fly ash disposal challenge and the limited availability of portland cement have the same solution: replace large amounts of portland cement with fly ash to create durable and sustainable concrete.

6 THE HANNOVER PRINCIPLES – DESIGN FOR SUSTAINABILITY

In 1991, as the planning of the World’s Fair was underway, the City of Hannover, Germany asked William McDonough and Michael Braungart to create sustainability principles to guide the large-scale development of EXPO 2000 in Hannover. “The Hannover Principles – Design for Sustainability” also include directives concerning the use of water. Although these guidelines were created for the World Fair, they are still a good tool to guide current and future development around the world [McDonough 1992].

Designers, planners, government officials, and all those who participate in the construction of new buildings and infrastructures should use the Hannover Principles. A new design philosophy has developed from these principles and should be included in sustainable systems and construction in the future. There are a number of examples of societies that have created sustainable and environmentally friendly communities. There is hope that the Hannover Principles will inspire development and improvements that are committed to sustainable growth with practical limits to create a sustainable and supportive future for communities and the world.

The Hannover Principles by William McDonough [McDonough 1992]:

1. Insist on rights of humanity and nature to co-exist; recognize interdependence; respect relationships between spirit and matter; accept responsibility for consequences of design; create safe objects of long-term value; eliminate the concept of waste; rely on natural energy flows; understand the limitations of design; and, seek constant improvement by the sharing of knowledge.

2. The Hannover Principles are not “cast-in-concrete”. They were devised to provide a tangible document that could evolve and be adapted as our understanding of our interdependence with nature becomes more important over time.

3. For sustainability, consider your actions on the following [McDonough 1992]: materials–use indigenous materials; land use–protect and create rich soil; urban context–preserve open spaces; water–use rainwater and gray-water (shower, sink, bath and laundry excess); wastes–recycle, air–create clean air; energy–use solar and wind energy, recycle waste energy; responsibility to nature–create silence; and, maintenance–minimize or eliminate for future generations.

Materials are critical to creating sustainable and responsible concrete designs. In order to ensure that the most effective and environmentally friendly materials are being used, the entire life-cycle of the structure should be taken into consideration. Material choice should include anticipation of the extraction, processing, transport, construction, operation, disposal, re-use, recycling, off-gassing and volatile organic compounds (VOC) associated with the material [McDonough 1992]. According to McDonough, constructions should be flexible to serve a variety of different needs (e.g., today’s storage building can be tomorrow’s school). Adapt materials that are sustainable in their process of extraction, manufacture, transformation, and degradation, as well as recyclability. Consider toxicity, off-gassing, finish, and maintenance. Recycling is essential. Make allowance for disassembly and reuse. Plan for reuse of the entire structure in the future. Minimize use of hazardous chemicals. Eliminate waste that cannot be part of a naturally sustainable cycle. Any solid wastes remaining must be dealt with in a non-toxic manner. Life-cycle costs must be studied, analyzed, and incorporated in planning and construction of a building or infrastructure. Life-cycle cost analysis process must evaluate energy use and environmental impact during the life of the product, process, and/or activity. This process must include extraction and processing of raw materials, manufacturing, transportation, maintenance, recycling, and returning to the environment. Costs and benefits must be evaluated and understood in both the short-term and long-term. Demolished concrete must be recycled. It can be readily used in new concrete for aggregates.

For the sustainability of the cement and concrete industries, use less water and portland cement in concrete production; and use more blended cements and tailor-made organic chemical admixtures. The devastation of air is a global problem, regardless of the locality in which the pollution is created [McDonough 1992]. The overall design of concrete structures must not contribute to atmospheric degradation. Those involved in the cement and concrete industries must evaluate ozone depletion and global warming throughout the construction and planning process. A major contribution to this effort is the use of more blended portland cement to minimize global climate change.

Water resources are being depleted by various uses [Bourg 2004]. Therefore, potable water should be conserved to serve life-sustaining needs rather than infrastructural needs. Rainwater and surface run-off...
water can be used as a water conservation method by recycling these water resources in construction instead of using potable water. Gray water should be recycled and used for grass, shrubs, plants, trees, and gardens; as well as for concrete production [McDonough 1992]. Furthermore, mixtures with less water should be developed with new technologies to create mortar and concrete containing a minimal amount of water. Benjamin Franklin said over 200 years ago in Poor Richard’s Almanac, “When the well’s dry, we know the worth of water.” Many facilities may have requirements that can be completed with non-potable water. By using non-potable water, a significant amount of money can be saved by avoiding or reducing potable water purchases and sewerage costs. To be as effective as possible, non-potable water for construction and building uses should be identified early in the planning and designing process to be most cost-effective. Four ways to utilize and recycle water are to reuse water on site for repeated cycles of the same task, treat and reuse water on site for multiple purposes, use gray water after solids have been eliminated, and collect non-potable water from sources such as rainwater, lakes, rivers and ponds for use in construction [Bourg 2004].

Energy efficiency, providing the same (or more) services for less energy, helps to protect the environment. When less energy is used, less energy is generated by power plants, thus reducing energy consumption and production. This in turn reduces GHGs and improves the quality of the air. Energy efficiency also helps the economy by saving costs for consumers and businesses. According to McDonough [1992]: (a) use buildings’ thermal inertia (e.g., concrete building’s mass allows it to retain heat); (b) use day lighting and natural ventilation; (c) use wind power and solar power; (d) recycle waste energy; (e) judiciously use color materials on surfaces; (f) reduce heat-islands in buildings; and, (g) manage and moderate micro-climates of buildings.

7 WASTE MATERIALS

Engineers, architects, planners, and builders should reuse industrial by-products and post-consumer wastes in concrete and other cement-based construction materials. Post-consumer wastes that should be considered for use in concrete include glass, plastics, tires, and demolished concrete, and clay bricks. To do this successfully, designers must watch for harmful hydration reactions of portland cement and changes in volume of concrete. The recycling of industrial by-products has been well established in the cement and concrete industries over the past couple of decades [UWM-CBU 2007]. The use of coal fly ash in concrete began in the 1930s, but volcanic ash has been used in mortar and primitive concrete for several millennia in Egypt, Italy, Mexico, and India. The use of by-products such as rice-husk ash, wood ash, silica fume and other pozzolanic materials, in addition to coal fly ash, can help to reduce the need for portland cement in addition to creating more durable concrete and reducing greenhouse gas emissions [Malhotra 2004, Mehta 2002, ACI 2004]. This will also contribute to the improvement of air quality, reduction of solid wastes, and sustainability of the cement and concrete industry [Mehta 2002].

In summary, for sustainability of the cement and concrete industries: use less portland cement; use less water; use application-specific high-quality, durable aggregates; and, use organic chemical admixtures. Fundamental laws of nature state that we cannot create or destroy matter; we can only affect how it is organized, transformed, and used. To manage natural resources, humanity must obey the rules of nature: use only what you need and never use a resource faster than nature can replenish it. Resources are extracted from the earth by 20% more than the earth produces. Therefore, what is consumed in 12 months will take 14.4 months to be replenished. The use of sustainable development procedures will reduce that rate [Time Magazine 2002]. “The issue is not environment vs. development or ecology vs. economy; the two can be (and must be) integrated” [Ricoh Company 2004].

8 CONCLUSIONS

As Kofi Annan, U.N. Secretary General, said in 2002, “We have the human and material resources needed to achieve sustainable developments, not as an abstract concept but as concrete reality” [Time Magazine 2002]. Professionals involved in the cement and concrete industries have the responsibility to generate lasting innovations to protect both the industries’ future viability and the health of our environment. Large volumes of by-product materials are generally disposed in landfills. Due to stricter environmental regulations, the disposal costs for by-products are rapidly escalating. Recycling and creating sustainable construction designs not only contributes to reduced disposal costs, but also aids in the conservation of natural resources. This conservation provides technical and economical benefits. It is necessary for those involved in the cement and concrete industries to eliminate waste and take responsibility for the life cycle of their creations. In order to be responsible engineers, it is necessary to think about the ecology, equity, and economy of our design [McDonough 1992]. Engineers must apply forethought into direct and meaningful action throughout our development practices. Sustainable designs must be used as an alternative
and better approach to traditional designs. The impacts of every design choice on the natural and cultural resources of the local, regional, and global environments must be recognized in the new design approaches developed and utilized by the cement and concrete industries.

REFERENCES


Swedish National Testing and Research Institute 2003. “Concrete for the Environment.” Published on Behalf of the Nordic Network Concrete for Environment, Boras, Sweden, 8 pages.


Closed Cycle Construction – A process for the separation and reuse of the total C&D waste stream

E. Mulder & L. Feenstra
TNOScience and Industry, Department of Separation Technology, Apeldoorn, The Netherlands

T.P.R. de Jong
Delft University of Technology, Faculty of Civil Engineering and Geosciences, Delft, The Netherlands

ABSTRACT: In The Netherlands, construction and demolition (C&D) waste is already to a large extent being reused. Especially the stony fraction is crushed and reused as a road base material. In order to increase the percentage of reuse of the total C&D waste flow to even higher levels, a new concept has been developed. In this concept, called ‘Closed Cycle Construction’, the processed materials are being reused on a higher quality level and the quantity of waste that has to be disposed of is minimized. For concrete and masonry, the new concept implies that the material cycle will be completely closed, and the original constituents (clay bricks, gravel, sand, cement stone) are recovered in thermal processes. The mixed C&D waste streams are separated and decontaminated. For this purpose several dry separation techniques are being developed. The quality of the stony fraction is improved so much, that this fraction can be reused as an aggregate in concrete. The new concept has several benefits from a sustainability point of view, namely less energy consumption, less carbon dioxide emission, less waste production and less land use (for excavation and disposal sites). One of the most remarkable benefits of the new concept is that the thermal process steps are fuelled with the combustible fraction of the C&D waste itself. Economically the new process is more or less comparable with the current way of processing C&D waste. On the basis of the positive results of a feasibility study, currently a pilot and demonstration project is being carried out. The aim is to optimise the different process steps of the Closed Cycle Construction process on a laboratory scale, and then to verify them on a large scale. The results of the project are promising, so far.

1 THE ‘CLOSED CYCLE CONSTRUCTION’ CONCEPT

In The Netherlands, construction and demolition (C&D) waste is already to a large extent being reused. The stony fraction is crushed and for more than 95% reused as a road base material. In order to increase the percentage of reuse of the total C&D waste flow to even higher levels, a new concept has been developed by TNO. In this concept, called ‘Closed Cycle Construction’, the processed materials are being reused on a higher quality level and the quantity of waste that has to be disposed of is minimized.

On the basis of the positive results of a feasibility study, currently a pilot and demonstration project is being carried out. The aim of this project is to optimise the different process steps of the Closed Cycle Construction process on a lab scale, and to verify them on a large scale. The project is managed by TNO, and carried out by a consortium of ten Dutch companies and four institutes (including TNO and the Delft University of Technology), and supported by the Dutch Government.

The new concept, called ‘Closed Cycle Construction’ is principally based on closing the material cycles for especially masonry and concrete in the construction sector. Besides, also the remaining mineral fractions of C&D waste are being re-utilised. In short this means that, after deconstruction/demolition of a building, concrete rubble, masonry debris and mixed stony rubble are separated and treated individually, gaining minerals that can be re-used as raw materials for the manufacture of new construction products (clay bricks and concrete). One of the most remarkable benefits of the new concept is that the thermal process steps are fuelled with the combustible fraction of the C&D waste itself.

In fact, in the Closed Cycle Construction concept four different C&D waste streams are distinctly processed, namely clean concrete rubble, clean
masonry debris, mixed rubble and mixed C&D waste (also including wood, plastic, plasterboard, glass, etc.). This is shown in figure 1. For concrete, this new technology involves a rotary kiln in which the uncontaminated concrete rubble is thermally treated at a temperature of about 700°C to dehydrate the cement stone. The concrete rubble pieces disintegrate and the original components are set free. After treatment, only 2% of hardened cement paste remains attached to the sand and gravel grains.

For masonry debris, the new technology consists of a three-step process. In the first step, the large pieces of debris are thermally treated at a temperature of about 550°C, to set free the majority of the original ceramic bricks. These whole bricks can be used for restoration purposes or for the construction of buildings in an old fashioned appearance. Subsequently, the remaining pieces of brick and mortar are physically separated. In the third step, the remaining ceramic fraction is crushed and reused as raw material for the production of new ceramic bricks.

To be able to process the entire supply of C&D waste, the above mentioned processes have to be implemented in an overall process. This overall process also includes process steps for the treatment of mixed C&D waste. The mixed C&D waste streams are separated and decontaminated. For this purpose several dry density separation techniques are being developed. The quality of the stony fraction is improved to a level that it can be re-used as an aggregate in concrete.

For demolition waste that has not been separated at the source, advanced detection and separation techniques are being developed, to sort out contaminants like gypsum and hazardous materials. The remaining material is divided into a heavy (stony) fraction and a light (combustible) fraction. The combustible fraction (wood, plastic, paper, bituminous roofing material) is cleaned up and prepared as a fuel for the thermal process steps.

2 THERMAL TREATMENT OF CONCRETE RUBBLE

To be able to close the material cycle for concrete completely, high quality raw materials must be produced from concrete rubble (Dijk 2002). These raw materials need to fulfil the criteria that are set for primary raw materials. For this reason, a thermal process has been developed for the treatment of concrete rubble.

The process step suitable for setting free the different components of concrete is based on the input of a combination of thermal and mechanical energy. As is
schematically shown in figure 2, the installation exists of the following unit operations:

- a jaw crusher to reduce the size of the coarse concrete rubble to below 10 cm;
- a magnet, to remove any reinforcement steel present in the concrete rubble;
- a rotary kiln for the combined input of thermal energy (to disintegrate the matrix) and mechanical energy (to further release the different concrete components);
- a vibrating screen and air separator to separate the following three fractions: coarse aggregate (primarily gravel), fine aggregate (sand) and cement stone.

The proposed process is based on the experiences of Mitsubishi, TNO and KEMA (Larbi et al. 2000, Shima et al. 1999). The heating step must be performed on a temperature of at least 700°C to obtain clean aggregate fractions, sand and gravel (Larbi et al. 2000). Preliminary investigations on a laboratory scale in which the concrete rubble was treated at 700°C, showed that from 1 ton of concrete 450 kg gravel (≥4 mm), 350 kg sand (<4 mm), 130 kg cement stone (<150 μm) and 10 kg reinforcement steel can be recovered (Larbi et al. 2000). The remaining 60 kg was originally present as hydration water, and was emitted as vapour.

If the concrete is treated at a relatively high temperature (>750°C) there is hardly any residual cement paste left attached to the aggregate grains. In that case, the gravel and sand are supposedly of the same quality as primary gravel and sand. The performance of the recovered aggregate however has to be verified experimentally (Mulder et al. 2002).

The dehydrated cement stone fraction can be used as a substitute for part of the Portland cement clinker in the cement production process. The cement stone can be directly fed to the Portland cement mill, or even added after the mill. This saves a lot of energy and raw materials.

3 THERMAL TREATMENT OF MASONRY DEBRIS

In co-operation with the Delft University of Technology, at TNO a PhD study was carried out into recycling options for masonry debris (Dijk 2002). The aim of the process development was closing the material cycle for ceramic clay bricks within the production chain. This resulted in a three-step process that is described in more detail in the next paragraphs.

The first step in the recycling of masonry debris is a thermal process, to recover whole bricks. The process is based on the difference in linear expansion coefficient between mortar and brick. This means that with increasing temperature, strains are built up in brick and mortar, causing shear stresses on the interface. Because of the fact that this interface, in general, is the weakest part of the masonry, this phenomenon leads to crack formation on the interface, setting free the whole bricks.
In the laboratory, optimum process conditions were determined. Especially the temperature required to separate all the mortar from the brick’s surface was investigated for different brick types and different mortar systems. It appeared that for cement dominated mortars, the best temperature was around 540°C. However, mortar types also containing lime, require even higher temperatures. In the case of feeding big lumps of masonry debris, this leads in that case to a relatively high percentage of cracked bricks. This is caused by the fact that the critical quartz solid phase transition temperature of 573°C must be passed twice, leading to internal stress. This high cracking percentage can be substantially decreased by a pre treatment, in which the individual bricks are mechanically separated. This is quite easy, because these mortar types are rather weak.

To verify the laboratory experiments, two experiments were carried out on semi-practical scale (one with a cement dominated mortar and one with a mortar also containing lime).

In both cases the bricks had a grey appearance at the bottom and top side. The facing sides however, were clean. The bricks were of good technical and environmental quality, in accordance with the Dutch standard NEN 2489 (dimensions and strength), and with the Dutch Building Materials Decree (leaching of heavy metals and salts).

4 SEPARATION OF MIXED C&D WASTE STREAMS

Mixed C&D waste streams will have to be separated and extensively cleaned up, in order to gain enough quality to reuse the different fractions. In the Closed Cycle Construction project the aim is to end up with a decontaminated mineral aggregate fraction that can
be reused in concrete and a decontaminated combustible fraction that can be used as a fuel in the thermal processes, described before. The challenge in this respect is to find the right combination of a cheap bulk separation technique (as a rough pre-concentration) and one or more automated sorting techniques for the further clean-up of the pre-concentrated fractions. This approach already proved successful in the processing of recycle glass and metallic scrap. An example is worked out in figure 6. In this example the dry density separator produces as heavies an aggregate of 98% purity and combustible lights of 80% purity. Assuming a typical concentration factor of 10 for a single stage industrial automatic sorting step, the final aggregate quality will be of 99.8% purity and the lights will have 98% purity. Light materials that are unsuitable for combustion in conventional processes, such as PVC, can be concentrated in a separate waste stream.

4.1 Dry separation techniques

With regard to the pre-contamination of the mixed C&D waste stream, several dry density separation techniques were investigated experimentally on a pilot scale (Jong & Fabrizi 2004). All investigated separators showed variable but satisfactory efficiency. The best choice depends on a trade-off between economy and required product quality, and on factors such as space requirements, capacity requirements, whether or not classification in narrow size fractions is possible, etc. The tests resulted in sufficient quantitative data to support a tailor made decision for a given operation.

The dry sand fluidised bed shows the highest product purities for 20 mm oversize and obtains product qualities that are similar to wet jigging. The principle ability to separate coarse material (larger than 50 mm) is unique for this dry technology and is of high relevance for C&D waste sorting. Unfortunately separation of larger sizes than 40 mm could not be investigated in the present project due to practical limitations of the applied equipment. Disadvantage is sand loss in the product and the lower efficiency for finer material (5 to 20 mm). However, in aggregates a certain sand level is tolerated and in combustible fractions it causes only a minor increase in ash content.

Although of lower separation efficiency, the air jig provides a favourable solution when prior classification into several size fractions is impractical and a significant fines percentage (2 to 20 mm) is present.
for which the fluidised bed is unsuitable. Both separators would also work well together, the jig treating the material up to 30 mm, the fluidised bed the 30 mm oversize. Prior removal of lights and fines with air sifting is recommended.

Ballistic separators and air tables require prior classification into rather narrow particle size fractions. A high screening efficiency is of paramount importance. For damp feed appropriate screening technology must be applied. Air tables show additional displacement of tile and glass fragments in the lights. Ballistic separation is very suitable as high volume pre-concentration wherever narrow classification can be done and there is sufficient head-room available.

4.2 Automatic sorting techniques

With regard to the automatic sorting techniques, required for the further clean-up of the pre-concentrated fractions, two techniques have been extensively investigated (de Jong et al. 2005). Both techniques have their own advantages.

Automatic colour sorting of the light and heavy fractions was investigated with the Scan & Sort CombiSense 1200 at the Department of Mineral Processing at RWTH Aachen, Germany. Its optical system incorporates a high-speed camera with 1 billion colours recognition and a special conductivity sensor permitting the identification of a variety of materials (see figure 7). It can handle mass streams of up to 40 tonnes per hour. Computer controlled nozzles blow out the detected materials. The camera is analyzing size, shape, colour and position of particles on the belt. The information is then used to generate impulses instructing the nozzles to blow out single particles or allow them to pass.

The results gained with the CombiSense exceeded expectations. It could well be used to increase product quality and avoid the presence of unwanted materials in the stream to be recycled.

Total wood recovery and grade from the light fraction are high, reaching values of 83% and 92% respectively. 3% of stones and tiles are separated with wood; by more effective density separation a large proportion of this heavy material could be removed and grade would be about 95%. It can be concluded that colour sorting can be effectively used for wood removal from mixed CDW.

Colour sorting also allows a good removal of gypsum form the heavy fraction showing a recovery of 94%. This is a good result because in recycle aggregate the presence of that mineral should be avoided due to its sulphate content. The grade is only 5.6% because of the presence of stones and tiles of similar colour in the concentrate. These could be easily removed by density separation in a second stage.

Glass recovery and grade are 96% and 56%. Thus the CombiSense separator would be also useful for...
glass removal from mixed building waste, where glass comprises the disturbing component.

A second automatic sorting technique that was investigated in the project, is a sorting based on Dual-energy X-ray transmission. On a lab-scale preliminary experiments were carried out, to ‘image’ the different components in mixed C&D waste. In figure 8 is shown that an effective split can be made between organic and inorganic materials, as well as an effective metal recognition in the same pass. Utilising dedicated imaging software specific components such as gypsum or asbestos can be removed as well.

In addition to the lab-scale experiments mentioned above, experiments were carried out with DE-X-ray sorting, on a larger scale (1 to 2 m³). Also here the main conclusion is that this type of automatic sorting is especially useful for the separation of organic and inorganic materials. Especially the sorting of wood at high purity and recovery is possible. Metal separation, also of parts that contain only smaller metal inserts, is well possible in addition.

As preparation for the tests the material (mixed C&D waste) was screened in a 20 × 40 mm and 40 × 80 mm fraction. The smaller and larger material was not sorted. The <20 mm material was only a small fraction; the fractions >80 mm cannot be sorted on the available DE-XRT sorter due to limitations in geometry and air jets. This is not a principle limitation, but a limitation of the experimental device.

The fractions of C&D waste were analysed on composition in the categories wood, stone, glass, gypsum, metal, plaster, textile, paper & cardboard and others (‘rest’). The DE-XRT sorting machine that was available at CommoDas GmbH in Wedel, Germany, was adjusted and trained to concentrate wood in the usual way.

The composition of the 20 × 40 mm input material and the component split after sorting are given in tables 1 and 2 respectively. The component split indicates the recovery; e.g. 43% of the input feed is recovered as wood in the wood concentrate, that is 47% of the feed input. The remaining 4% of wood is lost in the reject. The results of the 40 × 80 mm fraction are more or less comparable.

The test results indicate that from mixed C&D waste in the 20 to 80 mm size range a wood fraction can be sorted out with an excellent quality for re-use as a secondary fuel, and without any pre-treatment other than screening and sifting.

5 BENEFITS OF ‘CLOSED CYCLE CONSTRUCTION’

The new concept as a whole has several advantages. The most illustrative advantages are:

- Closing the material cycles for concrete and masonry within their own chain. With regard to the framework of sustainable development, this fulfils one of the objectives of the Dutch government. It also fits into the industries policy of producer’s responsibility and long-term raw materials availability.
- Recovery of high-grade raw materials for the production of new concrete and ceramic bricks means higher profits for the C&D waste treatment industry, because of higher prices for their products. The production of high-grade raw materials also reduces the excavation of primary materials, such as sand, gravel, clay and marl.
- Utilising the combustible fraction of demolition waste as a fuel doubles the environmental profit. First, in reducing the amount of waste to be disposed of, and second in reducing the amount of fuel that is required for the thermal treatment of concrete and masonry rubble.
- Re-use of the recovered cement stone fraction in the production of new cement (as replacement of part of the Portland cement clinker) leads to
less excavation of marl, and less carbon dioxide emissions.

- The integrated character of the process finally implies a reduction in transport costs. This means less fuel consumption and less exhaust gases.

REFERENCES


Effect of GRP industrial waste on durability of cement mortars

F. Tittarelli & G. Moriconi
Department of Materials and Environment Engineering and Physics, Università Politecnica delle Marche, Via Brecce Bianche, Ancona, Italy

ABSTRACT: Certain durability aspects of sustainable mortars manufactured by the addition of Glass Reinforced Plastic (GRP) industrial waste have been investigated. Mechanical and porosimetric characterization, water absorption and shrinkage measurements were carried out on cement mortars manufactured by replacing 0%–10%–15%–20% of the aggregate volume with GRP waste. An increase in porosity (25%) and, consequently, a decrease in mechanical strength (up to 40% depending on the addition rate), as well as higher deformability, were detected on the mortars manufactured by GRP waste addition. Moreover, unexpectedly, capillary water absorption and drying shrinkage of GRP mortars resulted in significantly lower values (70% and 50%, respectively) than those of the reference mortar manufactured without GRP addition. This decrease could involve enhanced durability, even if only in a supporting role.

1 INTRODUCTION

A judicious use of resources, achieved by the use of by-products and waste materials, and a lower environmental impact, achieved through reduced carbon dioxide emission and reduced natural aggregate extraction from quarries, represent two main actions that meet sustainable building development [Malhotra 2003].

Concrete can be a viable solution to environmental problems making the re-use of solid waste products from other industries possible in concrete production. This will reduce the need to landfill these materials while still maintaining an acceptable, and sometimes even better, concrete quality [Damtoft et al. 2003, Hendriks & Janssen 2003].

Glass Reinforced Plastic (GRP) is a composite material made of glass fibres dispersed in a resin, usually polyester, widely used in several fields from building to furniture to boats. Every year, in Western Europe, GRP processing produces 40000 tons of industrial waste [Kelderman 2000]. In Italy this waste is landfilled, due to the difficulty of separating the glassy part from the polymeric matrix. Concrete made with recycled glass [Dyer & Dhir 2001, Shao et al. 2000, Corinaldesi et al. 2003, Ambrosie & Pera 2003, Quian et al. 2003] or polymeric addition [Zhao 1995] has already been proposed in the literature. In particular, the polyester concrete is particularly resistant to chemical agents as well as to thermal cycles and can be useful for light weight constructions [Zhao 1995]. Therefore, the feasibility of re-using GRP industrial waste in manufactured concrete elements could be considered.

With this aim, the feasibility of re-using GRP industrial waste coming from a shipyard to manufacture concrete was evaluated. In particular, three types of GRP wastes are produced in general: GRP bars, 1 meter long and 1–2 cm thick, coming from faulty products; small pieces, 1–2 cm in size, and a fine powder, about 0.1 mm in size.

In previous works [Tittarelli et al. 2004, Tittarelli & Moriconi 2005] the finest GRP waste was physically and chemically characterized in order to outline compatibility issues with cement, if any.

In this work cement mortars manufactured by partially replacing the aggregate volume with GRP waste powder were investigated in terms of mechanical and porosimetric properties, water absorption and shrinkage measurements.

Future research should deal with possible leaching of toxic substances in deionized water.

2 EXPERIMENTAL

2.1 Materials

A commercial portland-limestone blended cement type CEM II/A-L 42.5 R according to the European Standards EN-197/1 was used. The Blaine fineness of cement was 0.415 m²/g and its density was 3.05 kg/m³.
The chemical composition of cement is reported in Table 1.

Natural sand with 5 mm maximum size was used as aggregate, with volume mass of 2620 kg/m³ and water absorption of 3.0%.

A GRP powder coming directly from a shipyard as an industrial waste was used (Fig. 1). Its chemical and physical characterization has already been reported in a previous work [Tittarelli et al. 2004, Tittarelli & Moriconi 2005].

A 30% aqueous solution of an acrylic-based superplasticizer was added, when required, in order to maintain the same workability level.

2.2 Specimens

Mortar mixtures with cement to sand ratio 1:3 (by weight) and W/C of 0.50 were manufactured by replacing the 0–10–15–20% of aggregate volume with the GRP industrial waste powder.

The use of GRP waste powder as partial aggregate replacement did not show any segregation problem and the resultant mixtures show similar characteristics to traditional ones. At 15% and 20% replacement rate, GRP appreciably decreases mortar workability and, at these rates, a superplasticizer was added at a dosage of 0.25% and 1% by weight of cement, respectively, in order to achieve comparable workability.

For each mixture, carried out according to UNI EN 1015-2, prismatic specimens (40 × 40 × 160 mm) were manufactured, wet cured for two days and air dried for 1 month at room temperature.

2.3 Tests

2.3.1 Mechanical characterization

Compressive and flexural strengths were determined on mortar specimens in conformance with UNI EN 1015-11 by observing strength loss for an increasing dosage of the GRP powder at different curing times.

2.3.2 Pore structure characterization

Three specimens for each mortar type were tested after 28 days of wet curing by means of the mercury intrusion technique, by avoiding the inclusion of coarser aggregate in the analyzed sample.

2.3.3 Capillary water absorption

Capillary water absorption tests were carried out on mortar specimens according to Italian Standard UNI 10859.

2.3.4 Water absorption at atmospheric pressure

Water absorption tests at atmospheric pressure were carried out on mortar specimens according to the procedure reported in UNI 7699.

2.3.5 Drying shrinkage test

Prismatic specimens (100 × 100 × 500 mm) were prepared according to Italian Standard UNI 6555-73 “Hydraulic Shrinkage Determination”. After one day of wet curing, the specimens were stored at constant temperature (20 ± 2°C) and constant relative humidity (70 ± 2%).

3 RESULTS

3.1 Mechanical characterization

The development with time of flexural strength of mortars manufactured with increasing GRP waste addition as a partial aggregate volume replacement is given in Figure 2. The results show that GRP addition produces a limited decrease in the flexural strengths of mortars, especially at longer ages, probably due to the presence of glass fibres.

On the other hand, aggregate replacement with GRP waste caused a strong reduction in compressive strength (up to 40% depending on the addition rate), due to the lower stiffness of the GRP powder with respect to natural sand and the low glass content, which was therefore unable to develop any pozzolanic behaviour (Fig. 3).

The stress/strain curves carried out on the specimens after one month of air curing (Fig. 4) showed...
that the partial aggregate replacement with GRP waste powder increased deformability of the mortars.

3.2 Pore structure characterization
The porosimetric results obtained by the mercury intrusion technique are reported in Table 2 and Figure 5.

Table 2. Porosimetric values of mortars manufactured with sand partially replaced by GRP at dosages of 0, 10, 20, 40% (by volume), respectively.

<table>
<thead>
<tr>
<th>GRP (%)</th>
<th>Porosity (%)</th>
<th>Mean radius (Å)</th>
<th>Specific surface (m²/g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>16</td>
<td>629</td>
<td>3.8</td>
</tr>
<tr>
<td>10</td>
<td>21</td>
<td>250</td>
<td>7.9</td>
</tr>
<tr>
<td>15</td>
<td>19</td>
<td>314</td>
<td>9.9</td>
</tr>
<tr>
<td>20</td>
<td>20</td>
<td>250</td>
<td>9.5</td>
</tr>
</tbody>
</table>

3.3 Water absorption at atmospheric pressure
Figure 6 shows the results of water absorption tests at atmospheric pressure obtained on mortars with increasing aggregate volume replacement by GRP addition. It is evident that, at the end of the test, as expected, the total amount of water absorbed increased with the GRP addition due to the higher open porosity of the cement mortar previously detected. However, unexpectedly, the curves showed no significant separation in the first hours of full immersion in water.

3.4 Capillary water absorption
The results of water absorption test were again confirmed and better highlighted during the capillary water absorption test. The relative capillary absorption coefficient ($CA_{rel}$), defined as the ratio between
the capillary absorption coefficient (CA) of the GRP mortar to that of the mortar without GRP addition
(Table 2), showed that GRP was able to decrease the capillary water absorption even in a more porous
cement matrix, of about 70%. The capillary absorption coefficient (CA) represents the slope of the initial
(up to \( t = 30 \text{ minutes} \)) linear part of the water absorption curve reported in Figure 7. Therefore, water
absorption is initially more effectively counteracted by the presence of GRP. This would indicate that, when a
mortar made with GRP is exposed to short periods of contact with water, as in the case of rain, it will not
absorb as much water as expected for a common mortar manufactured with the same water-cement ratio.

The relative capillary absorption index (IC_{rel}) is defined as the ratio of the area subtended by the water
absorption curve of the GRP mortar to the area of the absorption curve of the reference mortar (Figure 7) and
it is related to the corresponding amount of water absorbed during the full contact time. As shown in
Table 3, when more prolonged contact between water and the cement matrix occurs, the value of IC_{rel} is lower
than 0.5. This implies that aggregate partial replacement by GRP addition is able to reduce the water
absorption by at least 50%.

These unexpected results could be explained by a certain “hydrophobic effect” produced by the GRP
addition on the cement mortar. This could be due to the polymeric part of the GRP, which may be able to
compensate for the higher porosity of the GRP mortar with respect to the reference mortar with the same
water-cement ratio. This is especially valid for short water contact periods.

### 3.5 Drying shrinkage test

Figure 8 shows the results obtained up to 1 month of storage at room temperature and constant relative humidity (70 ± 2%). It is evident how, unexpectedly, the drying shrinkage of the mortar with GRP waste addition at the dosage rate of 10% by aggregate volume came out to be almost 50% of the drying shrinkage of reference mortar without GRP, in spite of higher porosity and lower mechanical performance of the GRP mortars.

Perhaps, the polymeric part of the GRP addition could play an important role, but further investigation
has to be carried on in order to verify this hypothesis.
Moreover, if shrinkage is restrained and no creep effects are considered, the lower the modulus of elasticity such as that of GRP mortar, the lower will be the induced elastic tensile stress for a given shrinkage strain. So, instead of comparing concretes on the basis of shrinkage strain, it may be better to compare them on the basis of the induced tensile stress. Multiplying the shrinkage strain after 28 days of drying by the elastic modulus at the same age, the results obtained for the four mortars are: 15.3 MPa for the reference mortar and 6.0 MPa, 11.8 MPa and 15.7 MPa for GRP mortars manufactured with sand partially replaced by GRP waste at dosages of 10–15–20% by volume, respectively. It is evident that the drying shrinkage behaviour may not be a problem, especially for the GRP mortar with 10% of volume aggregate replacement.

4 CONCLUSIONS

An increase in porosity (25%) and consequently, a decrease in mechanical strength (up to 40% depending on the addition rate), as well as higher deformability, was detected on the mortars manufactured by GRP waste addition. Nevertheless, unexpectedly, capillary water absorption and drying shrinkage of GRP mortars resulted significantly lower (70% and 50%, respectively) than those of the reference mortar made without any GRP addition, which may involve enhanced durability, such that this is not only related to mechanical strength.

These results demonstrated the potential of re-using an abundant industrial by-product, at present landfilled, to manufacture durable precast elements without any structural target, contemporarily matching an environmental issue.

REFERENCES


Engineering properties of oil palm shell (ops) hybrid concrete for lightweight precast floor slab

C.H. Ng, Z. Ideris, S.P. Narayanan, M.A. Mannan & V.J. Kurian
School of Engineering and Information Technology, Universiti Malaysia Sabah, Sabah, Malaysia

ABSTRACT: An experimental study on the engineering properties of OPS hybrid concrete for precast floor slab was conducted. Conventional crushed stone as partial replacement of OPS was employed. Three conditions of curing to simulate the laboratory curing condition, practical curing condition, and without curing are applied. The slump, demoulded density, compressive strength, flexural strength, splitting tensile strength and modulus of elasticity were determined. OPS concrete with 30% crushed stone as coarse aggregate fulfilled the minimum strength requirement for lifting/transportation and at service. Since the air dry density of this concrete is 2000 kg/m³, it can be accepted as lightweight concrete. Hence, OPS hybrid concrete with crushed stones can be used to manufacture lightweight precast floor slab. The results of this ongoing research on the potential use of OPS concrete for lightweight precast floor slabs are reported.

1 INTRODUCTION

Precast systems using structural lightweight concrete have several potential advantages over normal weight concrete (NWC), including reduced dead load of the structure offers a reduction in foundation size, smaller lifting equipments for placing precast panels, floor slabs requiring smaller supporting systems, lower density, making it particularly suitable with large pour area, high thermal insulation for buildings, inherent fire resistance, as well as reduced construction cost due to time saving and flexibility of design (Pang & Lo 2005). Oil palm shell (OPS) concrete has been investigated for a number of years (Mannan & Ganapathy 2001, 2002; Mannan et al 2002, 2006, Teo et al 2005, 2006a) and also its potential in structural concrete production (Teo et al 2006b). Its application in precast floor slab is yet to be studied.

With OPS as the only coarse aggregate, the modulus of elasticity of such concrete is approximately one third of the NWC (Mannan & Ganapathy 2002). Therefore, in this study, for the application of precast floor slab conventional crushed stone as partial replacement of OPS was employed to produce OPS hybrid concrete. Three conditions of curing simulated the laboratory curing condition, practical curing condition, and without curing. The slump, demoulded density, compressive strength, flexural strength, splitting tensile strength and modulus of elasticity were determined. OPS concrete with 30% crushed stone as coarse aggregate fulfilled the minimum strength requirement for lifting/transportation and service condition. The minimum strength requirement of precast slabs at the time of lifting/transporting is 10–15 N/mm² (The Government of the Hong Kong Special Administrative Region & Halfen Limited 2003, BS EN 13747 2005) and at service is 35 N/mm² (if reinforced) to 40 N/mm² (Smorgon Steel Group 2004, Elliott 2006a, b).

2 MATERIALS AND METHODS

2.1 Materials

Malaysian ordinary Portland cement (ASTM Type I) is used as binder, sand as fine aggregate and, OPS and crushed granite as coarse aggregate. The properties of OPS, granite and sand are shown in Table 1. Testing of aggregates is done as per relevant BS or ASTM.

2.2 Mix proportion

For 1.0 m³ of concrete, 450 kg of cement, 629 kg sand, 341 kg OPS, 313 kg crushed granite and 7.5 litres super plasticizer is employed. Free water/cement (W/C) ratio is 0.38. OPS aggregate used in this investigation is in saturated surface condition.

2.3 Test methods

The compressive strength of standard cubes (100 mm) and cylinder (100 mm × 200 mm) is determined as
Beams of 100/500 mm size for flexural test (ASTM C 78-02), and 150°/300 mm size cylinders for splitting tensile (ASTM C 496) and modulus of elasticity tests (ASTM C 469-02) are employed. The performance of OPS hybrid concrete under three curing conditions are studied (Table 2). For C-1 curing, the samples are demoulded after one day, kept in water for 27 days and then tested whereas for C-2 the samples are demoulded after a day, kept in water for 6 days and left without curing for 21 days and then tested. The C-1 simulates the usual laboratory curing which is an ideal condition of curing while C-2 curing conforms to that recommended for ASTM Type-1 cement. C-3 indicates no curing.

Tests for compressive strength are carried out at 1, 3, 7, 28 days whereas for flexural strength and splitting tensile strength at 3, 7 and 28 days. Modulus of elasticity is taken at 28 days only. For all tests, each result is the average of three samples.

3 RESULTS AND DISCUSSION

3.1 Slump and demoulded density

The slumps obtained (BS 1881, Part 102) is in the range of 90–100 mm. Maximum slump, as per ASTM C143 with tolerances as established by ASTM C94, for concrete to be vibrated shall be 100 mm with respect to either normal weight or structural lightweight concrete lab.

The demoulded density is between 1,985 to 2,130 kg/m³. The air-dry density can be calculated by subtracting the mass of water lost to the air, typically between 50 to 150 kg/m³ for lightweight concrete with normal weight fine aggregate (Neville, 2005).

### Table 1. Properties of OPS, crushed granite and river sand.

<table>
<thead>
<tr>
<th>No</th>
<th>Property of aggregates</th>
<th>Unit</th>
<th>Coarse aggregate</th>
<th>Fine aggregate</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Maximum size</td>
<td>mm</td>
<td>12.50</td>
<td>19.00</td>
</tr>
<tr>
<td>2</td>
<td>Specific gravity (SSD)</td>
<td>~</td>
<td>1.34</td>
<td>2.63</td>
</tr>
<tr>
<td>3</td>
<td>Water absorption capacity (24 hrs)</td>
<td>%</td>
<td>21.30</td>
<td>2.04</td>
</tr>
<tr>
<td>4</td>
<td>Moisture content as received</td>
<td>%</td>
<td>11.68</td>
<td>0.78</td>
</tr>
<tr>
<td>5</td>
<td>Los Angeles abrasion value</td>
<td>%</td>
<td>2.90</td>
<td>11.85</td>
</tr>
<tr>
<td>6</td>
<td>Bulk density, compacted</td>
<td>kg/m³</td>
<td>646</td>
<td>1481</td>
</tr>
<tr>
<td>7</td>
<td>Fineness modulus</td>
<td>~</td>
<td>6.30</td>
<td>6.94</td>
</tr>
<tr>
<td>8</td>
<td>Aggregate impact value, AIV</td>
<td>%</td>
<td>6.60</td>
<td>12.60</td>
</tr>
<tr>
<td>9</td>
<td>Aggregate crushing value, ACV</td>
<td>%</td>
<td>7.12</td>
<td>15.95</td>
</tr>
</tbody>
</table>

### Table 2. Curing conditions for OPS hybrid concretes.

<table>
<thead>
<tr>
<th>No</th>
<th>Curing symbol</th>
<th>Duration of curing (days)</th>
<th>Mould</th>
<th>Water</th>
<th>Room</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C-1</td>
<td>1</td>
<td>27</td>
<td>~</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>C-2</td>
<td>1</td>
<td>6</td>
<td>21</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>C-3</td>
<td>1</td>
<td>~</td>
<td>27</td>
<td></td>
</tr>
</tbody>
</table>

### Table 3. Cube strength for OPS hybrid concrete at different ages and curing conditions.

<table>
<thead>
<tr>
<th>No</th>
<th>Curing symbol</th>
<th>1 day</th>
<th>3 days</th>
<th>7 days</th>
<th>28 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C-1</td>
<td>17.30</td>
<td>29.50</td>
<td>35.97</td>
<td>38.53</td>
</tr>
<tr>
<td>2</td>
<td>C-2</td>
<td>17.30</td>
<td>29.50</td>
<td>35.97</td>
<td>37.45</td>
</tr>
<tr>
<td>3</td>
<td>C-3</td>
<td>17.30</td>
<td>28.05</td>
<td>32.20</td>
<td>34.76</td>
</tr>
</tbody>
</table>

### Table 4. Cylinder Compressive strength for OPS hybrid concrete at different ages and curing conditions.

<table>
<thead>
<tr>
<th>No</th>
<th>Curing symbol</th>
<th>3 days</th>
<th>7 days</th>
<th>28 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C-1</td>
<td>15.70</td>
<td>20.60</td>
<td>21.57</td>
</tr>
<tr>
<td>2</td>
<td>C-2</td>
<td>15.70</td>
<td>20.60</td>
<td>21.17</td>
</tr>
<tr>
<td>3</td>
<td>C-3</td>
<td>14.45</td>
<td>18.25</td>
<td>19.35</td>
</tr>
</tbody>
</table>

respect to either normal weight or structural lightweight concrete lab.

The performance of OPS hybrid concrete under three curing conditions are studied (Table 2). For C-1 curing, the samples are demoulded after one day, kept in water for 27 days and then tested whereas for C-2 the samples are demoulded after a day, kept in water for 6 days and left without curing for 21 days and then tested. The C-1 simulates the usual laboratory curing which is an ideal condition of curing while C-2 curing conforms to that recommended for ASTM Type-1 cement. C-3 indicates no curing.

Tests for compressive strength are carried out at 1, 3, 7, 28 days whereas for flexural strength and splitting tensile strength at 3, 7 and 28 days. Modulus of elasticity is taken at 28 days only. For all tests, each result is the average of three samples.
The 28-day compressive strength of cubes and cylinders were higher under C-1 curing condition compared to C-2 and C-3. These results are similar to that reported by Mannan & Ganapathy (2002).

The 28-day cube compressive strength is approximately 62–69% higher that of the OPS concrete produced (Mannan & Ganapathy 2001) due to the addition of 30% crushed granite as partial replacement.

It is also observed that the 28-day cylinder compressive strength of OPS hybrid concrete is approximately 56% of its cube compressive strength.

3.3 Flexural strength

The flexural strength of OPS hybrid concrete at different ages and curing regimes is shown (Table 5). The 28-day flexural strength of OPS hybrid concrete is approximately 13% to 15% of its compressive strength. This is higher compared to flexural strength of OPS concrete reported by Mannan & Ganapathy (2002) because 30% of OPS have been replaced by crushed granite. The relation between the flexural and compressive strength for NWC depends on the type and properties of coarse aggregate used (Neville 2005). The flexural strength of NWC ranges between 11% and 23% of the compressive strength.

3.4 Splitting tensile strength

The splitting tensile strength at different ages and curing conditions is shown (Table 6). The strength of OPS hybrid concrete under C-1 curing condition is higher than that under C-2 and C-3 curing condition. Similar to flexural strength, the splitting tensile strength is dependent on the curing condition. Also, OPS hybrid concrete exhibits higher splitting tensile strength than that of OPS concrete.

The 28-day splitting tensile strength of OPS hybrid concrete under C-1 and C-2 curing is approximately 9% of its compressive strength. In general, the tensile strength of NWC ranges from 7% to 11% of its compressive strength, with an average of 10% (Hassoun & Al-Manaseer 2005).

3.5 Modulus of elasticity

Under C-1, the modulus of elasticity (28 days) of OPS hybrid concrete is in the range of $2.03–2.19 \times 10^4$ N/mm$^2$. Only C-1 curing condition is employed, as it is not believed to affect the modulus of elasticity (Neville 2005). This modulus of elasticity is higher than that of the OPS concrete reported by Mannan & Ganapathy (2002). Kett (2000) pointed that the $E_c$ of NWC ranges from 1.4 to $4.1 \times 10^4$ N/mm$^2$.

Modulus of elasticity can also be calculated from the ACI 318 (2005) using Equation 1 in SI units:

$$E_c = 0.043w^{0.5} \sqrt{f'_c} \tag{1}$$

where $E_c = $ Modulus of elasticity; $w = $ unit weight of concrete (between 1400 to 2500 kg/m$^3$); and $f'_c = $ ultimate strength of a standard cylinder.

From Equation 1, modulus of elasticity of OPS hybrid concrete is $1.91 \times 10^4$N/mm$^2$.

4 CONCLUSIONS

The conclusions drawn from this investigation are summarised below:

1. The 28-day compressive strength of OPS hybrid concrete ranges between 34.76 and 38.53 N/mm$^2$. The cylinder compressive strength is about 56% of the cube compressive strength.
2. The 28-day flexural strength of OPS hybrid concrete is between 4.52 and 5.79 N/mm$^2$ averaging 14% of its compressive strength.
3. The 28-day splitting tensile strength for OPS hybrid concrete is between 3.19 and 3.50 N/mm$^2$. It is approximately 9% of its 28-day compressive strength.
4. The 28-day modulus of elasticity of OPS hybrid concrete is in the range of $2.03–2.19 \times 10^4$ N/mm$^2$.
5. Generally, the OPS hybrid concrete obtains better engineering properties than that of OPS concrete under the C-1 curing, which is an ideal curing condition.
6. OPS hybrid concrete fulfils the minimum strength requirement of precast slabs at the time of
lifting/transporting and at the time of service. Hence, the use of OPS hybrid concrete for lightweight precast concrete floor slab is recommended.

ACKNOWLEDGEMENTS

This work is supported through research grant LPIPM: CREAM/UPP04-02-10-04-11 from the Construction Industry Development Board of Malaysia.

REFERENCES

ASTM, C 29M-97, Standard test method for bulk density, (“unit weight”), and voids in aggregate, Annual Book of ASTM Standards.
ASTM, C127-04, Standard test method for density, relative density (specific gravity), and absorption of coarse aggregate, Annual book of ASTM Standards.
Durability of concrete mixed with fine recycled aggregates

Salomon Levy
PhD, Professor University UNINOVE. Chairman of TC MAB Technical Committee for Safety Environment of the Brazilian Concrete Institute IBRACON

Paulo Helene
Full Professor. Universidade de São Paulo. Escola Politécnica. Av. Prof. Almeida Prado, Brazil. President of the Brazilian Concrete Institute IBRACON

ABSTRACT: In this paper an analysis was conducted on the influence of use of 20%, 50% and 100% contents of two types of fine recycled aggregates (from demolish concrete and from old masonry) on the following properties of concrete: water absorption by immersion, pore volume, carbonation and resistivity. This analysis was not restricted to compare mixtures from a table of results. All the analysis was based on concrete family behavior. The Mix Design Nomograms (MDN) was made for concrete families: one as reference and six others with different recycled contents to replace the natural fine aggregate. The MDN obtained for the properties studied permitted compare class 20 MPa, 30 MPa and 40 MPa concretes. The results obtained indicate that the carbonation depth reached a minimum and the resistivity reached a maximum when 50% contents of fine recycled masonry aggregates were used. However, this tendency was not repeated for pore volume and water absorption. Thus, it should conclude that the durability like a carbonation and resistivity are physical-chemical phenomena that, in this case, were inversely related to water absorption and volume of pores.

1 INTRODUCTION

At the present time, several research investigations can be found conducted in worldwide (SANTOS, 2007), (ETXEBERRIA, 2006), (COLLINS, 2000) supporting the production and the use of concrete with recycled aggregate from the technical and economic point of view.

This work was conducted with the aim of understanding the essential properties for predicting the behavior of concretes produced with recycled aggregates during your service life. The target was understanding more about the durability of concretes prepared with recycled aggregates.

The use of fine recycled aggregates for producing concretes with structural purposes has been discussed worldwide, but many authors (VINCKE, J 1994) and (RILEM, 1994) recommend that such aggregates should not be utilized in structural concretes, due to their high water absorption.

This paper shows one analysis of the influence of recycled aggregates on the durability of concretes. To reach that answer 4 (four) properties were selected, such as water absorption, pore volume, carbonation and the resistivity properties intimately relates to the durability of a concrete and, consequently of a structure.

Wirquin et al (2000) reported that the processes of water absorption in recycled aggregate and in natural aggregate concretes are similar and obey the same laws. Also Mehta & Monteiro (1994) reported that the water, as a primary agent, is able to deteriorate natural and artificial materials, such as concrete.

Water, ions and gas penetrating the concrete porosity can also accelerated the concrete degradation kinetics during the service life of the structure.

2 PURPOSE

This work aims to analyze the influence of two types of fine aggregates, one from recycled concrete, and the other from recycled masonry on four (4) properties of concrete: water absorption, pore volume, carbonation and resistivity. These properties has directly influence in the durability of a concrete.

3 MATERIALS AND METHODS

To conduct this work, seven concrete families were prepared, three with fine recycled masonry aggregates, three with fine recycled concrete aggregates and a reference concrete with natural quartz river sand.
Table 1. Composition of the aggregates used in the preparation of the concretes.

<table>
<thead>
<tr>
<th>Concrete family</th>
<th>Fine aggregates content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference</td>
<td>100% natural quartz river sand</td>
</tr>
<tr>
<td>FRCA* 20–80</td>
<td>20% FRCA – 80% natural quartz river sand</td>
</tr>
<tr>
<td>FRCA 50–50</td>
<td>50% FRCA – 50% natural quartz river sand</td>
</tr>
<tr>
<td>FRCA 100–0</td>
<td>100% FRCA – 0% natural quartz river sand</td>
</tr>
<tr>
<td>FRMA** 20–80</td>
<td>20% FRMA – 80% natural quartz river sand</td>
</tr>
<tr>
<td>FRMA 50–50</td>
<td>50% FRMA – 50% natural quartz river sand</td>
</tr>
<tr>
<td>FRMA 100–0</td>
<td>100% FRMA – 0% natural quartz river sand</td>
</tr>
</tbody>
</table>

FRCA* fine recycled concrete aggregate.
FRMA** fine recycled masonry aggregate.

The proportions of the aggregates used in the preparation of the seven families are indicated in Table 1.

3.1 Materials

Blended Portland Cement, comprised of 35% blast furnace slag, with a Blaine fineness of 385 m²/kg, density of 2990 kg/m³ and compressive strength at 28 days of 49 MPa (7105 psi) was used in all concrete mixtures.

The fine natural aggregate used was natural washed quartz river sand, with a density of 2650 kg/m³, fineness modulus of 2.6, and water absorption of 1.8% at a saturated dry surface condition (SDS), according with ASTM C270.

The coarse natural aggregate used was crushed granite gravel, with a density of 2700 kg/m³, maximum size of 25 mm, fineness modulus of 7.0, and 0.8% water absorption at a SDS. Both are usual and sound aggregates in the Sao Paulo city region.

The fine recycled concrete aggregate (FRCA) that were used had been obtained from crushing old concrete structures, six months old. These old concrete presented compressive strength of 25 MPa, (3626 psi) and water cement ratio W/C of 0.66. The FRCA had a density of 2320 kg/m³, maximum particle size of 2.4 mm; fineness modulo of 2.5, and 10.3% water absorption at SDS.

The fine recycled masonry aggregate (FRMA) had been obtained from crushing one-year-old clay brick walls covered with bastard mortar (cement, calcium hydroxide and natural sand). The FRMA were composed of 76% clay brick, and 24% bastard mortar, by weight. The density was 1890 kg/m³, maximum particle size of 2.4 mm, fineness modulo of 2.5, and 13.0% water absorption at SDS.

In order to obtain the recycled aggregates, the demolished old concrete and masonry were passed through a jaw crusher and the resulting product was later subjected to a sieving operation.

The fractions corresponding to fine and coarse aggregates were used to produce the six (6) recycled concrete families (see Table 1), always with the aggregates at SDS condition. The results were compared to those measured for a reference concrete family, produced exclusively with natural aggregates.

3.2 Concrete mix proportion

Three dry aggregate/cement ratios by mass were used to compose each concrete family: 3/1; 4.5/1 and 6/1. All mixtures had the same fresh workability measured by slump test (70 ± 10) mm, what means, these concretes has the workability enough for many applications like, for instance, columns in buildings. For each mixture the recycled aggregate replace 0%, 20%, 50% and 100% of natural fine aggregates by mass. In this way, the total of seven concrete families and 21 different mix proportions were produced.

The main tests were performed beginning at 28 days. All specimens were cured in a standard humid chamber during their first 14 days; after that, they were kept exposed to laboratory ambient conditions (55% to 65% RH and 20° to 26°C).

3.3 Properties measured

The following properties were measured: compressive strength by ASTM C39; water absorption and total pore volume by ASTM C642; accelerated carbonation by RILEM CPC-18 method, and resistivity by ASTM G57. In addition, the cement content, water content, W/C ratio by mass, entrapped air, slump test and fresh concrete density were measured.

4 DATA RESULTS

Table 2 presents the data results for each mixture.

5 CONSTRUCTING THE MDN

A Mix Design Nomogram (MDN) was constructed for each of the seven (7) concrete families, one for the reference concrete family, three for concrete made with FRCA and three for concrete made with FRMA, as presented in Fig. 1, as an example.

Then, the corresponding MDN was used to obtain the W/C for the desired compressive concrete strength of 20 MPa, 30 MPa and 40 MPa, (2900 psi; 4350 psi and 5800 psi) at 28 days, for each family. The results are presented in Table 3.
Table 2. Result for each mixture.

<table>
<thead>
<tr>
<th>Concrete families</th>
<th>Aggregate replacement</th>
<th>Mix proportion (cement: sand river: fine recycled aggregate: coarse aggregate: W/C)</th>
<th>$f_c$ at 28d MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1.3 1.4,5 1.6</td>
<td>1.3 1.4,5 1.6</td>
</tr>
<tr>
<td>Natural</td>
<td></td>
<td>1: 1.24: 0.0: 1.76: 0.40 1: 2.08: 0.0: 2.42: 0.51 1: 2.92: 0.0: 3.08: 0.75</td>
<td>48.5 41.9 26.5</td>
</tr>
<tr>
<td>FRCA 20</td>
<td></td>
<td>1: 0.99: 0.25: 1.76: 0.46 1: 1.64: 0.416: 2.42: 0.58 1: 2.34: 0.58: 3.08: 0.69</td>
<td>56.1 36.7 27.5</td>
</tr>
<tr>
<td>FRCA 50</td>
<td></td>
<td>1: 0.62: 0.62: 1.76: 0.44 1: 1.04: 1.04: 2.42: 0.61 1: 1.46: 1.46: 3.08: 0.75</td>
<td>46.3 33.8 26.96</td>
</tr>
<tr>
<td>FRMA 20</td>
<td></td>
<td>1: 0.0: 1.24: 1.76: 0.47 1: 0.0: 2.08: 2.42: 0.61 1: 0.0: 2.92: 3.08: 0.80</td>
<td>46.6 30.2 23.3</td>
</tr>
<tr>
<td>FRMA 50</td>
<td></td>
<td>1: 0.99: 0.25: 1.76: 0.39 1: 1.64: 0.416: 2.42: 0.55 1: 2.34: 0.58: 3.08: 0.70</td>
<td>49.0 38.2 26.6</td>
</tr>
<tr>
<td>FRMA 100</td>
<td></td>
<td>1: 0.62: 0.62: 1.76: 0.45 1: 1.04: 1.04: 2.42: 0.56 1: 1.46: 1.46: 3.08: 0.74</td>
<td>49.5 40.0 29.4</td>
</tr>
<tr>
<td>FRMA 100</td>
<td></td>
<td>1: 0.0: 1.24: 1.76: 0.50 1: 0.0: 2.08: 2.42: 0.66 1: 0.0: 2.92: 3.08: 0.81</td>
<td>42.8 36.6 27.0</td>
</tr>
</tbody>
</table>

Table 3. Water cement ratio from MDN for desired compressive strength.

<table>
<thead>
<tr>
<th>Concrete families: compressive strength at 28d: Aggregate replacement</th>
<th>W/C ratio 20MPa</th>
<th>30MPa</th>
<th>40MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural</td>
<td>0%</td>
<td>0.91</td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>20%</td>
<td>0.80</td>
<td>0.66</td>
</tr>
<tr>
<td>FRCA</td>
<td>50%</td>
<td>0.92</td>
<td>0.69</td>
</tr>
<tr>
<td></td>
<td>100%</td>
<td>0.87</td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>20%</td>
<td>0.86</td>
<td>0.65</td>
</tr>
<tr>
<td>FRMA</td>
<td>50%</td>
<td>0.95</td>
<td>0.73</td>
</tr>
<tr>
<td></td>
<td>100%</td>
<td>1.03</td>
<td>0.76</td>
</tr>
</tbody>
</table>

Figure 1. Strength MDN for concrete family 100%–0% FRCA.

Figure 2. Water absorption and total volume pore for concrete family FRCA 100%–0%.

Figure 2 shows the correspondence between pore volumes and water absorption with W/C. Figure 3 shows the carbonation behavior and Fig. 4 presents the resistivity property as a function of W/C ratio.

Knowing each W/C ratio for each compressive strength, were possible to construct the correspondence between the durability properties (water absorption; pore volume; carbonation and resistivity) with the amount of fine recycled aggregate replacing natural aggregates, as showed in Fig. 5, Fig. 6, Fig. 7, and Fig. 8. Also was possible to find the cement content, resistivity, carbonation, pore volume and water absorption for each W/C to reach the specified compressive strength as showed in Tables 4 and 5.
The water absorption, as well as the total pore volume, increases with the amount of natural aggregates replaced by recycled aggregates. This behavior was expected, since the absorption of the fine recycled aggregates, FRCA and FRMA, was 6 to 10 times higher than the natural aggregates.

The carbonation depth, for the same compressive strength, reached an optimal value when 50% of the natural aggregate was replaced by FRMA. When the natural aggregate was replaced by FRCA, no significant alterations were recorded in the carbonation depth, independently of the replacement content.

When using FRCA or FRMA, also with 100% of replacement, the carbonation depth is still lower when compared to reference concrete families made by natural aggregates (see Fig. 3). The explanation of this behavior is supported by the highest cement content of recycled concretes to achieve the same compressive concrete strength of natural aggregates concrete. Fine recycled aggregate concretes need a higher cement content to achieve the same compressive strength as compared to reference concrete, as shown in Fig. 3 and Fig. 4.

![Figure 3](image1.png)  
Figure 3.  Carbonation MDN for concrete reference family and 100% FRCA.

![Figure 4](image2.png)  
Figure 4.  Resistivity for concrete family 100%–0% FRCA.

![Figure 5](image3.png)  
Figure 5.  Evolution of water absorption with the amount of natural aggregate being replaced by FRCA and FRMA to achieve 20 MPa, 30 MPa and 40 MPa at 28 days.
Figure 6. Evolution of total pore volume and carbonation depth as a function of replacement of natural aggregate by FRCA and FRMA to achieve 20 MPa, 30 MPa and 40 MPa at 28 days.

Figure 7. Evolution of carbonation depth as a function of replacement of natural aggregate by FRCA and FRMA to achieve 20 MPa, 30 MPa and 40 MPa at 28 days.

Figure 8. Evolution of resistivity with the amount of natural aggregate being replaced by FRCA and FRMA to achieve 20 MPa, 30 MPa and 40 MPa at 28 days.
This higher alkaline reserve acts to protect the concrete surface against carbonation mechanism. Clifton (1993) reported that the positive effect of the alkaline reserve in relation to carbonation can increase the service life of concrete structures, because they last longer without depassivation of reinforcement, which means an increase in the corrosion initiation period (Tuutti 1982).

Concerning resistivity, the Comité Euro-International du Béton (bulletin CEB 192) presents criteria for assessing the reinforcement corrosion probability as a function of concrete resistivity, classifying concretes from 5 kΩ.cm to 10 kΩ.cm, as high corrosion probability and concretes with more than 10 kΩ.cm as low and negligible risks of steel reinforcement corrosion. All the resistivity measurements were conducted under the same conditions. Comparing data results with CEB criteria, only when the FRCA replace 100% of natural aggregate the corrosion probability can be considered high. In all the others mixtures the corrosion probability is low or negligible.

All the properties investigated – absorption, total pore volume, carbonation and resistivity – could be expected to present consistent behavior. For the concrete family produced with the replacement of 50% of natural aggregate by FRMA, minimum values were obtained for carbonation and maximum values were obtained for resistivity. The reason for this observation can be justified why the replacement of natural aggregate reach better compaction, the high density.

### 7 CONCLUSIONS

By applying the very clear concepts proposed for experimental concrete mix design by Helene & Monteiro (1993) to the properties studied in this work, it was possible to observe and compare the behavior of concrete families and not only between isolated mixtures.

Under the conditions of this research it is possible to conclude that:

Concrete made with recycled aggregates (20%; 50% and 100% replacement) from old masonry or from old concrete can have the same fresh workability and can achieve the same 28-day compressive strength as concrete made with natural aggregates in the range from 20 MPa to 40 MPa. The carbonation depth always decreased when the replacement was 20% or 50%. For FRMA concrete...
family this improvement in behavior also occurred when the replacement was 100%. This behavior shows that carbonation depth strongly depends on the chemical composition of the concrete and not only on its physical aspects.

When 50% of the natural fine aggregate was substituted by FRMA, the resistivity reached maximum values.

The replacement of natural aggregates by recycled ones has been shown to be technically viable; however, it will be necessary to take into account that the recycled aggregates had water absorption about 10 times higher than that of natural aggregates, and this fact increased both total pore volume and water absorption of the concrete.

When the natural aggregate is replaced by the 20% or 50% of recycled aggregates from old concrete or old masonry the resulting recycled concrete may have the same, and sometimes better, behavior than the reference concrete made with natural aggregates. This fact justifies the efforts to use these concretes, which can contribute to the preservation of the environment and can achieve the same final performance at probably lower costs than those of ordinary concretes.

This investigation shows that it is possible to evaluate the influence of recycled aggregates in the depth of carbonation concrete. Also shows that the traditional approach by pore volume is not enough for understand the phenomena. The CO₂ gas penetration depends not only from porosity but also from concrete materials, which means chemicals aspects.

The use of Mix Design Nomogram (MDN) introduced by Helene & Monteiro (1993), allows the researchers to make a correct and relevant comparison between the different concrete families, adopting the same reference compressive concrete strength, instead of the usual poor comparison between individual mixture results.

The results obtained in this work point towards the effective possibility of using fine recycled aggregates in the production of new concretes, since the maximum substitution limits of 50% are complied with.

REFERENCES


An experimental investigation on the viability of using fine concrete recycled aggregate in concrete production

S.R. Sarhat
The High Institute for Comprehensive Professions, Ghadames-Libya

ABSTRACT: This paper presents an experimental investigation on the feasibility of full or partial replacement of fine natural aggregate (FNA) with fine recycled concrete aggregate (FRA) in concrete production. The studied variables were the FRA substitution ratio, the water-cement ratio, and the effect of presence of materials finer than 0.075 mm in FRA. Workability, compressive and splitting tensile strength were measured. The experimental results indicate that the replacement of FNA with FRA is feasible. Only marginal effects were noticed on workability and splitting tensile strengths with substitution of up to 45%, which even led to an increase in the compressive strength. It can be concluded from the results that the water–cement ratio is a key factor that affects the workability and the strength of concrete made with FRA. Also, the results show that the presence of up to 10% of materials finer than 0.075 mm in FRA does not affect workability or strength characteristics in the case of partial replacement of fine aggregate.

1 INTRODUCTION

Using recycled aggregates larger than 4.75 mm in production of new concrete is a well-studied topic all over the world, while the fine portion from demolition of concrete, which represents about 30% to 60% of the crushed demolition materials [Solayman 2005], is disposed of. The main reason for this situation is that the fines exhibit high water absorption ratios, which lead to a weak performance of the concrete matrix [Wainwright et al. 1994].

Some countries are suffering from a lack of appropriate natural fine aggregate for construction projects. In Libya, for example, the natural sands are too fine and usually do not achieve the grading requirements according to the BS: 882–1992 standards; however, people use it because they simply have no other alternatives. Also, sand extraction both in riverbeds and on the seacoast has had, sometimes, serious impacts [Evangelista & de Brito 2004]. To find solutions for the fine waste resulting from concrete crushing and to minimize natural sand extraction, which subsequently would minimize the environmental impact of this extraction, more research efforts are required.

In the present investigation, a research program was achieved to study the feasibility of full or partial replacement of fine natural aggregates with fine recycled concrete aggregates (FRA).

2 EXPERIMENTAL PROGRAM

The studied parameters in this investigation were:

- The substitution ratio of natural sand with fine recycled aggregates.
- The water-cement ratio.
- The effect of the presence materials finer than 0.075 mm in FRA.

2.1 Characterization of fine recycled concrete aggregates

FRA were obtained by crushing concrete parts resulting from demolition of an old concrete building built in 1970. The concrete was crushed in a large crushing plant used to obtain coarse and fine aggregates from crushing natural stones. Considerable efforts were made to keep the obtained fine recycled aggregates without dirty. The grading curve of the FRA lay in grading zone 2 according to BS: 882–1992.

The fine natural aggregates used in this investigation were a common type of natural sand in Libya, which always has a grading curve in zone 3. The main characteristics of the FNA and FRA are presented in Table 1.

2.2 Mixture proportions

The mixture proportions, expressed by mass as cement, coarse aggregate, and fine aggregate were
1:1.75:3.3 for all mixes. These proportions are recommended by the municipal regulations in Libya to obtain a structural concrete for normal residential and public buildings. The methodology of absolute volume was used to find the concrete composition for each batch. Ordinary Portland cement was used for all the mixes; and crushed coarse limestone aggregates, having a surface specific gravity of 2600 kg/m³, water absorption of 2.5%, and maximum size of 15 mm, were used.

Mixtures used in this investigation can be divided into three groups. The first group was prepared to investigate the feasibility of replacing FNA with FRA. Six concrete mixtures with various substitution ratios (0.0%, 15%, 30%, 45%, 60%, 100%) were cast and tested. The water-cement ratio (W/C) was fixed at 0.6. In the second group, four mixtures were used to study the effect of the presence of materials finer than 0.075 mm in FRA. Fine materials contents of 0.0%, 10%, 20% and 30% were investigated. A W/C of 0.65 and an FRA replacement ratio of 45% were used in all the four mixtures of this group. Tables 1 to 3 show the mixtures compositions used in 1 m³ for all groups.

Although the obtained FRA has only 5% fine materials content, this factor was studied in this investigation for two reasons. First, it was found recently in Libya that sand portion resulting from crushing natural stones in the crushing plants contains 8% to 30% materials finer than 0.075 mm [Mansur et al. 2004]. Secondly, these crushing plants, without any doubts, will be used to get coarse and fine recycled aggregates from crushing old concrete if a considerable efforts are made by the municipal authorities and people to adopt recycling. In this case, the FRA may contain high percents of fine materials.

The workability of each mixture was evaluated through the standard slump test. Also, for each mixture, six 150 x 150 x 150 mm cubes were produced for measurement of compressive strength, and six cylinders with 150 mm diameter and 300 mm height were produced for measurement of splitting tensile strength.

<table>
<thead>
<tr>
<th>Table 1. Properties of FNA and FRA.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Properties</td>
</tr>
<tr>
<td>Surface dry specific density (kg/m³)</td>
</tr>
<tr>
<td>Compacted bulk density (kg/m³)</td>
</tr>
<tr>
<td>Water absorption (%)</td>
</tr>
<tr>
<td>% Materials finer than 0.075 mm %</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 2. Mixtures compositions (for 1 m³) and test results of specimens cast for studying FRA substitution.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimens</td>
</tr>
<tr>
<td>% FRA Replacement</td>
</tr>
<tr>
<td>Cement (Kg)</td>
</tr>
<tr>
<td>FNA (Kg)</td>
</tr>
<tr>
<td>FRA (Kg)</td>
</tr>
<tr>
<td>Coarse aggregate (Kg)</td>
</tr>
<tr>
<td>Water (Kg)</td>
</tr>
<tr>
<td>Slump (mm)</td>
</tr>
<tr>
<td>Compressive strength MPa</td>
</tr>
<tr>
<td>Indirect tensile strength MPa</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 3. Mixtures compositions (for 1 m³) and test results of the specimens cast for studying water-cement ratio.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimens</td>
</tr>
<tr>
<td>Water-cement ratio</td>
</tr>
<tr>
<td>Cement (Kg)</td>
</tr>
<tr>
<td>FNA (Kg)</td>
</tr>
<tr>
<td>FRA (Kg)</td>
</tr>
<tr>
<td>Coarse aggregate (Kg)</td>
</tr>
<tr>
<td>Water (Kg)</td>
</tr>
<tr>
<td>Slump (mm)</td>
</tr>
<tr>
<td>Compressive strength MPa</td>
</tr>
<tr>
<td>Splitting tensile strength MPa</td>
</tr>
</tbody>
</table>
After demolding at an age of one day, the specimens were cured at 20°C and 100% relative humidity. All tests were carried out based on BS:1881–1993.

2.3 **Concrete mixing**

The production process, based on the procedures explained by [Roose & Massivbau 1999] was fixed for all mixtures. First, the aggregates were mixed with 20% of the water for exactly two minutes. After that they were covered with a PE-foil for ten minutes. The cement and the rest of the water were added and mixed for another two minutes before casting into moulds. Due to the high water-cement ratio used in all mixtures, no superplasticizers were used.

### 3 TEST RESULTS

#### 3.1 Substitution of FNA with FRA

The workability index used in this investigation, slump, was greatly affected by increasing the substitution ratio. The results obtained are presented in Figure 1. There was a gradual decrease in slump as the substitution ratio increased. Also, it is clear from Figure 1 that, up to 45% substitution ratio, values of the slump were acceptable and it was possible to get a workable mixture.

Table 1 and Figure 2 show obviously that the various substitution ratios of FRA led to small increases in the compressive strength of all mixtures. These results can be explained by two probable reasons. Firstly, the FRA used in this investigation was well graded in comparison with the FNA. Secondly, the FRA may possess a significant quantity of cement (both hydrated and non-hydrated), thus it is possible that part of this cement adds up to the cement from the mixture, increasing the global cement ratio of the mixture [Katz 2003]. The best improvement in compressive strength was achieved by replacing FNA with 30% FRA, which led to a 5% increase. The splitting tensile strength decreased slightly as the substitution ratio of FNA with FRA was increased. However, no distinct trend is visible.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>F0F</th>
<th>F10F</th>
<th>F20F</th>
<th>F30F</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Fine materials</td>
<td>0.0</td>
<td>10</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>Cement (Kg)</td>
<td>340.78</td>
<td>340.78</td>
<td>340.78</td>
<td>340.78</td>
</tr>
<tr>
<td>FNA (Kg)</td>
<td>328</td>
<td>328</td>
<td>328</td>
<td>328</td>
</tr>
<tr>
<td>FRA (Kg)</td>
<td>268.37</td>
<td>241.5</td>
<td>214.7</td>
<td>187.86</td>
</tr>
<tr>
<td>Materials finer than 0.075 mm</td>
<td>0.0</td>
<td>26.8</td>
<td>53.67</td>
<td>80.52</td>
</tr>
<tr>
<td>Coarse aggregate (Kg)</td>
<td>1124.6</td>
<td>1124.6</td>
<td>1124.6</td>
<td>1124.6</td>
</tr>
<tr>
<td>Water (Kg)</td>
<td>221.5</td>
<td>221.5</td>
<td>221.5</td>
<td>221.5</td>
</tr>
<tr>
<td>Slump (mm)</td>
<td>55</td>
<td>50</td>
<td>35</td>
<td>15</td>
</tr>
<tr>
<td>Compressive strength MPa</td>
<td>41</td>
<td>42.5</td>
<td>39</td>
<td>37</td>
</tr>
<tr>
<td>Splitting tensile strength MPa</td>
<td>3.2</td>
<td>3.35</td>
<td>3</td>
<td>2.85</td>
</tr>
</tbody>
</table>

Figure 1. Effect of FRA substitution ratio on workability.

Figure 2. Effect of FRA substitution ratio on the compressive strength.
3.2 Water-cement ratio

The results show that the W/C is still a prime factor which affects the workability and other mechanical properties of concrete containing FRA. It is clear from Figures 3 to 5 that increasing the W/C resulted in a considerable increasing in workability while this was also accompanied by a significant decreasing in compressive and splitting tensile strengths. A W/C of 0.6 was enough to get a workable mixture with a reasonable strength without using super-plasticizers or other additives. For W/C less than 0.6 it was difficult to get an acceptable workability without using additives.

3.3 Materials finer than 0.075 mm

It is known that a high content of materials finer than 0.075 mm may jeopardize the quality of the concrete, so this content is limited by standards. For example, according to BS: 882–1999, The maximum acceptable limit for materials finer than 0.075 is 16%.

It can be concluded from Figures 6 to 8 that up to 10% content of materials finer than 0.075 mm had a marginal effect on workability and even led to an increase in the compressive and splitting tensile strength.

![Figure 5. Effect of water-cement ratio on the splitting tensile strength.](image)

![Figure 6. Effect of microfines content on the compressive strength.](image)

![Figure 7. Effect of microfines content on the splitting tensile strength.](image)

![Figure 3. Effect of water-cement ratio on workability of concrete containing FRA.](image)

![Figure 4. Effect of water-cement ratio on the compressive strength.](image)
strength of concrete made with 45% FRA substitution ratio, while increasing the fine materials content to 20% and 30% led to considerable decreases in these properties. The compressive strength decreased by 4.8% and 9.8%, respectively. Also, 4.7% and 11% decreases in splitting tensile strength were recorded. Workability was the property most affected by increasing the content of fine materials in the FRA. The slump decreased by 75% when the content of fine materials increased to 30%.

4 CONCLUSIONS

The following conclusions can be reached from the present investigation concerning the feasibility of using fine recycled concrete aggregates:

– The replacement of natural fine aggregate with fine recycled concrete aggregate is feasible especially when the natural aggregates do not achieve the grading requirements.
– The substitution ratio of FRA, the water-cement ratio and the content of fine materials are key parameters for concrete made with FRA.
– With a substitution ratio of up to 45%, FRA has no major impact on workability and splitting tensile strength; and may even lead to an increase in the compressive strength.
– Water-cement ratio is a prime factor and it affects the properties of concrete made with FRA. For a substitution ratio of 45%, increasing the water-cement ratio led to a significant decrease in workability, compressive strength, and splitting tensile strength.
– When FRA was partially used, up to 10% content of materials finer than 0.075 mm had only a marginal effect on workability and led to increase the compressive and splitting tensile strength. In contrast, higher contents of these microfines (20% and 30%), noticeably decreased workability and strength characteristics.

ACKNOWLEDGEMENT

This experimental work was supported by the University of Al-Jabel Al-Gharby in Libya. Thanks to Mohammed Mosa, Hatem Jalal and Mohammad Balgho, who carried out a part of this work in their graduation project under the supervision of the author.

REFERENCES

Effect of different recycled aggregates on mortar performance

V. Corinaldesi, S. Monosi & G. Moriconi
Università Politecnica delle Marche, Ancona, Italy

ABSTRACT: This work was aimed at evaluating both mechanical and rheological behaviour of cementitious joint mortars prepared with three different kinds of recycled aggregate: one is made of concrete scraps obtained as rejected material from precast concrete production, one is based on recycled bricks and the last one is a miscellany of rubble from a demolition waste recycling plant. The experimental results showed that mechanical strength of the mortars prepared by substituting the natural sand with each one of these recycled aggregates is significantly lower than the reference. Nevertheless, the bond strength at the interface between mortar and brick was higher for mortars prepared with recycled aggregates and, in particular, when the fine aggregate demolition waste recycling was used.

1 INTRODUCTION

In Italy, every year more than 20 million tonnes of construction and demolition waste are produced. They are mainly constituted of debris coming from demolished structures made of masonry or reinforced concrete. At present, only 10% of this material is recycled and the remainder is disposed of in landfills for inert materials if it is not unlawfully disposed of by other means.

This work is aimed at evaluating the possibility of reusing waste from building demolition as aggregate for joint mortars. The annual production of joint mortar can be estimated as approximately 1 million cubic meters; therefore the total amount of material potentially recyclable could be significant.

A previous study [Sani et al. 2005] carried out on leaching behaviour of cementitious mixtures prepared with recycled aggregates coming from the same source showed that the environmental impact of these mortars can be sustainable.

In order to evaluate the quality of joint mortars made of recycled aggregate, both mechanical behaviour of cementitious mortars and rheological behaviour of cementitious pastes were evaluated. The mortars were prepared with three different kinds of recycled aggregate.

Finally, the interaction between the mortar and brick, in terms of bond strength developed at the interface mortar-brick, was studied.

2 MATERIALS

A commercial portland-limestone blended cement type CEM II/B-L 32.5 R according to EN-197/1 was used.

3 MIXTURE PROPORTIONS

3.1 Mortars

The proportions of the mortar mixtures are given in Table 2. The cement to sand ratio was 1:3 (by mass);
the water content of each mortar was set to achieve the same consistency of 110 ± 5 mm, evaluated according to EN 1015-3. Obviously, the higher the water absorption of the aggregate fractions (see Table 1), the higher the water amount necessary to reach the target consistency (see Table 2).

Table 1. Characterization of the recycled aggregate fractions.

<table>
<thead>
<tr>
<th>Aggregate type</th>
<th>REF</th>
<th>REC</th>
<th>BRI</th>
<th>CON</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volumetric mass in s.s.d.* (kg/m³)</td>
<td>2.59</td>
<td>2.42</td>
<td>2.18</td>
<td>2.38</td>
</tr>
<tr>
<td>Water absorption (%)</td>
<td>3.1</td>
<td>7.8</td>
<td>17.3</td>
<td>10.4</td>
</tr>
<tr>
<td>Fraction passing 150 µm sieve (%)</td>
<td>10</td>
<td>8</td>
<td>20</td>
<td>10</td>
</tr>
<tr>
<td>Blaine fineness of the fraction under 150 µm (m²/g)</td>
<td>0.145</td>
<td>0.793</td>
<td>0.474</td>
<td>0.483</td>
</tr>
</tbody>
</table>

* s.s.d. = saturated surface-dried.

Table 2. Mortar mixture proportions (kg/m³).

<table>
<thead>
<tr>
<th>Mixture</th>
<th>REF</th>
<th>REC</th>
<th>BRI</th>
<th>CON</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water/Cement</td>
<td>0.62</td>
<td>0.82</td>
<td>1.22</td>
<td>0.78</td>
</tr>
<tr>
<td>Water</td>
<td>280</td>
<td>370</td>
<td>550</td>
<td>350</td>
</tr>
<tr>
<td>Cement</td>
<td>450</td>
<td>450</td>
<td>450</td>
<td>450</td>
</tr>
<tr>
<td>Natural sand</td>
<td>1350</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Recycled rubble</td>
<td>–</td>
<td>1350</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Recycled brick</td>
<td>–</td>
<td>–</td>
<td>1350</td>
<td>–</td>
</tr>
<tr>
<td>Recycled concrete</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>1350</td>
</tr>
</tbody>
</table>

Table 3. Dosages of cement paste ingredients (g).

<table>
<thead>
<tr>
<th>Mixture</th>
<th>CEM+ REF</th>
<th>CEM+ REC</th>
<th>CEM+ BRI</th>
<th>CEM+ CON</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water/Cement</td>
<td>0.62</td>
<td>0.82</td>
<td>1.22</td>
<td>0.78</td>
</tr>
<tr>
<td>Water</td>
<td>280</td>
<td>370</td>
<td>550</td>
<td>350</td>
</tr>
<tr>
<td>Cement</td>
<td>450</td>
<td>450</td>
<td>450</td>
<td>450</td>
</tr>
<tr>
<td>Natural sand (&lt;150 µm)</td>
<td>135</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Recycled rubble (&lt;150 µm)</td>
<td>–</td>
<td>108</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Recycled brick (&lt;150 µm)</td>
<td>–</td>
<td>–</td>
<td>270</td>
<td>–</td>
</tr>
<tr>
<td>Recycled concrete (&lt;150 µm)</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>135</td>
</tr>
</tbody>
</table>

Table 4. Results of the mechanical tests on mortars at different curing times.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>M-REF</th>
<th>M-REC</th>
<th>M-BRI</th>
<th>M-CON</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexural strength (MPa) after 1 day</td>
<td>2.8</td>
<td>1.6</td>
<td>1.6</td>
<td>1.9</td>
</tr>
<tr>
<td>Strength (MPa) after 7 days</td>
<td>5.7</td>
<td>4.4</td>
<td>3.8</td>
<td>5.4</td>
</tr>
<tr>
<td>Compressive strength (MPa) after 28 days</td>
<td>6.6</td>
<td>5.0</td>
<td>4.7</td>
<td>6.9</td>
</tr>
</tbody>
</table>

4 MECHANICAL CHARACTERIZATION OF MORTARS

For each mortar mixture nine prismatic specimens (40 × 40 × 160 mm) were manufactured and wet cured at 20°C. The compressive and flexural strengths were evaluated after 1, 7 and 28 days of curing according to EN 196-1.

The results are reported in Table 4. Mortars containing recycled aggregates developed lower compressive and flexural strengths with respect to the reference cementitious mortar, particularly when the type ‘BRI’ aggregate was used. A 28-day compressive strength value higher than 20 MPa could be reached only for the mortar made with type ‘CON’ recycled aggregate. In this case, the shape of the concrete particles, sharp-cornered, plays a positive role in improving bond strength at the interface between the aggregate and surrounding cement paste.

5 RHEOLOGICAL CHARACTERIZATION OF PASTES

The rheological behaviour of the cement pastes was tested every 15 minutes up to one hour after the ingredients were mixed.
The experimental apparatus was a rotating rheometer based on coaxial rotary cylinders with a slowly increasing shear rate \( D \) ranging from 0 to 100 s\(^{-1}\). The walls of the concentric cylinders were not smooth but roughened in order to reduce (if not totally eliminate) the ‘slip’ phenomenon, i.e., the development of a water-rich layer close to the rotating cylinder which produces a lubricating effect, making flow easier and not representative of the bulk material [Saak et al. 2002].

The rheological behaviour was described by means of the Bingham flow model:

\[
\tau = \tau_y + \eta \cdot D
\]

where \( \tau \) is the shear stress (Pa), \( \tau_y \) is the yield stress (Pa), \( \eta \) is the plastic viscosity (Pa s) and \( D \) is the shear rate (s\(^{-1}\)).

The Bingham curves for each cement paste were extrapolated from experimental tests. The slope of the down-curve (decreasing shear rate) was used to calculate the plastic viscosity, while the intercept at zero shear rate was used to calculate the yield stress.

Thixotropy (which is the property of certain gels, such as cement paste, to be rigid when left standing but to become fluid when put into movement) was calculated as the area included between the up-curve and the down-curve.

The results obtained after 30 minutes from mixing of the ingredients are reported in Table 5.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>CEM+ REF</th>
<th>CEM+ REC</th>
<th>CEM+ BRI</th>
<th>CEM+ CON</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield stress (Pa)</td>
<td>49.4</td>
<td>6.3</td>
<td>3.9</td>
<td>7.6</td>
</tr>
<tr>
<td>Plastic viscosity (Pa s)</td>
<td>0.67</td>
<td>0.07</td>
<td>0.04</td>
<td>0.08</td>
</tr>
<tr>
<td>Thixotropy (Pa s)</td>
<td>-182</td>
<td>-855</td>
<td>-955</td>
<td>-522</td>
</tr>
</tbody>
</table>

6 MORTAR-BRICK BOND STRENGTH EVALUATION

In this work a test method, derived from the draft EN 1052-3 [CEN 1996], was followed in order to evaluate the bond strength developed during the shearing of a masonry unit with respect to the other along a mortar layer 10 mm thick. In this way the masonry behaviour in the absence of normal stress was investigated, corresponding to the constant term in the Mohr-Coulomb friction law.

The tested model, shown in Figure 2, is composed of three bricks; it has a symmetric structure, thus avoiding eccentric loads. For this purpose the geometry of the model was always kept under careful control. The configuration of the model was chosen to avoid the influence of lateral deformations of the brick portions emerging from the joined portion. The applied load (L) was measured and at the same time the vertical displacement of the central brick (\( \delta \)) was also monitored.

The 250 × 120 × 55 mm bricks used for the triplet test were characterized by a total open porosity of 43%, by a water absorption value after 24 hours of 19% and by an Initial Rate of Absorption value very close to 2 kg/m\(^2\)/min, which is the recommended value [Groot & Larbi 1999] for obtaining the maximum bond strength value.

Generally, only one joint was found cracked at the end of the test, so that the bond strength was calculated by dividing the maximum load by twice the fracture area where brick and mortar were in contact (approximately 120 × 200 mm); the test results are reported in Table 6.

A good mortar-brick adhesion depends mainly on the quality of the interfacial zone, which is indicated by higher bond strength values. In this regard, the recycled-aggregate mortars, in spite of poor mechanical behaviour (see Table 4), showed higher mortar-brick bond strength than the reference mortar (see Table 6), confirming previous results [Moriconi et al. 2003].
The presence of recycled material lowers the yield stress value as previously pointed out by means of rheological tests (cf. Table 5 and [Moriconi et al. 2003]); in this way the mortar could better permeate the brick surface assuring a physical interlock and, as a consequence, an improved bond.

In particular, the mortar prepared with type ‘REC’ aggregate shows a very high mortar-brick bond strength value (about three times that of reference mortar). On the basis of these results, the simultaneous presence of recycled concrete and brick seems to cause a synergistic effect.

Moreover, some tests carried out on masonry assemblies [Corinaldesi & Moriconi 2006] showed that their mechanical performance is mainly dependent on the bond strength between the mortar and brick and is generally improved when masonry is manufactured with recycled-aggregate mortars.

7 CONCLUSIONS

The experimental results show that mortars containing recycled aggregates developed a lower mechanical strength with respect to the reference cementitious mortar, particularly when the fraction based on recycled bricks was used.

Nevertheless, the bond strength at the interface between the mortar and the brick is higher for mortars prepared with recycled aggregates and, in particular, when the aggregate fraction coming from recycling of demolition waste is used.

For joint mortars, the mechanical performance of the overall mortar-brick system, strictly related to the mortar-brick adhesion, makes the mortar bond strength certainly more important than its mechanical strength.

In conclusion, the use of materials coming form C&D waste recycling instead of sand for the production of jointing mortars proved to be profitable not only for the obvious environmental advantages, but also in terms of improvement of the mortar-brick interface, which is generally recognized as the weak chain link of the masonry assemblage.

REFERENCES


Table 6. Results of the triplet tests.

<table>
<thead>
<tr>
<th>Mortar type</th>
<th>M-REF</th>
<th>M-REC</th>
<th>M-BRI</th>
<th>M-CON</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bond strength (MPa)</td>
<td>0.39</td>
<td>1.14</td>
<td>0.52</td>
<td>0.98</td>
</tr>
</tbody>
</table>
Researching recycled aggregate structural concrete for buildings

N. Morohashi & T. Sakurada

Department of Architecture and Architectural Engineering, College of Industrial Technology, Nihon University, Narashino, Chiba, Japan

ABSTRACT: This study used recycled aggregate concrete for floor slabs, which are plane members with a large surface area subject to drying, and added a drying shrinkage reducing agent to mitigate drying shrinkage. The relationship between use of this agent and the influence of drying shrinkage on cracking was investigated. The result found that, when a drying shrinkage reducing agent was added to recycled aggregate concrete-based floor slabs, not only were drying shrinkage cracks lessened but also post-yield strength against bending increased to give satisfactory bending properties. This suggests that recycled aggregate concrete could be used for members of reinforced concrete structures.

1 INTRODUCTION

In recent years in Japan, there has been a growing need for recycling materials in the building sector, particularly the chunks of concrete produced when reinforced concrete structures are demolished. This study reviews the possibility of using recycled aggregate concrete for members of reinforced concrete structures.

Previous studies reported that drying shrinkage cracks developing in reinforced concrete were caused by drying shrinkage due to fugitive emission of moisture from within the concrete [Architectural Institute of Japan 2003; Shrinkage Cracking in Reinforced Concrete Structures]. Tanano et al. [2006] reported in their research for wall members that more drying shrinkage cracks develop when recycled aggregate concrete is used in reinforced concrete than when normal concrete is used. This suggests that drying shrinkage cracks developing in recycled aggregate concrete may affect durability. Hence, this study reviews the effect of drying shrinkage on cracking, using recycled aggregate concrete for floor slabs, which are plane members whose large surface area is subject to airing, and then adding a drying shrinkage reducing agent to mitigate drying shrinkage. The effect of this agent on strength against bending of floor slabs as structural members is also examined.

2 SUMMARY OF EXPERIMENTS

2.1 Specimen

The mixing ratios of the concrete prepared in this study are shown in Table 1 and the series of floor slab concrete are shown in Table 2. Recycled aggregate concrete used in this research was grouped into two series: R series of 100% recycled coarse aggregate with 100% natural sand, and FR series of 100% recycled coarse aggregate with 100% recycled sand. The recycled aggregate use in this study was manufactured by crushing dismantled concrete with a jaw crusher at a recycled aggregate concrete plant. Each series was further reviewed for the dependence of the effect on whether or not a drying shrinkage reducing agent had been added. Floor slabs were tested for bending at the material age of 5 weeks, when no drying shrinkage cracks had yet developed, and also after one year had elapsed, during which drying shrinkage cracks developed. Figure 1 illustrates the shape of a specimen. Floor slab reinforcements were double reinforcement (double layred), with D10 (SD295A) used for the
main ones. To review the bonding force between recycled aggregate concrete and reinforcements, a lap splice of 20 db (200 mm) in lapped length was used for three bottom reinforcements and the remaining two were made through bar arrangement. With the coverage thickness of 30 mm both for top and bottom surfaces, concrete was flat-cast from the top. To identify the drying shrinkage of recycled aggregate concrete, 100 × 100 × 400 mm specimens for length changes used in the JIS-A-1129 method of testing for changes in concrete length were prepared together with the floor slabs for measuring the rate of drying shrinkage.

2.2 Drying shrinkage reducing agent

For this research, a glycol ether-based drying shrinkage reducing agent was added at the rate of 10 kg/m³ on site. After addition, the mixture was stirred at high speed for about 2 minutes. Regarding the properties of fresh concrete with drying shrinkage reducing agent added, it was found that both the air content and slump tended to be lower than those of concrete to which no such agent had been added.

3 TRANSITION OF CONCRETE STRENGTH AND YOUNG’S MODULUS

The transition of compressive strength is shown in Figure 2 and that of Young’s modulus is shown in Figure 3. Actual measurements of compressive strength and Young’s modulus for recycled aggregate concrete tended to increase with increasing material age. Recycled aggregate concrete with drying shrinkage reducing agent added showed both higher compressive strength and Young’s modulus than in the case without this agent. This is probably because air content decreased and made the concrete more dense. On the other hand, actual measurements of Young’s modulus tended to become lower than the values determined using a standard formula for the calculation of reinforced concrete structures [Architectural Institute of Japan 1999; AIJ Standard for Structural Calculation of Reinforced Concrete Structures].

4 RATE OF DRYING SHRINKAGE

Figure 4 shows the transition of the rate of drying shrinkage. The length-change specimens’ rates of

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Series/ the rate of substitution (%)</th>
<th>Age of flexural experiment</th>
<th>Drying shrinkage reducing agent</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) SR</td>
<td>SR series/ R</td>
<td>5 weeks</td>
<td>non addition addition</td>
</tr>
<tr>
<td>2) A-SR</td>
<td>recycled coarse aggregate 100</td>
<td>SRK</td>
<td>non addition addition</td>
</tr>
<tr>
<td>3) SRK</td>
<td>recycled sand 0</td>
<td>1 year</td>
<td>non addition addition</td>
</tr>
<tr>
<td>4) A-SRK</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5) SFR</td>
<td>FR series/ recycled coarse 100</td>
<td>SFR</td>
<td>non addition addition</td>
</tr>
<tr>
<td>6) A-SFR</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7) SFRK</td>
<td>aggregate 100 recycled sand 100</td>
<td>SFRK</td>
<td>non addition addition</td>
</tr>
<tr>
<td>8) A-SFRK</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

A: Drying shrinkage reducing agent, 10 kg/m³.

Table 2. Series of concrete of floor slab.
drying shrinkage sharply increased until a material age of 27 weeks to above the target limits [Architectural Institute of Japan 2003. JASS 5] of $800 \times 10^{-6}$ and stabilized thereafter. Recycled aggregate concrete with drying shrinkage reducing agent added tended to have lower rates of drying shrinkage than in the case without this agent. On the other hand, the rates of drying shrinkage of the floor slabs showed no increase until a material age of 3 weeks. This is probably because there was no fugitive emission of moisture from within the concrete, since they underwent wet-compress curing. The rates of drying shrinkage of the floor slabs tended to become lower than those of the specimens for length change. This is probably because the shrinkage of floor slabs was restricted by the arrangement of the reinforcement.

5 DRYING SHRINKAGE CRACKS ON FLOOR SLABS

5.1 Drying shrinkage cracks

Figure 5 shows the drying shrinkage cracks that had developed at one year. Drying shrinkage cracks were periodically observed by visually checking the top surface of the floor slabs. Drying shrinkage cracks started from the periphery of the floor slabs with their large surface area subject to airing and, as the material age advanced, extended inward. At one year, floor slabs to which no drying shrinkage reducing agent had been added showed many fine drying shrinkage cracks, whereas floor slabs with agent showed fewer drying shrinkage cracks. Thus, the smaller rate of drying shrinkage tended to cause fewer drying shrinkage cracks.

5.2 Width and depth of drying shrinkage cracks

Figure 6 shows the width and depth of the drying shrinkage cracks, which were measured after visually selecting representative cracks. Figure 6 a) shows that the width of drying shrinkage cracks in recycled aggregate concrete changed little up to one year. The depth of drying shrinkage cracks at one year was about 0.02 mm to 0.04 mm. Figure 6 b) shows that the depth of drying shrinkage cracks was 9.5 mm to 17.0 mm, so it can be assumed that none of the cracks had reached the reinforcements in the floor slabs.
6 BENDING PROPERTIES

6.1 Maximum flexural cracks at allowable unit stress for long-sustained loading

Table 7 lists the results of the flexural experiments. Figure 7 shows the maximum flexural crack width at allowable unit stress for long-sustained loading on the FR series which gave large rates of drying.

Flexural crack widths were determined by measuring flexural cracks which developed in the net-flexural floor slab section along bottom reinforcement lines under a microscope (graduated at 0.02 mm minimum). At one year, the maximum flexural crack width was within the target RC standard limit of 0.25 mm. Since crack widths on the bottom surface (tensile edge) of a plane member are subjected when considering its durability, the maximum flexural crack width on the distorted bottom surface is assumed to be larger. Floor slabs to which a drying shrinkage reducing agent had been added tended to suffer smaller maximum flexural crack widths than in the case without such agent. Maximum flexural crack widths at one year tended to be larger than those at 5 weeks.

6.2 Load-deflection curve

Figure 8 shows the load-deflection curve. The load was repeatedly applied in the positive direction only while gradually raising the main reinforcement stress intensity by fixed time each time. Post-member yield control of deflection was based on a yield deflection of \( \delta_y \). Strength against bending tended to be larger than that at 5 weeks. This is probably

<table>
<thead>
<tr>
<th>Specimens</th>
<th>( \sigma_B ) (N/mm(^2))</th>
<th>( P_y ) (kN)</th>
<th>( \delta_y ) (mm)</th>
<th>( P_{\text{max}} ) (kN)</th>
<th>( \delta_{\text{max}} ) (mm)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) SR</td>
<td>27.5</td>
<td>57.0</td>
<td>3.89</td>
<td>78.8</td>
<td>40.02</td>
<td>F</td>
</tr>
<tr>
<td>2) A-SR</td>
<td>35.2</td>
<td>66.9</td>
<td>3.61</td>
<td>97.4</td>
<td>40.00</td>
<td>F</td>
</tr>
<tr>
<td>3) SRK</td>
<td>32.8</td>
<td>59.7</td>
<td>4.25</td>
<td>89.6</td>
<td>38.67</td>
<td>F</td>
</tr>
<tr>
<td>4) A-SRK</td>
<td>40.3</td>
<td>64.9</td>
<td>4.02</td>
<td>95.4</td>
<td>40.11</td>
<td>F</td>
</tr>
<tr>
<td>5) SFR</td>
<td>30.3</td>
<td>52.5</td>
<td>4.03</td>
<td>79.6</td>
<td>40.00</td>
<td>F</td>
</tr>
<tr>
<td>6) A-SFR</td>
<td>33.6</td>
<td>59.8</td>
<td>2.88</td>
<td>74.0*</td>
<td>38.04</td>
<td>FS</td>
</tr>
<tr>
<td>7) SFRK</td>
<td>34.8</td>
<td>59.2</td>
<td>5.28</td>
<td>89.4</td>
<td>40.11</td>
<td>F</td>
</tr>
<tr>
<td>8) A-SFRK</td>
<td>38.3</td>
<td>59.4</td>
<td>3.85</td>
<td>90.1</td>
<td>24.08</td>
<td>FS</td>
</tr>
</tbody>
</table>

\( \sigma_B \): compressive strength.

F: flexural failure FS: bond splitting failure after yield.

* A-SFR: load of split failure 69.9(kN), \( \delta = 20.00 \text{(mm)} \).
because the compressive strength of concrete was larger at one year than at 5 weeks. Figure 9 shows load-deflection envelope curves. The rigidity of drying shrinkage reducing agent-added floor slabs before the material yielded was larger than in the case without such agent.

After the member yielded, the load increased with increasing deflection in all floor slabs, demonstrating satisfactory strength against bending, while in the case with a drying shrinkage reducing agent added to the FR series, A-SFRK suffered bond splitting failure.

### ACKNOWLEDGEMENTS

The authors would like to acknowledge the support of the Tokyo Disposal Association for Construction Waste and F Company of admixture manufacturer.

### REFERENCES

Architectural Institute of Japan 2003. Shrinkage Cracking in Reinforced Concrete Structures – Mechanisms and Practice of Crack Control


Architectural Institute of Japan revised 1999. AIJ Standard for Structural Calculation of Reinforced Concrete Structures – Based on Allowable Stress Concept

Recycling of concrete in precast concrete production

V. Corinaldesi & G. Moriconi
Università Politecnica delle Marche, Ancona, Italy

ABSTRACT: This work aimed at evaluating the influence of the use of recycled concrete aggregates on the mechanical performance of precast concrete. Recycled concrete was obtained as scraps of the precasting process itself and characterized by physical properties quite close to those of ordinary aggregates, due to the high strength class of the original concrete, which was in the range 50–55 MPa. Several substitution percentages were tried, between 5% and 30% by weight of the total amount of aggregate used for preparing concrete, in order to establish the maximum replacement percentage that could allow the same concrete strength class to be maintained. For this purpose, several concrete mixtures were prepared using rejected concretes coming from different precasting plants. Concrete performance was evaluated after 1, 7 and 28 days of wet curing, in terms of compressive strength. The results obtained showed that the quality of the old concrete influences the maximum substitution percentage that can be tolerated. When the original strength class is 55 MPa, the substitution can be carried out at a percentage of up to 30%. In addition, the drying shrinkage of two concrete mixtures prepared with 70% of one of these recycled concrete fractions was monitored for 100 days of curing at a relative humidity of 65% and a temperature of 15°C. The results obtained were encouraging, particularly when a shrinkage-reducing admixture was added to the concrete mixture in addition to a water-reducing admixture.

1 INTRODUCTION

At present, it is widely accepted that economic development must be sustainable, as far as minimisation of environmental impacts and safe-guarding of non-renewable resources is concerned. In particular, the opportunity of re-using solid waste materials as a replacement for those extracted from quarries in building construction is becoming more and more widespread [Bassan & Galimberti 1995, Bassan 1996, Corinaldesi & Moriconi 2002a, Corinaldesi et al. 2001, Moriconi et al. 2003]. This approach is consistent with the concept of solid waste, accepted in the European Community as well as in the individual member Countries, as a new resource that can be recycled in the production process and not simply disposed of in landfills.

The main objective of the research program is to demonstrate that scraps coming from the manufacture of precast, reinforced concrete (R/C) or prestressed concrete (P/C) elements, can be re-used as RAs to partially replace natural aggregates from quarries in the production of concrete for manufacturing new reinforced concrete elements with unchanged mechanical performance.

The goal would be remarkable from an environmental point of view, implying concrete life-cycle completion, as well as an economic achievement [Corinaldesi & Moriconi 2001].

In fact, the disposal of cementitious waste produced during manufacturing of precast concrete elements implies growing problems related to increasing transportation costs and increasing charges for their delivery to the landfill. On the other hand, their downcycling as backfill in road or pavement sub-bases, for example, even if excellent from a technical and economical point of view, can be a concern due to the potential leaching of hazardous substances derived from the cement, at least as far as Italian Standards are concerned [Carrescia et al. 2004].

On the other hand, the recycling of waste materials as aggregates for new concrete in the prefabrication process could be considered ideal. Moreover, since the waste material volume in prefabrication plants is relatively low, limited replacement of natural with recycled aggregate (RA) could be sufficient to manage all of the production scraps, at the same time gaining further environmental advantage, in terms of reduced extraction from quarries.

The use of nontraditional materials to manufacture concrete can affect the mechanical behavior as well as the durability of the final products, as suggested by the results of the research so far carried out [Corinaldesi

Recent research developments have highlighted two very important aspects that justify this work. The first one is related to the durability of RA concrete, which was shown to be higher than that of an ordinary concrete [Corinaldesi & Moriconi 2002b, Corinaldesi et al. 2002b]. This result can be ascribed to two factors: (i) the filling effect, and sometimes pozzolanic reaction, of the fine RA fractions and (ii) the improvement of the interface between RA and the surrounding cement paste, which is better the higher the concrete content of the RA. The second aspect is related to the mechanical strength loss of RA concrete, which is strongly dependent on the RA quality; in fact, it is completely cancelled out when RA consists of concrete belonging to a strength class equal to or higher than that of the new concrete in which they will be introduced [Coppola et al. 1995].

2 MATERIALS AND METHODS

2.1 Preparation and set up of the recycled-concrete aggregate fractions

The first step was cataloguing, based on the concrete original strength class and structural use reported in Table 1, the various grain size fractions of production scraps received from different prefabrication plants.

Each recycled-concrete fraction grading has been characterized as received, before laboratory crushing to a maximum grain size of 10 mm. The grain size distribution of each crushed fraction was then determined without any preliminary sieving. The results reported in Figure 1 show that a single crushing step is sufficient to achieve a standard grain size distribution, regardless of the original grain size distribution, even when this is almost monomodal with a relatively high maximum grain size, as shown in Figure 2.

Moreover, the crushing step appears necessary to achieve a grain size distribution consistent with optimal aggregate grading when crushed recycled-concrete fractions are combined with ordinary aggregate fractions.

Finally, the RA shape obtained after crushing was optimal for use in concrete manufacturing, according to Italian European Standards UNI EN 933-3 and UNI EN 933-4.

2.2 Chemical and physical characterization of the recycled-concrete aggregate

The physical properties of the recycled-concrete aggregate fractions were determined after laboratory grinding.

The water content of each fraction was determined by drying according to UNI EN 1097-5. The aggregates’ apparent densities and their water absorptions were then determined by following the methods in UNI EN 1097-6. The results obtained are reported in Table 2.

Alkali-silica reactivity tests were also carried out according to ASTM C289. No reactivity of the examined fractions, all characterized by a very low value of dissolved silica, could be observed.

2.3 Recycled-aggregate concrete mixtures

The influence of recycled-concrete aggregate on workability of fresh concrete and compressive strength of hardened concrete, was evaluated on twelve mixtures laboratory prepared by varying the RA origin and the natural aggregate replacement percentage (see Table 3).

The reference mixture (Ref, 0% replacement) was prepared by combining a natural sand (0–5 mm) and two crushed natural stone fractions (2–10 mm and 5–25 mm, respectively).

Test mixtures were prepared by using B, D, E, F and H recycled-concrete crushed fractions (characterized by grain size distributions within 0–12 mm, as shown in Figs. 1–2) by replacing the finer of the two natural crushed fractions (2–10 mm) at percentages varying from 5 to 30% by mass of the total aggregate amount.

Water to cement ratio (0.5) and fresh workability (fluid consistency) were kept constant for all the mixtures as well as the dosage of an acrylic-based superplasticizer equal to 1% by mass of cement.

2.4 Compression tests on recycled-aggregate concretes

Compressive strength tests were carried out on 100 mm cubic specimens after 1, 7 and 28 days room curing.
The results are reported in Figures 3, 4 and 5 respectively. Except the B fraction, with lower strength class and higher water absorption, the replacement of natural crushed aggregate with recycled concrete did not cause significant changes in the 1-day, 7-day and 28-day compressive strengths, independently of the RA origin and replacement percentage.

2.5 Drying shrinkage tests

In order to evaluate the behavior of concretes containing recycled concrete as aggregate in terms of drying shrinkage, two concrete mixtures were prepared with the same water to cement ratio of 0.45 and the same workability (170 mm slump, corresponding to fluid consistency), by varying the kind of chemical admixture.

Table 2. Main physical properties of the recycled-concrete aggregate.

<table>
<thead>
<tr>
<th>Fraction</th>
<th>Water absorption (%)</th>
<th>Specific gravity in s.s.d. (*) conditions (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>8.34</td>
<td>2.31</td>
</tr>
<tr>
<td>B</td>
<td>9.21</td>
<td>2.16</td>
</tr>
<tr>
<td>C</td>
<td>7.70</td>
<td>2.25</td>
</tr>
<tr>
<td>D</td>
<td>5.56</td>
<td>2.34</td>
</tr>
<tr>
<td>E</td>
<td>7.26</td>
<td>2.24</td>
</tr>
<tr>
<td>F</td>
<td>7.63</td>
<td>2.22</td>
</tr>
<tr>
<td>G</td>
<td>7.86</td>
<td>2.18</td>
</tr>
<tr>
<td>H</td>
<td>8.34</td>
<td>2.21</td>
</tr>
<tr>
<td>I</td>
<td>9.97</td>
<td>2.07</td>
</tr>
</tbody>
</table>

(*) s.s.d. = saturated surface-dried.
In Table 4 the mixtures proportions of these concretes are reported.

The mixture indicated as ‘H-70%’ was prepared with the same acrylic-based superplasticizer previously employed, at the dosage of 0.8% by mass of cement in order to achieve a 25% water reduction. This concrete was prepared by using the recycled-concrete aggregate type ‘H’ as coarse aggregate (see Table 1, Table 2 and Fig. 2). 70% by volume of this aggregate fraction was combined with 30% of natural sand in order to guarantee optimal grain size distribution of the whole inert fraction according to the Bolomey particle size distribution curve.

The ‘H-70% + SRA’ mixture was prepared by using a multifunction superplasticizer obtained by attaching a shrinkage reducing group (SRA) to the polycarboxylate polymer. In this way both water

---

### Table 3. Concrete mixture proportions (dosages expressed as kg/m³ of concrete).

<table>
<thead>
<tr>
<th>Mixture</th>
<th>W/C</th>
<th>Water</th>
<th>Cement</th>
<th>Superpl. Admix.</th>
<th>Natural sand (0–5 mm)</th>
<th>Natural fine crushed fraction (2–10 mm)</th>
<th>Natural coarse crushed fraction (5–25 mm)</th>
<th>Recycled-concrete crushed fraction (0–12 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ref</td>
<td>0.50</td>
<td>175</td>
<td>350</td>
<td>3.5</td>
<td>535</td>
<td>535</td>
<td>715</td>
<td>–</td>
</tr>
<tr>
<td>B-5%</td>
<td>0.50</td>
<td>175</td>
<td>350</td>
<td>3.5</td>
<td>535</td>
<td>89</td>
<td>715</td>
<td>82 (type B)</td>
</tr>
<tr>
<td>B-10%</td>
<td>0.50</td>
<td>175</td>
<td>350</td>
<td>3.5</td>
<td>535</td>
<td>178</td>
<td>715</td>
<td>165 (type B)</td>
</tr>
<tr>
<td>B-30%</td>
<td>0.50</td>
<td>175</td>
<td>350</td>
<td>3.5</td>
<td>535</td>
<td>–</td>
<td>715</td>
<td>490 (type B)</td>
</tr>
<tr>
<td>D-5%</td>
<td>0.50</td>
<td>175</td>
<td>350</td>
<td>3.5</td>
<td>535</td>
<td>89</td>
<td>715</td>
<td>88 (type D)</td>
</tr>
<tr>
<td>D-10%</td>
<td>0.50</td>
<td>175</td>
<td>350</td>
<td>3.5</td>
<td>535</td>
<td>178</td>
<td>715</td>
<td>177 (type D)</td>
</tr>
<tr>
<td>E-5%</td>
<td>0.50</td>
<td>175</td>
<td>350</td>
<td>3.5</td>
<td>535</td>
<td>89</td>
<td>715</td>
<td>85 (type E)</td>
</tr>
<tr>
<td>E-10%</td>
<td>0.50</td>
<td>175</td>
<td>350</td>
<td>3.5</td>
<td>535</td>
<td>178</td>
<td>715</td>
<td>170 (type E)</td>
</tr>
<tr>
<td>F-5%</td>
<td>0.50</td>
<td>175</td>
<td>350</td>
<td>3.5</td>
<td>535</td>
<td>89</td>
<td>715</td>
<td>85 (type F)</td>
</tr>
<tr>
<td>F-10%</td>
<td>0.50</td>
<td>175</td>
<td>350</td>
<td>3.5</td>
<td>535</td>
<td>178</td>
<td>715</td>
<td>170 (type F)</td>
</tr>
<tr>
<td>H-5%</td>
<td>0.50</td>
<td>175</td>
<td>350</td>
<td>3.5</td>
<td>535</td>
<td>89</td>
<td>715</td>
<td>83 (type H)</td>
</tr>
<tr>
<td>H-10%</td>
<td>0.50</td>
<td>175</td>
<td>350</td>
<td>3.5</td>
<td>535</td>
<td>178</td>
<td>715</td>
<td>165 (type H)</td>
</tr>
</tbody>
</table>

---

Figure 3. Compressive strength after 1 day of curing of recycled-aggregate concretes.
dosage and concrete shrinkage can be reduced. It was added at a dosage of 1.6% by mass of cement, in order to achieve the same water reduction (25%) obtained in the previous concrete mixture.

In both concretes, the aggregate fractions were previously water soaked. This operation is particularly important for the recycled concrete aggregate, which had a water absorption of 8.34%, as shown in Table 2.
Absorbed water can provide an internal water supply, thus reducing drying shrinkage of concrete.

After 28 days of curing, the compressive strength values, measured on 100 mm concrete cubes were 45 MPa and 49 MPa for the ‘H-70%’ and the ‘H-70% + SRA’ concrete respectively.

For the evaluation of concrete drying shrinkage, three prismatic specimens (100 by 100 by 500 mm) were cast for each concrete mixture, wet cured for 2 days at a temperature of 15°C, then cured at the same temperature and a relative humidity of 65% up to 100 days. The results obtained are reported in Fig. 6.

By comparing drying-shrinkage strains, the effectiveness of the superplasticizer incorporating a shrinkage reducing group is evident after 100 days: concrete shrinkage was decreased by 75%. The measured value of drying shrinkage was less than $-250\mu m/m$ after 100 days of testing. Therefore, if previously water soaked, the recycled-concrete aggregate seems to perform satisfactorily in terms of drying shrinkage.

3 CONCLUSIONS

Preliminary results obtained on mechanical performance of concrete made by replacing crushed natural aggregate with recycled concrete, discarded by prefabrication plants and re-used on the spot, confirm that making RA concrete for structural use is feasible and cheap. This practice could provide an adequate solution to environmental issues, such as increasingly limited availability of material from quarries and landfill space. Furthermore, this approach would make concrete a more sustainable material by lowering its environmental impact.
The results obtained showed that the quality of the old concrete influences the maximum substitution percentage which can be tolerated. When the original strength class is 55 MPa, the substitution can be carried out at a percentage of up to 30%.

In addition, the drying shrinkage behavior of concretes prepared with 70% of one of these recycled concrete fractions was good, particularly when a shrinkage-reducing admixture was added to the concrete mixture beyond that of simply a water-reducing type.

ACKNOWLEDGEMENTS

The writers acknowledge the grant of the ministry of University and Research of Italy (MIUR), as well as the contribution of the Association of Cement Producers (AITEC) and of the Precasting Companies Precompressi Centro-Nord, Generale Prefabbricati and RDB.

REFERENCES


Concrete paving products made with recycled demolition aggregates

K. Tang, M.N. Soutsos & S.G. Millard
Department of Engineering, University of Liverpool, Liverpool, UK

ABSTRACT: The potential for using construction and demolition waste (C&DW) derived aggregate in the manufacture of concrete paving blocks is being investigated at the University of Liverpool. Market research indicated that economic benefits might be achieved by using recycled demolition aggregates in precast concrete paving products. The mechanical properties, i.e., compressive and tensile splitting strength, of paving blocks made with recycled demolition aggregates were also investigated. Levels of aggregate replacement were determined that produced blocks with similar mechanical properties to those of blocks made using only natural aggregates, without requiring an increase in the cement content.

1 INTRODUCTION

In 1999, C&DW in the EU amounted to 450 million tonnes [Symonds, 1999], approximately 25% of all waste generated that year. Only 25% of this C&DW was recycled or re-used. In 2005, the UK produced 72.5 million tonnes of C&DW [EIONET, 2005] (including excavated soils) of which only 35% was recycled [Communities and local government, 2005]. During this time, 270 million tonnes of quarried aggregates were extracted [ODPM, 2006]. As a waste reduction measure, the UK Government aims to increase the use of recycled and secondary aggregates from 40 million tonnes up to 55 million tonnes by 2006 and up to 60 million tonnes by 2011 [Collins et al. 2005]. Recycled aggregates are currently used primarily for low-grade applications such as granular sub-base of road pavements. There is therefore a need to investigate a high value market for C&DW derived aggregates.

As precast concrete paving blocks are low-profit-margin products, block-making factories will be interested in C&DW derived aggregates if the price is lower than that of quarried aggregate. The price of natural aggregates (see Table 1) is likely to increase, especially after the introduction of the Aggregate Levy Tax. This will encourage companies to consider using recycled demolition aggregate which will almost certainly be cheaper. In areas such as North West England, where precast factories are situated close to urban areas and local natural aggregate resources are limited [Soutsos et al. 2004], use of recycled demolition aggregates may also reduce transportation costs.

Concrete paving blocks were selected for investigation for the following reasons:
- High usage. It is estimated that 100 square feet (= 9 m²) per person of paving blocks are installed annually in Europe [Concrete network, 2006].
- The risk of steel corrosion damage resulting from chloride contaminated C&DW will not be an issue because conventional paving blocks are not reinforced with steel.
- The production of concrete paving products is a manufacturing process where the supply and storage of materials is more easily managed than with concrete used for in-situ construction.

The use of crushed concrete- and masonry-derived aggregates to replace both coarse and fine natural aggregates in concrete building blocks has already been investigated at the University of Liverpool [Soutsos et al. 2004]. Building blocks using up to 60% replacement of quarried coarse aggregate with concrete-derived C&DW aggregate achieved a similar compressive strength to those made using 100%...
quarried limestone aggregate, without requiring an increase in the cement content. Poon and Chan studied the use of recycled crushed clay brick and recycled concrete aggregate in the manufacture of concrete paving blocks [Poon et al., 2006]. The paving blocks had a compressive strength between 25 and 63 MPa at 28 days, depending on the percentage of recycled material used. Both studies also showed a different detrimental influence on the mechanical properties depending on whether recycled concrete- or recycled masonry-derived aggregates were used. For this reason, recycled concrete and recycled masonry were investigated separately in this project.

2 AVAILABLE OF C&DW-DERIVED AGGREGATE

The market demand for C&DW-derived aggregates can be assured in an urban area such as Liverpool, where on-going replacement of infrastructure is associated with high costs of C&DW disposal. Currently, demolition of around 20,000 houses is planned in Merseyside as part of a ‘housing market renewal’ [Clover, 2005] scheme. As part of a regeneration plan to build a dual carriageway road into Liverpool city centre, hundreds of houses will be demolished by 2008 [BBC, 2006].

The concrete C&DW that was crushed to produce aggregate for this study came from a reinforced concrete tower block (see Fig. 1). The masonry C&DW came from the demolition of a row of council houses (see Fig. 2). Demolition waste material was transported and crushed at a local crushing plant. The crusher was unable to crush all of this material into the required size despite utilizing a ‘choke feed’ technique [Soutsos et al. 2004]. Even with this technique only 30% of the total demolition material supplied was crushed to the required coarse (6 mm) and fine (4 mm to dust) aggregate size.

3 MATERIALS & EXPERIMENTAL METHODS

3.1 Recycled aggregates used in this project

The use of recycled aggregate in designated concrete mixes is now permitted by BS 8500-2:2002 [BSI, 2002]. This is the first UK specification for the use of C&DW derived aggregates in concrete. The grading of 6 mm recycled aggregate from both concrete and masonry demolition waste was found to be similar to that of quarried limestone (see Fig. 3). However, recycled 4 mm-to-dust aggregate derived from masonry demolition waste was found to be significantly finer than natural medium graded sand. The converse was
found to be true for fine recycled aggregate derived from concrete demolition waste (see Fig. 3). Recycled aggregate from both concrete and masonry demolition wastes had a much higher water absorption and a lower density (see Table 2). The lower density and higher water absorption of C&DW-derived aggregates were expected to be problematic [Soutsos et al. 2004].

3.2 Preliminary trials – development of ‘vibro-compaction’ technique

The block-making industry uses a heavy vibrating compactor to compress the concrete into a block mould. In the first phase of the project, the factory procedure was replicated in the laboratory by using a vibration hammer to compact the concrete from above. However, preliminary trials using a range of water-cement ratios indicated that, despite using similar mix proportions to the factory, the target compressive and tensile splitting strengths of 49 MPa and 3.6 MPa, respectively, were not achieved. A study of the compressive strength results also indicated that for a water-cement ratio below 0.30, full compaction was not achieved (see Fig. 4). Another approach was investigated to improve the compaction, which involved increasing the compressive force and dispensing with vibration. A pressure of up to 15 N/mm² was applied to the fresh concrete to compact the specimens. The resulting low compressive strength indicated that compression alone was insufficient and that simultaneous compression and vibration was necessary to achieve the required compaction.

Efforts were then focused on modifying the existing frame used with the vibration hammer to enable compression and vibration from both above and below. A small metal table was converted to a vibrating table by attaching a clamp-on-vibrator.

This table was used in conjunction with the vibration hammer so that specimens could be vibrated above and below, while they were being compacted (see Fig. 5). The use of this combined vibro-compaction technique, together with the use of a superplasticiser, enabled the wet density of paving blocks to be increased from

![Figure 4. Compressive strength (7-day) vs water-cement ratio, using vibration compaction from above.](image)

![Figure 5. Combination of vibrating table and vibration hammer to cast concrete paving blocks.](image)

| Table 2. Relative density and water absorption (% of dry mass). |
|----------------------------------|-----------------|-----------------|-----------------|-----------------|
|                                  | Fine aggregate  | Coarse aggregate |
|----------------------------------|-----------------|-----------------|-----------------|-----------------|
| Relative density (Saturated and Surface Dry) | Sand | Concrete | Masonry | Limestone | Concrete | Masonry |
|                                  | 2.44            | 2.25            | 2.42            | 2.69            | 2.38            | 2.26            |
| Relative density (oven dry)      | 2.41            | 1.82            | 2.01            | 2.67            | 2.27            | 2.11            |
| Moisture Absorption              | 1.5             | 13.56           | 13.42           | 0.65            | 7.24            | 8.83            |
2230 kg/m³ (achieved by vibro-compaction only) to 2390 kg/m³. Compressive strengths greater than 49 MPa and tensile splitting strengths greater than 3.6 MPa were achieved at 28 days. The original texture of concrete paving blocks cast in the laboratory using the improved vibro-compaction technique compared well with that of paving blocks obtained from the factory (see Fig. 6).

4 THE EFFECT OF RECYCLED DEMOLITION AGGREGATE ON THE MECHANICAL PROPERTIES OF CONCRETE PAVING BLOCKS

4.1 Series I: Concrete-derived C&DW aggregate

After successfully replicating the industrial block-making procedure in the laboratory, the replacement of quarried limestone with concrete-derived C&DW aggregates was studied. The mix proportions of the control mix using only natural aggregates was converted to a volumetric ratio and natural aggregate was replaced by an equal volume of C&DW-derived aggregate. This volumetric replacement approach allows for the different density of recycled aggregates and enables production of blocks of the same physical size from different mixes (see Table 3 & 4). Each series of mixes commenced with an initial cement content of 230 kg/m³. This was incrementally raised to 380 kg/m³ to yield specimens with a range of cement contents but the same replacement aggregate ratio.

It was thought that the fine aggregate, i.e., 4 mm-to-dust, would have the most detrimental effect on compressive strength [Soutsos et al. 2004]. Studies were planned using a replacement of either coarse or fine aggregate alone, to quantify the relative effects of each. Concrete paving blocks use a high ratio of 4:1 fine/coarse aggregate in order to achieve a high-quality surface finish. It was found that a reduction in the water-cement (W/C) ratio was required to obtain a comparable compressive strength for a block using concrete-derived aggregate (see Fig. 7). A reduction

Figure 6. Cross-section texture: industry and laboratory cast specimens.

Table 3. Mix proportions (kg/m³) for concrete paving blocks – Replacement of coarse natural aggregate with masonry-derived coarse aggregate.

<table>
<thead>
<tr>
<th>Mix Type</th>
<th>Coarse aggregate (6 mm)</th>
<th>Fine aggregate (4 mm-to-dust)</th>
<th>Free W/C</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cement</td>
<td>Limestone</td>
<td>Masonry</td>
</tr>
<tr>
<td>Control (100% quarried aggregate)</td>
<td>230</td>
<td>380</td>
<td>1521</td>
</tr>
<tr>
<td></td>
<td>305</td>
<td>380</td>
<td>1521</td>
</tr>
<tr>
<td></td>
<td>380</td>
<td>380</td>
<td>1521</td>
</tr>
<tr>
<td>25% Coarse replaced</td>
<td>230</td>
<td>285</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>305</td>
<td>285</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>380</td>
<td>285</td>
<td>80</td>
</tr>
<tr>
<td>50% Coarse replaced</td>
<td>230</td>
<td>190</td>
<td>160</td>
</tr>
<tr>
<td></td>
<td>305</td>
<td>190</td>
<td>160</td>
</tr>
<tr>
<td></td>
<td>380</td>
<td>190</td>
<td>160</td>
</tr>
<tr>
<td>75% Coarse replaced</td>
<td>230</td>
<td>95</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td>305</td>
<td>95</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td>380</td>
<td>95</td>
<td>240</td>
</tr>
<tr>
<td>100% Coarse replaced</td>
<td>230</td>
<td>319</td>
<td>1520</td>
</tr>
<tr>
<td></td>
<td>305</td>
<td>319</td>
<td>1520</td>
</tr>
<tr>
<td></td>
<td>380</td>
<td>319</td>
<td>1520</td>
</tr>
</tbody>
</table>
Table 4. Mix proportions used for concrete paving blocks – Replacement of fine fraction of aggregates with concrete-derived aggregates.

<table>
<thead>
<tr>
<th>Mix Type</th>
<th>Cement</th>
<th>Coarse aggregate (6 mm)</th>
<th>Fine aggregate (4 mm-to-dust)</th>
<th>Free W/C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control (100% quarried aggregate)</td>
<td>230</td>
<td>380</td>
<td>1521</td>
<td>0.39</td>
</tr>
<tr>
<td></td>
<td>305</td>
<td>380</td>
<td>1521</td>
<td>0.36</td>
</tr>
<tr>
<td></td>
<td>380</td>
<td>380</td>
<td>1521</td>
<td>0.33</td>
</tr>
<tr>
<td>25% Fine replaced</td>
<td>230</td>
<td>380</td>
<td>1140</td>
<td>0.38</td>
</tr>
<tr>
<td></td>
<td>280</td>
<td>380</td>
<td>1140</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>330</td>
<td>380</td>
<td>1140</td>
<td>0.33</td>
</tr>
<tr>
<td></td>
<td>380</td>
<td>380</td>
<td>1140</td>
<td>0.31</td>
</tr>
<tr>
<td>50% Fine replaced</td>
<td>230</td>
<td>380</td>
<td>760</td>
<td>0.41</td>
</tr>
<tr>
<td></td>
<td>280</td>
<td>380</td>
<td>760</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td>330</td>
<td>380</td>
<td>760</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>380</td>
<td>380</td>
<td>760</td>
<td>0.33</td>
</tr>
<tr>
<td>75% Fine replaced</td>
<td>230</td>
<td>380</td>
<td>380</td>
<td>0.39</td>
</tr>
<tr>
<td></td>
<td>280</td>
<td>380</td>
<td>380</td>
<td>0.36</td>
</tr>
<tr>
<td></td>
<td>330</td>
<td>380</td>
<td>380</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td>380</td>
<td>380</td>
<td>380</td>
<td>0.33</td>
</tr>
<tr>
<td>100% Fine replaced</td>
<td>230</td>
<td>380</td>
<td>1402</td>
<td>0.42</td>
</tr>
<tr>
<td></td>
<td>280</td>
<td>380</td>
<td>1402</td>
<td>0.38</td>
</tr>
<tr>
<td></td>
<td>330</td>
<td>380</td>
<td>1402</td>
<td>0.36</td>
</tr>
<tr>
<td></td>
<td>380</td>
<td>380</td>
<td>1402</td>
<td>0.34</td>
</tr>
</tbody>
</table>

Figure 7. Effect of W/C on 7-day strength of blocks using different replacement levels (%) of concrete-derived fine aggregate.
Figure 8. Effect of replacement level (%) on strength of blocks using concrete-derived fine aggregate. (All mixes contained 380 kg/m³ of cement. The free water had to be increased with increasing replacement levels to maintain the same workability).

Figure 9. Effect of water cement ratio on 7-day strength of blocks using different replacement levels (%) of masonry-derived coarse aggregate.
in the water-cement ratio also necessitated an increase in the cement content to obtain a high-quality surface finish. It appears that the consistency of the mix is dependent on the free water content, i.e., the free water content needs to be constant for all water-cement ratios. This observation is consistent with the design of normal concrete mixes. The total water content will of course increase to account for the higher percentages of water absorption of the recycled demolition aggregate.

Fig. 8 shows that the detrimental effect on the compressive strength from replacing natural fine aggregates with concrete-derived C&DW aggregates is relatively small, with a reduction 10% in the 28-day strength at a replacement level of 75%. However, the resulting tensile strength showed considerable variability and hence a conservative recommendation of a maximum replacement level of 60% for fine concrete-derived C&DW aggregates is made. Further experimental work is being conducted to investigate the combined use of both coarse and fine concrete derived C&DW aggregates. It is to be hoped that these studies will confirm and further increase confidence in the above replacement level recommendations.

4.2 Series II: Masonry-derived C&DW aggregate

The replacement of newly quarried limestone aggregate with masonry-derived C&DW aggregate was investigated separately from concrete-derived aggregate.

It was again found that a lower W/C is needed if blocks made using masonry-derived C&DW aggregate are to have the same strength as blocks using quarried limestone. A lower W/C also necessitated an increase in cement content. The dependence of the mix consistency on the free W/C using masonry-derived C&DW aggregate was similar to that observed using concrete-derived aggregate (see Fig. 9). It was concluded that a replacement level of up to 60% for the coarse masonry derived aggregate could be recommended (see Fig. 10).

5 CONCLUSIONS

With the introduction of the Aggregate Levy Tax, economic benefits might be achieved from using recycled demolition aggregates in the manufacture of precast concrete paving blocks. Concrete paving blocks, with a replacement of up to 60% of coarse natural aggregates

![Figure 10. Effect of replacement level (%) on strength of blocks using masonry-derived coarse aggregate. (All mixes contained 380 kg/m³ of cement. The free water had to be increased with increasing replacement levels to maintain the same workability).](image-url)
with recycled masonry-derived aggregates, can still achieve equivalent strength without the need for increased cement content. It was also concluded that a replacement of 60% of the fine natural aggregate with concrete-derived aggregates could be recommended.

ACKNOWLEDGEMENTS

The authors wish to thank the Veolia Environmental Trust and the Flintshire Community Trust Ltd. (AD Waste Ltd.) for funding this project. The authors are also grateful to the following industrial collaborators for their assistance: Clean Merseyside Centre, Marshalls Ltd., Forticrete Ltd., Liverpool City Council, Liverpool Housing Action Trust, RMC Readymix Ltd., W.F. Doyle & Co. Ltd. and DSM Demolition Ltd.

REFERENCES

Using tires rubber waste for modification of concrete properties

G. Skripkiūnas, A. Grinys & M. Daukšys
Kaunas University of Technology, Building Materials and Structures Research Center, Department of Building Materials, Kaunas, Lithuania

ABSTRACT: Utilization of used tires or other vulcanized rubber waste is an important ecological problem for all countries. Rubber waste has long-term decay in natural conditions due to high durability for lot of climate or soil exposure and biological attacks. Used tires or other vulcanized rubber waste in most countries are burned up, but this has negative effect on environment. The aim of investigation was to study the possibilities to use granulated rubber waste as elastic additive for Portland-cement concrete.

Used tires or other vulcanized rubber waste were crumbed into different sizes fractions 0/0.5 mm, 0/1 mm and 0/3 mm. The rubber additive was used as fine aggregate replacement in concrete mixtures by 3.2 per cent of aggregates mass. Previous results indicate that using elastic additives from rubber waste increase freezing and thawing resistance of concrete. This additive works as damping agent reducing the stresses inside the concrete. The influence of different fractions granulated tires rubber waste on the properties of fresh and hardened concrete was tested in this investigation. The effect of this additive on technological properties, air content in fresh concrete, density, compressive strength, porosity, abrasive resistance and deformation properties of concrete was investigated.

The experimental results indicate the reduction of compressive strength and modulus of elasticity of concrete for all fractions of rubber waste. Rubber waste has no influence on capillary porosity of hardened concrete but increases air content and closed porosity of concrete and has positive effect on freezing and thawing resistance of concrete.

The results of investigation showed that crumbed rubber waste additive could be used for modification of Portland-cement concrete properties and utilization of used tires or other vulcanised rubber waste.
porosity parameters and freezing and thawing resistance of concrete were tested.

2 MATERIALS

Portland cement CEM I 42.5R was used for investigation. Water content for normal consistency cement slurry 24.5 per cent, specific area of cement particles 371 m²/kg. For fine aggregate was used sand fraction (fr.) 0/4 mm with bulk density 1560 kg/m³. In the investigation, part of sand was replaced by the rubber waste from used tires (3.2 per cent from aggregate by mass). For coarse aggregate was used crushed granite fr. 2/8 mm and 11/16 mm with bulk density 1480 kg/m³ and 1510 kg/m³ respectively. Coarse aggregate content in all concrete mixtures was the same. Plasticizing admixture based on polycarboxilie polymers was used with density of solution 1040 kg/m³. In all mixtures content of plasticizing admixture was the same – 0.6 per cent from cement content.

Mechanically crushed rubber waste from used tires was used. Rubber waste was classified to fractions 0/0.5; 0/1 and 0/3. The technological equipment for used tires rubber waste processing is installed in JSC “Metaloidas” Siauliai, Lithuania. The density of crushed rubber waste particles is 1020 kg/m³.

3 TEST METHODS

Cement and aggregates were dosed by weight while water and chemical additives were dosed by volume. Chemical additives in the form of solutions were mixed with water used in preparation of concrete mixtures. Concrete mixtures were mixed for 3 min. in the laboratory in forced type concrete mixer.

The consistency and density of concrete mixture were determined by EN 12350-2 [Standard 1999] and 12350-6 [Standard 1999b]. Concrete specimens – cubes 100 × 100 × 100 mm were cured in conditions according EN 12390-2 [Standard 2000c] and tested after 28 days. Density of concrete was determined by EN 12390-7 [Standard 2000d], compressive strength – by EN 12390-3 [Standard 2001e], abrasive resistance – by LST L 1428.15 [Standard 2006f], porosity of concrete – by water absorption kinetics [Standard 1978g] and freezing – thawing resistance of concrete – by volumetric freezing after immersing in 5 per cent NaCl solution [Standard 2005h].

In order to examine the effect of crumbed rubber waste on the characteristics of concrete mixture and hardened concrete four concrete mixtures were proportioned and mixed under laboratory conditions. Concrete mixtures with crumbled rubber particles of fraction 0/0.5, 0/1, 0/3 (3.2 per cent from aggregate by mass) and control mixture without rubber additives were tested. Proportions of concrete mixtures are presented in table 1.

4 RESULTS

4.1 Characteristics of mixtures

It was determined that rubber waste affects the properties of concrete mixture. In concrete mixtures containing rubber waste less value of density is obtained, however air content is higher in this concrete. The decrease of density of concrete mixture due to rubber waste applied may be explained by higher total air content in concrete mixtures. Lower density of concrete mixture is obtained due to less density of rubber waste particles compared with sand particles density.

It was determined that the slump of mixtures with fine rubber waste particles (fr. 0/0.5 and fr. 0/1) was decreased by 2–2.5 cm from the point of view of mixtures without rubber waste; while the slump of the mixture with coarse rubber waste (fr. 0/3) was 1 cm higher compared to the mixture without rubber additives (Fig. 2). Using fine rubber waste particles (fr. 0/0.5 and fr. 0/1), the decrease of the slump in concrete mixtures may be explained by more complicated surface texture and large specific surface of rubber waste particles than that of the control mixture with fine sand aggregates (0/4 fr.). Rubber particles of more complicated texture and large specific surface absorb larger amount of water than round particles of

<table>
<thead>
<tr>
<th>Rubber waste fraction</th>
<th>Rubber waste, kg</th>
<th>Cement, kg</th>
<th>Crushed granite 2/8 fraction, kg</th>
<th>Crushed granite 11/16 fraction, kg</th>
<th>Sand 0/4 fraction, kg</th>
<th>Superplastycizer, %</th>
<th>Water, l</th>
</tr>
</thead>
<tbody>
<tr>
<td>0/0.5</td>
<td>22.8</td>
<td>451</td>
<td>380</td>
<td>569</td>
<td>821</td>
<td>0.6</td>
<td>158</td>
</tr>
<tr>
<td>0/1</td>
<td>22.8</td>
<td>451</td>
<td>380</td>
<td>569</td>
<td>821</td>
<td>0.6</td>
<td>158</td>
</tr>
<tr>
<td>0/3</td>
<td>22.8</td>
<td>451</td>
<td>380</td>
<td>569</td>
<td>821</td>
<td>0.6</td>
<td>158</td>
</tr>
<tr>
<td>–</td>
<td>–</td>
<td>451</td>
<td>380</td>
<td>569</td>
<td>881</td>
<td>0.6</td>
<td>158</td>
</tr>
</tbody>
</table>

Table 1. Proportions of concrete mixtures.
fine aggregate sand (0/4 fr.). This results in the decrease in the slump characteristics of concrete mixtures. Using coarse fraction rubber waste (fr. 0/3) in mixtures, the increase in the slump from the point of view of control mixture was obtained due to the increased coarseness of fine aggregates.

Using fine fraction rubber additives (fr. 0/0.5 and fr. 0/1) air content in concrete mixture increases up to 2.6–2.8%, while using coarse fraction rubber additive (fr. 0/3) air content remains the same as that contained in concrete mixture without rubber additives (Fig. 3). As rubber waste is light and porous aggregates, their application results in increased air content in concrete mixtures.

4.2 Properties of hardened concrete

Dependence of concrete compressive strength on the coarseness of rubber additive is presented in Fig. 4. From this figure can see that concrete compressive strength decreases when rubber waste is added. Mostly (about 20 per cent) concrete compressive strength decreases when fr. 0/1 rubber additive is used; when fr. 0/0.5 and fr. 0/3 rubber additives are used concrete compressive strength decreases less (10–13 per cent). Accepted that air content of concrete with fr. 0/3 rubber additives is less than in other mixtures with rubber additives, it can be stated that increasing coarseness of rubber additives results in decreased compressive strength of concrete.

The abrasion resistance of concrete with different rubber waste particles fraction are presented in Fig. 5. As can be see, the coarseness of rubber waste additive has no effect on abrasion resistance of concrete.
4.3 Porosity and durability of concrete

Parameters of concrete with different rubber waste particles fraction porosity are presented in Fig. 6, Fig. 7, Fig. 8, Fig. 9.

Having added rubber waste into concrete no changes in the distribution of pores according to their size is observed (Fig. 6), while the average size of pores changes a little more (Fig. 7). Having added fr. 0/0.5 and fr. 0/3 of rubber additive the pores become slightly larger, while having added fr. 0/1 rubber additive the pores remain of the same size as those in concrete without rubber additives.

Rubber waste additive does not increase the open (capillary) porosity of concrete, and using fine rubber additive (fr. 0/0.5) it even reduces (Fig. 8). However, using rubber additives the close porosity is much higher than that of concrete without any rubber additives due to larger amount of air contained in such kind of concrete mixtures and close pores contained in rubber particles themselves (Fig. 9).

Concrete freezing and thawing resistance can be predicted using the criterion $K_F$. According to this criterion a concrete is considered to be frost resistant when the volume of closed pores exceeds the increase volume of water in concrete capillary pores during the water freezing [Sheikin et al. 1979]:

$$K_F = \frac{P_u}{0.09 \cdot P_o}$$  \hspace{1cm} (1)

where $P_u$ – closed porosity of concrete (air pores); $P_o$ – open porosity of concrete (capillary pores).

After calculation criterion $K_F$ according to closed and open porosity of concrete it was obtained that concrete with fine rubber waste additives (fr. 0/0.5 and 0/1 mm) has the highest value of this criterion. The value of this criterion is about 2.6–2.9 times higher than that of concrete without rubber additives. When concrete has coarse rubber waste additives (fr. 0/3), the frost resistant criterion is equal to 2.1 and is also higher than that of concrete without rubber additives, that have frost resistance criterion equal to 1.0.

The freezing and thawing resistance of concrete with fr. 0/0.5, fr. 0/1, 0/3 rubber waste additives and without any rubber waste additives was determined by volumetric freezing in the air and thawing in 5 per cent NaCl solution. After 610 freezing and thawing cycles, samples were checked visually (Fig. 10). No cracks and fragmentation was observed in samples with rubber additives, but in samples without rubber additives a network of cracks was observed in the
angles of concrete cubes which indicate the beginning of concrete fragmentation.

5 CONCLUSIONS

1. Fine rubber waste additives (fr. 0/0.5 and 0/1) increase the slump of concrete mixture, while coarse rubber waste additives (fr. 0/3) do not change the amount of air contained in concrete mixture.

2. Fine rubber additives (fr. 0/0.5 and 0/1) increase the air content in the concrete mixture; while coarse rubber additives (fr. 0/3) do not change the amount of air contained in concrete mixture.

3. The concrete strength is decreasing while the coarseness of rubber waste additives is increasing.

4. Rubber waste additives and their coarseness have no effect on concrete abrasion resistance characteristics.

5. According to porosity parameters the most significant resistance to freezing and thawing resistance predicted for concrete with fine rubber waste additives (fr. 0/0.5). Increasing coarseness of rubber additives cause the decrease in the resistance of concrete to freezing and thawing.

REFERENCES


89

Figure 10. Concrete samples with different fractions of rubber waste additives and control sample after 610 cycles of freezing and thawing.
GOST 12730.4-78 standard. 1978g. Concretes. Methods of determination of porosity parameters.
Employment of waste sand to compose fibre reinforced cement composites

J. Katzer
Technical University of Koszalin, Poland

ABSTRACT: This lecture describes the research on concrete based on waste sand obtained from hydrograding of all-in-aggregate in the Central Pomerania in Poland. Concrete was modified by replacing from 0 to 25% of cement for the silica fume and adding from 0 to 2% of steel fibres. The received results of the research on the fresh concrete mixtures and properties of hardened composite are presented on charts. The author noted a considerable improvement in compression strength and watertightness of the examined concrete mixes.

1 INTRODUCTION

Natural aggregates in Central Pomerania in Poland are of glacial origin and they occur in a form of sands and all-in-aggregates. In the region there are 38 documented and described pit deposits (Katzer 2003, Katzer & Kobaka 2005a). The main mineral component of Central Pomerania aggregate is quartz and crystalline rock, dominated by granite. In the Figure 1 you can see a photograph of magnified Pomeranian sand. White grains are quartz, black grains are granite. The majority of grains have an ellipsoidal shape. The rest of grains have a spherical or flaky shape. As far as smoothness of the grain surface is concerned fine aggregate is composed of angular and partially subrounded grains. Grading of sands and all-in-aggregate of Central Pomerania described by Katzer (2003) is summarized in Table 1.

In the mentioned pit deposits, fractions from 0 to 4.0 mm constitute 97% by weight of all aggregate. Documented deposits of aggregate in this region reach over 100 billion of kilograms. Approximately half of it is constituted by deposits hydrograded during the exploitation. Waste sand is a by-product of hydrograding natural all-in-aggregate. During the process of hydrograding, all-in-aggregate is divided into gravel and sand. There is a deficit of gravel in the region, therefore gravel received during hydrograding of all-in-aggregate is constantly being sold. Sand received during the process of hydrograding, due to its excessive amount, is called waste sand. This sand, because of the excess of fine aggregate in the Central Pomerania region, is stored on continuously growing wasteheaps close to the aggregate pits. Figures 2 and 3 show the constantly growing wasteheaps of waste sand near the aggregate pit in Sepolno Wielkie located near the city of Bialy Bor in the Central Pomerania region.

A bar chart of grading of aggregate in Central Pomerania is presented in Figure 4. There are three kinds of aggregate available in the region: natural sand, waste sand and all-in-aggregate. As it is shown in Figure 4, in comparison to natural sand, waste sand is characterized by considerably higher grain-size distribution, a smaller amount of stone dust and a higher content of minerals and crystal rocks than the natural sand obtained from the same aggregate pit, which is a significant advantage from the point of view of designing and future cement composites production. Apart from that, this sand has lower factors of graining changeability and a higher content of crystal rocks than all-in-aggregate out of which it was sifted. Waste

Figure 1. Waste sand.
sand sifted during the process of hydrograding is washed and as a result deprived of clay and other fine material. It has a great significance because specific surface of aggregate is strongly related to the amount of fine particles present in it. Waste sand is also free from other deleterious substances such as salt contamination, organic impurities and unsound particles, usually present in natural aggregate. For the last 25 years in Central Pomerania there have been several research programmes concerning designing, production technology and technical parameters of concrete based on the waste sand (Dondelewski & Maciek 1995, Maciek et al. 1978, Maciek & Dondelewski 1980, Maciek & Dondelewski 1981, Marcinkowski & Dondelewski 1980, Marcinkowski & Dondelewski 1984). These research programmes concerned concretes produced in the same way as ordinary concrete without any admixtures or additives. It has been established that concrete based on waste sand is characterized (in comparison to ordinary concrete) by relatively high cavity, porosity and absorption, law resistance to local mechanical damage, abrasion and acid attack. It is also less frost and water resistant.

The grading characteristics of aggregate directly influences parameters of the concrete based on it. Using fine aggregate as a main component of concrete affects its behaviour. The big surface area of the fine aggregate determines the amount of water necessary

---

Table 1. Sands and all-in aggregates with their known graining parameters.

<table>
<thead>
<tr>
<th>Ordinal number</th>
<th>Name of deposit</th>
<th>0.0 mm–0.5 mm [%]</th>
<th>0.5 mm–2.0 mm [%]</th>
<th>2.0 mm–32.0 mm [%]</th>
<th>Hummel fineness modulus</th>
<th>Specific surface [m²/kg]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Brzezno</td>
<td>84.6</td>
<td>14.1</td>
<td>1.3</td>
<td>54.4</td>
<td>10.295</td>
</tr>
<tr>
<td>2</td>
<td>Jawory</td>
<td>25.9</td>
<td>46.9</td>
<td>6.6</td>
<td>79.1</td>
<td>6.423</td>
</tr>
<tr>
<td>3</td>
<td>Kalisz Pom.</td>
<td>59.1</td>
<td>39.8</td>
<td>10.0</td>
<td>74.8</td>
<td>8.022</td>
</tr>
<tr>
<td>4</td>
<td>Klepino</td>
<td>55.1</td>
<td>31.6</td>
<td>13.3</td>
<td>81.3</td>
<td>32.094</td>
</tr>
<tr>
<td>5</td>
<td>Piekarz</td>
<td>58.5</td>
<td>37.5</td>
<td>4.0</td>
<td>69.1</td>
<td>8.096</td>
</tr>
<tr>
<td>6</td>
<td>Pniewo</td>
<td>89.5</td>
<td>10.1</td>
<td>0.4</td>
<td>49.2</td>
<td>11.652</td>
</tr>
<tr>
<td>7</td>
<td>Siánow</td>
<td>39.8</td>
<td>54.2</td>
<td>6.0</td>
<td>79.8</td>
<td>6.437</td>
</tr>
<tr>
<td>8</td>
<td>Siemianowice</td>
<td>67.7</td>
<td>29.5</td>
<td>2.8</td>
<td>64.4</td>
<td>8.600</td>
</tr>
<tr>
<td>9</td>
<td>Świecianowo</td>
<td>22.8</td>
<td>38.5</td>
<td>2.3</td>
<td>70.6</td>
<td>7.044</td>
</tr>
<tr>
<td>10</td>
<td>Świdwinek</td>
<td>95.5</td>
<td>4.5</td>
<td>0.0</td>
<td>28.5</td>
<td>20.551</td>
</tr>
<tr>
<td>11</td>
<td>Welnianka</td>
<td>85.6</td>
<td>12.7</td>
<td>1.8</td>
<td>49.0</td>
<td>12.412</td>
</tr>
<tr>
<td>12</td>
<td>Węgorzewo</td>
<td>49.5</td>
<td>46.2</td>
<td>4.3</td>
<td>74.9</td>
<td>7.221</td>
</tr>
<tr>
<td>13</td>
<td>JastrowieI</td>
<td>49.8</td>
<td>49.0</td>
<td>1.2</td>
<td>71.7</td>
<td>6.975</td>
</tr>
<tr>
<td>14</td>
<td>Mielniko</td>
<td>58.3</td>
<td>40.9</td>
<td>0.8</td>
<td>67.9</td>
<td>7.684</td>
</tr>
<tr>
<td>15</td>
<td>Parzeczko</td>
<td>52.4</td>
<td>46.6</td>
<td>1.0</td>
<td>69.6</td>
<td>7.561</td>
</tr>
<tr>
<td>16</td>
<td>Sępólno</td>
<td>24.3</td>
<td>65.9</td>
<td>8.9</td>
<td>92.4</td>
<td>4.656</td>
</tr>
<tr>
<td>17</td>
<td>Bobolicek</td>
<td>33.2</td>
<td>55.6</td>
<td>11.2</td>
<td>88.1</td>
<td>5.565</td>
</tr>
<tr>
<td>18</td>
<td>Chlebowo</td>
<td>46.2</td>
<td>39.6</td>
<td>14.2</td>
<td>80.2</td>
<td>7.755</td>
</tr>
<tr>
<td>19</td>
<td>Cieszeniewo</td>
<td>34.1</td>
<td>54.8</td>
<td>11.1</td>
<td>86.0</td>
<td>5.860</td>
</tr>
<tr>
<td>20</td>
<td>Golce</td>
<td>43.1</td>
<td>53.7</td>
<td>3.3</td>
<td>76.0</td>
<td>6.718</td>
</tr>
<tr>
<td>21</td>
<td>JastrowieII</td>
<td>24.1</td>
<td>56.2</td>
<td>19.6</td>
<td>98.6</td>
<td>4.653</td>
</tr>
<tr>
<td>22</td>
<td>Karso</td>
<td>17.0</td>
<td>76.5</td>
<td>6.5</td>
<td>84.5</td>
<td>4.376</td>
</tr>
<tr>
<td>23</td>
<td>Kuznica Czarn.</td>
<td>47.9</td>
<td>40.8</td>
<td>11.3</td>
<td>82.5</td>
<td>6.658</td>
</tr>
<tr>
<td>24</td>
<td>Ledyczek</td>
<td>34.8</td>
<td>53.5</td>
<td>11.7</td>
<td>85.8</td>
<td>6.290</td>
</tr>
<tr>
<td>25</td>
<td>Mielec</td>
<td>41.4</td>
<td>45.8</td>
<td>12.8</td>
<td>83.7</td>
<td>6.650</td>
</tr>
<tr>
<td>26</td>
<td>Nowogarddek</td>
<td>39.8</td>
<td>50.6</td>
<td>9.6</td>
<td>81.8</td>
<td>6.187</td>
</tr>
<tr>
<td>27</td>
<td>Osowo</td>
<td>27.1</td>
<td>58.6</td>
<td>14.3</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>28</td>
<td>Polanow</td>
<td>23.8</td>
<td>57.9</td>
<td>18.4</td>
<td>98.1</td>
<td>4.826</td>
</tr>
<tr>
<td>29</td>
<td>Babino</td>
<td>47.5</td>
<td>46.6</td>
<td>5.9</td>
<td>77.5</td>
<td>6.170</td>
</tr>
<tr>
<td>30</td>
<td>Ratajki</td>
<td>12.6</td>
<td>71.8</td>
<td>15.5</td>
<td>104.4</td>
<td>3.752</td>
</tr>
<tr>
<td>31</td>
<td>Rzeszynkowo</td>
<td>26.0</td>
<td>63.7</td>
<td>10.2</td>
<td>91.6</td>
<td>5.109</td>
</tr>
<tr>
<td>32</td>
<td>Siecino</td>
<td>45.3</td>
<td>39.0</td>
<td>15.7</td>
<td>82.1</td>
<td>7.492</td>
</tr>
<tr>
<td>33</td>
<td>Spore</td>
<td>55.5</td>
<td>38.8</td>
<td>5.3</td>
<td>72.6</td>
<td>7.410</td>
</tr>
<tr>
<td>34</td>
<td>Siemianowice</td>
<td>23.5</td>
<td>64.8</td>
<td>11.7</td>
<td>93.7</td>
<td>4.578</td>
</tr>
<tr>
<td>35</td>
<td>Stary Chwalim</td>
<td>29.2</td>
<td>55.1</td>
<td>15.7</td>
<td>92.1</td>
<td>5.423</td>
</tr>
<tr>
<td>36</td>
<td>Walkowice</td>
<td>16.1</td>
<td>70.4</td>
<td>13.5</td>
<td>99.0</td>
<td>4.533</td>
</tr>
<tr>
<td>37</td>
<td>Warblewo</td>
<td>19.6</td>
<td>54.0</td>
<td>26.4</td>
<td>104.2</td>
<td>4.308</td>
</tr>
<tr>
<td>38</td>
<td>Zelkowo</td>
<td>20.0</td>
<td>70.5</td>
<td>9.6</td>
<td>94.9</td>
<td>4.621</td>
</tr>
</tbody>
</table>

Total 38
to wet all the solids. More water is needed (comparing to ordinary concrete) to maintain the same workability of the fresh concrete mix what was stated in previous publications by Brzezicki (1997) and Hudson (1998). Despite the limitations mentioned above, for the last 25 years there have been numerous attempts to produce different precast elements from concrete based on waste sand. The main reason for these attempts is the general availability of waste sand in Central Pomerania, its very low price and short distances for the transportation of the aggregate. Numerous civil engineering applications of fine aggregate concretes presented for example by Katzer & Kobaka (2005a) have proved that meticulously made fine aggregate concrete is characterized by satisfactory strength and durability in order to be applied in civil engineering as a standard construction material. Today fine aggregates find more and more applications in the production of different kinds of cement composites such as steel fibre reinforced concretes (SFRC), ferrocement and precast elements with dense reinforcement. This issue was widely described and discussed by many researchers e.g. Johnston (2001), Maidl (1995), Nawy (1996) and Neville (1995).

The facts mentioned above forced the author to prepare a research programme of concretes based on the waste sand and modified by silica fume (SF) and steel fibre (Vf) to increase their durability and strength characteristics.

### 2 RESEARCH PROGRAMME

Fine aggregate concrete was composed of Portland Cement 32.5 (CEM I, 400 kg/m³) and waste sand. Properties of the used waste sand are summarized in Table 2. The water to cement ratio (w/c) was equal to 0.50. The examinations were carried out with the help of an experimental design. The author used the central composite experimental design based on Plackett-Burman design thoroughly described by Hicks (1982), Mann (1950) and Schenck (1979). The subject of the research was defined as complex material whose interior structure was unknown and unavailable for an observer. The observer knew the input parameters which were the addition of steel fiber (Vf), and silica.
fume (SF). The output parameters were strength and other properties of concrete. The applied experimental design is presented in Figure 5 and in Table 3. The objectivity of the experiments was assured by the choice of the sequence of the realization of specific experiments from a table of random numbers. All calculations connected with specifying a correlation coefficient and graphic interpretation of the model were carried using a statistical computer programme widely described by Bayramov (2004), Borovikov & Borovikov (1998) and Lee-Ing & Chung-Ho (2002). Polynomial fit was used to achieve contour plots. Fitted functions were characterized by a correlation coefficient equal to at least 0.9.

Eleven concrete mixes were modified by the addition of silica fume (from 0% to 25%), and reinforced by steel fibre (from 0% to 2%). A photograph of magnified raw silica fume used to the tests is presented in Figure 6.

Table 3. Experimental design values of input factor.

<table>
<thead>
<tr>
<th>Number of experiment</th>
<th>Number of realization</th>
<th>$x_1$ in code values</th>
<th>$x_2$ in code values</th>
<th>SF [%]</th>
<th>Vf [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>-1.000</td>
<td>-1.000</td>
<td>5.0</td>
<td>0.4</td>
</tr>
<tr>
<td>2</td>
<td>5</td>
<td>-1.000</td>
<td>-1.000</td>
<td>20.0</td>
<td>0.4</td>
</tr>
<tr>
<td>3</td>
<td>9</td>
<td>1.000</td>
<td>-1.000</td>
<td>5.0</td>
<td>1.6</td>
</tr>
<tr>
<td>4</td>
<td>8</td>
<td>1.000</td>
<td>-1.000</td>
<td>20.0</td>
<td>1.6</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>-1.682</td>
<td>0.000</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>6</td>
<td>9</td>
<td>1.682</td>
<td>0.000</td>
<td>25.0</td>
<td>1.0</td>
</tr>
<tr>
<td>7</td>
<td>3</td>
<td>0.000</td>
<td>-1.682</td>
<td>12.5</td>
<td>0.0</td>
</tr>
<tr>
<td>8</td>
<td>4</td>
<td>0.000</td>
<td>1.682</td>
<td>12.5</td>
<td>2.0</td>
</tr>
<tr>
<td>9,10,11</td>
<td>7,6,11</td>
<td>0.000</td>
<td>0.000</td>
<td>12.5</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Eleven concrete mixes were modified by the addition of silica fume (from 0% to 25%), and reinforced by steel fibre (from 0% to 2%). A photograph of magnified raw silica fume used to the tests is presented in Figure 6.

Figure 5. The scheme of the arrangement of measuring points.

Figure 6. Raw silica fume used to the tests.

Figure 7. The geometry of the employed steel fibre in mm ($a = 5$ mm, $b = 3$ mm).
fine aggregate and modified by silica fume (Katzer & Piatek 1999).

The specimens were in a form of cubes and beams. From each concrete mix 30 cube specimens 150·150·150 mm, 6 cylinder specimens φ150·500 mm and 6 beam specimens 100·100·500 mm were made. The research programme covered the examinations of features of the concrete samples after 120 days of curing because of high silica fume reactivity in time. This reactivity has a significant influence on concrete parameters examined after 28 days of curing.

The research programme was divided into three main stages. The first stage covered the testing of properties of fresh concrete mix. The second stage covered the testing of compressive strength, flexural strength, deflection of the beams and establishing stress-strain curves. The third stage covered testing of watertightness after different cycles of compressive load.

The examination of watertightness of concrete samples does not reflect watertightness of concrete composite in an actual construction. Each, even the simplest construction works within constantly changing conditions, where loads appear and disappear in time. The influence of the changeability of static loads in time on concrete parameters has already been well known, especially as far as ultimate compressive strength is concerned (Lenkiewicz & Pidek 1970). Keeping the above facts in mind, the author has worked out a watertightness test procedure after a cycle of loads. Specimens from each concrete mix were divided into three independent groups, which were subjected to different cycles of compressive load.

The cycle no. 1 was an eightfold load until the moment of causing stress of 45% of breaking stress and unload of samples. The samples were loaded with an increasing force at a uniform rate (the speed of growing of stress 0.5 ± 0.1 MPa/s). In these conditions the achieved pulse frequency was 1/35 Hz. The course of the first cycle of load was presented in Figure 8.

The cycle no. 2 was a single load of the samples until the moment of causing stress of 90% of breaking stress and unload of the samples.

The cycle no. 3 was a comparative cycle therefore there was no pre-load of the samples. These samples were treated as control samples. After the cycle of load, samples were dried in temperature +60°C.

Drying the samples was to eliminate the influence of absorption of vapour from air by samples during their curing on watertightness examination. Then samples were subjected to the watertightness evaluating. Watertightness of fine aggregate concrete was tested using an apparatus which supplies water from below and can test six samples under a constant pressure of 0.2 MPa to 1.2 MPa simultaneously. The used apparatus is shown in Figure 9 and widely described by Katzer (2006).

The test was carried out under constant pressure of 1.2 MPa within 72 hours. After this period of time samples were split and the depth of water penetration was measured. The results were presented with the use of a leaking speed parameter \( k_v \) which was described by Katzer (2004) and counted according to the Equation (1):

\[
k_v = \frac{x_{\text{max}}}{(2 \sum h_i t_i)}
\]

where \( x_{\text{max}} \) = maximum depth of water penetration in meters, \( h_i \) = pressure of water in meters H₂O, \( t_i \) = time of lasting of the pressure in seconds.

The expression of watertightness with the use of parameter \( k_v \) allows to compare watertightness of both samples which were thoroughly penetrated by water during the experiment and the ones which remained tight until the completion of it. One may also compare
the watertightness of samples examined with varied methods.

To realize the flexural strength examination under four point loading, two concentrated loads were applied on the samples in spacing 1/3 of the span. The scheme of four point loading of the concrete beam is presented in Figure 10. The flexural strength examinations were carried out on specimens 100·100·500 mm. The establishing stress-strain curves was carried out on specimens φ150·500 mm with axial compression. The examination was finished after having achieved about 0.9 failure load.

3 RESULTS OF THE RESEARCH

Consistency of fresh concrete mixture determined with the help of Vebe procedure varied from 7 to 11 seconds. The maximum amount of air in analyzed fresh concrete mixtures was equal to 4%. Most of the analyzed mixtures was characterized by amount of air from 2.5% to 3%. Density of fresh concrete mixture modified by silica fume and steel fibre is described by the Equation 2.

$$\rho_{\text{vm}} = 2064.21 + 4.98x + 51.31y - 0.12x^2 - 0.49xy + 0.41y^2$$  \hspace{1cm} (2)

where \(x = \text{SF} \,[\%]\), \(y = \text{Vf} \,[\%]\).

Density of fresh concrete mixture increase together with the increase of steel fibre addition. Plain concrete is characterized by density equal to 2064 kg/m\(^3\), and concrete modified by 2% of steel fibre is characterized by density equal to 2200 kg/m\(^3\). Addition of silica fume does not seem to influence this property of fresh concrete mixture.

The results of the ultimate compressive strength examination are shown in Figure 11. Both silica fume and steel fibre cause the growth in ultimate compressive strength, and both of their influences accumulate. The growth of ultimate compressive strength appears mainly because of the addition of steel fibre. Above a certain optimum amount of the steel fibre addition (about 1.5%) ultimate compressive strength of the examined composites slightly decreases.

The results of flexural strength examinations are presented in Figure 12. The shape of the presented surface shows a big influence of the steel fibre addition on acquiring quasi-plastic features by the composite. Concrete mix without any silica fume admixture has the biggest flexural strength. During the flexural strength examination deflection was measured at the imposed force \(F = 2.25 \text{kN}\). The results of the deflection examination are shown in Figure 13.
Figure 14 shows the interdependence $\sigma$-$\varepsilon$ for chosen composites.

Watertightness of fine aggregate concrete composites subjected to cycles of pre-load no. 1 is shown in Figure 15. Watertightness of the described composites increases both in relation to the addition of SF as well as of $V_f$. The increases of watertightness are at a uniform rate in both directions and they are close to a linear relation. A comparative concrete mix has $k_V = 340 \cdot 10^{-12}$ m/s and the most tight of the maximum additions SF and $V_f$ $k_V = 120 \cdot 10^{-12}$ m/s. The equation (3) describes the surface pictured in Figure 15.

$$k_{V\text{c}} = 341.63 - 8.65x + 0.06x^2 - 51.09y - 9.41y^2 + 1.82xy$$

(3)

Watertightness of fine aggregate concrete composites subjected to the cycles of pre-load no. 2 is shown in Figure 16. Watertightness increases mainly with regard to the increasing addition of $V_f$. The addition of SF influences the improvement of watertightness of the described composites but its significance is very limited in this case. The non-modified concrete mix has $k_{V} = 3985 \cdot 10^{-12}$ m/s, and the concrete mix with maximum addition of SF and $V_f$ was characterized by $k_{V} = 680 \cdot 10^{-12}$ m/s. The equation (4) describes the surface pictured in Figure 16.

$$k_{V\text{c}} = 3985 - 39.7x + 0.8x^2 - 1542.2y + 102.6y^2 - 2.9xy$$

(4)

Watertightness of fine aggregate concrete composites which were not subjected to any preload is shown in Figure 17. Watertightness of concrete without any additions expressed with the use of parameter equals 230 $\cdot 10^{-12}$ m/s. Modifying the concrete mix solely with steel fibre reinforcement allowed to achieve watertightness of 103 $\cdot 10^{-12}$ m/s. Modifying the concrete mix solely with silica fume allowed to achieve 39 $\cdot 10^{-12}$ m/s. It is visible in Figure 17 that the addition of reinforcement worsens the tightness attained with silica fume when its quantity exceeds 10%. With the increase of the amount of silica fume the tightness of the fine aggregate concrete composite grows, especially in concrete mixes of a high content of reinforcement. With the addition of more than 15% of silica fume all concrete mixtures are characterized by considerable tightness, which stabilizes at a level of 25 $\cdot 23 \cdot 10^{-12}$ m/s.
4 DISCUSSION

Ultimate compressive strength (which was 24 MPa for plain concrete) grew by over 100% (to 50 MPa) for the optimal concrete mixture. All concretes were made of Portland Cement 32.5 (400 kg/m³) without admixtures. It proves that there are yet other possibilities of achieving significantly bigger strengths. For this feature the combination of the two additives in the cured composite is very effective.

The results of flexural strength examinations disclose the effectiveness of steel fibre addition behaviour with this kind of loading. Plain concrete mix obtains the strength 2.75 MPa, and concrete reinforced by 2% of steel fibre equals 6 MPa. Despite of the passing 120 days of curing, silica fume additive worsens significantly flexural strength obtained thanks to the steel fibre itself. Despite this negative tendency concrete mixes of maximum 15% of silica fume and from 1.5% to 2% of steel fibre additive are characterised by much higher strength than plain mixture.

The deflection measurement, similar to the flexural strength described above, shows the acquirement of quasi-features by SFRC composite. The deflection of composite modified by 25% of silica fume and 2% of steel fibre is five times smaller than the deflection of composites modified by silica fume alone.

Watertightness of fine aggregate fibre reinforced composites without any preload reaches the level from $k_V = 23$ to $230 \cdot 10^{-12}$ m/s. Composites which were subjected to the pre-load cycle of load no. 1 are characterised by watertightness from $k_V = 120$ to $340 \cdot 10^{-12}$ m/s, and composites subjected to the preload cycle of load no. 2 from $k_V = 680$ to $3985 \cdot 10^{-12}$ m/s. If one assumes the watertightness of composites without load as a point of reference then the relation of watertightness of not modified fine aggregate concrete, loaded with the cycle of load no. 1 and no. 2 is 1:5:29. A similar relation for a composite modified with a maximum addition of SF and $V_f$ is 1:1.5:1.7. It is observable in the above proportions how significant is the influence of the addition of steel fibre and the kind of preload onto watertightness of fine aggregate concrete composites.

There is an increasing influence of the amount of additions of steel fibres on sustaining tightness of fine aggregate concrete composites with the increase in preload. The addition of silica fume so much tightening the structure of concretes subjected to no preload loses its primary meaning with the increasing load. In Figure 17 one may see that watertightness of fine aggregate concretes depends mainly on the amount of addition of SF. In Figure 15, after the preload causing 45% of breaking stress Watertightness in a relative extent depends on the amount of the addition of SF and $V_f$. With load causing 90% of breaking stress Watertightness of fine aggregate concrete composites already significantly depends on the amount of the addition of $V_f$.

Concrete, “working” in a construction carries most frequently loads causing about half of the breaking stress and to a certain extent changing in time. Bearing this in mind the most important for the application in engineering is watertightness of fine aggregate concrete composites under cycles of preloads no. 1. In such a case Watertightness of fine aggregate concrete composite modified only by silica fume examined in a traditional way is very high. The same concrete subjected to the cycle of loads no. 1 loses about 80% of its primary watertightness and in result it fails to meet the primarily requirements as far as watertightness is concerned.

5 CONCLUSIONS

The presented research on discussed types of concretes let the author state that:

1. The addition of both silica fume and steel fibre improves ultimate compressive strength.
2. Steel fibre improves flexural strength of the examined concretes and it decreases deflection and linear strain.
3. Despite 120 days of curing silica fume does not improve flexural strength and it does not decrease strain under axial load.
4. The relatively high watertightness of the described composites after the cycle of loads is achieved due to the addition of steel fibres.
5. Subjecting samples to the cycles of preload has an extended influence on their later watertightness.
6. Watertightness of not loaded fine aggregate concrete composites depends greatly on the addition of SF.
7. Watertightness examined on samples not preloaded does not allow for any assessing of watertightness of preloaded ones.
8. The addition of steel fibres allows to sustain a relatively high watertightness in fine aggregate concrete composites subjected to preload.
9. It is possible to obtain composites, on the basis of waste sand, which are characterized by very high chosen features.

REFERENCES


Hicks, C.R. 1982. *Fundamental concepts in the design experiments*, Holt Rinehart and Winston, New York, NY, USA.


Lee-Ing, T. & Chung-Ho, W. 2002. *STATISTICA V5.5 and Basic Statistic Analysis*, TasngHai Publisher, Taiwan.


PORTLAND CEMENT CONCRETE USING HIGH LEVELS OF MICROFINES

P.N. QUIROGA
Escuela Colombiana de Ingeniería, Bogotá, Colombia

D.W. FOWLER
International Center for Aggregates Research, The University of Texas at Austin, TX, USA

ABSTRACT: Aggregates are the most widely used building material. Nearly all roadways and a large percentage of other buildings and infrastructure utilize aggregates, the primary component in bases, Portland cement concrete, and asphalt concrete. In the U.S., nearly 3 billion tons of aggregates are produced each year. An increasingly larger percentage of aggregates are obtained by crushing; supplies of natural sands and gravels are becoming increasingly difficult to obtain due to environmental limitations and depletion of resources. Manufactured aggregates include relatively high percentages of microfines, i.e., aggregates smaller than 75-µm that traditionally have been viewed as a waste material. Specifications generally limit the amount of microfines in most aggregate applications. Considerable research has been conducted with the objective of demonstrating that higher amounts of microfines can be used to produce quality building materials. This presentation will focus on applications to Portland cement concrete. Results of research will be presented showing how aggregates with high microfine contents can be optimized for use in concrete; the results of strength and durability properties of high fines concrete will be compared to traditional concretes made with natural sands.

1 INTRODUCTION

Since 1996 the International Center for Aggregate Research (ICAR) has conducted industry-sponsored research projects related to concrete made with manufactured aggregates with high microfines at the University of Texas at Austin [Ahn & Fowler 2001, Quiroga & Fowler 2004]. Many types of aggregates have been used to make mortar and concrete mixtures with different amounts and types of microfines. The effect of different variables such as shape and texture of aggregate, amount of microfines, and type and amounts of admixtures and cementitious materials on the performance of fresh and hardened mortar and concrete has been observed. The results of some of the studies performed in this research will be presented as follows.

2 FRESH MORTAR AND CONCRETE

The workability of mortar and concrete mixtures was evaluated in a quantitative way by means of traditional tests such as flow and slump as well as in a qualitative way by evaluating properties such as cohesion, compactability and finishability. In a few cases different devices like vibrating tables or rheometers were used to assess workability. In general, it was observed that slump and flow decrease with the amount of microfines.

Figure 1 shows the flow for mortar mixtures with a fixed water-cement ratio (W/C). Manufactured fine aggregates (MFA) resulted in lower flows but some of these values are just slightly lower than for the natural sand control mixture, even for microfine contents as high as 15 percent. For microfines contents higher than 7 percent some mixtures had flow values lower than 70 percent of the control mixture. Flow tends to decrease with increasing amounts of microfines, although for less than about 10 percent the rate of decrease is low. For microfines contents higher than 15 percent, the flow was very low for the materials used.

For some mixtures a high range water reducer admixture (superplasticizer) based on carboxylic acrylic ester co-polymers was used. Figure 2 shows the dosage of the superplasticizer required to achieve a given slump for a set of concrete mixtures with different amounts of microfines from trap rock (TR). All these mixtures had the same W/C of 0.41, the same amount of paste and the same type of aggregate. The dosage of superplasticizer increased with the amount of microfines but the increase was low for microfines content of 10% or lower.
The type of microfines affects the performance of concrete. To evaluate that effect, some mixtures were made with the same W/C (0.41), the same paste content, the same type of aggregate (plus 75 µm sieve size) but with different type of microfines. Microfines obtained by sieving fine aggregate from three different sources were used. Figure 3 shows the dosage of superplasticizer required to reach 120-mm slump for mixtures without microfines, mixtures with 15% trap rock microfines (TR), mixtures with 15% granite microfines (GR) and 15% limestone microfines (LS). The superplasticizer demand increases for mixtures with 15% microfines, but it also varies with the type of microfines.

In this case, the superplasticizer demand for limestone microfines was significantly lower than for granite and trap rock microfines. Besides the effect on superplasticizer demand, it was observed that the effect of these three types of microfines on other aspects of workability such as cohesion, finishability and compact ability, evaluated qualitatively, was different.

3 HARDENED CONCRETE

As for hardened concrete, compressive strength, flexural strength, abrasion resistance, chloride ion permeability, and drying shrinkage of concrete mixtures with and without microfines were evaluated. In general, the performance in terms of strength and durability of concrete mixtures with manufactured aggregate with high microfines was equal or better than mixtures with natural sand with no microfines.

Figures 4 and 5 show the 28-day compressive and the 28-day flexural strengths of a set of eleven concrete mixtures with the same W/C (0.53) and the same paste content. These mixtures had the same coarse aggregate but different fine aggregates: one natural sand without microfines and ten crushed sands with microfines content as indicated in Table 1. In general, mixtures with manufactured aggregate with high microfines had compressive strengths slightly higher and flexural strengths significantly higher than the control mixture with natural sand.

Figure 6 shows for a different set of concrete mixtures the flexural strength for different types of aggregates (plus 75 µm sieve size) and different types and amounts of microfines. As expected, strength depended significantly on the type of aggregate but it was not affected significantly by the amount and type of microfines.

Drying shrinkage did not increase significantly in mixtures with high microfines as compared with
mixtures without microfines. For a set of concrete mixtures with water cement ratio of 0.41, the 28-day drying shrinkage of mixtures containing fine aggregate with 20% microfines, was 0.029%, just 11% higher than mixtures with no microfines (0.026 %) as can be seen in Figure 7.

As for resistance to chloride ion penetration, in general, concrete with manufactured aggregate with high microfines had less chloride ion permeability than concrete without microfines, as can be seen in Figure 8.

4 MICROFINES CHARACTERIZATION

From some of the results previously presented it can be concluded that the type of microfines has a significant effect on the performance of concrete. Furthermore, in the screening process made at the beginning of the research, it was found that some types of microfines may result in very high water demand and their use in concrete might be questioned. Consequently, microfines should be characterized to evaluate if they can be used for concrete or to determine the maximum amounts in which they can be used.

The methylene blue value (MBV) test [Yool 1998, Dumitru 1999] and the water demand test using the Vicat apparatus [Dewar 1999] were used in the attempt to find a proper test for microfines. The MBV measures the amount of methylene blue dye that can be absorbed by a powder material such as microfines or clay particles. Some clay such as smectites is able to absorb great amounts of methylene blue giving high MBV. It is believed that microfines contaminated with clays or with very small particles that could increase water demand will result in high MBV.

The flow of a set of mortar mixtures was compared with the MBV of the microfines of the different aggregates used. It was observed that flow tends to decrease with increasing MBV, although the correlation is not
very strong \((R^2 = 0.40)\) as can be seen in Figure 9. However, four data points in the left bottom part of the figure indicate that low MBVs do not ensure good workability. The mixture with very low flow and very high MBV that is shown in the right bottom of the figure suggests that MFA with very high MBVs might not be suitable for concrete.

The Vicat water demand test run for microfines is similar to the Vicat test used for consistency of cement pastes. The purpose is to measure the amount of water required to reach the target consistency. It is expected that the higher the water demand in the test, the higher the water demand in concrete. The packing degree, \(\phi\), defined as the volume of solids divided by the total volume, is related to the water demand by the following equation:

\[
\phi = \frac{V_{\text{solids}}}{V_{\text{total}}} = \frac{1}{1 + \frac{W_w}{W_{mf}} \cdot SG_{mf}}
\]

Where:
- \(W_w\) = weight of water
- \(W_{mf}\) = weight of microfines or powder
- \(SG_{mf}\) = specific gravity of microfines or powder

Three types of microfines: traprock (TR), granite (GR) and limestone (LS) were characterized with this test. Figure 10 shows the relation between the amounts of superplasticizer required to reach a 120-mm slump in a set of concrete mixtures made with these three microfines and the packing degree of microfines. From these results it was concluded that this test might be a good indicator of the microfines suitability for concrete.

5 GRADING OPTIMIZATION

Research has shown that aggregate grading plays a key role on the performance of Portland cement concrete, particularly in the fresh state (i.e. workability). There is general agreement that proper grading with the right amount of each fraction size will result in concrete with good workability, low water demand, and low paste content. In other words, a good aggregate grading will result in economical and durable concrete since cement is one of the most expensive components of concrete, and cement paste is the concrete component that shrinks and that is more easily attacked by acids, sulfates, and the environment in general.

Aggregate with a high packing degree (volume of solids to total volume) has low voids, so a smaller amount of paste is needed for filling these voids. It has been found that aggregate packing degree is related to aggregate grading. Good gradings have high packing degree and conversely poor grading tend to have low packing degrees. In fact some methods to optimize aggregate grading are based on packing concepts.

Some methods have been devised to optimize aggregate grading. Many ideal size distributions such as the “0.45 power chart” have been proposed since the beginning of concrete technology, and a number of procedures have been developed in order to optimize aggregate grading in practical ways. For example, the “18–8” specification has been used in the United States for some years. The aim is to limit the maximum and minimum amounts of aggregate fractions as shown in Figure 11. However, this method alone does not ensure good grading since aggregate blends that fit the limits still can have an excess or deficit of coarse or fine particles.

Shilstone [2002] has proposed the use of the “0.45 power chart” along with the “coarseness chart”. A good grading will usually be near to the ideal straight line of the 0.45 power chart (Figure 12). For the coarseness chart, aggregate is divided in three fractions: large, Q, intermediate, I, and fine, W. Large aggregate is composed by the plus 3/8-in (9.5 mm) sieve particles, intermediate is composed by the minus 3/8-in and plus N4, and fine is composed by the minus N4 and plus N200 sieve particles. The coarseness chart gives the
relationship between the modified workability factor (WF), and the coarseness factor (CF), that can be computed from equations (2) and (3)

\[ WF(\%) = W \times 100 + (C - 6) \times 2.5 \]  

(2)

Where:
\[ W = \text{minus N8 sieve particles} \]
\[ C = \text{amount of cement in sacks per cubic yard of concrete} \]

\[ CF(\%) = 100 \frac{Q}{Q + I} \]  

(3)

Where:
\[ Q = \text{Plus 3/8-in. sieve particles} \]
\[ I = \text{Minus 3/8 in. and plus N8 sieve particles} \]

Figure 12 presents the coarseness chart which is divided into five zones. The diagonal bar, which separates zone V from the others, also separates rocky mixtures from sandy mixes. Mixtures in zone I are prone to segregation. Mixtures in zone IV have too much fine aggregate and are likely to crack, yield low strength, and segregate during vibration. Zone II is the desirable zone. Zone III is an extension of zone II for maximum size of aggregate of 0.5 in. or less.

Although a uniform blend should have material retained in every sieve, Shilstone [2002] points out that the amount retained in a single sieve is not as important as the amounts retained in two adjacent sieves. Accordingly, sieves are grouped in clusters of two: 2 in. and 1-1/2 in., 1 in. and 3/4 in., N4 and N8, N16 and N30, and N50 and N100.

Excess material in one sieve in a cluster can compensate for a deficiency on the other sieve of the cluster. Deficiencies in two adjacent sieves in different clusters can be acceptable, as long as there is a peak at the other sieve of the cluster. It is in the N4-N8 cluster where the effect of shape and texture is more significant.

All minus N200 sieve materials are classified as paste. The combination of paste and fine aggregate (minus No 8) is considered as mortar. According to Shilstone, mortar is the most important part of the mixture because it is the key to strength, durability, workability, finishability, pumpability, wearability and cost. Shilstone states that "most problems are caused by sand grading that increase or decrease the amount of minus No 8 sieve particles". The “mortar factor," defined as the volume of the minus N8 sieve material (including aggregate, cement, water and air), allows the engineer to have a better idea of the mixture behavior.

6 HIGH MICROFINES AGGREGATE GRADING OPTIMIZATION

The workability of more than 100 concrete mixtures with high microfines was evaluated both quantitatively (by means of the slump test and a flow test on a vibrating table) and qualitatively. The aggregate grading was analyzed using the “18–8” specification, the “0.45 power chart”, and the “coarseness chart”.

Based on these results and on the performance in the field of some concrete mixtures with high microfines the following guidelines were developed:
It is recommended to perform the methylene blue test using the Ohio DOT standard (Appendix A.1) on microfines. A high MBV usually indicates a high probability of claylike and harmful particles. Microfines with MBV higher than 6 should be further evaluated using petrographic examination or chemical analysis.

For microfines and cementing materials the packing density should be obtained in saturated conditions. The Vicat test (ASTM C 187) is recommended for that purpose. The wet packing density values provide a comparison of different types of microfines and gives an indication of the effect of microfines on concrete water demand. The wet packing density can be determined by means of equation 1.

A modified version of the “18–8” specification (Figure 13) is recommended for high microfines concrete since the original specification is intended for concrete with no microfines.

As for the coarseness chart, the recommendation is to divide the desired zone II in three sub zones: II-a, II-b, and II-c (Figure 14). When no superplasticizer is used the recommended sub zones are II-a and II-b. When superplasticizer is used, the recommended sub zones are II-b and II-c. Finally, it is recommended that aggregate grading follow the 0.45 power chart.

7 CONCLUSIONS

From the results of these projects the following conclusions can be made.

- Good quality concrete in terms of strength and durability can be made from manufactured fine aggregate with high microfines content.
- Characterization tests to assess microfines suitability for concrete are required since some microfines might be harmful for concrete.
- More research is required to establish the suitability of the methylene blue and the water demand tests to characterize microfines for use in concrete.
- Proper grading and water reducing admixtures can be used to control the tendency of sand with high microfines to increase the water demand.

REFERENCES


Utilization of organo-modified reservoir sludge as fine aggregates in cement mortars

W.Y. Kuo, J.S. Huang, T.E. Tan & C.Y. Chou

Department of Civil Engineering, National Cheng Kung University, Tainan, Taiwan

ABSTRACT: Disposal of reservoir sludge has become a major problem in most of principal reservoirs in Taiwan. For the purposes of economy and ecology, we attempt to study the feasibility of using reservoir sludge as a substitute for a portion of fine aggregates in cement mortars. Through a cationic-exchange reaction with cationic surfactants, reservoir sludge in Taiwan mainly composed of smectite clay is first organo-modified. As a partial replacement of fine aggregates, the percentage of organo-modified reservoir sludge (OMRS) added in cement mortars ranges from 1% to 100%. The compressive strengths and permeability ratios of cement mortars with various percentages of OMRS particles were measured and then compared to those of plain cement mortars. Experimental results indicate that it is possible to replace up to 30% by weight of fine aggregates by OMRS particles in cement mortars for normal practice. Furthermore, excellent waterproof of cement mortars is achieved when the percentage of OMRS particles is within the range of 5–50%. Meanwhile, the microstructure of cement mortars is characterized by using SEM, EDS and MIP to evaluate the effects of OMRS particles on the improvements of mechanical properties of cement mortars.

1 INSTRUCTIONS

In Taiwan, more than 40 principal reservoirs with a total effective storage volume of approximately $1.6 \times 10^9 \text{m}^3$ have been consecutively constructed for the purposes of flood control, irrigation and water supply. Over the past decades, most of the principal reservoirs have been experiencing an annual 0.5% reduction of total effective storage volume due to the poor geological nature in their drainage basins. To increase the capacity of public water supply and ensure the safety of reservoirs, some renovation and improvement programs have been approved and are currently being undertaken by the government authorities in Taiwan. Therefore, the disposal and utilization of huge amount of evacuated reservoir sludge are of importance and become urgent when both economy and ecology are concerned.

Most of the reservoir sludge in Taiwan is mainly composed of gravel, sand and more than 60% smectite clays containing $\text{SiO}_2$, $\text{Al}_2\text{O}_3$ and $\text{CaO}$. Among them, gravel and sand can be separately used as coarse and fine aggregates in concretes. But, the water absorption of smectite clays causes detrimental expansion to a certain extent when they are mixed with water and thus cannot be directly used in concretes. The disposal and utilization of reservoir sludge have been paid less attention as compared to sewage sludge which has been reused as artificial lightweight aggregates [Bhat & Reid 1989; Tay & Yip 1989; Yip & Tay 1990], foamed materials [Wang et al. 2005], manufacture bricks [Tay 1987a] and a partial replacement for cement in concretes [Monzo et al. 1996, 1999; Pandey & Sharma 2000; Tay 1987b,c; Tay & Show 1991]. Nevertheless, reservoir sludge is expected to be much less hazardous as compared to municipal sewage sludge with some heavy metals or toxic substances and could be extensively used as construction and building materials.

Smectite clays such as montmorillonite, saponite and hectorite are the primary compounds of reservoir sludge in Taiwan. The typical chemical structure of smectite clays consists of an edge-shared octahedral sheet with aluminum or magnesium hydroxide and two silicate tetrahedral sheets with sodium or calcium cations; the sodium and calcium cations could be chemically replaced by a cationic-exchange reaction. Through the cationic-exchange reaction with organic surfactants, reservoir sludge in Taiwan can be organo-modified [Kuo et al. 2007]. The organo-modified reservoir sludge (OMRS) particles are hydrophobic and then can be utilized as a substitute for a portion of fine aggregates in cement mortars. The hydrophobic OMRS
particles, which can prevent water from penetrating the interlayer regions between silicate sheets in smectite clays, can be characterized by using thermo-gravimetric analysis (TGA). By conducting a series of compression and permeability tests on cement mortars with different percentages of organo-modified reservoir sludge, the feasibility of using reservoir sludge as fine aggregates in cement mortars is evaluated. Also, the microstructures of cement mortars containing different percentages of OMRS particles are characterized by using mercury intrusion porosimetry (MIP) and EDS to provide more physical insights.

2 MATERIALS AND METHODS

2.1 Preparation of OMRS particles

The reservoir sludge (RS) used here was evacuated from the A-Kung-Tien reservoir in Kaohsiung County of southwestern Taiwan. It is found that more than 60% of the reservoir sludge is smectite clays and its specific gravity is 2.72. The cation exchange capacity (CEC) of the reservoir sludge determined from atomic absorption spectrometer analyzer is 25 meq/100 g. The dosage of cationic surfactant used for the cation-exchange reaction of a reservoir sludge sample can be calculated from the following equation:

\[ X = \frac{1.1W(Ce/100)MW}{100Q} \]

Wherein, \( X \) is the dosage of cationic surfactant, \( Ce \) is the CEC value of the reservoir sludge sample expressed in a unit of meq/100 g, \( W \) is the total weight of the reservoir sludge sample, \( MW = 185.35 \text{ g/mol} \) is the molecular weight of cationic surfactant, dodecylamine \( C_{12}H_{27}N \), and \( Q = 1 \) is the number of valence electron. Besides, extra 10% of required amount of cationic surfactant is added to ensure that any reservoir sludge sample can be thoroughly organo-modified through the cation-exchange reaction, leading to a coefficient of 1.1 given in Equation (1).

At first, 500 g reservoir sludge was stirred in a beaker with 4000 ml distilled water at room temperature. Twelve hours later, 25 g dodecylamine \( C_{12}H_{27}N \), calculated from Equation (1), was stirred in another beaker with 1000 ml distilled water at higher temperature. After the cationic surfactant was completely dissolved in water, the pH value of mixing water was adjusted to about 3–4 by the addition of 1 M hydrochloride acid solution. Then, the resulting solution was slowly poured into the beaker with reservoir sludge suspension solution. In order to complete the cation-exchange reaction, the compound solution was vigorously stirred at room temperature for additional 12 hours. Next, the organo-modified reservoir sludge in the compound solution was washed and filtered repeatedly at least three times to get rid of the excess of organic ions. Finally, the OMRS slurry was dehydrated at 110°C for 24 hours and then ground into OMRS particles. The size distribution of OMRS particles was determined from a sieve analysis; more than 80% of the OMRS particles are between 75 \( \mu m \) and 600 \( \mu m \). Before the OMRS particles are used as fine aggregates in cement mortars, it is necessary to check if the hydrophilic reservoir sludge becomes hydrophobic after being organo-modified by the cationic surfactant dodecylamine. Hence, thermo-gravimetric analyses (TGA) on the RS and OMRS particles were separately conducted and then compared to each other.

2.2 Preparation of OMRS cement mortars

To evaluate the validity of OMRS particles used as a replacement for fine aggregates in cement mortars, various mix proportions of cement mortars with OMRS particles are proposed and listed in Table 1.

The constituents of the cement mortars we studied here are normal portland cement, quartz sands, OMRS particles and water. For the cement mortars we produced, the ratio of quartz sands and cement was fixed and equal to 2.75. The percentage of OMRS particles, expressed as a replacement fraction of the total weight of quartz sands in a mixture proportion of cement mortar, ranges from 1% to 100%; the percentages of OMRS particles considered here are 0, 1, 5, 10, 20, 30, 40, 50, 80 and 100%. From Table 1, it is noted that the required mixing water for the cement mortars with the same consistency increases with increasing percentage of OMRS particles.

2.3 Test methods of cement mortars

The effects of OMRS particles on the compressive strengths of cement mortars are evaluated by

<table>
<thead>
<tr>
<th>Dosage of OMRS(%)</th>
<th>W/C</th>
<th>OMRS (kg/m³)</th>
<th>Sand (kg/m³)</th>
<th>Cement (kg/m³)</th>
<th>Water (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.485</td>
<td>0</td>
<td>1375</td>
<td>500</td>
<td>240</td>
</tr>
<tr>
<td>1</td>
<td>0.49</td>
<td>14</td>
<td>1361</td>
<td>500</td>
<td>245</td>
</tr>
<tr>
<td>5</td>
<td>0.50</td>
<td>69</td>
<td>1306</td>
<td>500</td>
<td>250</td>
</tr>
<tr>
<td>10</td>
<td>0.53</td>
<td>138</td>
<td>1237</td>
<td>500</td>
<td>265</td>
</tr>
<tr>
<td>20</td>
<td>0.57</td>
<td>275</td>
<td>1100</td>
<td>500</td>
<td>285</td>
</tr>
<tr>
<td>30</td>
<td>0.62</td>
<td>413</td>
<td>962</td>
<td>500</td>
<td>310</td>
</tr>
<tr>
<td>40</td>
<td>0.70</td>
<td>550</td>
<td>825</td>
<td>500</td>
<td>350</td>
</tr>
<tr>
<td>50</td>
<td>0.80</td>
<td>688</td>
<td>687</td>
<td>500</td>
<td>400</td>
</tr>
<tr>
<td>80</td>
<td>1.25</td>
<td>1100</td>
<td>275</td>
<td>500</td>
<td>625</td>
</tr>
<tr>
<td>100</td>
<td>1.64</td>
<td>1375</td>
<td>0</td>
<td>500</td>
<td>820</td>
</tr>
</tbody>
</table>

* Flow table spread value of cement mortar was 100%.
conducting a series of compression tests. The wet cement mortars containing OMRS particles were cast into 50 mm × 50 mm × 50 mm cubic steel molds. One day later, the cubic cement mortar specimens were removed from steel molds and cured in water at room temperature for additional 2, 6 or 27 days. Cubic cement mortar specimens cured in water were dried and then trimmed before compression testing. The 3-day, 7-day and 28-day cement mortar specimens were loaded under a constant rate of 300 KPa/sec in compression tests. On the other hand, the effects of OMRS particles on the permeability of cement mortars are evaluated by employing JIS A1404 test method for waterproof agent of cement for concrete construction. The disk steel molds for permeability tests have an inner diameter of 150 mm and a height of 40 mm. The disk cement mortar specimens were removed from steel molds after 48 hours curing. The disk cement mortar specimens were cured at a temperature of 20 ± 3°C and a humidity of more than 80% for additional 19 days and then oven-dried to a constant weight at 80°C before permeability testing. During permeability testing, a constant hydraulic pressure of 200 KPa was imposed on the bottom surface of each disk cement mortar specimen. After permeability testing, both the top and bottom surfaces of each disk cement mortar specimen were wiped and then its resulting weight was recorded and used to calculate the increase of total weight. The permeability ratio (PR) is defined as the ratio of the increase of total weight of any cement mortar specimen containing OMRS particles to that of plain cement mortar specimen with a water-cement ratio of 0.485. When the PR value of a cement mortar specimen containing OMRS particles is less than 1.0, its waterproofing performance is better than plain cement mortar. On the contrary, the waterproofing performance of a cement mortar specimen is worse than plain cement mortar when its PR value is larger than 1.0.

The pore size distributions of 28-day cement mortar specimens containing various percentages of OMRS particles were measured by using mercury intrusion porosimetry. The cement mortar specimens with a dimension of 10 mm × 10 mm × 15 mm were first dried in an oven heated up to 110°C and then placed in a chamber of MIP. The percentages of OMRS particles we studied here were 0%, 10%, 50% and 100%. The variation of total weight for each cement mortar specimen was recorded as the pressure in the chamber was increased from zero up to roughly 400 MPa. Consequently, the pore size distribution of each cement mortar specimen was obtained from MIP measurement. In addition, the locations of OMRS particles distributed in 28-day cement mortar specimens were identified by using EDS with point-mapping images.

3 RESULTS AND DISCUSSIONS

3.1 Characterization of OMRS particles
Experimental results of thermogravimetric analyses (TGA) on the RS and OMRS particles are shown in Figure 1. The total weight of each sample for TGA tests is initially about 50 mg at the starting temperature of 50°C and then decreases consecutively when the test temperature is heated up from 50°C to 1000°C at a constant rate of 30°C/min under nitrogen atmosphere. Based on the derivative weight curves of RS and OMRS particles, it is found that the weight percentage of absorbed water in the interlayer regions between silicate sheets of smectite clays in RS particles, evaporated at around 150°C, is roughly 2% and much larger than that in OMRS particles. From Figures 1, it is also seen that the cationic surfactant dodecylamine in the interlayer regions between silicate sheets of smectite clays for OMRS particles is decomposed at the temperatures from 300°C to 500°C with a loss of around 5% total weight. Moreover, the weight loss curves for both RS and OMRS particles at the temperatures from 500°C to 1000°C are similar. Therefore, it can be concluded that the OMRS particles we used here have been organo-modified and become hydrophobic before they are mixed with water, quartz sands and cement.

3.2 Compressive strength
The measured 3-day, 7-day and 28-day compressive strengths of cement mortar specimens containing various percentages of OMRS particles are listed in Table 2 and shown in Figure 2; in each case, three...
cement mortar specimens were measured and then averaged. From Figure 2, it is quite consistent that either the 3-day, 7-day or 28-day compressive strengths of cement mortar specimens decrease with increasing percentage of OMRS particles. According to the magnitude of 28-day compressive strengths, the cement mortar specimens with different percentages of OMRS particles can be divided into three groups.

28-day compressive strengths of cement mortar specimens with 1%–30% OMRS particles are more than 25 MPa. Thus, cement mortar specimens with 1%–30% OMRS particles are classified as group I. It is noted that the 7-day compressive strengths of cement mortar specimens with 1% and 5% OMRS particles could be up to 95% of their 28-day compressive strengths. Meanwhile, the 28-day compressive strengths of cement mortars with 1% and 5% OMRS particles are close to those of plain cement mortars. In addition, the 7-day compressive strengths of cement mortars with 10%, 20% and 30% OMRS particles are around 72% of their 28-day compressive strengths. As compared to plain cement mortars, the 28-day compressive strengths of cement mortars with 10%, 20% and 30% OMRS particles drop within the range of 16%–32%. The decreasing 28-day strengths of cement mortars with OMRS particles result from the increasing water-cement ratios to attain a constant flow table spread value of 100% when the percentage of OMRS particles is increased. Besides, the hydration of cement mortars is apparently retarded due to the hindrance of hydrophobic OMRS particles to water diffusion when the percentage of OMRS particles is more than 10%.

Similar results can be found in group II including the cement mortars with 40% or 50% OMRS particles. In group II, the 28-day compressive strengths of cement mortars are more than 15 MPa but less than 25 MPa, and the 7-day compressive strengths of cement mortars only reach 66% of their 28-day compressive strengths. Again, the higher water-cement ratios required for the cement mortars with 40% and 50% OMRS particles to have a flow table spread value of 100% result in a significant decrease of 28-day compressive strengths.

A dramatic reduction of compressive strengths occurs for the cement mortars with a percentage of 80% or 100% OMRS particles, denoted as group III. Because of the excessively high water-cement ratios required to keep the consistency of cement mortar slurry with an extra amount of hydrophobic OMRS particles, the development of compressive strengths for the cement mortars becomes much slower even after 7 days curing. As expected, the 28-day compressive strengths of cement mortars with 80% and 100% OMRS particles are tremendously reduced and could be less than 10 MPa.

Experimental results of groups I suggest that OMRS particles can be used in normal-strength cement mortars. If the percentage of replacement for fine aggregates is less than 30%, the 28-day compressive strengths of cement mortars drop at a maximum of 30% but are still higher than 25 MPa which satisfies the design requirement of normal-strength cement mortars. Meanwhile, experimental results of group II indicate that the 28-day compressive strengths of cement mortars are within a range of 15–25 MPa when the percentage of replacement for fine aggregates varies from 30% to 50%. Thus, the cement mortars in group II can be used as intermediate-strength cement mortars in minor structural facilities and components. In group III, the 28-day compressive

<p>| DOSAGE OF | PERMEABILITY | COMPRATIVE STRENGTH (MPA) |</p>
<table>
<thead>
<tr>
<th>OMRS (%)</th>
<th>RATIO</th>
<th>3-DAY</th>
<th>7-DAY</th>
<th>28-DAY</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1</td>
<td>31.7</td>
<td>42.4</td>
<td>44.7</td>
</tr>
<tr>
<td>1</td>
<td>0.72</td>
<td>33.4</td>
<td>42.1</td>
<td>43.8</td>
</tr>
<tr>
<td>5</td>
<td>0.17</td>
<td>30.2</td>
<td>39.6</td>
<td>42.3</td>
</tr>
<tr>
<td>10</td>
<td>0.09</td>
<td>21.4</td>
<td>29.1</td>
<td>37.6</td>
</tr>
<tr>
<td>20</td>
<td>0.06</td>
<td>18.2</td>
<td>26.9</td>
<td>35.4</td>
</tr>
<tr>
<td>30</td>
<td>0.04</td>
<td>14.1</td>
<td>19.7</td>
<td>30.2</td>
</tr>
<tr>
<td>40</td>
<td>0.03</td>
<td>9.2</td>
<td>13.0</td>
<td>22.5</td>
</tr>
<tr>
<td>50</td>
<td>0.05</td>
<td>6.5</td>
<td>11.5</td>
<td>15.0</td>
</tr>
<tr>
<td>80</td>
<td>0.64</td>
<td>2.3</td>
<td>3.6</td>
<td>4.7</td>
</tr>
<tr>
<td>100</td>
<td>1.27</td>
<td>1.1</td>
<td>1.5</td>
<td>2.2</td>
</tr>
</tbody>
</table>

Figure 2. Compressive strengths of cement mortars containing various dosages of OMRS particles at different curing ages of 3, 7 and 28 days.
strengths of cement mortars are lower than 10 MPa. Hence, cement mortars in group III can be used as controlled low-strength materials (CLSM) in construction if the percentage of replacement for fine aggregates is higher than 80%.

3.3 Permeability ratio

Experiment results of permeability tests on cement mortars with various percentages of OMRS particles are listed in Table 2. The PR value is dramatically reduced when the percentage of OMRS particles in cement mortars is increased up to only 5%. Hence, it is obvious that OMRS particles used in cement mortars not only act as fine aggregates but also act as diffusion barriers to provide excellent waterproof performance. As the percentage of OMRS particles is increased from 10% up to 50%, the PR value of cement mortars become much lower than that of plain cement mortars. However, the total volume of accessible pores becomes more continuous as the percentage of OMRS particles is 10%. Meanwhile, the pore sizes of cement mortars containing 50% OMRS particles are mainly within a range from 0.01 µm to 0.05 µm and the total volume of accessed pores is much lower than that of plain cement mortars. The reason for higher porosity and larger pore size could be due to the difficulty in mixing for cement mortars with 100% OMRS particle even their water-cement ratio is increased to 1.64.

Although the total volume of accessed pores is reduced for cement mortars containing 10% OMRS particles, their compressive strengths are lower than those of plain cement mortars. The reason for those is primarily due to the retardation of cement hydration and lower strength of OMRS particles as compared to quartz sands. When the percentage of OMRS particles is higher than 10%, the 3-day, 7-day and 28-day compressive strength of cement mortars are much lower than those of plain cement mortars. Nevertheless, lower PR values of cement mortars containing more than 10% OMRS particles can be achieved.

The EDS raw data of plain cement mortars and cement mortars containing 10%, 50% and 100% OMRS particles are listed in Tables 3–6, respectively. From Tables 3–6, it is found that the Si/Ca ratio increases with increasing percentage of OMRS particles; the Si/Ca ratios of cement mortars containing 0%, 10%, 50% and 100% OMRS particles are 0.17, 0.36, 0.54 and 0.79, respectively. Meanwhile, the content of element Al increases with increasing percentage of OMRS particles. For instance, the EDS scattered molecules of Si and Ca point-mapping images for cement mortars containing 0% and 50% OMRS particles are shown in Figures 4 and 5, respectively. It is also found that some white points in the Si charts are absent in the Ca charts of Figures 5; these white points in the Si charts are identified as OMRS particles. The above finding verifies that OMRS particles mainly
composed of $SiO_2$ and $Al_2O_3$ can disperse well in the cross-sections of cement mortars even though the dosage of OMRS particles is up to 50%. As a result, the OMRS particles organo-modified by dodecylamine molecules with single hydrophobic carbon chain are easier to disperse in cement mortars than those by quaternary ammonium chloride molecules with two

Table 3. EDS raw data for the cross-section of a plain cement mortar.

<table>
<thead>
<tr>
<th>Element</th>
<th>Weight (%)</th>
<th>Atomic(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mg</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Al</td>
<td>2.72</td>
<td>3.81</td>
</tr>
<tr>
<td>Si</td>
<td>13.40</td>
<td>18.00</td>
</tr>
<tr>
<td>K</td>
<td>2.78</td>
<td>2.68</td>
</tr>
<tr>
<td>Ca</td>
<td>77.80</td>
<td>73.23</td>
</tr>
<tr>
<td>Ti</td>
<td>0.47</td>
<td>0.37</td>
</tr>
<tr>
<td>Mn</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Fe</td>
<td>2.84</td>
<td>1.92</td>
</tr>
<tr>
<td>Ni</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Cu</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Total</td>
<td>100</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 4. EDS raw data for the cross-section of a cement mortar containing 10% OMRS particles.

<table>
<thead>
<tr>
<th>Element</th>
<th>Weight (%)</th>
<th>Atomic(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mg</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Al</td>
<td>5.37</td>
<td>7.23</td>
</tr>
<tr>
<td>Si</td>
<td>22.09</td>
<td>28.55</td>
</tr>
<tr>
<td>K</td>
<td>5.16</td>
<td>4.79</td>
</tr>
<tr>
<td>Ca</td>
<td>60.81</td>
<td>55.08</td>
</tr>
<tr>
<td>Ti</td>
<td>0.66</td>
<td>0.50</td>
</tr>
<tr>
<td>Mn</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Fe</td>
<td>5.91</td>
<td>3.84</td>
</tr>
<tr>
<td>Ni</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Cu</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Total</td>
<td>100</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 5. EDS raw data for the cross-section of a cement mortar containing 50% OMRS particles.

<table>
<thead>
<tr>
<th>Element</th>
<th>Weight (%)</th>
<th>Atomic(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mg</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Al</td>
<td>9.27</td>
<td>12.16</td>
</tr>
<tr>
<td>Si</td>
<td>26.75</td>
<td>33.69</td>
</tr>
<tr>
<td>K</td>
<td>4.65</td>
<td>4.21</td>
</tr>
<tr>
<td>Ca</td>
<td>49.52</td>
<td>33.70</td>
</tr>
<tr>
<td>Ti</td>
<td>0.98</td>
<td>0.73</td>
</tr>
<tr>
<td>Mn</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Fe</td>
<td>8.57</td>
<td>5.43</td>
</tr>
<tr>
<td>Ni</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Cu</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Total</td>
<td>100</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 6. EDS raw data for the cross-section of a cement mortar containing 100% OMRS particles.

<table>
<thead>
<tr>
<th>Element</th>
<th>Weight (%)</th>
<th>Atomic(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mg</td>
<td>1.67</td>
<td>2.35</td>
</tr>
<tr>
<td>Al</td>
<td>11.83</td>
<td>15.05</td>
</tr>
<tr>
<td>Si</td>
<td>30.58</td>
<td>37.39</td>
</tr>
<tr>
<td>K</td>
<td>5.69</td>
<td>5.00</td>
</tr>
<tr>
<td>Ca</td>
<td>38.68</td>
<td>33.15</td>
</tr>
<tr>
<td>Ti</td>
<td>1.09</td>
<td>0.78</td>
</tr>
<tr>
<td>Mn</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Fe</td>
<td>9.77</td>
<td>6.01</td>
</tr>
<tr>
<td>Ni</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Cu</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Total</td>
<td>100</td>
<td>100</td>
</tr>
</tbody>
</table>

Figure 4. EDS scattered molecules of $Si$ and $Ca$ point-mapping images of a plain cement mortar.
carbon chains [Kuo et al. 2006]. Hence, a considerable amount of OMRS particles without forming clusters can be used as fine aggregates in cement mortars.

4 CONCLUSIONS

The feasibility of using reservoir sludge for replacement of fine aggregates in cement mortars is studied here. Since smectite clays are the primary compounds of reservoir sludge in Taiwan, reservoir sludge are first organo-modified before they are mixed with water. Thermogravimetric analyses indicate that hydrophilic reservoir sludge become hydrophobic through a cationic-exchange reaction. The experimental results show that the compressive strengths of cement mortars decrease as the percentage of OMRS particles is increased. If the percentage of OMRS particles is less than 30%, the 28-day compressive strengths of cement mortar are higher than 25 MPa which satisfies the design requirement of normal-strength cement mortars. Meanwhile, the compressive strengths of cement mortars are within a range of 15–25 MPa when the percentage of OMRS particles varies from 30% to 50%. Thus, the cement mortars containing 30%–50% OMRS particles can be used as intermediate-strength cement mortars in minor structural facilities and components. However, the compressive strengths of cement mortars with more than 80% OMRS particles are lower than 10 MPa.

Furthermore, the permeability ratios of cement mortars are dramatically reduced when the percentage of OMRS particles is increased up to only 5%. Hence, OMRS particles used in cement mortars not only act as fine aggregates but also act as diffusion barriers to provide excellent waterproof performance. As the percentage of OMRS particles is increased from 10% to 50%, the PR values of cement mortars become much lower. But, the PR values increase significantly when the percentage of OMRS particles is larger than 80%.

From MIP measurements, it is found that the pore size distribution and the total volume of capillary pores in cement mortars are dramatically influenced by the percentage of OMRS particles we introduced. As compared to plain cement mortars, the volume of intermediate pores ranging from 0.2 µm to 2 µm is significantly reduced for cement mortars containing OMRS particles. However, the total volume of capillary pores in cement mortars containing 100% OMRS particles become higher due to the difficulty in mixing. Therefore, it is verified that the capillary pores in cement mortars can be partially filled if adequate amount of OMRS particles is used. The EDS raw data and scattered molecules point-mapping images for cement mortars containing OMRS particles also show that the OMRS particles organo-modified by dodecylamine molecules with single hydrophobic carbon chain are easier to disperse in cement mortars.

However, the strength of single OMRS particle is lower than quartz sand. When the percentage of OMRS particles is higher than 10%, the compressive strengths of cement mortars become weaker even though their total volume of accessed pores is lower as compared to plain cement mortars. Therefore, 5% OMRS particles used as a substitute for fine aggregates in cement mortar is preferred when workability, strength and permeability are sought. Cement mortars containing 10% to 30% OMRS particles can provide sufficient compressive strengths and excellent waterproofing performance if appropriate water-cement ratio is used.

Figure 5. EDS scattered molecules of Si and Ca point-mapping images of cement mortar containing 50% OMRS particles.
REFERENCES


Study on fly ash-saturation in cementitious materials

Long Guangcheng, Liu Yunhua & Xie Youjun
School of Civil Engineering and Architecture, Central South University, Changsha, China

ABSTRACT: The concept of fly ash-saturation is put forward based on the index of maximum compressive strength and optimum strength factor of unit fly ash in mortar and concrete system, respectively. Experiments are also carried out to investigate the effect of water to binder ratio (w/b) and type of fly ash on fly ash-saturation under full hydration condition in mortar system. The results indicate that the value of fly ash-saturation in mortar is influenced by w/b and type of fly ash. In investigated area, fly ash-saturation in mortar and concrete ranges from 20% to 30% of total binder by weight. The fly ash-saturation for optimum compressive strength of mortar/concrete differs from one based on optimum unit strength factor of fly ash.

1 INTRODUCTION

It is well known that the utilization of fly ash in cement-based materials is of great importance not only for improving application technology of industrial by-products but also for upgrading properties of cement-based materials [Mehta 1999]. For several decades, much efforts has been focused on developing some new technologies of high value-added utilization of natural or industrial waste such as fly ash. Many achievements related to effects of fly ash on properties of cement-based materials have been documented [Mehta 1999, SHI Huisheng et al 2004, Long et al 2005, Babu K G et al 1996, Baoju et al 2003]. However, most of results are obtained from experiments under a limited age condition. In terms of hydration theory of cement and the physical and chemical action of minerals admixtures including fly ash in concrete, effect of fly ash on properties of concrete/mortar is time-dependent. And therefore the results gained within limited age might not reflect the whole or long-term effects of fly ash on properties of cement-based materials. Furthermore, the information related to strength contribution of unit fly ash on properties of cement-based materials is limited. In order to know systematically the efficiency of fly ash and to develop some new methods for high value-added utilization of fly ash in concrete, it is necessary to study the reasonable amount range of fly ash in concrete and the relationship between content of fly ash and properties of concrete with fly ash.

On basis of above analysis and plenty of practices, it follows that under ‘full hydration’ condition, there exists a certain threshold of fly ash to make cement-based materials system obtain optimum properties such as best strength or lowest permeability etc. and to let fly ash exerts best contribution to cement-based materials. In this paper, this threshold is named as fly ash-saturation. Obviously, the fly ash-saturation is affected by many factors such as properties of cementitious materials system, type of fly ash, water to binder (cement and mineral admixture) ratio (w/b) etc. In addition, it is noted that the concept of ‘full hydration’ for fly ash-saturation in this paper is the adequately hydration under corresponding to w/b condition and is not necessary to complete hydration of each cement or fly ash particle, which is evaluated by index of the highest value of compressive strength of sample.

In order to assess the strength contribution of unit fly ash and efficiency of fly ash in cement-based materials, the index of unit strength-factor, $\lambda$, is put forward. $\lambda$ is defined as strength contribution of unit fly ash in sample with fly ash to one of unit cement in control sample ratio. In this study, it is assumed that the strength of sample with fly ash is composed of two parts of strength contributions of fly ash and cement. And strength contribution of cement in sample with fly ash can be calculated by one of unit cement in control sample multiplying mass percent of cement in sample. It is expected that the result is not be affected by such assumption. So, one can obtain the equation (1) as followings.

$$\lambda = \frac{R - (100 - \alpha) \gamma_c}{\alpha \gamma_c} \quad (\alpha > 0)$$

where: $\gamma_c$ is named as strength contribution of unit cement, obtaining from compressive strength of control sample dividing by 100; $\alpha$ is the replacement of...
cement by fly ash, expressed by weight percent. R is the compressive strength of sample with fly ash. It is obvious that the limit of $\lambda$ is equal to 1 when $\alpha$ is zero.

In following section, the fly ash-saturation under full hydration condition is investigated by experiment on basis of the consideration of optimum strength of mortar and concrete and the best unit strength-factor of fly ash in cement-based materials.

2 EXPERIMENTAL DETAILS

2.1 Raw materials

Cement used is Grade 32.5 Ordinary Portland cement according to Chinese Standard GB 175-1999. Its chemical compositions are listed in Table 1. Three types of fly ash such as grade I (FAI), grade II (FAII) and one denoted as UFA was used. UFA is derived from grade II fly ash by grinding. Their chemical compositions and physical properties are given in Table 1 and Table 2. The tree types of fly ash in this paper are similar to class F according to ASTM standard. A naphthalene sulphonic acid was used to improve workability of sample.

River sand with fineness modulus of 2.2 was used for the mortar specimen. Its size distribution is $0.16/4.75\,\text{mm}$. For the concrete samples, gravel with a maximum diameter of 25 mm was employed as coarse aggregate. Its apparent density was 2700 kg/m$^3$. River sand with a fineness modulus of 2.8 and an apparent density of 2650 kg/m$^3$ was used as fine aggregate.

2.2 Sample preparation and test

Mortar sample is prepared by stainless steel mold with 40 mm $\times$ 40 mm $\times$ 160 mm. The weight ratio of sand to binder (cement and fly ash) is 2.5 for all mortar samples. The flowability for all mortar samples ranges from 135 mm to 150 mm measured by flow table test according to ASTM C230-90.

Concrete sample is made by stainless steel mold with 100 mm $\times$ 100 mm $\times$ 100 mm. Mix proportion of raw materials in concrete mixture is: 500 kg/m$^3$ binder; 610 kg/m$^3$ sand; 1230 kg/m$^3$ coarse aggregate; 140 kg/m$^3$ water; The slump for all concrete samples ranges from 10 mm to 30 mm. Water-reducer is added to improve workability of mortar and concrete sample.

In order to let cement and fly ash ‘fully hydrate’ within a limited age, the following procedure of curing treatment for mortar sample was chosen: sample was demolded at 1 day after casting and cured in water with a temperature of 22°C for four days and then cured in water with a temperature of 60°C for two days. For Concrete sample, two curing conditions are used. One is standard curing condition ($20\pm2^\circ\text{C}$, RH $\geq 95\%$) for 90d. The other is heat water with a temperature of 60°C curing condition for 12h after demolded at 1d and then standard curing until 90d age. Due to the consideration of side effects of excess high temperature on microstructure and properties of sample, heat water with a temperature of only 60°C is chosen as a rapid curing condition. The details for curing treatment can be referred documents [LONG Guangcheng et al 2005].

3 RESULTS AND DISCUSSION

3.1 Effects of amount of fly ash on strength of mortar and unit strength factor under fully hydration condition

Figure 1 shows the result of effect of amount of fly ash FAI on compressive strength of mortar with a w/b of 0.30 and its unit strength factor under specific condition.

Figure 1 shows the result of effect of amount of fly ash FAI on compressive strength of mortar with a w/b of 0.30 and its unit strength factor under specific condition.

From result shown in Figure.1, one can see that under investigated condition, the compressive strength of mortar firstly increases with increasing amount of fly ash and then gradually reduces after more than 30% replacement of cement by fly ash. The variation of flexural strength of mortar is similar to that of compressive strength of mortar. This shows that there is an optimum amount of fly ash to make compressive or flexural strength of mortar come up to maximum under specific condition. This optimum amount of fly ash is about 30% in this experiment. That is to say, fly ash-saturation is 30% replacement of cement by fly ash with respect to maximum strength index of mortar. In addition, it can be found that the relationship between variation of strength of mortar and amount of fly ash under investigated experimental condition differs from that of under standard curing condition ($20\pm2^\circ\text{C}$ and RH $\geq 95\%$) within 28 days age. The reason is that
under standard curing condition the efficiency of fly ash cannot be brought into play completely with a limited age such as 28 days age.

Further, from the results given in Figure 1b, unit strength factor of fly ash varies with various content of fly ash in mortar. There exists an optimum content of fly ash, i.e. 20%, to make its unit strength factor maximum. The unit strength factor of fly ash reduces greatly with increasing amount of fly ash when the replacement of cement by fly ash is more than 30%. However, even if the replacement of cement by fly ash is up to 60% the unit strength factor of fly ash is still equal to about 1, which shows a considerable contribution of fly ash to compressive strength of mortar. This also indicates that, under full hydration condition, unit fly ash has a high strength contribution to cement-based materials. At the same time, one can also see that the optimum content of fly ash for maximum compressive strength of mortar differs from that for maximum unit strength factor of fly ash. This may be the reason that, on the one hand, the addition of fly ash can improve adaptability of mortar/concrete to high temperature curing [LIU Baoju et al 2005]. On the other hand, fly ash particles can play a physical filling role and a chemical reaction effect on cementitious materials system. A lack of fly ash particles will lead to an insufficient area available for the precipitation of hydrates and an excess of fly ash particles will contribute to the dispersion of the cement grains, which both result in a reduction of compressive strength of sample [Martin Cyr et al 2006]. Obviously, the effect of fly ash on sample varies with amount of fly ash. When the total physicochemical effects of fly ash comes up to the best the compressive strength of sample is the highest. And it is the corresponding optimum amount of fly ash.

3.2 Effect of w/b on fly ash-saturation in mortar

Figure 2 shows the effects of w/b on fly ash-saturation (FAI) of mortar system. From the result given in figure 2a, it can be seen that the compressive strength of mortar decreases with increasing w/b when the replacement of cement by fly ash is up to 60%. However, when the replacement of cement by fly ash is less than 50% the compressive strength of mortar is still high. This shows that fly ash has a good adaptability to high temperature curing. At the same time, one can also see that the optimum content of fly ash for maximum compressive strength of mortar differs from that for maximum unit strength factor of fly ash. This may be the reason that, on the one hand, the addition of fly ash can improve adaptability of mortar/concrete to high temperature curing [LIU Baoju et al 2005]. On the other hand, fly ash particles can play a physical filling role and a chemical reaction effect on cementitious materials system. A lack of fly ash particles will lead to an insufficient area available for the precipitation of hydrates and an excess of fly ash particles will contribute to the dispersion of the cement grains, which both result in a reduction of compressive strength of sample [Martin Cyr et al 2006]. Obviously, the effect of fly ash on sample varies with amount of fly ash. When the total physicochemical effects of fly ash comes up to the best the compressive strength of sample is the highest. And it is the corresponding optimum amount of fly ash.

Figure 1. Effect of content of fly ash FAI on strength and unit strength factor of mortar with a w/b of 0.30.

Figure 2. Effect of w/b on fly ash-saturation in mortar.
mortar reduces with increase of w/b. Moreover, the amount of fly ash for maximum compressive strength of mortar varies with w/b. When w/b is equal to 0.3, the compressive strength of mortar comes up to maximum when the replacement of cement by fly ash FAI is 30%. However, after w/b is more than 0.34 the compressive strength of mortar with 20% fly ash is up to maximum, i.e. fly ash-saturation varies with w/b of mortar with respect to compressive strength of mortar. This may be the reason that under a low w/b condition, mortar with a higher content of fly ash will have a higher water to cement ratio. Therefore, more cement particles can be hydrated and fly ash particles play fully physical and chemical role in mortar system, which results in a maximum compressive strength of mortar.

Fig. 2b shows that there is a reducing tendency of unit strength-factor with increasing w/b. Moreover, for all investigated w/b, the unit strength factor of fly ash reached maximum when replacement of cement by fly ash is 20%. This indicates within investigated area, fly ash-saturation in mortar is different for various assessing indices.

3.3 Effects of type of fly ash on fly ash-saturation

The physical and chemical effect of fly ash on properties of cement-based materials depends on characteristics of hydration environment and properties of fly ash. As to a defined hydration environment, the physical and chemical effect of fly ash is determined by type of fly ash. Figure 3 gives the experimental result of effect of fly ash type on fly ash-saturation in mortar with a w/b of 0.3.

Figure 3 shows that type of fly ash greatly influenced compressive strength of mortar. And the optimum content of fly ash for maximum compressive strength of mortar differs from each other with respect to type of fly ash. Moreover, the most noteworthy is that the strength of mortar with UFA is considerable large and the one of mortar with FAII is much less compared with mortar with FAI. UFA is derived from FAII by grinding treatment. This indicates that mechanical grinding treatment to fly ash can remarkably improve strength contribution of fly ash on cement-based materials system. This may result from the improvement of size distribution and chemical reaction of UFA particles by mechanical treatment. Therefore UFA show good physical filling effect and chemical reaction role in sample compared with FAII.

The result shown in Figure 3b indicates that the variation of unit strength factor of fly ash with replacement of cement by fly ash is similar to each other for three types of fly ash. Furthermore, the fly ash-saturation is the same for three types of fly ash and it is about 20%. Compared with FAI or UFA, the unit strength-factor of FAII is much less. The less chemical activity and larger particles diameter of FAII is responsible for this result.

3.4 Fly ash-saturation in concrete with respect to maximum compressive strength

Compared with mortar, concrete system includes coarse aggregate and its microstructure differs from that of mortar. Therefore, the physical and chemical effects of fly ash on concrete may be different from that of mortar and so does the fly ash-saturation. With respect to concrete system, the influences of replacement of cement by fly ash on compressive strength of concrete and its unit strength factor are given in Figure 4.

From the result shown in figure 4a, one can see that there exists an optimum amount of fly ash to make compressive strength of concrete reach maximum whether under standard curing of 90 days age condition or under heat water curing for 12 h and standard curing until 90 days age condition. In this experiment, the optimum content of fly ash is the same, i.e. 20% both for two kinds of curing condition. In addition, it
is noted that addition of fly ash into concrete improved significantly the compressive strength. This is so called improvement of steam curing adaptability of concrete by addition of fly ash[LIU Baoju et al 2005] This will also result in a high unit strength factor of fly ash when fly ash was added into concrete system under investigated heat curing condition (see Figure 4b). Figure 4b shows that the unit strength factor of fly ash is more than 2.0 within 20% replacement of cement by fly ash and then rapidly reduces after addition amount of fly ash is more than 20%.  

4 CONCLUSIONS

As mentioned above, fly ash-saturation in cementitious materials system is a very complicated issue, which depends on w/b of concrete or mortar, characteristics of hydration system, fly ash type and aggregates type etc. Moreover, there are many kinds of fly ash including various physical properties such as fineness and chemical properties. Therefore, this issue can not be solved clearly by a simple paper with limited space. From present study, following conclusions can be obtained.

1) There exists a fly ash-saturation range for maximum strength of mortar and concrete or maximum strength-factor of unit fly ash in cement-based materials under fully hydration condition. The proposition of fly ash-saturation is reasonable and valid.

2) Parameter of water to binder ratio and type of fly ash can greatly influence fly ash-saturation in cementitious materials. In this investigated area, fly ash-saturation ranges from 20% to 30% replacement of cement by fly ash for mortar or concrete system.

3) Fly ash-saturation for maximum compressive strength of cementitious materials differs from that for maximum strength-factor of unit fly ash.

4) Mechanical grinding treatment to fly ash can remarkably improve its strength contribution on cement-based materials.

REFERENCES

Martin Cyr, Philippe Lawrence & Erick Ringot 2006. Efficiency of mineral admixtures in mortars: Quantification of the physical and chemical effects of fine admixtures in relation with compressive strength, Cement and Concrete Research, 36(2): 264–277
Ultra fine fly ash concrete

Bruce K.T. Kandie & Ewan A. Byars
CMRU, Department of Civil and Structural Engineering, University of Sheffield, UK

ABSTRACT: This paper reports on the use of Ultra-Fine Fly Ash (UFFA) used as up to 50% partial replacement for cement in concrete. UFFA is a new generation fine coal fly ash, which is double-classified to obtain the finer particles from the classified bulk. The influence of UFFA on fresh properties (flow, setting time and air content), porosity of mid to high strength, medium workability concrete mixes (designed for equal 28-day strength) are reported and compared with concrete mixes made with OPC, Micro-Silica (MS) and seven BS 3892 Part 1 fly ashes (PFA).

The study shows that UFFA has significantly higher pozzolanicity than all UK fly ashes tested and significantly reduces the water demand and air content of concrete. The compressive strength results indicate that a partial replacement of OPC with UFFA can not only reduce the total binder content required to obtain target strength at specific ages irrespective of curing, but also increases the efficiency of the combination binder by up to 50% at later ages.

The porosity of concrete made with UFFA all showed improvement compared to equal strength concrete made with OPC and UK PFA, irrespective of curing. A high strength of 130 N/mm² was attained at 28 days for a blend of UFFA/MS/OPC concrete with a combination of selected aggregates sizes proportions.

1 INTRODUCTION

Portland cement is the principle hydraulic binder used in modern concrete practice and manufactured by energy intensive process that consuming up to 5.29 GJ/t, mostly obtained from burning of fossil fuel, is consumed [Khurana et al. 2002]. The production of every tonne of Portland cement emit 0.89 to 1.1 tonne of carbon dioxide, this contribute to the greenhouse gas emission that is responsible for global warming [Bouzoubaa et al 1998 and Turanli et al 2005].

The current production of fly ash at UK power plants is in the range of 8–10 Mt per annum. Approximately 50% of this is used in industrial applications and the rest requires disposal. The world situation is similar with an average of 40% usage [McCarthy et al 1999].

Ultra fine fly ash (UFFA) is a new style of fly ash. Which is double classified at source to produce a product consisting at only the very finest fly ash particles, typically with a mean 3 μm and a specific surface area of 30,000 cm²/g.

The aim of this investigation was to characterise the fresh, engineering and durability performance of UFFA concrete in comparison to concrete made with OPC, a range of UK fly ashes and micro silica.

2 EXPERIMENTAL INVESTIGATION

2.1 Materials

The cement was a Portland cement confirming to BS 12 1996, class 42.5 N. Seven UK Class F fly ashes confirming to BS 3892, Part 1: 1997. Micro silica was an aqueous slurry (50/50 by weight) and UFFA used was a commercially available class F fly ash confirming to BS 3892 Part 1: 1997. Their chemical compositions are show in Table 1. The aggregate was 20 and 10 mm river aggregates and sand are shown in Table 2. A naphthalene-based superplasticizer to BS 5075, Part 3 was used to achieve a concrete workability level at 125 mm slump. The properties of the aggregates are given in Table 2. Figure 1 show the various types of fly ashes researched on.

2.2 Concrete mixes

A series of concrete mixes were prepared for grades 40–80 N/mm², with w/b ratios of 0.21–0.56 and a 125 mm slump target. UFFA was used in the proportion of 0–50% of the total cementitious materials. The cementitious materials tested included PC42.5N [BS12:1992], commercially available UFFA to BS
Part 1, 7 UK fly ashes to BS 3892: Part 1 and a Micro silica in slurry form (Table 1). The fresh samples were covered with polyethylene sheet to prevent evaporation. All the specimens were cast in 100 mm steel cube mould and were cured in water to air environment after demoulding after 1 day.

2.3 Curing conditions

The curing conditions used are listed in Table 2.

2.4 Determination of particle distribution

A laser Malvern Mastersizer E was used to measure the particle size and the surface area of the various pozzolans and cementitious materials.

2.5 Determination of loss on ignition

The loss of ignition was determined in accordance to BS EN 196-2: 1995 at a temperature of 950°C for duration of 1 hour.

2.6 Determination of the setting time

The setting time was determined in accordance to BS EN 196-3.

2.7 Determination of water demand

This test was undertaken in accordance to BS 3892: Part 1: 1993. Fine, medium and coarse sand was mixed in accordance to BS EN 196: Part 1: 1987.

Table 1. Chemical properties of the cementitious materials used.

<table>
<thead>
<tr>
<th>PERCENTAGE COMPOSITION</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Chemical composition</strong></td>
</tr>
</tbody>
</table>

Table 2. The properties of the aggregates.

<table>
<thead>
<tr>
<th>Properties of aggregates</th>
<th>Coarse aggregates</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>% Passing</strong></td>
<td><strong>Sieve no. (μm)</strong></td>
</tr>
<tr>
<td><strong>Sieve no. (μm)</strong></td>
<td></td>
</tr>
<tr>
<td>4.75</td>
<td>98</td>
</tr>
<tr>
<td>2.36</td>
<td>86</td>
</tr>
<tr>
<td>1.18</td>
<td>76</td>
</tr>
<tr>
<td>0.60</td>
<td>62</td>
</tr>
<tr>
<td>0.30</td>
<td>20</td>
</tr>
<tr>
<td>0.15</td>
<td>6</td>
</tr>
<tr>
<td>0.075</td>
<td>5</td>
</tr>
<tr>
<td>FM</td>
<td>2.51</td>
</tr>
</tbody>
</table>

* Provided by the manufacturer.
2.8 Determination of porosity of concrete

The porosity of all the mixes was determined using vacuum saturation apparatus [RILEM Recommendations 1984].

2.9 Determination of the compressive strength

The compressive strength in accordance to BS 1881: Part 116: 1983 on three 100 mm cubes at 7 and 28 days of full water immersion samples.

3 RESULTS AND DISCUSSIONS

3.1 Particle size distribution

Table 4 shows the particle size distribution for the various materials analysed. From this table, it is clear that UFFA has higher specific surface area than the UK. BS 3892 Part 1 PFA and GGBS which was in the range of 340 to 715 m²/kg and 1,448 m²/kg respectively, but SM had the highest surface area. UFFA had a fineness of 0% with a mean particle size of 2.32 µm and 90% of the particles were 5.25 µm, this was less than that obtained from BS 3892 Part 1 PFA and GGBS but similar to MS. The fineness of the various fly ashes showed relationship with its content in respect particle fineness (Table 4). Figure 2 (a) show there is a linear relationship between the mean particle size and the fineness of fly ash and similarly with every increase in fineness there is an increase in the carbon content (Figure 2 (b)). The relationship between the specific surface area and the mean particle size was not linear but a curve, which was decreasing with the increase in the mean particle size due to the fly ash particle being more coarser in it morphology.

A global fly ash characteristic may be prediction from figure 2 by knowing one of the elements of any type of fly ash. The mathematical for the curve or linear relationships for the fly ashes particles characteristics: AMD verses Sp. Surface Area is given by equation $y = 7503.8x^{-0.95}$ ($R^2 = 0.97$), AMD verses Fineness by equation $y = 0.7232x^{-2.9}$ ($R^2 = 0.95$) and Carbon content verse Sp. Surface Area by equation $y = 1169x^{0.61}$ ($R^2 = 0.92$).
3.2 Water demand

Figure 3 shows the water demand for cement, various pazzolans and the cementitious materials that were used in this investigation. UFFA had the lower water demand and 29% was the maximum water reduction at 95% replacement level of PC42.5N. The seven UK fly ashes had a variation of water demand and these were between the upper and lower bound zone. GGBS

Table 4. Mineral admixture laser particle properties.

<table>
<thead>
<tr>
<th>Cementitious material</th>
<th>Sp. surface area m²/Kg</th>
<th>Carbon content (%)</th>
<th>Fineness &gt;45(µm) (%)</th>
<th>Particle diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>D[ν,0.1]</td>
</tr>
<tr>
<td>UFFA</td>
<td>2,893</td>
<td>0.23</td>
<td>0</td>
<td>0.4</td>
</tr>
<tr>
<td>PFA1</td>
<td>715</td>
<td>3.1</td>
<td>6.5</td>
<td>1.5</td>
</tr>
<tr>
<td>PFA2</td>
<td>359</td>
<td>4.9</td>
<td>11.1</td>
<td>3.37</td>
</tr>
<tr>
<td>PFA3</td>
<td>509</td>
<td>5.8</td>
<td>10.1</td>
<td>2.14</td>
</tr>
<tr>
<td>PFA4</td>
<td>681</td>
<td>2.5</td>
<td>6.3</td>
<td>1.58</td>
</tr>
<tr>
<td>PFA5</td>
<td>448</td>
<td>6.1</td>
<td>14.3</td>
<td>2.43</td>
</tr>
<tr>
<td>PFA6</td>
<td>499</td>
<td>3.3</td>
<td>9.3</td>
<td>2.24</td>
</tr>
<tr>
<td>PFA7</td>
<td>340</td>
<td>4.7</td>
<td>14.2</td>
<td>3.65</td>
</tr>
<tr>
<td>MS*</td>
<td>4,952</td>
<td>–</td>
<td>0</td>
<td>0.25</td>
</tr>
<tr>
<td>GGBS*</td>
<td>1,448</td>
<td>–</td>
<td>0.5</td>
<td>0.9</td>
</tr>
</tbody>
</table>

* for comparison.

Figure 2. Fly ash physical characteristics.
showed a slight reduction in the water demand up to a maximum reduction of 4% while MS increased the water demand as its partial replacement level of PC 42.5 N was increased, with an increase of 20 and 71 for 10% and 30% replacement level. Figure 4 (a) and (b) shows that the specific surface area, unburned carbon content and the replacement level were the factor responsible for the high and low water demand for the UFFA, seven UK fly ashes, GGBS and MS.

We note that as the specific surface area for the fly ashes increased, there was a low water demand, similarly as the replacement of cement was increased there was a low water demand. This was directly opposite by the increase in carbon content increasing the water demand. This clearly shows that the spherical particle shape, the low level of carbon and the fineness of UFFA may be the influencing factors to this reduction in water. This was not similar to SM despite it having a high surface area and its irregular surface texture resulted to this high water demand. GGBS having a high surface area did not show any significant water reduction.

### 3.3 Workability

All the concrete mixes were designed for a slump of 120 mm for an equal 28 days compressive strength. The workability was measured at different rates using
the Tattersall two-point workability test and the flow table. From table 5 and figure 5, we note that as the percentage of UFFA replacement of cement increased, there was a reduction in yield value (g) and plastic viscosity (h). This reduction of g and h due to UFFA replacement levels is as a result of an UFFA supplying additional lubricant round the grains of aggregates, due to the “ball bearing effect” of the ultra fine particle size, spherical shape and low carbon content of UFFA. The application of superplasticizers also aided the decrease in both g and h value for all the cementitious materials. The workability of concrete containing UFFA was enhanced compared to BS 3892 Part 1 PFA and that of MS. Despite the fineness of MS, the workability was low, this may be as a resulted due to MS irregular shape, high surface area and its high reactivity, resulting in a high g and h value due to MS reacting with the mixing water right from the moment it comes in contact with water and a little of the mixing water is left for lubricating the concrete despite the application of the superplasticizer.

Figure 6 (a) Shows the workability of concrete in terms of flow due to UFFA application. It can be noted that as the UFFA content is increased from 0% to 50% by weight of OPC, the flowability increases, this may be attributed to the ultra fineness of this fly ash compared to the UK BS 3892 Part 1 PFA and MS. Similarly, there was a significant reduction in the air content as compared to the control mixes. But there was a variation in the air content for mixes cast for

<table>
<thead>
<tr>
<th>Mixes</th>
<th>Two-Point Test Results</th>
<th>% Air Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade 40N/mm²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>OPC</td>
<td>2.96</td>
<td>0.48</td>
</tr>
<tr>
<td>UFFA-05</td>
<td>1.95</td>
<td>0.393</td>
</tr>
<tr>
<td>UFFA-10</td>
<td>1.59</td>
<td>0.383</td>
</tr>
<tr>
<td>UFFA-30</td>
<td>1.22</td>
<td>0.38</td>
</tr>
<tr>
<td>UFFA-50</td>
<td>0.71</td>
<td>0.336</td>
</tr>
<tr>
<td>BS 3892 Part 1 PFA-30</td>
<td>1.4</td>
<td>0.395</td>
</tr>
<tr>
<td>MS-10</td>
<td>2.03</td>
<td>0.392</td>
</tr>
<tr>
<td>Grade 80N/mm²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>OPC</td>
<td>3.34</td>
<td>0.465</td>
</tr>
<tr>
<td>UFFA-05</td>
<td>0.72</td>
<td>0.276</td>
</tr>
<tr>
<td>UFFA-10</td>
<td>0.65</td>
<td>0.275</td>
</tr>
<tr>
<td>UFFA-30</td>
<td>0.29</td>
<td>0.274</td>
</tr>
<tr>
<td>BS 3892 Part 1 PFA-30</td>
<td>0.42</td>
<td>0.294</td>
</tr>
<tr>
<td>MS-10</td>
<td>1.25</td>
<td>0.418</td>
</tr>
</tbody>
</table>

Figure 5. Workability of fresh concrete.
low and high strength, high strength concrete attaining higher air content than the low strength. This is due to the different superplasticizers used for low and high strength concrete. Complast 111 superplasticizer was applied at 0.2% by weight of the cementitious material for high strength concrete. This superplasticizer aided the workability of concrete but also exhibited the air-entraining phenomena for high strength concrete as shown on Figure 6 (b).

### 3.4 Pozzolanicity

The pozzolanic activity of the fly ashes was determined in terms of strength activity index shown on Figure 6. UFFA pozzolanicity was higher than UK BS 3892 Part 1 and OPC. The seven UK fly ashes pozzolanic indexes are between the lower and upper bound zone. UFFA pozzolanic activity was enhanced by the ultra-fine particle size, high content of CaO, the low carbon content and it high reactivity compared to those of the seven UK fly ashes (as shown in Table 1 and 3).

### 3.5 Porosity

The porosity of the concrete specimens was determined after 28 days and 6 months of curing for the three different curing environments: E1, E2 and E3. The results are shown on Table 6. Porosity was factored with respect to the porosity attained by OPC concrete at 28 days of water cured (E1) samples for both low and high strength concrete. From the results we can deduce that the curing environment had a significant effect on the porosity of the cementation concrete. Water cured (E1) samples had the lowest porosity compared to both partially water cured then air-cured samples (E2) and air-cured samples (E3). The order of porosity improvement was $E1 > E2 > E3$.

The application of fly ash and MS lowered the porosity of the concrete compared to the control mixes. UFFA concrete had a lower porosity than both the UK BS 3892 Part I PFA and MS concrete. This may be due to the physical and chemical properties of UFFA over these other cementitious material.

Table 6. The typical factored porosity with respect to 28 days water cured OPC.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Curing Environment</th>
<th>Porosity of Concrete (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UK Fly ash</td>
<td>28 days</td>
</tr>
<tr>
<td></td>
<td>Upper bound</td>
<td>E1</td>
</tr>
<tr>
<td>OPC</td>
<td></td>
<td>1.00</td>
</tr>
<tr>
<td>UFFA05</td>
<td></td>
<td>0.96</td>
</tr>
<tr>
<td>UFFA10</td>
<td></td>
<td>0.92</td>
</tr>
<tr>
<td>UFFA30</td>
<td>UK Fly ash</td>
<td>0.62</td>
</tr>
<tr>
<td>UFFA50</td>
<td>Lower bound</td>
<td>0.59</td>
</tr>
<tr>
<td>UK BS 3892 Part 1 PFA 1 30</td>
<td></td>
<td>0.79</td>
</tr>
<tr>
<td>UK BS 3892 Part 1 PFA 2 30</td>
<td></td>
<td>0.89</td>
</tr>
<tr>
<td>MS05</td>
<td></td>
<td>0.90</td>
</tr>
<tr>
<td>MS10</td>
<td></td>
<td>0.87</td>
</tr>
</tbody>
</table>

E1: Water curing; E2: 3 days Water curing then Air curing and E3: Air curing.
Table 4 shows the improvement of concrete porosity due to the application of this cementation material. UFFA concrete improved the permeation of the overall concrete compared to the control. This improvement was indicated by the further reduction in porosity of the concrete with the increasing content of UFFA replacement level of OPC. 10% replacement level of OPC was in the same domain in improvement of the permeability of concrete as compared to 30% UK BS 3892 Part 1 PFA. Period of curing (from 28 to 6 months) also reduced the porosity of the concrete over the 3 curing environments. This improvement of concrete permeability due to the application of UFFA over both UK BS 3892 Part 1 PFA and MS. May be due to the due to the considerable pore refinement as a result of the addition of UFFA content and the formation of pozzolanic reaction products with the progress of cement hydration. The application of UFFA being more reactive than the UK BS 3892 Part 1 PFA would react with the calcium hydroxide in the water filled capillary channel to produce calcium silicate and aluminate hydrates of the same or similar type that are formed in normal hydration on cement. UFFA consumes more of the calcium hydroxide in the pozzolanic reaction compared to UK fly ash and 10% MS. The calcium hydroxide is converted to water insoluble hydration product, filling the capillaries, thereby reducing the permeability on the cementitious concrete and this is further reduced at 6 months of curing compared to 28 days of curing.

UFFA at 30% replacement level concrete cured in E2 environment at 28 days was better than the UK fly ashes cures in E1. The porosity of this concrete was exactly the same as that of 10% UFFA and both 5% and 10% MS concretes cured in E1 environment at 6 months.

Improvement in concrete porosity by UFFA compared to MS concretes may be said to be at the same range at 6 months at 5–10% replacement levels of OPC from the results in Table 4 in all the three curing environment.

4 CONCRETE COMPRESSIVE STRENGTH

The water contents for the mixes are given in Table 7. The incorporation of cementitious material resulted in

Table 7. Composition and the compressive strength of concretes.

<table>
<thead>
<tr>
<th>MIX CODE</th>
<th>OPC</th>
<th>UFFA</th>
<th>MS</th>
<th>Save</th>
<th>w</th>
<th>w/b</th>
<th>20 mm</th>
<th>15 mm</th>
<th>10 mm</th>
<th>5 mm</th>
<th>Sand</th>
<th>Age of testing (days)</th>
<th>Compressive strength (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC</td>
<td>522</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.23</td>
<td>770</td>
<td>0</td>
<td>340</td>
<td>0</td>
<td>778</td>
<td>7</td>
<td>84.6</td>
</tr>
<tr>
<td>UFFA/MS 1*</td>
<td>368</td>
<td>79</td>
<td>79</td>
<td>153</td>
<td>100</td>
<td>0.19</td>
<td>770</td>
<td>0</td>
<td>340</td>
<td>0</td>
<td>702</td>
<td>7</td>
<td>97</td>
</tr>
<tr>
<td>UFFA/MS 2**</td>
<td>368</td>
<td>79</td>
<td>79</td>
<td>153</td>
<td>100</td>
<td>0.19</td>
<td>0</td>
<td>770</td>
<td>340</td>
<td>0</td>
<td>702</td>
<td>7</td>
<td>98.4</td>
</tr>
<tr>
<td>UFFA/MS 3***</td>
<td>368</td>
<td>79</td>
<td>79</td>
<td>153</td>
<td>100</td>
<td>0.19</td>
<td>0</td>
<td>1110</td>
<td>0</td>
<td>156</td>
<td>546</td>
<td>7</td>
<td>102.5</td>
</tr>
</tbody>
</table>

Figure 7. Pozzolanicity of the pozzolans.
a reduction in water-to-binder ratio of the UFFA/MS concrete mixes. The results of strength are shown in the same table. The strength of UFFA/MS concrete is higher than those of OPC concrete. This strength development showed that crushed gravel is better than river gravel. Due to the river gravel shearing faster than the crushed gravels. A combination of 15 mm crushed gravel and 5 mm of sub sand resulted in the highest strength which was 129.5 N/mm² at 526 kg/m³ binder at 28 days of testing. This was higher than 118.5 N/mm² attained by Domone and Soutsos [1995] and Tsartsari [2002] at 28 days. The 5 mm coarse aggregates seemed to fill the voids between the coarse aggregates, a view that is supported by Kaplan in 1959 [Goltermann et al. 1997] who stated that compressive strength of concrete is higher than that of mortar and indicated that the mechanical interlocking of the coarse aggregate contributes to this. The binder of UFFA/MS had 153 kg/m³ cement saving.

5 CONCLUSION

This article dealt with the detailed study on UFFA compared to UK BS 3892 Part 1 PFA and MS, the finding of which may be summarized as follows.

UFFA has significantly higher specific surface and fineness than a range of UK PFAs tested.

UFFA improved the workability of concrete more than conventional PFAS tested and this effect tended to increase with the increasing replacement of cement with UFFA.

At the upper limits of conventional replacement level (40%) UFFA reduced water content by 23% whilst the best UK PFA tested reduced water demand by 11%.

The pozzolanicity of UFFA was higher when measured using standard mixes to BS3892 at all ages tested and the difference increased with time.

The application of UFFA reduced the porosity of concrete significantly, particularly with extended water curing.

The highest strength achieved was 129.5 N/mm² by a blend of UFFA and MS at a cement replacement level of 15%. The w/b ratio was 0.19 and the aggregates used were crushed gravel of maximum size 15 mm.

ACKNOWLEDGEMENT

I dedicate this paper to the late Dr Ewan A. Byars, for his helpful comments during the investigation. A vote of thanks goes to my parents, Sheffield University department of Civil and Structural Engineering and Kenya government for their moral support and sponsorships.

REFERENCES

British Standards Institution. BS 12 [1996].
British Standards Institution. BS 3892, Part 1 [1997].
British Standards Institution. BS 1881, Part 5 [1970].
British Standards Institution. BS EN 196-2 [1995].
British Standards Institution. BS EN 196-3 [1995].


Achieving sustainable construction through use of fly ash in concrete – An Indian experience

Manish Mokal
Hindustan Construction Co. Ltd., Hincon House, Vikhroli (W), L.B.S. Marg, Mumbai

ABSTRACT: Manufacturing of normal portland cement, a major ingredient in concrete industry contributes significantly to the CO₂ emissions. The paper mentions briefly the efforts on part of Hindustan Construction Company Ltd, the leading Indian construction company; to advocate and maximize the use of fly ash as a supplementary cementitious material in concrete. The paper includes case study on use of High Volume Fly Ash Concrete incorporating 55% fly ash at Bandra Worli Sea Link Project in Mumbai and Self Compacting Concrete incorporating 45% fly ash at Rajasthan Atomic Power Project near Kota.

Of the many technical benefits, the most significant benefit of use of fly ash in concrete is enhancement of durability of concrete, due to almost nil permeability, which in turn increases the service life of the concrete. The other technical benefits associated with use of fly ash in concrete are contribution to increase in workability, reduction in heat of hydration and better sulphate/chloride resistance. The utilization of fly ash also reduces the consumption of cement in concrete thereby reducing its production to that amount.

Fly Ash concrete is a classic example for attaining sustainability in concrete construction along with the benefits of increasing the durability and service life of the structure.

1 INTRODUCTION

Construction is a major economic activity. In India, it accounts for over 6% of Gross Domestic Product (GDP). It covers a wide range of activities from making simple dwelling units to construction of highly complex power stations, dams, bridges, roads & highways and so on. Inevitably, a construction project has to deal with environmental aspects. The threat of global warming is real. According to a report by Environmental Protection Agency (EPA) of the U.S.A. Green house gases are accumulating in the earth’s atmosphere as a result of human activities, causing global mean surface temperatures and sub-surface ocean temperatures to rise. The challenge for the civil engineering community in the near future will be to realize projects in harmony with the concept of sustainable development. This involves the use of high-performance materials produced at reasonable cost with the lowest possible environmental impact.

Traditionally, construction activities have relied on using normal portland cement. Production of one tonne of normal portland cement releases approximately one tonne of CO₂ into the atmosphere. India is the second largest producer of cement in the world. With the high emphasis given to cover the backlog in infrastructure construction by the government, the demand for cement is likely to rise sharply in coming years.

Using only normal portland cement in concrete has many problems in itself. Further, current demand of sustainable construction by reducing the environmental impact and increasing the durability of the structure cannot be achieved.

There are many supplementary cementitious materials (SCM) used in producing concrete, like, Fly Ash, Ground Granulated Blast Furnace Slag, Silica Fume, Rice-husk Ash. A conscious use of these materials, especially the abundantly available fly ash in India, would help conserve natural resources. It would also save energy required for producing normal portland cement, help the cause of environment, provide superior concrete structures and benefit the clients.

This paper discusses three cases studies on utilisation of fly ash in concrete.
2 CASE STUDY 1 – USE OF FLY ASH FOR TREMIE SEAL CONCRETE AT BANDRA-WORLI SEA LINK PROJECT

2.1 Project description

Bandra-Worli Sea Link Project is a prestigious project that envisages construction of 5.86 km long eight-lane freeway to be built across the sea. Most of the concrete specified is high strength, high performance type.

The distinctive features of the Bandra-Worli Sea Link Project are:

- Out of 5.86 km length, the length of bridge portion is 4.0 km including 500 m cable stayed portion.
- Two independent 4 lane Cable stayed bridge of 500 m length on single pylon of 126 m height.
- Two independent 4 lane Cable stayed bridge of 350 m length at Worli end.
- Bridge will be supported on friction piles. The piles will be drilled and concreted in the sea-bed, 8–10 m below the depth of water.
- Two curves on the sea link, for which different types of trapezoidal segments (in plan) are necessary.

2.2 Requirement of Tremie seal concrete

For concreting of the pile cap, a cofferdam was driven into the ground and the depth was filled with sand up to a level of 2 metres below the bottom of the pile cap. These 2 metres were filled with Tremie seal concrete of M30 grade. The basic purpose of this tremie concrete was to provide an impermeable working platform for the pile cap construction.

As per the project specification, the compressive strength requirement is 30 MPa at 28 days. As the concrete was required to be placed underwater, it necessitates the concrete to be cohesive enough with required workability to be placed with Tremie pipe/pump. The location of the placing point was in deep sea and the placing procedure for tremie seal concrete required a slump of 170 ± 10 mm after 3 hours. Initially the tremie seal concrete was designed with OPC only. Subsequently, use of fly ash concrete was proposed and after conducting lab and field trials, the mix was adopted in field.

2.3 Materials

2.3.1 Normal portland cement

Normal portland cement of 53 Grade conforming to IS 12269-1987 was selected for use in concrete.

2.3.2 Fly Ash

ASTM Class F Fly Ash conforming to requirement of ASTM C-618-00 from field 3 of Dahanu Thermal Power Plant was utilized after conducting necessary physical and chemicals tests.

Table 1. Mixture proportioning of conventional concrete & Fly Ash Concrete for Tremie Seal (BWSL).

<table>
<thead>
<tr>
<th>Ingredients (kg/m³)</th>
<th>Conventional concrete</th>
<th>Fly ash concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (OPC 53 Gr)</td>
<td>396</td>
<td>180</td>
</tr>
<tr>
<td>Fly Ash (Dahanu)</td>
<td>0</td>
<td>220 (55%)</td>
</tr>
<tr>
<td>Total Cementitious material</td>
<td>396</td>
<td>400</td>
</tr>
<tr>
<td>Water</td>
<td>178</td>
<td>135</td>
</tr>
<tr>
<td>W/cm ratio</td>
<td>0.45</td>
<td>0.34</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>905</td>
<td>1000</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>835</td>
<td>930</td>
</tr>
<tr>
<td>Admixture (HRWR)</td>
<td>5.5</td>
<td>10</td>
</tr>
</tbody>
</table>

2.3.3 Aggregate

Crushed basalt rock with a maximum size of 20 mm was used as coarse aggregate. Fine aggregate consisted of crushed sand manufactured from basalt rock and natural sand from River Ambika. The individual aggregates were blended to get the desired combined grading.

2.3.4 Chemical admixture

Sulfonated Naphthalene Formaldehyde based high-range water-reducing Admixture confirming to ASTM C494-99 Type G was used.

2.4 Mixture proportions

The calculated quantities per cubic metre of concrete for conventional mixture with normal portland cement only and mixture with fly ash are given in Table 1.

2.5 Field application

Concrete was produced in 90 m³/hr capacity batching & mixing plants situated onshore. The concrete after mixing at batching plant was discharged through conveyor into the transit mixers mounted on barges, which then hauled it to the placing point. The concrete after arrival at placing point was taken in buckets, which discharged them into concrete pumps located on jack up platform. The concrete was finally placed by means of a pump. A total 2000 cubic metre of tremie seal concrete was placed for the construction of this tremie seal under the pylon pile cap. Figure 1 shows the view of completed tremie seal concrete.

2.6 Field test results

The workability of concrete at placement point was determined using slump cone and flow table. At placing point the slump was 150 mm and flow was 150%. Required numbers of 150 mm cubes were cast for testing at various ages in order to study the strength
development behavior of this concrete. Samples were also cast to determine water permeability of concrete as per DIN 1048 & resistance to chloride ion penetration as per ASTM C1202. The test results are listed in Table 2. The rate of development of strength at different ages is shown in Figure 2.

2.7 Interpretation of test results

Table 2 shows the properties of hardened concrete for tremie seal concrete for only normal portland cement mixture and mixture with fly ash. Though the concrete was required to fulfill the 28 days criteria, study was conducted on both concrete mixes to judge the improvement of the performance in later ages. The following points confirm the beneficial effect of fly ash usage in concrete.

The fly ash concrete achieving 36.5 MPa at 28 days attained strength of 63.9 MPa after 91 days. Whereas the strength of normal portland cement concrete which achieved 45.1 MPa at 28 days increased only to 47.1 MPa in 91 days. Equivalent cube strength of cores extracted after 1 year from the actual structure indicated strength of 69.9 MPa. The calcium hydroxide produced during the initial cement hydration later reacts with the silica in fly ash to produce more C-S-H gel resulting in continuous gain in strength with time.

The permeability of fly ash concrete tested as per DIN 1048 indicated penetration of 42.5 mm after 28 days & was reduced to 0 mm penetration after 91 days as against 60 mm and 32 mm for OPC concrete. Production of additional C-S-H gel due to secondary hydration makes the concrete denser. This reduces the inter pore connectivity thereby reducing the permeability of concrete.

Specimens tested for chloride ion penetration as per ASTM C 1202 after 28 & 91 days indicated values of 2000 & 972 coulombs. These values indicate “low” & “very low” penetrability respectively. The RCPT values obtained for OPC concrete indicate 5500 & 3000 coulombs at 28 & 91 days which lie in high and moderate penetrability. The result obtained from the core extracted from the actual structure is 423 coulombs after 365 days. This indicates that concrete becomes progressively resistant to chloride ion penetration i.e. more durable with passage of time.

2.8 Conclusion

The tremie seal requires to withstand an uplift pressure of approx. 5 metres head. This requires the concrete to be impermeable to allow it to be used as a working platform for pile cap concrete. The tremie seal concrete made showed that the concrete was impermeable and it gave a virtually dry working platform as shown in Fig. 1 due to incorporation of Fly Ash.

Table 2. Hardened concrete properties of conventional concrete & Fly Ash Concrete for Tremie Seal (BWSL).

<table>
<thead>
<tr>
<th>Properties</th>
<th>Conventional concrete</th>
<th>Fly ash concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength (MPa)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 days</td>
<td>20.8</td>
<td>10.2</td>
</tr>
<tr>
<td>7 days</td>
<td>27.4</td>
<td>18.5</td>
</tr>
<tr>
<td>14 days</td>
<td>37.2</td>
<td>20.3</td>
</tr>
<tr>
<td>28 days</td>
<td>45.1</td>
<td>36.5</td>
</tr>
<tr>
<td>56 days</td>
<td>46.2</td>
<td>47.2</td>
</tr>
<tr>
<td>91 days</td>
<td>47.1</td>
<td>63.9</td>
</tr>
<tr>
<td>180 days</td>
<td>–</td>
<td>64.8</td>
</tr>
<tr>
<td>Equivalent cube strength of core @ 365 days</td>
<td>–</td>
<td>69.9</td>
</tr>
<tr>
<td>Durability Properties</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water Permeability @ 28 D</td>
<td>60</td>
<td>42.5</td>
</tr>
<tr>
<td>Water Permeability @ 91 D</td>
<td>32</td>
<td>0</td>
</tr>
<tr>
<td>RCPT @ 28 D</td>
<td>5500 coulombs</td>
<td>2000 coulombs</td>
</tr>
<tr>
<td>RCPT @ 91 D</td>
<td>3000 coulombs</td>
<td>972 coulombs</td>
</tr>
<tr>
<td>RCPT of cores @ 365 D</td>
<td>–</td>
<td>423 coulombs</td>
</tr>
</tbody>
</table>

Figure 1. Completed view of Tremie Seal Concrete with 55% fly ash.

Figure 2. Rate of gain of strength of conventional & Fly Ash concrete (FAC) for tremie seal.

133
3. CASE STUDY 2 – USE OF FLY ASH FOR STRUCTURAL MEMBERS AT RAJASTHAN ATOMIC POWER PROJECT

3.1 Project description

The project consists of two reactor units intended to generate 2 × 220 MW of electricity. The scope of work for HCC covers construction of Main plant buildings comprising of reactor building, reactor auxiliary, turbine and control buildings, tunnels and trenches and miscellaneous works.

The construction activities for above-mentioned structures involved are:

- Reinforced concrete works in raft, isolated footings, concrete blocks, equipment foundations, slab, beams, walls, hatches, hatch covers, pre-cast plank and trenches with M25 grade of concrete
- Prestressed concrete work in inner and outer containment wall and dome with M45 grade of concrete
- Heavy concrete work in calandria vault with integral carbon steel liner, fuelling machine service area, heavy concrete blocks and heavy grout
- Miscellaneous works.

3.2 Requirements of structural concrete

Use of fly ash was proposed in structural concrete. The grade of concrete was M25 and the target strength to be achieved was 32 MPa. Several trials were conducted with different percentages of fly ash replacements to check the suitability of use of fly ash in structural concrete. Initially concrete with 25% fly ash was approved. After checking the performance of 25% fly ash concrete, the concrete mix with 40% fly ash was approved and is being used at site. The mix with 40% fly ash indicated better long-term strength and water permeability results that led to taking decision for its use.

3.3 Materials

3.3.1 Normal portland cement

Normal portland cement of 43 Grade conforming to IS 8112-1989 was selected for use in concrete.

3.3.2 Fly ash

ASTM Class F Fly Ash conforming to requirement of ASTM C-618-99 from field 3 of Kota Thermal Power Plant was utilized after conducting necessary physical and chemicals tests.

3.3.3 Aggregate

Crushed quartzite rock with a maximum size of 20 mm was used as coarse aggregate. Fine aggregate consisted of crushed sand manufactured from quartzite rock and natural sand from River Jhalawar.

3.4 Mixture proportions field results

The calculated quantities per cubic metre of concrete along with fresh and hardened concrete properties are as given in Table 3.

The individual aggregates were blended to get the desired combined grading.

3.3.4 Chemical admixture

Sulfonated Naphthalene Formaldehyde based high-range water-reducing Admixture confirming to ASTM C494-99 Type G was used.

3.5 Interpretation of results

It was observed that the strength of 40% fly ash mixture increased from 37.8 MPa at 28 days to 48.6 MPa at 56 days. The rate of increase in strength for 40% fly ash mixture was quite appreciable with compressive strength values as high as 53.5 MPa at 91 days and 66.2 MPa at 180 days. The mixture with 0% fly ash had no appreciable increase in strength from 28 to 56 days. The permeability of mixture with 40% fly ash was 13 mm against 33 mm for mixture with 0% fly ash at 28 days. Progressive reduction in the RCPT value of 40% fly ash mixture was observed. Both the findings indicate that the fly ash concrete was more durable than the concrete without any fly ash.

3.6 Conclusion

The strength achieved with 0% fly ash mixture was 43.91 MPa against the required target strength of

Table 3. Mixture proportioning of conventional concrete & Fly Ash Concrete for structural members (RAPP).

<table>
<thead>
<tr>
<th>Ingredients (kg/m³)</th>
<th>Mix with 0% Fly Ash</th>
<th>Mix with 25% Fly Ash</th>
<th>Mix with 40% Fly Ash</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (OPC 43 Gr)</td>
<td>320</td>
<td>260</td>
<td>204</td>
</tr>
<tr>
<td>Fly Ash (Kota)</td>
<td>0</td>
<td>80</td>
<td>136 (40%)</td>
</tr>
<tr>
<td>W/Cm</td>
<td>0.5</td>
<td>0.47</td>
<td>0.45</td>
</tr>
<tr>
<td>Superplasticiser</td>
<td>5.2</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>Slump (mm)</td>
<td>120</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Compressive strength (MPa)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7 days</td>
<td>31.77</td>
<td>26.29</td>
<td>19.7</td>
</tr>
<tr>
<td>28 days</td>
<td>43.91</td>
<td>41.12</td>
<td>37.8</td>
</tr>
<tr>
<td>56 days</td>
<td>44.57</td>
<td>43.10</td>
<td>48.6</td>
</tr>
<tr>
<td>91 days</td>
<td>–</td>
<td>–</td>
<td>53.5</td>
</tr>
<tr>
<td>180 days</td>
<td>–</td>
<td>–</td>
<td>66.2</td>
</tr>
<tr>
<td>Permeability (mm)</td>
<td>33</td>
<td>22</td>
<td>13</td>
</tr>
<tr>
<td>RCPT @ 28 days</td>
<td>–</td>
<td>–</td>
<td>1958</td>
</tr>
<tr>
<td>RCPT @ 91 days</td>
<td>–</td>
<td>–</td>
<td>855</td>
</tr>
</tbody>
</table>
32 MPa. Though there was a scope to economise the mix, the same could not be done due to the restriction of minimum cement content and maximum water cement ratio. The mix with 40% fly ash not only satisfied the required target strength but also indicated better durability. Using fly ash as replacement of cement helps in economizing the concrete mixture and achieving better durability.

4 CASE STUDY 3 – USE OF FLY ASH FOR SELF-COMPACTING CONCRETE (SCC) FOR COLUMNS, BEAMS AND OTHER STRUCTURAL ELEMENTS IN RAJASTHAN ATOMIC POWER PROJECT

4.1 Requirements of SCC

The columns, beams and other structural elements in the nuclear power plant have dense reinforcement. Due to this lot of honeycombing was being observed after deshuttering. SCC was proposed as a solution to rectify the problem. The grade of concrete was M25 and target strength to be achieved was 32 MPa. The concreting methodology required concrete to satisfy self-compacting properties after 1 hour.

4.2 Material

The raw material used was same as that for structural concrete except for the admixtures. For achieving self-compacting properties in fresh state a combination of three different types of admixtures were used. A polycarboxylate ether based admixture was used to achieve the workability, retarder to achieve the required retention and a viscosity-modifying agent to achieve the required viscosity.

4.3 Mixture proportions and test results

Initial mixture proportions were taken as recommended in the Guidelines for Self Compacting Concrete as given in EFNARC. Slight modifications were made to incorporate available local materials. Accordingly, various laboratory trials were conducted to arrive at an optimum combination of materials for achieving desired fresh and hardened state properties. The details of the final mixture proportions are as given in table 4.

4.4 Conclusion

The columns and beams in the nuclear power plant are heavily congested with reinforcement making it difficult to vibrate the concrete properly. Use of SCC worked as an ideal solution to build a structure free of honeycombing. Use of fly ash in concrete enhanced the rheology of concrete making it more cohesive and flowable. With conventional concrete the normal lift of column is 3 m. With the use of SCC it was able to do concrete of 10 m height in single lift. This increases the progress of work.
5 GENERAL REMARKS

The above case studies prove the utility of fly ash for different types of concrete applications. If the same concrete had been done with only OPC, amount of cement consumption would have been much higher. For the above applications HCC has reduced cement consumption by 7500 MT. Using fly ash in concrete not only improves the durability of the structure in a cost effective manner but also helps in conserving the natural resources and reduces the green house gases.

REFERENCES

ACI 211.1-97, Standard Practice for selecting Proportion of Normal, heavy weight and Mass Concrete
ACI 304.2 R-96, Placing concrete by pumping methods
ACI 211.3 R, Guide for selecting proportions for No Slump Concrete
V. M. Malhotra, High Volume Fly Ash Concrete
EFNARC, Specification and Guidelines for Self-Compacting Concrete
1 INTRODUCTION

The fundamental objective of this research project was to develop mixture proportioning information and field construction experience with a high recycled content in-situ mixed roller compacted concrete for a 1.416 hectare [3.5 acre] bottom ash and synthetic gypsum outdoor storage pad (Figure 1). The Class C fly ash and bottom ash used in this project were produced at We Energies' Pleasant Prairie Power Plant located in Kenosha County, Wisconsin. Ingredients of the concrete produced consisted of recycled concrete coarse aggregate, bottom ash fine aggregate, Class C fly ash and either Portland cement or slag cement yielding a 93% or 100% recycled content, respectively. Compressive strengths attained 21 MPa [3,100 psi] for the Portland cement/Class C fly ash blend at an age of 1 year and 12–14 MPa [1,700–2,000 psi] for the Slag cement/Class C fly ash blend.
The eco-pad will be monitored for performance to determine if it is a long term solution for large storage areas such as manufacturing and commercial warehousing storage yards, terminals, dockyards, and other large pavements that do not require a hard troweled finish.

1.1 Objectives and scope

The primary objective of this research project was to develop a “green” alternative paving material and construction process for environmentally friendly large outdoor storage area pavements. Ninety Three to 100% recycled content was achieved for the pavement and long term performance is being evaluated of this potentially sustainable process. The project was changed during construction where slag cement was substituted in place of the portland cement after the first day of paving due to a shortage of available portland cement and the late time of the Wisconsin construction season. This field change provided the additional 100% recycled content dimension of this research study.

1.2 Relevant publications

A literature search was conducted and rather than compiling an exhaustive review of roller compacted concrete and high volume concrete literature, some important publications were reviewed and are listed in the references.

2 LABORATORY TESTING

A laboratory mix analysis was performed to determine the mixture proportioning. For this particular mix analysis, the effectiveness of the proposed cementitious (portland cement and Class C fly ash mixture) and aggregate (recycled concrete and bottom ash) materials were assessed by evaluating the materials grainsize analysis, aggregate blending, moisture-density relationship, and moisture-strength relationships.

Representative samples of the proposed materials which included recycled concrete from Racine County (which is located in southeast Wisconsin), bottom ash and Class C fly ash from We Energies Pleasant Prairie...
Power Plant, and portland cement from LaFarge were obtained. Laboratory testing of the samples included moisture content testing (ASTM D2216), grain size analysis (ASTM D422), moisture density relationship by modified proctor method (ASTM D1557), and compressive strength analysis (ASTM D1633). The moisture density relationship for each mixture was performed in general accordance with ASTM D1557, except that the five lift requirement was replaced with three lifts. A 102 mm [4 in] diameter split mold was used for compressive strength determination per ASTM D1633 to facilitate the removal of each specimen with minimal disturbance to the samples. Upon completion, the specimens were sealed in plastic bags and cured for seven (7) days at 100°F in accordance with ASTM C-593 to approximate conditions of 28-day cure periods. Following curing, the samples were capped with a gypsum cap and the compressive strength was determined using a constant drive calibrated load frame. The test specimens have a height to diameter ratio of 1.15:1; therefore, the strengths obtained provide a relative measure of strength rather than the measure of compressive strength such as would be obtained for specimens having a height to diameter ratio of 2:1. In general 1.15 height to diameter ratio samples will produce a slightly higher strength than that achieved with a height to diameter ratio of 2.

The preliminary mixture proportioning testing was performed in two (2) phases. Initially, samples of the proposed recycled concrete (coarse aggregate) and the bottom ash (fine aggregate) were tested to determine their optimum blend for grainsize distribution and density. The second phase consisted of mixing the selected aggregate blend with varying amounts of the blended cementitious binder material for determination of the mixture’s optimum density and strength characteristics.

3 LABORATORY TESTING RESULTS

Evaluation of the proposed in-situ mixed roller compacted recycled concrete mixture was performed based on coarse and fine aggregate consisting of 38 mm [1½ in] top size recycled concrete from Payne and Dolan in Racine, Wisconsin and bottom ash from We Energies Pleasant Prairie Power Plant (P4). The blended cementitious material consisted of 50% portland cement from LaFarge and 50% Class C fly ash from We Energies P4.

Based on the grain size analysis of the proposed recycled aggregate, the coarse aggregate is described as a poorly to well-graded crushed concrete with about 48–67% gravel, 31–45% sand, and 2.6–6.6% silt/clay sized particles. The initial sample of recycled concrete was somewhat more open graded with limited fines compared to the more densely graded samples supplied during the second phase of the laboratory and field test. This was also evident in the aggregate blends, Moisture Density, and Moisture Strength test results. The fine aggregate is described as a bottom ash with about 4–6% gravel, 77–85% sand, and 13–17% silt/clay sized particles. Results of the unit weight tests on the coarse and fine aggregate indicated loose unit weights of 1584 to 1682 kg/m3 [99 to 105 pcf] and 1041 kg/m3 [65 pcf], respectively. Grain size analysis test results are shown on Table 1 and Figure 2.

Results of the coarse/fine aggregate blends 50/50, 60/40, 70/30, and 80/20 generally indicate a poorly graded aggregate with about 35–54% gravel, 41–57% sand, and 5–8% silt/clay blends producing the higher densities. Based on the blended aggregate test results the 60/40 sized particles. The compacted unit weights of the blends ranged from 1646.9 to 1747.8 kg/m3 recycled concrete/bottom ash blend was selected for

| Table 1. Summary of grain size analysis tests recycled aggregates Eco-pad pavement MES Project No. 7-45151. |
|---|---|---|---|---|
| Sample no. | Bottom ash | Recycled concrete |
| Grainsize analysis, mm [in/ sieve #] | BA1 | BA2 | RC1 | RC2 | RC3 (field) |
| 38.1 [1½] | – | – | 100 | – | – |
| 25.4 [1] | – | – | 95.7 | 100 | 100 |
| 19.05 [3/4] | 100 | 100 | 85.0 | 93.4 | 94.2 |
| 12.7 [¼] | 99.0 | 98.5 | 65.2 | 75.5 | 79.1 |
| 9.5 [3/8] | 98.1 | 96.7 | 54.6 | 66.1 | 69.4 |
| 4.76 [#4] | 95.6 | 94.0 | 33.5 | 48.4 | 51.8 |
| 2.0 [#10] | 92.3 | 91.2 | 20.3 | 37.1 | 35.8 |
| 0.84 [#20] | 83.5 | 83.8 | 12.1 | 27.9 | 24.2 |
| 0.42 [#40] | 67.0 | 70.1 | 7.5 | 20.6 | 17.1 |
| 0.147 [#100] | 26.7 | 36.1 | 3.5 | 9.4 | 8.7 |
| 0.074 [#200] | 12.6 | 16.7 | 2.6 | 6.6 | 6.6 |
the moisture-density and moisture-[102.8 to 109.1 pcf] with the 60/40 and 70/30 strength relationship testing. Table 2 provides a summary of the aggregate trial blending tests.

4 CEMENTITIOUS MATERIALS

Chemical and physical data available on the cementitious materials used on this project are shown on Table 3. The Class C fly ash was supplied from We Energies Pleasant Prairie Power Plant which is also the site of the eco-pad construction project. The slag cement and portland cement were both provided by Lafarge NA. The cementitious materials were preblended at Lafarge’s terminal located in Milwaukee, Wisconsin.

The 1646.9 to 1747.8 kg/m3 [102.8 to 109.1 pcf] with the 60/40 and 70/30 blends produced the higher densities as shown on Table 1. Based on the blended aggregate test results, the 60/40 recycled concrete/bottom ash blend was selected for the moisture-density and moisture-strength relationship testing.

Evaluation of the compression strength potential of the preliminary selected aggregate blend 60/40 included performing moisture-density and moisture-strength relationship testing by modified Proctor method as previously described. Results of the moisture-density and moisture-strength test results are summarized in Table 4.

The test results indicated that the 18% cementitious blend content exhibited the highest compressive strength. However, since the increase in strength from the 15–18% specimen was less than 0.03 MPa [5 psi], the 15% cementitious blend content was selected for economic and sustainability reasons on the project.

Additional tests were also performed with varying aggregate blends when the second sample of recycled concrete showed a somewhat denser gradation. Therefore, aggregate blends of 50/50, 60/40 and 70/30 recycled concrete/bottom ash were mixed with a constant 15% cementitious blend content to determine their moisture-density and moisture-strength relationship. Results of the tests are summarized in Tables 5 and 6.

Results of the moisture-density and moisture-strength tests indicate the 70/30 aggregate blend with 15% blended cementitious content exhibited the higher strength and density. This is likely due to the material’s denser graded nature which allows for a more compact arrangement of particles which also

---

Table 2. Summary of aggregate trial blending tests.

<table>
<thead>
<tr>
<th>Sample no. grainsize analysis, mm [in/sieve #]</th>
<th>BA1</th>
<th>RC1</th>
<th>50/50</th>
<th>60/40</th>
<th>70/30</th>
<th>80/20</th>
</tr>
</thead>
<tbody>
<tr>
<td>38.1 [1½]</td>
<td></td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>25.4 [1]</td>
<td></td>
<td>95.7</td>
<td>97.6</td>
<td>97.4</td>
<td>97.0</td>
<td>96.6</td>
</tr>
<tr>
<td>19.05 [¾]</td>
<td>100</td>
<td>85.0</td>
<td>92.5</td>
<td>91.0</td>
<td>89.5</td>
<td>88.0</td>
</tr>
<tr>
<td>12.7 [⅜]</td>
<td>99.0</td>
<td>65.2</td>
<td>82.1</td>
<td>78.7</td>
<td>75.3</td>
<td>72.0</td>
</tr>
<tr>
<td>9.5 [⅝]</td>
<td>98.1</td>
<td>54.6</td>
<td>76.4</td>
<td>72.0</td>
<td>67.7</td>
<td>63.3</td>
</tr>
<tr>
<td>4.76 [#4]</td>
<td>95.6</td>
<td>33.5</td>
<td>64.6</td>
<td>58.3</td>
<td>52.1</td>
<td>45.9</td>
</tr>
<tr>
<td>2.0 [#10]</td>
<td>92.3</td>
<td>20.3</td>
<td>56.3</td>
<td>49.1</td>
<td>41.9</td>
<td>74.7</td>
</tr>
<tr>
<td>0.84 [#20]</td>
<td>83.5</td>
<td>12.1</td>
<td>47.8</td>
<td>40.7</td>
<td>33.5</td>
<td>26.4</td>
</tr>
<tr>
<td>0.42 [#40]</td>
<td>67.0</td>
<td>7.5</td>
<td>37.3</td>
<td>31.3</td>
<td>25.4</td>
<td>19.4</td>
</tr>
<tr>
<td>0.147 [#100]</td>
<td>26.7</td>
<td>3.5</td>
<td>15.1</td>
<td>12.8</td>
<td>10.5</td>
<td>8.1</td>
</tr>
<tr>
<td>0.074 [#200]</td>
<td>12.6</td>
<td>2.6</td>
<td>7.6</td>
<td>6.6</td>
<td>5.6</td>
<td>4.6</td>
</tr>
<tr>
<td>USCS</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SM</td>
<td></td>
<td></td>
<td>SP-SM</td>
<td>SP-SM</td>
<td>GP-GM</td>
<td>GP</td>
</tr>
<tr>
<td>GW</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unit Weight (Loose), kg/m³ [pcf]</td>
<td>1041.3</td>
<td>1586</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Unit Weight (Compacted), kg/m³ [pcf]</td>
<td></td>
<td></td>
<td>1646.9</td>
<td>1744.6</td>
<td>1747.8</td>
<td>1733.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>[102.8]</td>
<td>[108.9]</td>
<td>[109.1]</td>
<td>[108.2]</td>
</tr>
</tbody>
</table>

- Compacted unit weight is based on modified proctor method (ASTM D1557) at moisture content of 15%.
- Loose unit weight is based on as received moisture content.
- Aggregate blends are based on initial bottom ash and recycled concrete samples submitted.
Table 3. Selected chemicals and physical composition of cementitious materials.

<table>
<thead>
<tr>
<th>Chemical data</th>
<th>Class C fly ash</th>
<th>Slag cement</th>
<th>Portland cement</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂ (silicon dioxide)%</td>
<td>40.35</td>
<td>35.68</td>
<td>20.7</td>
</tr>
<tr>
<td>Al₂O₃ (aluminum oxide)%</td>
<td>18.94</td>
<td>10.00</td>
<td>4.8</td>
</tr>
<tr>
<td>Fe₂O₃ (iron oxide)%</td>
<td>5.18</td>
<td>0.58</td>
<td>2.7</td>
</tr>
<tr>
<td>SiO₂ + Al₂O₃ + Fe₂O₃ %</td>
<td>64.47</td>
<td>46.26</td>
<td>28.2</td>
</tr>
<tr>
<td>CaO (calcium oxide)%</td>
<td>21.65</td>
<td>38.62</td>
<td>65.4</td>
</tr>
<tr>
<td>MgO (magnesium oxide)%</td>
<td>3.81</td>
<td>11.2</td>
<td>2.5</td>
</tr>
<tr>
<td>SO₃ (sulfur trioxide)%</td>
<td>1.93</td>
<td>2.41</td>
<td>2.4</td>
</tr>
<tr>
<td>LOI (loss on ignition)%</td>
<td>0.45</td>
<td>–</td>
<td>1.61</td>
</tr>
<tr>
<td>Na₂O (sodium oxide)%</td>
<td>1.78</td>
<td>0.35</td>
<td>–</td>
</tr>
<tr>
<td>K₂O (potassium oxide)%</td>
<td>1.25</td>
<td>0.42</td>
<td>–</td>
</tr>
<tr>
<td>Available Alkalis (as equivalent Na₂O%)</td>
<td>1.3</td>
<td>–</td>
<td>0.53</td>
</tr>
</tbody>
</table>

Physical data

| Fineness, amount retained on #325 sieve%  | 13.6           | 1.0         | 5.3             |
| Specific gravity                         | 2.52           | 2.95        | 3.15            |

Table 4. Summary of density/strength tests of 60/40 RC/BA aggregate blend with varying cement contents.

<table>
<thead>
<tr>
<th>Mixture ID</th>
<th>Moisture-density relationship</th>
<th>Moisture-strength relationship</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Optimum moisture %</td>
<td>Maximum dry density, kg/m³ [pcf]</td>
</tr>
<tr>
<td>60/40 RC/BA @12% PC/FA</td>
<td>15.5</td>
<td>1762.2 [110.0]</td>
</tr>
<tr>
<td>60/40 RC/BA @15% PC/FA</td>
<td>16.5</td>
<td>1762.2 [110.0]</td>
</tr>
<tr>
<td>60/40 RC/BA @18% PC/FA</td>
<td>14.5</td>
<td>1778.2 [111.0]</td>
</tr>
</tbody>
</table>

Table 5. Summary of aggregate trial blending tests.

<table>
<thead>
<tr>
<th>Sample no. grainsize, mm [in/sieve #]</th>
<th>BA2</th>
<th>RC2</th>
<th>50/50</th>
<th>60/40</th>
<th>70/30</th>
</tr>
</thead>
<tbody>
<tr>
<td>38.1 [1/2]</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>25.4 [1]</td>
<td>–</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>19.05 [5/8]</td>
<td>100</td>
<td>94.2</td>
<td>96.7</td>
<td>96.0</td>
<td>95.4</td>
</tr>
<tr>
<td>12.7 [3/8]</td>
<td>98.5</td>
<td>79.1</td>
<td>87.0</td>
<td>84.7</td>
<td>82.4</td>
</tr>
<tr>
<td>9.5 [5/16]</td>
<td>96.7</td>
<td>69.4</td>
<td>81.4</td>
<td>78.3</td>
<td>75.3</td>
</tr>
<tr>
<td>4.76 [#/4]</td>
<td>94.0</td>
<td>51.8</td>
<td>71.2</td>
<td>66.6</td>
<td>62.1</td>
</tr>
<tr>
<td>2.0 [#/10]</td>
<td>91.2</td>
<td>35.8</td>
<td>64.2</td>
<td>58.7</td>
<td>53.3</td>
</tr>
<tr>
<td>0.84 [#/20]</td>
<td>83.8</td>
<td>24.2</td>
<td>55.9</td>
<td>50.3</td>
<td>44.7</td>
</tr>
<tr>
<td>0.42 [#/40]</td>
<td>70.1</td>
<td>17.1</td>
<td>45.4</td>
<td>40.4</td>
<td>35.5</td>
</tr>
<tr>
<td>0.147 [#/100]</td>
<td>36.1</td>
<td>8.7</td>
<td>22.8</td>
<td>20.1</td>
<td>17.4</td>
</tr>
<tr>
<td>0.074 [#/200]</td>
<td>16.7</td>
<td>6.6</td>
<td>11.7</td>
<td>10.6</td>
<td>9.6</td>
</tr>
<tr>
<td>USCS SM</td>
<td>6P-6M</td>
<td>SP-SM</td>
<td>SP-SM</td>
<td>SP-SM</td>
<td>SP-SM</td>
</tr>
<tr>
<td>Unit Weight (Loose), kg/m³ [pcf]</td>
<td>1041.3</td>
<td>1682.1</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

- Loose unit weight is based on as received moisture content.
- Aggregate blends are based on second set of Bottom Ash and Recycled Concrete samples submitted.
produce a higher density. Higher density also generally produces higher strengths.

Subsequently, it was elected to construct the field test pavement with a 127 mm [5 in] and 76 mm [3 in] thick recycled concrete/bottom ash aggregate blend and a 15% blended cementitious content. Construction of the pavement would consist of placing 76 mm [3 in] of bottom ash and 127 mm [5 in] of recycled concrete prior to pre-blending. This ratio by volume was expected to be similar to the 70/30 ratio by weight performed in the preliminary lab testing phase and also reflected the practical level of precision in construction.

5 TEST PAVEMENT CONSTRUCTION
OVERVIEW

Based on the laboratory test data, an in-situ mixing process was developed and a test pavement constructed. A 305 mm [12 in] thick compacted bottom ash base grade was established for the proposed eco-pad test pavement. The in-situ mixing phase consisted of placing 76 mm [3 in] thickness of bottom ash across the proposed pavement area. Then 127 mm [5 in] of recycled concrete was placed over the bottom ash. The recycled concrete and bottom ash were then pre-mixed to a depth of 203 mm [8 in] with an asphalt reclaimer. The cementitious materials supplier provided a 50/50 blend of portland cement and Class C fly ash that was placed dry with a vane feeder spreading truck over the previously mixed aggregate blend. The cementitious material blend was spread at a rate of 59.7 kg/m² [110 pounds per square yard]. This rate was based on a 15% dry unit weight basis of the maximum dry density of the laboratory blended mixture. Moisture conditioning (addition of water) was not generally required on this project due to relatively wet site conditions experienced due to a rainy period prior to mixing. After mixing, the aggregate and cementitious materials mixture was compacted with a large vibratory sheepsfoot compactor, graded, and final rolling was accomplished with a smooth drum roller. A recommended mixture moisture content of ±2% for optimum strength (9 to 13%) was recommended along with a maximum delay period from mixing of the cementitious material to compaction. Compaction of the in-situ mixture was specified at 95 percent of the maximum dry density as determined by the modified Proctor method.

Climate conditions during the in-situ mixing were challenging during construction. Due to a regional cement shortage, construction was delayed into late October and early November when temperatures were cold in Wisconsin. This was complicated by a rainy period that made obtaining optimum moisture content for compaction a challenge at the beginning of the project. A bottleneck in construction operation was the rate at which the cementitious material could be blended at the terminal and delivered to the project. An operating issue at the cement terminal also threatened to delay the project because cement could not be unloaded. However, ground granulated slag cement was available at the terminal and substituted for portland cement, thus adding another interesting dimension to this project.

5.1 Concrete mixing and construction process

In summary, an in-situ mixing process was developed for use on this project. Once the 305 mm [12 in] thick bottom ash base grade was established, an additional 76 mm [3 in] thick layer of bottom ash was spread with dump trucks and a road grader over the pad. Next, a 127 mm [5 in] thick layer of recycled concrete coarse aggregate was placed over the bottom ash. An asphalt reclaimer was used to mix the aggregates in place, followed by a road grader and compacted. Due to a rainy period, additional moisture was not required. The cementitious materials supplier provided a pre-blended 50/50 class C fly ash and portland cement mixture that was placed dry with a vane feeder spreader truck over the compacted aggregate surface. An asphalt reclaimer followed to mix the cementitious and aggregate materials. Water can be added through the reclaimer, if needed, to obtain optimum moisture conditions for compaction. 95% modified proctor densities were then achieved with the use of a sheepsfoot compactor followed by a vibratory smooth drum roller and an asphalt-like finish texture was obtained. Saw cuts on a 6 m [20 ft] grid

<table>
<thead>
<tr>
<th>Mixture Id</th>
<th>Optimum moisture %</th>
<th>Maximum dry density, kg/m³ [pcf]</th>
<th>Optimum moisture %</th>
<th>Maximum strength, MPa [psi]</th>
</tr>
</thead>
<tbody>
<tr>
<td>60/40 RC/BA @12% PC/FA</td>
<td>13.5</td>
<td>1770 [110.5]</td>
<td>12.6</td>
<td>10 [1460]</td>
</tr>
<tr>
<td>60/40 RC/BA @15% PC/FA</td>
<td>12.5</td>
<td>1842 [115.0]</td>
<td>12.4</td>
<td>13 [1880]</td>
</tr>
<tr>
<td>60/40 RC/BA @18% PC/FA</td>
<td>10.5</td>
<td>1922 [120.0]</td>
<td>11.2</td>
<td>13.2 [1920]</td>
</tr>
</tbody>
</table>
followed the next day. A curing compound was applied following the saw cuts and finally an elastomeric joint filler was applied to seal the saw cut joints. Climate conditions were challenging during construction. Due to a regional cement shortage, construction was delayed into late October and early November, when temperatures are beginning to get cold in Wisconsin. This was complicated by a rainy period that made attaining an optimum moisture content for compaction a challenge at the beginning of the project. The bottleneck in the construction operation was the rate at which the cementitious materials could be blended at the terminal and delivered to the project. An operating issue at the cement terminal also threatened to further delay the project because cement could not be unloaded. However, ground granulated blast furnace slag cement was available at the terminal and substituted for the portland cement thus adding another interesting dimension to this research project. A call to the slag cement association indicated that they were not aware of a prior use of a 50/50 fly ash/slag cementitious blend without portland cement being used in construction. The slag and class C fly ash binder pavement combined with the recycled aggregates provided a 100% recycled material content in approximately two thirds of the pavement area.

6 FIELD TESTING

Field testing of the materials used in the construction of the eco-pad test pavement were performed in three stages. The initial stage consisted of performing grain-size analysis on samples of the field blended aggregates. A laboratory mixture analysis of the field aggregate blend with 15% of the blended cementitious material was also performed to establish laboratory moisture-density and moisture-strength relationships. The second stage of the testing was performed during the field mixing of the blended aggregate and cementitious materials. The field testing consisted of performing field density testing by nuclear gauge method (ASTM D2922) during the compaction phase to assess the in-situ moisture content and percent compaction achieved. In addition, samples of the in-situ mixture were obtained and compacted in the field by the modified Proctor method. The field molded specimens were delivered to the laboratory and cured for a period of 7 to 365 days to assess the compressive strength development of the mixture.

The final phase of the testing included obtaining in-situ core specimens after approximately one year to assess the in-place strength of the in-situ recycled concrete mixture. The cores were obtained with a rotary type drill with a diamond impregnated core barrel in general accordance with ASTM D42. Samples were subsequently air dried for 7 days, capped with a gypsum capping compound and compressive strengths were determined in accordance with ASTM C39.

7 RESULTS OF FIELD TESTING

Evaluation of the in-situ recycled concrete mixture constructed in the eco-pad pavement was based on the 127 mm [5 in] recycled concrete to a 76 mm [3 in] bottom ash aggregate blend and 15% blended cementitious (50% portland cement/50% Class C fly ash) material or 15% blended (50% slag cement/50% Class C flyash) material.

Results of the grainsize analysis of the individual bottom ash and recycled concrete samples used on-site indicated gradations similar to the results obtained in the laboratory testing phase. Results of the field blended aggregate samples also are similar to those obtained in the laboratory test phase. Results of the grainsize analysis of the lab and field aggregate blends are summarized in Table 7.

Results of the moisture density relationship testing indicated a higher maximum dry density at about the same optimum moisture content as in the preliminary laboratory mix proportioning phase. This is likely due to a well-graded sample resulting in a more densely compacted mixture. The higher result in the compressive strength may also be due to the higher density characteristics and lower optimum moisture contents. Subsequently, two (2) additional samples of the previously sampled and combined field blended aggregate were mixed in the lab, one (1) with 15% PC/FA and the other with 15% SC/FA cementitious blend to further assess the moisture-density and moisture-strength relationships. Results of the tests on the PC/FA blend showed similar results to those of the PC/FA blend of the first aggregate field blend mixture. Results of the SC/FA cementitious blend also provided results that were similar to those of the first aggregate field blend mixture. Results of the SC/FA cementitious blend resulted with similar moisture density relationships but with lower strengths, 11 MPa [1600 psi] vs. 15.3 and 18.6 MPa [2225 and 2700 psi]. This is likely due to the fact that the slag cement contained less CaO and also generally develops its strength at a slower rate than portland cement. The second phase of the field testing included performing field moisture and density testing during the placement and compaction phase of the construction. In summary, the field blended aggregate had moisture contents initially of 14 to 19 percent, which was 0 to 5 percent above their recommended mixing range of 9 to 13.

However, during the mixing process the moisture contents were generally found to range from 10 to 16 percent based on the in-place field density testing. The field density testing also indicated an in-place
Table 7. Comparison of aggregate blend gradations.

<table>
<thead>
<tr>
<th>Aggregate blend</th>
<th>Grainsize analysis, mm [in/sieve#]</th>
<th>60/40 blend</th>
<th>70/30 blend</th>
<th>Field blend</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC/BA</td>
<td>38.1 [1½]</td>
<td>100</td>
<td>100</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>25.4 [1]</td>
<td>97.4</td>
<td>97</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>19.05 [%]</td>
<td>91.0</td>
<td>89.5</td>
<td>96</td>
</tr>
<tr>
<td></td>
<td>12.7 [%]</td>
<td>78.7</td>
<td>75.3</td>
<td>84.7</td>
</tr>
<tr>
<td></td>
<td>9.5 [3/8]</td>
<td>72.0</td>
<td>67.7</td>
<td>78.3</td>
</tr>
<tr>
<td></td>
<td>4.76 [#4]</td>
<td>58.3</td>
<td>52.1</td>
<td>66.6</td>
</tr>
<tr>
<td></td>
<td>2.0 [#10]</td>
<td>49.1</td>
<td>41.9</td>
<td>58.7</td>
</tr>
<tr>
<td></td>
<td>0.84 [#20]</td>
<td>40.7</td>
<td>33.5</td>
<td>50.3</td>
</tr>
<tr>
<td></td>
<td>0.42 [#40]</td>
<td>31.3</td>
<td>25.4</td>
<td>40.4</td>
</tr>
<tr>
<td></td>
<td>0.147 [#100]</td>
<td>12.8</td>
<td>10.5</td>
<td>20.1</td>
</tr>
<tr>
<td></td>
<td>0.074 [#200]</td>
<td>6.6</td>
<td>5.6</td>
<td>10.6</td>
</tr>
</tbody>
</table>

Table 8. Summary of density/strength tests based on field blended aggregate samples.

<table>
<thead>
<tr>
<th>Mixture ID</th>
<th>Moisture-density relationship</th>
<th>Moisture-strength relationship</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Optimum moisture %</td>
<td>Maximum dry density, kg/m³ [pcf]</td>
</tr>
<tr>
<td>60/40 RC/BA @12% PC/FA</td>
<td>10.5</td>
<td>1986.5 [124.0]</td>
</tr>
<tr>
<td>60/40 RC/BA @15% PC/FA</td>
<td>10.0</td>
<td>1986.5 [124.0]</td>
</tr>
<tr>
<td>60/40 RC/BA @18% PC/FA</td>
<td>11.0</td>
<td>1954.4 [122.0]</td>
</tr>
</tbody>
</table>

Table 9. Summary of field compressive strength test results.

<table>
<thead>
<tr>
<th>Sample no.</th>
<th>FS-1</th>
<th>FS-2</th>
<th>FS-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample date</td>
<td>11-04-04</td>
<td>11-05-04</td>
<td>11-08-04</td>
</tr>
<tr>
<td>Continuous blend</td>
<td>PC/FA</td>
<td>SC/FA</td>
<td>SC/FA</td>
</tr>
<tr>
<td>Field molded samples</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moisture content, %</td>
<td>13.6</td>
<td>14.9</td>
<td>13.1</td>
</tr>
<tr>
<td>Dry Density, kg/m³ [pcf]</td>
<td>1941.6–1956 [121.2–122.1]</td>
<td>1919.2–1948 [119.8–121.6]</td>
<td>1933.6–1954.4 [120.7–122]</td>
</tr>
<tr>
<td>Compaction, %</td>
<td>99–100</td>
<td>96–98</td>
<td>97–98</td>
</tr>
<tr>
<td>Compressive strength, MPa [psi]</td>
<td>11.1 [1620]</td>
<td>1.3 [185]</td>
<td>1.0 [145]</td>
</tr>
<tr>
<td>7 day (air)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7 day (oven)</td>
<td>16.8 [2440]</td>
<td>1.3 [195]</td>
<td>1.2 [175]</td>
</tr>
<tr>
<td>56 day (air)</td>
<td>17.4 [2525]</td>
<td>1.8 [265]</td>
<td>1.7 [240]</td>
</tr>
<tr>
<td>180 day (air)</td>
<td>22.6 [3280]</td>
<td>6.8 [985]</td>
<td>6.2 [900]</td>
</tr>
<tr>
<td>365 day (air)</td>
<td>29.9 [4325]</td>
<td>18.4 [2675]</td>
<td>6.9 [2455]</td>
</tr>
<tr>
<td>365 day (corrected)*</td>
<td>27.1 [3930]</td>
<td>16.8 [2435]</td>
<td>15.4 [2235]</td>
</tr>
<tr>
<td>Field core samples</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>365-day (air)</td>
<td>21.7 [3150]</td>
<td>13.6 [1970]</td>
<td>12.0 [1735]</td>
</tr>
</tbody>
</table>

Note: The molded field samples have a height to diameter ratio of 1.15:1 compared to the length to diameter ratio of 2:1 for the field core samples. The 1.15 H/D samples therefore result in a somewhat higher strength than would be achieved with the 2 H/D core samples. A correction factor of 0.91 was therefore applied to obtain a corrected strength value on the molded 365 day sample for comparison purposes.
compaction ranging from 92 to 99 percent with an average of 96.5 percent.

Results of the field molded compressive strength specimens are summarized in Table 9 and Figure 3. In summary, the field molded samples of the PC/FA cementitious blend indicated compressive strengths (16.8 MPa [2440 psi] at 28 days and 17.4 MPa [2525 psi] at 56 days) that are similar to those of the laboratory mixtures with the field blend aggregates (15.3 MPa [2225 psi] and 18.6 MPa [2700 psi]) and somewhat higher than these with the laboratory blended aggregates (13.0 MPa [1880 psi] and 13.2 MPa [1920 psi]). The field molded samples with the SC/FA cementitious blend indicated compressive strengths on the order of 1.3 MPa [195 psi] and 1.2 MPa [175 psi] at 28 days which turned out to be much less than the laboratory mixture which yielded a strength of 11 MPa [1600 psi] using the accelerated core method. This is probably due to the much lower curing temperatures of the field samples and the fact that slag cement generally develops strength at a slower rate at lower temperatures.

The 365 days test results indicated compressive strengths on the order of 30 MPa [4325 psi] and 18 MPa [2565 psi] for the PC/FA and SC/FA mixtures, respectively.

The final phase of the field testing included obtaining field core samples from the eco-pad pavement section after one year of field curing. Results of the core strength tests indicated an average compressive strength of 12.7 MPa [1852 psi] and 12.8 MPa [1852 psi] for the PC/FA and SC/FA mixtures. In comparing these results to the molded field samples, it must be recognized that the molded specimens have a height to diameter ratio of 2. Therefore, the molded samples will yield a somewhat higher strength value. Correcting the shorter molded samples with a correction factor of 0.91 as suggested in ASTM C42, the molded samples would indicate strengths of 27.1 MPa [3930 psi] and 16.1 MPa [2334 psi], respectively. The test results for the field molded and cored samples are summarized in Table 9.

8 CONCLUSIONS

1) Based on the results of the analysis, when the recycled concrete and bottom ash selected are blended from 127 mm [5 in] and 76 mm [3 in] thick layers respectively and mixed with a 15% blended (50% portland cement/50% Class C Fly ash) cementitious material by dry weight, properly mixed, moisture conditioned and compacted, a compressive strength on the order of 21.4 MPa [3100 psi] in one year was attainable.

2) When the aggregate blend is mixed using slag cement in lieu of portland cement in the cementitious material blend, a compressive strength on the order of 11.7 to 13.8 MPa [1700 to 2000 psi] in one year was attainable.

3) Some variability in the test data due to variations of the individual materials was observed and subsequently showed some variability in the results obtained. Therefore, a mixture analysis for the specific ingredients of each mixture should be undertaken for each project.

4) A “green” alternative paving material and construction process for environmentally friendly and economic large outdoor storage area pavements has been developed with a 93 to 100% recycled content. Long term performance will be evaluated of this pavement that was produced in a sustainable manner.

REFERENCES


1 INTRODUCTION

Coal thermal power largely depends on coal, one of fuel resources, which has a worldwide distribution centering on the Pacific Region, contributing to stable supply. Because it excels other fossil fuels such as LNG economically as well, coal is expected to play a significant role as a basic supplier of energy next to nuclear power and also as an intermediate supplier. Thus, the development of coal thermal power plant is underway throughout Japan. Consequently, more coal ash has been released from power plants year by year. In the year of 2004, the amount of the coal ash from power plants reached 7.13 million tons, of which approximately 12% has been thrown away in ash dumps and the rest of it, approximately 88%, has been recycled in portland cement/concrete area and civil engineering/construction area. Raw material for portland cement, however, occupies a high rate approximately 68% of the overall recycled, showing distorted proportion in recycling. In addition, the difficulty to secure ash dumps midst recent growing awareness of environmental preservation and the increasing tendency to build recycle-oriented society makes it more important to develop further techniques to make the most of coal ash.

On the other hand, due to the growing tendency toward improvement of work environment and sanitary management for workers in tunnel construction, the Welfare, Health and Labor Ministry has tightened dust regulations on spraying shotcrete, giving a boost to development of many dust reducing materials. Under these circumstances, the Shikoku Electric Power group has succeeded in developing high-performance shotcrete in order to improve workability, namely less dust and less rebound, to develop strength and to enhance durability, by producing the highly functional fly ash (here after, HFA) by sorting and extracting round and fine particles from raw particles of coal ash with windpower-utilized advanced classification technique and mixing HFA in shotcrete.

2 CHARACTERISTICS OF HFA

One of the main factors to change quality of fly ash is the increase of poor shaped fly ash particles due to the mixture of coals from different foreign countries or the change in combustion conditions resulted from countermesures for environmental preservation [Kanatsu et al. 1998]. Removal of these irregular coarse particles causing quality change with wind-power-utilized sophisticated classification technique to sort and extract high-quality round and fine particles less than 20μm can bring highly functional fly ash, which has much higher quality and function than original fly ash.
The HFA is approved as Class I of JIS A 6201 Concrete Fly Ash revised in February 1999 (Table 1). The HFA has much better characteristics than original particles before classification such as smaller quality change, more round particles and higher pozzolanic reaction. Mixing the HFA into general concrete, quality of concrete, such as decrease of water content, improvement of long age strength, and improvement of water-tightness, is improvable by the effects of ball bearing and fine particles. About these properties, the HFA has obtained technical evaluation from the authorities [A Concrete Admixture "FINASH". 1994] and at present around 10,000 tons of HFA has been used in public works mainly in the Shikoku region in Japan.

3 MODEL TUNNEL SPRAY TESTS

Prior to spray tests at the actual road tunnel, tests at the model tunnel are conducted in order to verify the effects of the shotcrete using HFA to improve workability and to develop strength.

3.1 Outline of spray tests [Ishii et al. 2001, Fukami et al. 2000]

3.1.1 Mixture and materials

The mixture of shotcrete used in the model tunnel spray tests consists of two types as shown in Table 2; plain standard mixture No. 1 which contains no HFA at all and the mixture No. 2 contains 100 kg/m³ HFA in place of fine aggregate. Mixing of 100 kg/m³ HFA contributes to moderate increase in viscosity as well as reduction in water content. The specification of concrete materials is shown in Table 3. In addition, HFA has high density in comparison with general fly ash so that there become few ratios of hollow particle with classification.

3.1.2 Spray tests

The spray tests are conducted at the model tunnel set up outside which is 3.5 m in height, 3.5 m in width and 7.0 m in length. A piston-pump-system SUPREMA (Max. discharge: 14 m³/h) is used for a spraying gun and a powder accelerator compressor (Max. discharge: 3.2 kg/min) is adopted for adding accelerator. In terms of piping, a 3-inch flexible hose is attached at the pump outlet piping, which extends approximately

---

**Table 1. Characteristic of HFA.**

<table>
<thead>
<tr>
<th>Classification</th>
<th>SiO₂ (%)</th>
<th>Wet percentage (%)</th>
<th>Loss on ignition (%)</th>
<th>Density (g/cm³)</th>
<th>Specific surface ratio* (cm²/g)</th>
<th>Flow value ratio (%)</th>
<th>Activity index* (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HFA</td>
<td>57.0</td>
<td>0.09</td>
<td>1.5</td>
<td>2.40</td>
<td>5570</td>
<td>112</td>
<td>91</td>
</tr>
<tr>
<td>JIS A 6201 (Class I)</td>
<td>≥45</td>
<td>1.0</td>
<td>≤3.0</td>
<td>≥1.95</td>
<td>≥5000</td>
<td>≥105</td>
<td>≥90</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>28-days 91-days</td>
</tr>
</tbody>
</table>

* The ratio with standard mortar.

**Table 2. Mixture proportions of shotcrete test at model tunnel.**

<table>
<thead>
<tr>
<th>Mixture no.</th>
<th>Water cement ratio (%)</th>
<th>Sand percentage (%)</th>
<th>Water</th>
<th>Cement</th>
<th>HFA</th>
<th>Fine aggregate</th>
<th>coarse aggregate</th>
<th>Set accelerating agent</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>61.7</td>
<td>60.0</td>
<td>222</td>
<td>360</td>
<td>–</td>
<td>1031</td>
<td>708</td>
<td>21.6</td>
</tr>
<tr>
<td>2</td>
<td>56.4</td>
<td></td>
<td>203</td>
<td>360</td>
<td>100</td>
<td>953</td>
<td>726</td>
<td>21.6</td>
</tr>
</tbody>
</table>

Note: Max. size of coarse aggregate: 15 mm. Slump: 120 ± 10 mm. Accelerator: 6% by mass of cement.

**Table 3. Materials used.**

<table>
<thead>
<tr>
<th>Materials</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland cement</td>
<td>Normal Portland cement, Density: 3.16 g/cm³</td>
</tr>
<tr>
<td>HFA</td>
<td>Density: 2.40 g/cm³, Specific surface area: 5570 cm²/g</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>Inland sand, Density: 2.59 g/cm³, FM: 2.73</td>
</tr>
<tr>
<td>Coarse aggregate</td>
<td>Crushed stone, Density: 2.66 g/cm³, FM: 6.19</td>
</tr>
<tr>
<td>Accelerator</td>
<td>Cement mineral, Density: 2.80 g/cm³</td>
</tr>
</tbody>
</table>
30 m to the end of the nozzle considering the length in actual tunnels.

Prior to the spray tests, two types of shotcrete mixture selected is produced in a ready-mixed concrete factory and transported to the test site by agitator truck. Here, the properties of fresh concrete are measured and, at the same time, samples for base concrete strength tests are prepared.

In the spray tests, at first samples are prepared by spraying the disc container and the panel frame (50 × 50 × 20 cm) set in the model tunnel. The former is for initial compressive strength tests (3 h and 4 h), and the latter is for preparing samples for compressive strength tests. Next, after a vinyl sheet is spread in the model tunnel, spraying is conducted manually to the arch part circumferentially in the range of 2 m and toward the tunnel inside in the range of 4 m at an spray angle 45° with keeping the distance of 1.5 m from the nozzle tip for 8 minutes (Spray: 1 m³), and the amount of dust is measured with a digital dust meter. At measurement, both mouths of the tunnel are covered with sheets to prevent rebound materials and dust from splashing out of the tunnel. The rebound rate is evaluated, after the completion of spray, by collecting rebound materials fallen down on the vinyl sheets and measuring the weight.

After the completion of spray tests, measurement of initial compressive strength is carried out in 3 and 24 hours with pull-out test method. At the same time, strength tests at the proper age are performed after a core sample (φ10 cm × 20 cm) is cut off from the panel frame for strength tests.

3.2 Results and discussion

The results of the model tunnel spray tests are presented in Figure 1–3.

3.2.1 Workability (amount of dust and rebound rate)
The amount of dust and the rebound rate are reduced to 63% and 47% respectively in the mixture No. 2 containing HFA 100 kg/m³ in place of fine aggregate, compared with the plain standard mixture No. 1 (Fig. 1). The results reveal that the workability of shotcrete is remarkably improved with less dust and less rebound by the mixture of HFA. This possibly shows that mixing HFA in shotcrete increases viscosity of concrete properly and the cohesion gained by the viscosity reduces separation and splash of fine particles as well as improves adhesion of concrete.

3.2.2 Strength development property

Initial compressive strength is measured in compliance with the test method of JSCE-G561, whereas compressive strength tests are conducted in accordance with JIS A 1107 after a core sample is cut off from the panel frame and cured at the temperature 20°C and the relative humidity 80% till the proper age. In addition, density of mixture No. 1 is 2.31 g/cm³, and the density of mixture No. 2 is 2.27 g/cm³.

The mixture No. 2, compared with the standard mixture No. 1, obtains 62% increased strength in 3 hours and 37% increased strength in 24 hours, which shows an excellent property to develop initial strength (Fig. 2). Since initial strength of shotcrete is proportional to the amount of accelerator added in general, a certain amount of accelerator (6% by mass of cement) is added to each mixture in these spray tests. It is considered that the excellent initial strength gained above is caused by further acceleration of hydration of portland cement by accelerator by the effect of fine particles of HFA.

The results of compressive strength tests at the medium and long age find that the strength of the mixture No. 2 at the age of 7, 28 and 91 days is increased 10%, 24% and 37% respectively compared with the mixture No. 1, which shows an excellent property to develop strength (Fig. 3). Thus, it is considered that the use of HFA causes densification of hardened concrete with the aid of gap-filling and pozzolanic
effects as well as decrease of water content, and eventually contributes to the improvement of strength development.

4 ACTUAL ROAD TUNNEL SPRAYTESTS

The model tunnel spray test results reveal that the use of HFA as an admixture of shotcrete made it possible to decrease the amount of dust and the rebound rate, and consequently to improve the strength development of shotcrete. Thus, the particular shotcrete is applied to actual road tunnel works in order to identify and verify the dust reducing effect, the strength development property and durability.

4.1 Outline of in-situ spray tests

4.1.1 Mixture and materials

The mixture of concrete and the specification of the concrete materials used in the actual road tunnel spray tests are shown in Table 4 and Table 5 respectively.

The in-situ spray tests are conducted on three types of mixture. The plain standard mixture No. 1 is the unit portland cement 360 kg/m³ which contains no HFA at all. The mixture No. 2 is the standard mixture with which conventional dust-reducing agent equivalent to 0.1% mass of portland cement is added. The mixture No. 3 contains 100 kg/m³ HFA in place of fine aggregate. The mixture No. 3 makes it possible to increase viscosity of concrete properly and to decrease the water content by 9 kg/m³ compared with the standard mixture.

4.1.2 Spray tests

The spray tests are performed using the on-site spray gun and equipment on the above three types of mixture. Review is made on workability by the measurement of the amount of dust and rebound weight during spraying, and also on the strength development property and the durability.

Prior to the spray tests, the necessary concrete is made at the on-site concrete plant and the fresh properties of slump and air content are measured. Then the concrete is transported to the working place in the underground by agitator truck.

The spray tests are conducted with an integral piston-pump-system spray gun (Max. discharge: 20 m³/h) by spraying 2 m³ concrete on the arch and side-wall parts of the half section of the tunnel at the speed of approximately 12 m³/h. The amount of dust released while spraying is measured at six points located 5, 10,

![Figure 3. Compressive strength in model tunnel tests.](image)

Table 4. Mixture proportions of shotcrete test at actual road tunnel.

<table>
<thead>
<tr>
<th>Mixture no.</th>
<th>Water cement ratio (%)</th>
<th>Sand percentage (%)</th>
<th>Water</th>
<th>Cement</th>
<th>HFA</th>
<th>Fine aggregate</th>
<th>Coarse aggregate</th>
<th>Conventional dust reducing agent</th>
<th>Set accelerating agent</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>61.4</td>
<td>60.3</td>
<td>221</td>
<td>360</td>
<td>–</td>
<td>1038</td>
<td>707</td>
<td>–</td>
<td>25.2</td>
</tr>
<tr>
<td>2</td>
<td>61.4</td>
<td></td>
<td>221</td>
<td>360</td>
<td>–</td>
<td>1038</td>
<td>707</td>
<td>0.36</td>
<td>25.2</td>
</tr>
<tr>
<td>3</td>
<td>58.9</td>
<td></td>
<td>212</td>
<td>360</td>
<td>100</td>
<td>943</td>
<td>718</td>
<td>–</td>
<td>25.2</td>
</tr>
</tbody>
</table>

Note: Max. size of coarse aggregate: 15 mm. Slump: 80 ± 20 mm. Accelerator: 7% by mass of cement.
and 15 m apart from the working place four times each (1 min/time) with a digital dust meter.

Besides, after spreading a blue sheet in front of the working place and spraying 2 m³ concrete, the weight of rebound materials is measured to calculate the rate of rebound.

Then, spraying is done after setting, in front of the working place, a container for the initial compressive strength tests and a panel frame for preparing samples for the compressive strength tests, the accelerated carbonation tests, and the drying shrinkage tests.

After the completion of spray tests, the initial compressive strength in 3 and 24 h are measured with pull-out test method. And also all kinds of tests are performed at the proper age on core samples cut off from the panel frame for the strength tests and the accelerated carbonation tests (Ø 10 cm × 20 cm), and for the drying shrinkage tests (Ø 10 cm × 10 cm × 40 cm).

Note three core samples same direction as the spraying direction for the accelerated carbonation tests and another three core samples perpendicular to the spraying direction for the drying shrinkage tests are cut out. In addition, density of mixture No. 1 is 2.30 g/cm³, density of mixture No. 2 is 2.29 g/cm³, and density of mixture No. 3 is 2.27 g/cm³.

The accelerated carbonation tests are conducted under the circumstances of 20°C in temperature, 60% in relative humidity, and 5% in carbon dioxide density on the core samples pulled out after 4 weeks’ cure sample in the 20°C water and another 4 weeks' cure sample in the 20°C normal air. Note the ages of the samples on which the carbonated thickness are measured are the core samples as old as 7, 28, and 56 days after being moved to the circumstance for the accelerated carbonation tests.

The drying shrinkage tests are performed according to JIS A 1129 on the pulled-out core samples after gauge plugs are stuck on them and they are cured for one week in the 20°C water. The shrinkage is measured on each testing sample aged 7, 14, 28, 56, 84 days after cure in water.

4.2 Results and discussion
The results of the actual road tunnel spray tests are presented in Figure 4–8.

4.2.1 Workability (amount of dust and rebound rate)
The mixture No. 3 containing 100 kg/m³ HFA in place of fine aggregate remarkably reduces the amount of dust to 53% compared with the plain standard mixture No. 1. Even compared with the Figure 76% achieved by the mixture No. 2 containing conventional dust reducing agent, the mixture No. 3 accomplishes much better dust-reducing effect. At the same time, the rebound rate of the mixture No. 3 greatly reduces to 48% compared with the mixture No. 1 and this Figure is even better than the mixture No. 2 reduction of 84%, revealing excellent rebound reducing effect (Fig. 4).

The results of these actual road tunnel spray tests, as well as the results of the model tunnel spray tests, find that the shotcrete using HFA in place of fine aggregate as an admixture shows the splendid effect to improve workability.

4.2.2 Strength development property
According to the test results for initial compressive strength, the strength of the mixture No. 3 in 3 and 24 hours is 186% and 93% higher respectively than that of the mixture No. 1, proving excellent strength development, whereas the strength of the mixture No. 2 in 24 h is almost the same as the mixture No. 1 (Fig. 5).

The results of the compressive strength tests at medium and long ages reveal that the strength of the mixture No. 3 at the age of 7, 28 and 91 days is 15%, 48% and 29% higher respectively than that of the mixture No. 1, showing good strength development.

Figure 4. Amount of dust and rebound rate in actual road tunnel tests.

Figure 5. Initial compressive strength in actual road tunnel tests.
Thus, as well as the results obtained from the model tunnel tests, the results of the in-situ spray tests at the actual road tunnel proves the excellent property of the shotcrete replacing fine aggregate with HFA to develop strength at initial, medium and long ages.

4.2.3 Durability (carbonation and drying shrinkage)

It is generally known that concrete is inclined to be carbonated when fly ash is used [Nakai et al. 2001]. But there is no problem in the durability, against carbonation, of mixture No. 3 containing the HFA in place of fine aggregate. One reason is because the value of carbonated thickness of the mixture No. 3 is almost as large as those of the standard mixture No. 1 and the mixture No. 2 containing conventional dust-reducing agent. And the other reason is that covering concrete is generally used in Japan (Fig. 7).

The shrinkage on the mixture No. 3 containing the HFA in place of fine aggregate is reduced to 60% of those on the standard mixture No. 1 and the mixture No. 2 with conventional dust-reducing agent, and the problem of drying shrinkage was vastly improved (Fig. 8).

5 CONCLUSIONS

This study is made to identify that the use of HFA as an admixture for shotcrete causes shotcrete to improve workability, enhance strength development property and durability.

Conventionally, anti-dust agent has been used for dust reduction, and many kinds of admixtures such as water reducing agent have been adopted for rebound control and strength enhancement according to each objective. Since the newly-developed HFA makes it possible to improve the workability of shotcrete as well as to enhance the strength development property and the durability, it is proved that HFA is one of the most suitable admixtures for shotcrete.

By this study, the following things became clear.

5.1 Effect to improve workability

(1) It is identified that the use of 100 kg/m³ HFA in place of fine aggregate brings proper viscosity as well as 4 ~ 8% reduction in water content (slump range: 8 ~ 12 cm) by the bearing effect of fine spherical particles compared with plain mixture.

(2) In the model tunnel spray tests, the amount of dust and the rebound rate reduce to approximately 60% and 50% respectively. Similarly, in the in-situ spray tests at the actual road tunnel, both the amount of dust and the rebound rate reduces to approximately 50%.

5.2 Effect to improve quality of hardened shotcrete

(1) The shotcrete using 100 kg/m³ HFA in place of fine aggregate shows remarkable improvement in compressive strength at initial, medium and long
ages because the hardened shotcrete becomes thicker with the aid of micro filler and pozzolanic effects by the HFA.

(2) The shotcrete using the 100 kg/m³ HFA in place of fine aggregate has durability (carbonation and drying shrinkage) equal or superior to plain mixture or admixture with conventional dust-reducing agent.

The spray tests results show that the use of the HFA improves the quality and workability of shotcrete as well as reduces the over-all amount of shotcrete with the reduction in rebound rate, and eventually it can cut approximately 10% of construction cost.

For this reason, shotcrete with HFA was adopted in approximately 40 tunnel construction in Shikoku region as of the end of March in 2006. Technique to make the most of coal ash released massively from coal thermal power plants as a resource is very significant to build the recycle-oriented society. Thus, the Shikoku Electric Power group will continue further research and development to expand the application of coal ash in a wide variety of fields.

REFERENCES


The use of fly ash for enhancing durability and sustainability

L.K.A. Sear
United Kingdom Quality Ash Association, Regent House, Bath Avenue, Wolverhampton, West Midlands, UK

ABSTRACT: Fly ash has been used for many years in a wide range of applications because it is able to offer many technical advantages such as enhanced durability and performance. For example it is able to improve the sulfate resistance, reduce chloride diffusion, prevent alkali silica reaction, and reduce heat generation, etc in cementitious applications. These benefits have been researched by many people with published papers totalling many thousands. However, it is only in recent years it is increasingly recognised that using fly ash also results in significant environmental and sustainability benefits, simply by replacing virgin aggregates, acting as a cementitious binder reducing overall greenhouse gas emissions, enhancing durability, extending a structures work life, etc. While considering these technical and environmental benefits, it is of a surprise that still a considerable proportion of fly ash produced is landfilled every year.

This paper will take an overview of the use of coal fired power station ash. It will review the wide range of applications for fly ash, looking at more recent developments in research, standards, specifications, ash processing, etc and the new ideas that have emerged for possible applications. The paper will consider the sustainability and environmental benefits of greater utilisation of the material. Also, those applications where little progress seems to be being made will be considered and reasons examined.

1. THE USE OF COAL FIRED POWER STATION PRODUCTS

Coal fired power stations produce a range of products which have been used in a wide variety of construction applications for many years. Typical usage patterns are shown in Figure 1, where 64% of the produced products are being used beneficially, either substituting for cement or virgin aggregate/material.

One of the most obvious has been the use of furnace bottom ash (FBA) that has established a role in the manufacture of lightweight concrete blocks for many years. As all UK coal fired power stations are ‘wet bottomed’, the FBA has proven to be a reliable coarse and fine aggregate in lightweight concrete block making, taking 100% of FBA production. In recent years, due to the contraction in the number of coal burning power stations and changes in burner technologies to reduce emissions, there is reduced production and a shortage of FBA. Block manufacturers are having to source similar materials, ranging from recovered bottom ash, incinerator bottom ash and even imported pumice from a variety of sources, including imports. Unfortunately, many of these options involve greater transport distances, processing or depletion of virgin aggregates adding to the environmental impact.

Many modern power stations now operate Flue Gas Desulfurisation plants, to reduce SOx emissions to air. The majority of these systems use limestone to chemically react with the flue gases producing gypsum. This is very high quality gypsum which has found a ready market in the manufacture of gypsum plasterboard, again taking 100% of production.

The remaining material produced in a coal fired power station is fly ash that is the material which is carried with the flue gases and extracted using electrostatic precipitators. All references to fly ash in this paper refer to pulverised coal fired power station ash; widely know as Pulverised Fuel Ash (PFA) in the UK. This should not be confused with fly ashes produced from other furnaces that will have differing properties and environmental impacts.

Approximately 50% of fly ash produced currently is being used in construction products [UKQAAs]. Many of these are cementitious applications covering cement manufacture as both kiln feed and blended cement, additions to concrete, grouting of mines and caverns, and aerated block manufacture. The other major applications are for fill and ground remediation.

The balance of the fly ash is disposed of in mono landfill. The ash is mixed with water (~15% by weight) so that it can be used in many applications or
transferred and disposed of in the landfill without causing dust problems, this process if called ‘conditioning’. As large mixers are required to mix the ash with water and transport distances can be fairly significant, there is an environmental impact associated with preparing for disposal, as well as the obvious impact of disposing of the ash. Some stations use a lagooning system to dispose of their ash. The ash is mixed with copious amounts of water and the resulting slurry being pumped for disposal in lagoons. These are eventually drained and the ash may then be used.

These landfill sites are eventually landscaped and returned to agriculture or for industrial and housing use. However, there is approximately some 55,000,000 tonnes of fly ash readily accessible from these disposal sites, with a further 60,000,000 tonnes of material that could be recovered with greater difficulty.

All recovered material from landfill sites would require some form of processing, at least screening to remove agglomerated and lumpy material. For some applications, for example for use in concrete, drying and classifying may be required to produce a sufficiently reactive product.

2 RESEARCH

Research into PFA/fly ash has been extensive over the years. The United Kingdom Quality Ash Association (UKQAA) bibliography contains 11,200 papers produced mainly since 1982 that all refer to fly ash. The considerable majority of these relate to the cementitious applications and in the main to concrete.

The properties of concrete using fly ash are well known. Fly ash is pozzolanic producing additional hydration products when in the presence of water and lime. It is these additional hydration products that enhance Portland cement concretes, reducing the permeability. As a result, the addition of fly ash to concrete improves resistance to alkali silica reaction, significantly reduces the permeability to chlorides, improves sulfate resistance, reduces the heat of hydration and the risk of thermal cracking, etc. Some of the very recent and more interesting research projects are summarised as follows.

2.1 Heat of hydration and thermal cracking

Two recent projects have been looking at heat of hydration and thermal cracking issues.
The University of Dundee [Dhir et al. 2006] has been testing the heat of hydration of modern cements, including those containing both Ground Granulated Blastfurnace Slag (GGBS) and PFA. The research has suggested that the fly ash is in effect having no effect on the maximum rate of heat production, irrespective of fly ash source, at normal curing temperatures. At higher temperatures the heat production due to the fly ash rises only marginally. These results are consistent with the normal understanding of the pozzolanic reaction.

The second project was to review CIRIA report 91 [CIRIA 1992], which was originally published in 1992. This revised report is due for publication in spring 2007. This report, authored by Bamforth, gives guidance on when thermal cracking is likely to occur and is based on the temperature differential of the concrete’s core to the exposed surface. The models have been refined, based on both Bamforth’s own work and Dundee University’s research and contains more information about critical steel contents, restraint, and compliance with Euro Code 2 (EC2).

It is clear that fly ash is a very effective material in reducing the risks of thermal cracking, see Figure 2, particularly when higher proportions (>30%) are used.

2.2 Resistance to sulfate attack

The ability of fly ash concrete to enhance the sulfate resistance of concrete has been recognised for many years within the UK. However, the discovery of the thaumasite form of attack some years ago in the M5 bridge foundations has led to a comprehensive review and more research.

Though much work has been done on PFA complying with BS3892 Part 1 on sulfate resistance, little had been done within the UK on unprocessed fly ash. This was increasingly more popular with the publication of EN450. During the research programme investigating the thaumasite form of attack carried out by both BRE [BRE 2003] and the University of Sheffield [Hill et al. 2003], unprocessed fly ash and greater proportions of ash in the concrete were tried. The UK QAA is also funding work with BRE, which is still ongoing, into the effect of the initial temperature and the resistance to thaumasite attack.

The result of these projects has been to conclude that a 25% fly ash component as a proportion of the total cementitious content does gives a limited resistance to the thaumasite form of attack. However, higher proportions, >36% and up to the maximum of 55% fly ash content, give a far more enhanced performance with no significant thaumasite attack being observed. The ongoing BRE project to look at curing temperatures as would be experienced in the ground, shows this has little effect other than what one would expect from concrete cured at the average temperature.

2.3 Soil stabilisation

The stabilisation of soils using lime has been practiced for some years in road construction. Using the existing soil as a suitable sub grade rather than use vast quantities of imported aggregates has gained favour in recent years as a method of reducing environmental impacts. The process has made many sites accessible and been successfully used.

One issue which has plagued this technique has been the risk of sulfate heave [Britpave 2006]. The lime being added to bind the clays, instead of reacting pozzolantically with the clay, reacts with sulfates present in the clay to form ettringite which expands and disrupts the road surface. As this can occur some weeks or months after the road is constructed, this can prove to be a very expensive problem.

To measure the sulfate content of clays and predict whether such a reaction may occur has been difficult and many contractors have opted for a proven remediation approach. It was found that GGBS [Higgins et al. 1998], when added in with the lime, is able to prevent sulfate heave occurring.

Little work has been carried out in the UK to assess whether fly ash/PFA is able to achieve the same affect, though fly ash has been shown in the USA to work successfully [Dermatas 2001]. Some preliminary UK work suggests that fly ash can prevent sulfate heave and a larger research project has been commissioned to investigate this further.

2.4 Hazardous waste treatment

With a ban on the disposal of sludge into landfill and the ever increasing costs of hazardous waste disposal, the stabilisation of hazardous wastes in cementitious systems has increasing attractions. Research is ongoing [UCL 2006] into whether additions like fly ash,
ggbs, silica fume, etc in combination with cement could be used to stabilise these wastes into a form suitable for disposal.

3 STANDARDS AND SPECIFICATIONS

Standards for fly ash/PFA have been around for 40 years. The majority of these standards related to concrete, and specifically in the UK for the mixer addition of ash. In recent times the cement and concrete industries have begun to use more fly ash, due to increasing pressure to reduce overall CO₂ emissions. The manufacture of Portland cement produces ~960 kg/tonne of CO₂ so there are big incentives to use a CO₂ neutral material like fly ash.

3.1 Cement and concrete

The cement industry has become a big user of fly ash both as a kiln feed and for blending into cement. Fly ash is used as a kiln feed as a source of silica, replacing the usual clay, marl or sand. Cement chemistry is a complicated area and in order to produce cement consistency of the fly ash is paramount. For blending in with Portland cement, the ash may be added as a Minor Addition Constituent (MAC), which can be up to 5% of cement, or in far greater proportions within blended cements with up to 55%. These additions comply with BS EN197-1 ‘Common Cements’ and are an effective way for a cement supplier to reduce the overall emissions associated with Portland cement manufacture.

Fly ash for use as a concrete addition at the mixer is normally supplied complying with EN450:2005 [BSI 2005], (Figure 3) where it can be counted as being part of the cement content of the concrete, see BS8500. EN450 describes two basic types of fly ash, Category N and Category S. The differences are described below.

3.1.1 EN450:2005 Category N fly ash

Category N (Normal) is fly ash that is taken direct from the power station electrostatic precipitators. EN450 imposes a series of quality control requirements on the ash, such as fineness, chemical properties, etc but this material would normally be controlled by a process of selection and rejection based on the various control parameters. As a result Category N fly ash is generally sourced straight from the power station silos without any processing. This could be considered as zero environmental impact at the factory gate from production and even possibly a negative impact if the energy associated with conditioning the ash for disposal is taken into account.

3.1.2 EN450:2005 Category S fly ash

Category S, which we shall call ‘special’ fly ash, is again dry ash from the station. In the majority of cases this is processed to remove the coarser ash particles within the material. Typically this is done with air swept classifiers and the process reduces the water demand and increases the strength of the resulting concrete by removing the misshapen and generally coarser fraction. This requires energy, typically 9.75 kWh per tonne of Category S fly ash, which equates to ~4.2 kg of CO₂ per tonne of product. However, in relation to the improvements in reactivity and water demand of the resulting concretes, classification is a positive environmental benefit.

3.2 Hydraulic bound mixtures in road construction

Hydraulically bound mixtures (HBM) are new to the UK, while they have a long history of use in other EU countries. They rely on the binding ability of a range of materials, but principally fly ash/PFA and/or ground granulated blastfurnace slag (GGBS) in combination with lime and/or cement. The aggregates may be recycled aggregates, road planings, incinerator bottom ash, or similar.

HBM’s are similar to Cement Bound Mixtures (CBM’s) in they are designed to produce a strong durable, hard wearing sub-base for asphalt wearing surfaces. However, HBM’s are slow setting and hardening and can be trafficked immediately [Britpave 2005]. Consisting mainly of recycled or by-product material they are very environmentally friendly in comparison with the virgin aggregates alternatives.

There are three standards in which fly ash may be used in hydraulically bound mixtures (HBM) for road construction as follows;

3.2.1 BS EN14227-1: Cement bound granular mixtures (CBGM) [BSI 2004a]

This standard specifies requirements, test methods and compliance criteria for cement bound granular mixtures. This permits the use of cements complying with BS EN197-1 [BSI 2000a] and Hydraulic Road Binders (HRB) complying with ENV13282 [BSI
and therefore can include fly ash as a constituent of the cement or HRB.

It contains the criteria for gradings and design purposes for CBGM's.

3.2.2 BS EN14227-3: Fly ash bound granular mixtures (FABM) [BSI 2004b]

This standard specifies the requirements for constituents, composition and laboratory performance of fly ash bound mixtures. Both siliceous and calcareous fly ashes may be used complying with Part 4. It refers to fly ashes added as a binder to the mixture on site. However, this standard permits the use of multiple blends of gypsum and GGBS plus other constituents such as sodium carbonate to aid setting.

Five differing FABM's are described:

- FABM 1 simply requires compliance with a grading envelope.
- FABM 2 specifies limits on grading, compacity and Immediate Bearing Index (IBI).
- FABM 3 is a fine aggregate mixture, i.e. there is no coarse aggregate, which has minimum IBI values.
- FABM 4 is a mixture where the supplier declares a grading, IBI, etc, similar to a proprietary mixture.
- FABM 5 is where fly ash is both the binder and the main constituent of the mixture, e.g. a lime: fly ash mixture would be a FABM 5.

Compressive strength classes may also be specified, but curing conditions and age are in accordance with the practices at the place of use. Compacity is defined as the absolute volume of the constituents calculated from the Particle Densities for each constituent divided by the apparent volume as measured by the Proctor dry density.

3.2.3 BS EN14227-4: Fly ash for hydraulically bound mixtures [BSI 2004c]

This details the requirements for fly ash for use in FABM's. The usual properties and limits are given, fineness, loss on ignition, sulfate content, calcium oxide, etc for which UK fly ashes would not have any difficulty in compliance.

The above standards are now incorporated within the Specification for Highway Works 800 [Highways Agency 2006a] series. The design for HBM is contained in Interim Advice Note 73/06 [Highways Agency 2006b], though there have been some issues with this design method, which at the time of writing are being resolved.

3.3 Regulated dangerous substances and REACH

Upon the horizon are two additional EU requirements that will impact on all materials, Registration, Evaluation, Authorisation and Restriction of Chemicals (REACH) and Regulated Dangerous Substances (RDS).

REACH [EU Commission 2006] is supposed to; ‘…provide a high level of protection of human health and the environment. At the same time, it will enhance the competitiveness of the EU chemicals industry by fostering innovation and ensuring high safety standards for its products.’

Under REACH, each producer and importer of chemicals in volumes of 1 tonne or more per year and per producer/importer – around 30,000 substances – will have to register them with a new EU Chemicals Agency, submitting information on properties, uses and safe ways of handling them.

In tandem with this RDS [Theilen 2004] is destined to control dangerous substances within EU standards. This is intended to remove environmental barriers to trade resulting from differing local environmental laws. Each EU standard will have list of dangerous substances, which will be tested using harmonised standards. Each country will be able to set its own limits for the RDS, but this in itself should not be a barrier to trade. Currently the work of TC351 is to assess existing test methods and harmonise a series of methods to enable REACH to work.

Apparently how REACH and RDS interact has not been decided by the EU Commission, or it could be construed that they were not aware of the two initiatives.

4 SUSTAINABILITY

While there are many changes in standards, and new directions in the applications for fly ash forthcoming, there is also the issue of sustainability. Increasingly clients, consultants and contractors are under pressure to reduce the environmental impact of construction and operation of a structure.

Fly ash/PFA has been used as a method of offsetting the environmental impacts by substitution. In most applications fly ash replaces virgin aggregate or Portland cement, both of which have an environmental impact in their production and are depleting virgin resources.

The UK power industry has produced fly ash since the 1950’s, much of which has been landfilled. Many hundreds of millions of tonnes of this fly ash are no longer accessible as the number of stations has reduced from in excess of 100 to only 18 coal fired stations. The sites have subsequently been redeveloped and the ash disposal sites used for industrial purposes and occasionally for housing. However, on the remaining coal fired power stations there is some 55,000,000 tonnes of fly ash readily available and a further 60,000,000 tonnes may be accessible if required. Barlow Mound [Drax Power Ltd 2006], see Figure 4, is an example of a large fly ash stockpile of in excess of 16,000,000 tonnes.
The production of electricity from coal is unlikely to cease in the near future. Potential development of clean coal technologies and the vast reserves of coal that exist within the world, mean that ash in some form or another will continue to be produced for many years. The current production of ash plus the existing stockpiles of fly ash form a readily available mineral resource for future generations.

Stockpile material would need extracting, screening, drying and possibly grinding or classification for some applications, e.g. for use in concrete, but they could be put to beneficial use. All these technologies already exist and, unlike some other secondary materials, there is no need to import fly ash as supply outstrips demand and large quantities of material are available on stock. At the current rates of utilisation stockpile material alone could last for 30 years.

4.1 Ash processing

Stockpile ash could form a significant resource for future generations, though much of this material would have to be processed in some manner. This may simply be screening the ash to remove agglomerated lumps but may extend to drying the ash, prior to further beneficiation processes. In addition to stockpile material, the pressure to reduce emissions from furnaces, including coal fired power stations has resulted in changes in the current production ash. The use of low NOx burners has increased Loss On Ignition (LOI) values in fly ash by $\sim 1.5\%$ in recent years. Further changes in EU emissions limits effective in 2007/8 mean that burners on power stations are being modified to further reduce NOx. The result will be even higher LOI [van den Berg 1998] in ash with much of production approaching the LOI limit for use in concrete.

The concrete standards within the UK set limits of LOI at 7.0\% and this will force ash producers to process the ash to remove excess carbon if they wish to remain in these markets. Various systems exist that are capable or producing very low carbon ash, which are reviewed in the following section.

4.1.1 Carbon reduction

There are two basic ways of removing carbon from ash, by separating the carbon using some form of electrostatic technique or by burning off the carbon.

Within the UK electrostatic separation using the Separation Technologies Industries (STI) [Gasiorowski & Bittner 2006] technique has been adopted. This system has a charged belt over which the ash passes. The carbon is attracted in one direction and the ash in another. Ash with LOI’s as low as 1.5\% has been produced with this system.

There are more complex methods of beneficiation that are being proposed at some stations.

4.1.2 Other beneficiation techniques

Proposals to build a Rocktron [Smalley et al. 2006] ash beneficiation plant at two power stations have been made. In this system, as well as reducing the LOI of the ash, a variety of other processes are carried out to separate cenospheres, produce a finer ash, extract the carbon, etc. In this manner it is proposed to produce a numbers of marketable products for the construction industry. As one can imagine such processing requires a high capital investment.

4.2 Enhancing durability

Using fly ash in many applications has both environmental and technical advantages. For example using 30\% fly ash as a replacement for Portland cement will reduce the risk of Alkali Silica Reaction [BRE 2004], improve the resistance to chloride attack of reinforcing, improve the sulfate resistance of the concrete [BRE 2005], reduce the permeability, etc. These benefits have been well documented over the years and are reflected in the British Standard for specifying concrete, BS8500 Parts 1 [BSI 2002a] and 2 [BSI 2002b].

As most concrete mixes are designed for 28 days strength using standard curing, in practice at later ages many fly ash concretes achieve far higher strength and lower permeabilities in situ than would be expected from the equivalent Portland cement concrete, as shown in Figure 5.

In practical terms fly ash concrete can be far more durable than implied by the 28 days strength, thus extending the life of the structure. By making structures last longer the environmental impact of building a construction is diluted. Many structures are now designed with 100 years life in mind, in which fly ash can play a major part in achieving this longevity. However, it is hard to find structures that have been required to or have lasted 100 years in practice.

Figure 4. Barlow Mound at Drax Power station, UK.
To take advantage of the benefits of fly ash concrete to ensure longevity requires considerable thought on the part of the designer, who must allow for flexibility of use and ease of maintenance. However, the Romans have proven longevity is possible with structures such as the Pantheon [Watson 2002] built between 118 and 128 AD as in Figure 6, the dome being a volcanic fly ash concrete structure.

4.3 Environmental benefits

The environmental benefits of using fly ash are relatively straightforward. Invariably fly ash is substituted for either virgin aggregate or Portland cement in all applications. However, there are often additional benefits associated with its technical performance.

4.3.1 Aggregate

Fly ash is used as an aggregate in the following applications [UKQAA 2006b]:

- Fill material – for embankments, raising levels, etc.
- Grouting – for filling of caverns, mines, stabilising poor ground, etc.
- Concrete – used as a filler aggregate when aggregate particle size distribution lacks fine material or a concrete has low cement content.
- As filler – can be used in a wide variety of applications to bulk out products, e.g. rubber, paint, etc.

Typically to produce 1 tonne of aggregate at the factory gate equates to ≈21 kg of CO₂. Transport [Parrott 1999] will be excluded from all these figures simply because it is very variable. Sometimes a quarry will be farther from a site than a power station and vice-versa.

The Concrete Industry Alliance [Parrott 1999] concluded that transporting the raw materials from the
source to the concrete plant and the concrete to the site accounted for \(-10\%\) of the environmental impacts of producing the concrete. Though of some significance, the basic calculations of impacts for materials are still valid using at the factory gate assumptions.

As \(-2,400,000\) tonnes of ash [UKQAA 2006a] are used predominately as filler we can conclude by a crude calculation that the use of fly ash reduces environmental impact by \(~56,500\) tonnes of CO₂ per annum. However, the situation is not so simple. The following example explains the problems.

To produce grouts the contractors prefer fly ash. There are good technical reasons why;

1. Fly ash grouts require far less cement for a given strength than for natural aggregate. This is because fly ash reduces the water content of the grout and is pozzolanic.
2. Fly ash has a lower particle density (\(~2.3\) kg/m³) than natural aggregate (\(~2.6\) kg/m³), so about \(13\%\) less material by weight is required.
3. Fly ash slows the setting time of the grout. This enables more grout to be placed and less injection holes to be drilled.
4. Fly ash grouts do not bleed significantly and can be pumped long distances. This makes them very efficient at filling of the void completely with minimal disruption.

These differences can be very substantial, with one grouting contract [Sear 2004] reporting that using fly ash reduces vehicles movements by \(40\%\) and material cost by \(\sim 50\%\) in comparison with those for Portland cement and virgin sand grout.

There are similar issues with ash as a fill material. The benefits with fly ash are that it compacts easily, is lightweight so reducing pressure on sub-soils, produces a stable embankment, naturally sheds rain water, has increasing strength with time, etc.

In aerated concrete block manufacture, a big user of fly ash, the ash is acting both as aggregate and binder. The natural alternative for making such blocks is ground sand, which uses virgin aggregate and energy.

In many applications fly ash, though being treated as an inert filler is in fact reacting pozzolaneously enhancing the performance of the material.

### 4.3.2 Cementitious binder

The use of fly ash as a partial substitute for Portland cement is the primary way of reducing environmental impacts. To manufacture 1 tonne of Portland cement requires \(450,000\) tonnes of CO₂ emissions. Using fly ash can significantly reduce the overall environmental impacts when used as a cementitious binder.

### 4.4 Reduction in CO₂ emissions using fly ash

Firstly let us consider the use of fly ash as a mixer addition to concrete. Fly ash is pozzolanic and of lower particle density than Portland cement, a direct replacement mass for mass in not possible. Normally an increase in total cementitious material is needed of about \(10\%\), depending on whether Category N or S ash is used, the strength required and the overall concrete gradation.

Table 1 shows the relative environmental benefits of using fly ash in concrete for a 40 MPa at 28 days concrete. It should be noted that while Category S fly ash is normally processed, the additional environmental burden is offset by the improved reactivity in the concrete mix. Higher proportions of ash, because of the need to compensate for the strength at 28 days, require greater total cementitious contents, offsetting the benefits.

As the pozzolanic reaction is rather a slow reaction at normal curing temperatures, specifying later ages for achieving a given strength does give further environmental benefits. Table 2 shows the results for a 40 MPa at 56 days concrete, with the additional benefits. For mixes with lower required strengths or other specific criteria higher benefits per m³ of concrete may be possible.

As some \(400,000\) tonnes of fly ash are used in concrete production p.a. as an addition in \(~4,200,000\) m³ of concrete, this would suggest an overall reduction in CO₂ emissions of at least \(250,000\) tonnes p.a. is due to the use of fly ash.

Our second consideration is the use of fly ash in cement manufacture. The cement industry is under a great deal of pressure to reduce CO₂ emissions as Portland cement, by the nature of its chemistry requires the calcining of calcium carbonate. Adding fly ash to cement as a kiln feed material, as a Minor Additional Constituent (MAC) or in the production of blended fly ash cement, reduces the overall environmental impact. Clearly using ash as a MAC or in blended cement has the greatest benefit as it is replacing Portland cement clinker.

The use of fly ash as a MAC will have the greatest environmental benefit as it is a direct weight for weight replacement for the Portland cement. This equates to \(\sim 40\) kg/tonne reduction of CO₂. As some \(11,300,000\) tonnes of cement are sold annually, this could amount to \(\sim 450,000\) tonnes of CO₂. For blended cements, similar calculations would apply as given in Table 1. However, it is not publicly known as to what proportion of fly ash is used for MAC/blended cement and what is used in kiln feed, so it is difficult to estimate the overall reduction in impacts.

From the above it is clear that significant reductions in emissions are already being gained by the use of fly ash in concrete and in cement.

### 4.5 Overall environmental benefits

It is difficult to estimate the overall benefits of using fly ash in reducing emissions to air, because detailed
data are not available. In addition how does one decide to assign whether a process is simply replacing inert virgin aggregate or benefiting from the poz- zolanic reactions and to what extent? It would seem reasonable to estimate that the minimum benefit of the use of fly ash in the UK in all applications equates to a reduction of \( \frac{1}{100000} \times 600,000 \) tonnes of \( \text{CO}_2 \) emissions per annum.

4.6 The future

Coal fired generation will continue for sometime and the production of ash products will also continue. It is clear that there are the supporting standards to encourage and increase their use in construction.

CONCLUSIONS

Fly ash/PFA has a wide range of applications. In cementitious uses it is able to enhance the durability significantly and for other applications it can replace virgin aggregates, usually as a filler aggregate.

These uses reduce the \( \text{CO}_2 \) footprint significantly, especially when used as both a cementitious material and filler, such as in grouting. It is estimated that this currently equates to over 600,000 tonnes per annum.

This paper can only provide a short overview of fly ash from coal fired power stations, however as a material it is able to enhance the durability of a product, reduce overall greenhouse gas emissions, has a long history of use and is readily available.

There are not many secondary or by-product materials that have such credentials.

REFERENCES


BRE 2003. The design of structural concrete to resist the thaumasite for of attack, Report 7–3 year confirmation report, DTI Partners In Innovation project no cc1879, September 2003. BRE, Watford, UK.

BRE 2004. Alkali silica reaction in concrete, BRE Digest 330, BRE, Watford, UK.

BRE 2005. Concrete in aggressive ground, BRE Special Digest 1, 3rd edition, BRE, Watford, UK.


BSI 2002a. BS EN 8500-1, 2002, Concrete – Complementary British standard to BS EN260-1, Part 1: Method of
Parrott L. 1999. Environmental impacts of transport relative to those of concrete, CIA Environmental Factsheet, UK Concrete Industry Alliance. Concrete Centre, Camberley Surrey, UK.
REACH Fact sheet, 2006, European Commission, for further information see; www.europa.eu.int/comm/environment/chemicals/reach.htm
Sear L K A 2004. UKQAA internal members report on Bosty Lane, Walsall grouting contract.

Cracking behavior of concrete containing fly ash due to drying shrinkage

Taeseok Seo, Yoshiteru Ohno & Takao Nakagawa
Ohno Lab, Dept. of Architectural Engineering, Osaka University, Osaka Japan

ABSTRACT: In order to investigate a cracking behaviour of concrete containing fly ash due to drying shrinkage, the restrained shrinkage cracking test of concrete specimens of ordinary Portland cement concrete and 20% fly ash concrete were performed. And drying shrinkage, creep, crack occurrence time, crack width and other characteristics were measured. Cracking tendency (crack occurrence time and crack width etc.) of fly ash concrete was almost the same as or a little superior to ordinary Portland cement concrete.

1 INTRODUCTION

Fly ash as an industrial by-product which is produced at a steam power plant is being used as concrete material, and there are many studies on the properties of fly ash concrete [Wang, A.G. et al. 2004, Hwang, K.R. et al. 2004], but study on the shrinkage cracking behavior under long term age of fly ash concrete is scares [Mehmet, G. et al. 2004]. In this paper, in order to investigate a cracking behaviour of fly ash concrete due to drying shrinkage, restrained shrinkage cracking test, drying shrinkage test, creep test, and other physical tests were carried out.

2 TEST OUTLINE

2.1 Concrete mixture

The material used in this investigation were ordinary Portland cement, fly ash(density:2.29), natural sand (dry density:2.57 g/cm³), crushed stone(dry density: 2.69 g/cm³) and Air Entraining water reducing agent.

Mix proportion of concrete is shown in table 1. The kinds of concrete mixture are ordinary Portland cement concrete of fly ash replacement ratio 0% and fly ash replacement ratio 20%.

2.2 Shape, kind and measuring method of specimen

The specimens shown in Fig.1 were made on accordance with standard[JIS A 1151 2002, Suzuki, K. et al. 1993]. The restrained steel ratio is 7.9%. Dry shrinkage and creep specimens (length:500 mm, width: 100 mm, thickness:100 mm) were made too. Two specimens were made according to concrete mixture. All specimens were stored in a controlled environment of 20±2°C and 60±5% relative humidity throughout the duration of the test. All strains of specimen (Drying shrinkage, creep, restrained steel) and crack width were measured by a contacting strain gauge (C.S.G) with precision of 1/1000 mm. 300 mm C.S.G was used to measure the strain of restrained steel and concrete, and 100 mm C.S.G was used to measure the crack width.

Table 1. Mix proportion of.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Amount of unit (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen</td>
<td>W/C (%)</td>
</tr>
<tr>
<td>F0</td>
<td>55</td>
</tr>
<tr>
<td>F2-I</td>
<td>69</td>
</tr>
<tr>
<td>F2-O</td>
<td>55</td>
</tr>
</tbody>
</table>

* F2-I: replace fly ash of 20% of cement to cement.
* F2-O: replace fly ash of 20% of cement to sand.
3 RESULTS AND DISCUSSION

3.1 Compressive strength, young’s modulus and split strength of concrete

Compressive strength, young’s modulus and split strength of concrete is shown in Table 2,3. Compressive strength of concrete was measured at the age of 7, 14, 28, 91 days and because the crack of concrete occurs at the early age, split strength was measured at the age of 7, 14, 21, 28 days. Compressive strength, young’s modulus split strength of specimen F2-O was higher than values of F0, F2-I. And values of F2-I was the smallest.

3.2 Strain of drying shrinkage and creep

The measurement of drying shrinkage was started at the age of 7 days, and creep test was started at the age of 14 days. The creep specimen was subjected to a compressive stress of 6 MPa. Strains of drying shrinkage and specific creep until the age of 400 days are shown in Figure 2, 3. The value of the drying shrinkage has decreased a little at the age of about 200~250 days by the bad condition of curing room. The strain of drying shrinkage of F2-O and F2-I containing fly ash was smaller than the value of ordinary Portland cement concrete specimen F0, and the values of F2-I and F2-O were almost same. It is thought that shrinkage strain of fly ash concrete specimen is smaller than ordinary Portland cement concrete specimen because of pozzolan effect by fly ash. In case of the specific creep strain, in inverse proportion to compressive strength (young’s modulus) of concrete, the specific creep strain of F2-O was the smallest, and the values of F2-I was the biggest.
3.3 **Restrained shrinkage cracking test**

Figure 4. shows crack condition of the specimen. All cracks penetrated the specimen. In the F0 and F2-O specimens, crack did not occur between measurement points (300 mm). The strains of restrained steel and concrete are shown in Figure 5. These strains were measured by 300 mm C.S.G. We see from Figure 5. that the strains of concrete and restrained steel are almost same before the crack occurs, and when crack occurs in the specimen, the strain of steel returns to almost zero. Figure 6. shows the restrained ratio of restrained shrinkage cracking specimen. The restrained ratio of all specimens are about from 0.7 to 0.75. Here, the restrained ratio is defined by Equation 1.

\[
R = \frac{\epsilon_{sh} - \epsilon_R}{\epsilon_{sh}}
\]  

(1)

Where \( R \): restrained ratio, \( \epsilon_{sh} \): free shrinkage strain, \( \epsilon_R \): restrained shrinkage strain

Table 3. summarizes the relationship of restrained concrete stress(\( \sigma_{rs} \)) and split strength when crack occurs. The experimental value of the restrained concrete stress was calculated by Equation 2., and the split strength was estimated by the linear interpolation of the measurement value. It can be seen from Table 3. that the restrained concrete stress, when the crack occurs, is about 50~60% of split strength(\( f_t \)). This result almost corresponds to the past study [Kanda, T. 2005]. And though the split strength of specimen F2-I was the smallest, the crack occurred lately. The reason why the crack of F2-I occurred lately might be that the relaxation of stress due to creep was the biggest. Figure 7. shows restrained concrete stress. The solid line in the figure indicates the analysis value by step by step method(Equation 3.) [Neville, A.M. 1970, AIJ 2006], and the standard of crack occurrence is the value considering the average value 0.55 of \( \sigma_{rs}/f_t \)(Table 3.) at the split strength of concrete.

Here, for the analysis, the values of the drying shrinkage, creep coefficient, young’s modulus and restrained ratio obtained from the experiment was used. Good agreement is shown between the analysis value and experiment value. It was conformed that the age of crack occurrence day could be predicted by the proper ratio of \( \sigma_{rs}/f_t \) and step by step method.

\[
\sigma_{rs} = \frac{E_y \cdot \epsilon_{sh} \cdot A_i}{A_c}
\]  

(2)

\[
\sigma_{rs} = \sum_{j=1}^{i} \frac{E(t_j)}{1+\phi(t_{j+1/2}/t_j)} R(t_j) \Delta \epsilon_{sh}(t_j)
\]  

(3)

Table 3. Restrained stress and split strength.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Day</th>
<th>( \sigma_{rs}/f_t )</th>
<th>Day</th>
<th>( \sigma_{rs}/f_t )</th>
</tr>
</thead>
<tbody>
<tr>
<td>F0</td>
<td>15</td>
<td>0.47</td>
<td>17</td>
<td>0.57</td>
</tr>
<tr>
<td>F2-I</td>
<td>19</td>
<td>0.62</td>
<td>17</td>
<td>0.56</td>
</tr>
<tr>
<td>F2-O</td>
<td>17</td>
<td>0.56</td>
<td>19</td>
<td>0.56</td>
</tr>
</tbody>
</table>
$E(t_j)$: concrete young’s modulus at time $t_j$, $\phi(t_{i+1/2}, t_j)$: creep coefficient at time $t_{i+1/2}$, $R(t_j)$: restrained ratio at time $t_j$, $\Delta \varepsilon_{sh}(t_j)$: change of free shrinkage at time $t_j$, $E_s$: steel youngis modulus, $\varepsilon_{st}$: strain of steel, $A_s$: section of steel, $A_c$: section of concrete

The change of crack width is shown in Figure 8. There were little differences of the crack width of the specimens. But the crack width of specimen F0 of ordinary Portland cement concrete was the biggest, and the crack width specimen F2-I and F2-O containing fly ash was almost same.

4 CONCLUSIONS

The result of this study is as follows.

1) It is thought that the strain of drying shrinkage of fly ash concrete specimens F2-I, F2-O was smaller than ordinary Portland cement concrete specimen F0 because of pozzolan effect by fly ash.

2) The crack occurrence day of specimen containing fly ash was a little late than ordinary Portland cement concrete specimen.

3) Since drying shrinkage strain of specimen F0 was bigger than fly ash concrete specimen (F2-I, F2-O), the crack width of specimen F0 of ordinary Portland cement concrete was the biggest, and the crack width specimen F2-I and F2-O containing fly ash was almost same.

In a word, although there was little difference of the crack width, it was confirmed that the crack performance of concrete containing fly ash was a little superior to the crack performance of ordinary Portland cement concrete.

REFERENCES


Suzuki, K. et al. [1993], Test method for cracking of concrete due to drying shrinkage, proceeding of the fifth international RILEM symposium, pp. 63–70, 1993


Potential uses for coal combustion by-products for sustainable construction materials

T.F. Vandivort & P.F. Ziemkiewicz
Combustion Byproducts Recycling Consortium, National Mine Land Reclamation Center, West Virginia University, Morgantown, WV, USA

ABSTRACT: More than 123 million tons of solid by-products are produced by coal-burning electric utilities each year in the United States [ACAA 2005]. In particular, the annual production of flue gas desulfurization (FGD) by-products continues to increase. As landfill space becomes more limited and expensive, there is incentive to find productive and sustainable uses for coal combustion by-products (CCBs). However, by-products utilization technology is not likely to be adopted by the construction industry unless it is more cost-effective than land filling and the resulting construction has sufficient thawing. Therefore, it is extremely important that the electric-utility industry provide guidance to CCB research and development programs. Likewise, government agencies and private-sector organizations that may be able to utilize these materials should also provide input.

A U.S. Department of Energy – National Energy Technology Laboratory-funded program, the Combustion Byproducts Recycling Consortium (CBRC), serves as an effective vehicle for acquiring and maintaining guidance from these diverse organizations so that the proper balance in research and development is achieved. The mission of the CBRC is to promote and support the commercially viable and environmentally sound recycling of CCBs for productive uses, through scientific research, development, and field testing. Since its inception in 1998, the CBRC has funded 52 CCB research projects nationwide with a total value exceeding $10 million. Projects have ranged from using CCBs in mine void filling, surface mine reclamation, soil remediation, road base materials and aggregates to marine structures, lagoon liners, transmission poles, and building products. This paper focuses specifically on the projects related to using CCBs for construction materials including wall panels, siding, bricks, masonry units, countertops, and wall and floor tiles. Project results and findings are presented.

1 INTRODUCTION

Each year over 123 million tons of solid by-products are produced by coal-burning electric utilities in the United States [ACAA 2005]. Annual production of flue gas desulfurization (FGD) by-products continues to increase as the result of more stringent sulfur emission restrictions. In addition, stricter limits on NOx emissions mandated by the 1990 Clean Air Act have resulted in utility burner/boiler modifications that frequently yield higher carbon concentrations in fly ash, which restricts the use of the ash as a cement replacement. If newer, “clean coal” combustion and gasification technologies are adopted, their by-products may also present a management challenge.

The objective of the Combustion Byproducts Recycling Consortium (CBRC) is to develop and demonstrate technologies to address issues related to the recycling of by-products associated with coal combustion processes. A goal of CBRC is that these technologies, by the year 2010, will lead to an overall ash utilization rate from the current 40% to 50% [ACAA 2005] by such measures as increasing the current rate of FGD by-product use and increasing the number of uses considered “allowable” under state regulations. Another issue of interest to the CBRC is to examine the environmental impact of both by-product utilization and disposal.

No by-product utilization technology is likely to be adopted by industry unless it is more cost-effective than land filling. Therefore, it is extremely important that the utility industry provide guidance to the R&D program. Government agencies and private-sector organizations that may be able to utilize these materials in the conduct of their missions should also provide input. The CBRC serves as an effective vehicle for acquiring and maintaining guidance from these diverse organizations so that the proper balance in the R&D program is achieved.

The CBRC is sponsored by the U.S. Department of Energy’s National Energy Technology Laboratory and managed by the National Mine Land Reclamation
Center at West Virginia University in Morgantown, West Virginia.

This paper will highlight the DOE-NETL-funded CBRC projects that are related to using CCBs for construction materials including wall panels, siding, bricks, masonry units, countertops, and wall and floor tiles. Projects focusing on large-scale commercial products including transmission poles and marine structures using CCB’s are also highlighted.

2 COAL COMBUSTION BY-PRODUCTS

Coal combustion by-products are formed in the process of burning coal for electricity in electric-generating power plants. The type of by-products formed depends upon the method of combustion and the raw materials burned. The resultant by-products are fly ash, bottom ash, flue gas desulfurization (FGD) materials, fluidized bed combustion (FBC) ash, and boiler slag [NETL 2006]. The three types of by-products mentioned throughout this paper include flue gas desulfurization (FGD) material, Class C ash, and Class F ash.

2.1 Fly ash

Fly ash is tiny particles (like talcum powder) of mineral residue from burning coal in power plant boilers. Fly ash is a silica, alumina, and calcium based material, or pozzolan, which is capable of forming a cementitious material when combined with water. [WRAG 2006]. ASTM C618 defines two classes of fly ash: Class C and Class F. Class F fly ash is formed when bituminous or anthracite coal is burned and meets the physical requirements and chemical composition of ASTM C618. Class C is fly ash produced when sub-bituminous coal or lignite is burned, has pozzolanic properties, some self-cementing properties, and meets the physical requirements and chemical composition of ASTM C618. [NETL 2006].

2.2 FGD material

FGD material is a solid residue which is produced when sulfur dioxide gas is removed from power plant boiler stack gasses during the flue gas desulfurization process. The FGD material develops in flue gas scrubbers when gaseous sulfur dioxide reacts with slurried limestone to produce calcium sulfite (gypsum). Some power plants further oxidize calcium sulfite to calcium sulfate, or gypsum. FGD gypsum has many uses including wallboard production, Portland cement production, soil amendments and fertilizer. Finding uses for FGD gypsum is more challenging. Both materials are wet when produced in the scrubbers and are dried for handling [NETL 2006].

3 OTHER RELATED RESEARCH

3.1 Cement/concrete

Since the 1950’s, coal fly ash has become more commonly and successfully used as a primary building constituent in building materials [Ward 2003] such as cement mixtures for the production of concrete [Butalia & Bargheiser 2004]; [Creamer 2003]; Jones et al. [2005]; McCarthy et al. [2005], not only in the United States but in other countries as well including Japan Ohnaka et al. [2005], South Africa [Kruger & Krueger 2005], Israel Nisnevich et al. [2005], the Netherlands [Saraber & van den Berg 2005], and Spain Leiva et al. [2005] Spanish research has focused on the ability of fly ash to improve the mechanical strength of concrete as well as its fire resistance Leiva et al. [2005]. In South Africa, fly ash is used as a filler in polymers and cement mixtures [Kruger & Krueger 2005]. In Japan, fly ash has been used primarily in cement mixtures. However, Japanese cement production has been decreasing and research is now focused on using fly ash as a substitute for soil and sand for civil engineering applications Ohnaka et al. [2005].

In the past, adding some strongly alkaline coal combustion byproducts (CCBs) to conventional concrete blends resulted in alkali-silicate reactions (ASR) that degraded the mixture. New blends have been developed that control ASR and allow for new markets for using CCBs in concrete applications [Colaizzi 2003]. Researchers have found that when fly ash is used as filler, the strength of the composite material increases making fly ash composite materials viable for concrete aggregates [White 2001].

Researchers have studied both Class C and Class F fly ash in cement mixtures. Studies have been performed on Class C fly ash to replace cement in ranges from 0–70% by weight. The mixtures have been tested for water-, air-, and chloride-permeabilities, and for compressive strength. The highest resistance to air-, water-, and chloride-permeability occurred with 55%, 35%, and 18–55% fly ash mixture, respectively Naik et al. [1996]. Other researchers blended a mixture containing Class C fly ash. The resultant cement was evaluated in accordance with ASTM C1157. The result indicated the cement-fly ash blend to have equal or more strength than standard cement [Wu & Naik 2003].

Researchers have evaluated how well concrete pavements made with large amounts of Class C and Class F fly ash have performed over the long-term. Three mixtures with up to 70% Class C and three mixtures with up to 67% Class F fly ash were evaluated. Results showed Class F contributed greater pozzolanic strength than did Class C fly ash Naik et al. [2003a, b]. Researchers in the Netherlands have also found Class F fly ash to be a good pozzolanic material in concrete production [Fraaij & Copuroglu 2005]. Researchers
have further discovered that: 1) up to 25% of ponded ash can be successfully mixed with non-air entrained concrete and 2) 5% Class F fly ash and up to 20% ponded ash can be successfully mixed with air-entrained concrete [Naik et al. 2003]. FGD sludge has also been used to produce cement. FGD sludge has also been used to produce cement. A mixture of 13% FGD, 24% clay, and 63% limestone resulted in a suitable rapid-setting cement [Bhattacheria 2001a].

Numerous research projects have been undertaken to improve the performance of fly-ash-containing concrete products. For instance, researchers at Montana State University have used Class C fly ash as the binder in place of Portland cement in concrete production. The resultant product performed exceptionally well in regards to workability and strength compared to traditional concrete made with Portland cement [Cross et al. 2005]. Researchers at Ohio State University have found the use of fly ash in structural concrete to result in a product with reduced permeability, increased durability, and lower corrosion potential. This may translate to safer and more cost-effective infrastructure [Butalia & Bargaheiser 2004].

A variety of CCB-containing products have been developed with equivalent or superior attributes, including production costs, than concrete comprised of blends containing natural materials. For instance, Flashag is a newly patented aggregate made from fly ash. Concrete made from this aggregate is lightweight, has high strength, high durability potential, and low porosity. Flashag concrete weighs less than 21% of either Dacite or Granite aggregate concretes; is 15–21% stronger, and better at resisting the formation of cracks associated with shrinkage than normal weight, high-strength concretes. Manufacturing Flashag is less expensive and easier than other technologies used to manufacture aggregates from fly ash [Kayali 2005a]. Another is MRT® Cement in Texas which has found a process that allows for over 80% fly ash be combined with other materials to create a rapid-hardening cement. These MRT® masonry cements can be used in stone block, brick, and stucco construction. MRT® products have been found to be cost-competitive with other cement products [Hicks 2003].

3.2 Masonry units/bricks/tiles

When fly ash is used as a filler, the strength of the composite material increases. This makes fly ash composite materials viable for masonry units and bricks [White 2001]. FGD has been used to produce aggregates that meet ASTM standards and were used to produce concrete masonry blocks [Wu et al. 2003]. Class F fly ash has been used in brick production and has shown that bricks containing the fly ash were comparable to standard building bricks [Chou et al. 2003]. Fly ash has also been added to lime and phosphogypsum to form hollow blocks for walls in construction applications. These hollow blocks are lightweight and have good thermal insulation. Fly ash-containing blocks serve as a viable alternative to concrete or clay blocks for use in building construction [Kumar 2003; Hughes et al. 1995].

One problem to overcome in manufacturing bricks using fly ash is the ability of the fly ash brick to resist freeze/thaw damage. Even though it is possible to produce fly ash bricks with high compressive strength, the resultant brick can be brittle and more susceptible to freeze/thaw damage. Researchers have discovered that by using an air-entrainment chemical in a similar manner to that used in air-entrained concrete, that a fly ash brick can be manufactured to sustain the 50-cycle freeze/thaw ASTM Standard [Liu et al. 2005].

A variety of masonry unit/brick products have been produced. For instance, FlashBricks are manufactured bricks using fly ash. These bricks are approximately 28% lighter than their clay brick counterparts which translate to lower transportation costs. Compressive and tensile strengths were 24% and 3 times that of standard clay bricks, respectively. FlashBrick is a similar red color to that of clay bricks, but has a rougher texture which may account for the 44% higher bond strength with mortar [Kayali 2005b].

Fly-ash-based tiles were made and found to be superior to commercial patio and wall tiles and comparable to commercial floor tiles. The resultant red color can be glazed with different textures and colors. Tiles with more than 60% Class F fly ash have withstood freeze/thaw tests. Production of these tiles can be achieved with available commercial equipment and processing methods [Bhattacheria et al. 2001b].

4 THE PROJECTS

4.1 Development of structural materials from sulfate-rich FGD scrubber sludge

Vivak Malhotra, Southern Illinois University, Principal Investigator

4.1.1 Background

Currently, the United States produces approximately 19 million tons of FGD scrubber sludge annually [ACAA 2005]. Most is landfilled near the power plants that produce it. In response to the 1990 Clean Air Act amendments, even more wet scrubber sludge is expected to be produced each year. Less than 4% of the wet scrubber sludge produced is used nationally each year [ACAA 2005]. Therefore, finding alternative uses for this material is necessary.

If FGD scrubber sludge is used to manufacture structural materials, it is necessary to know how pressure, temperature, and other factors impact the
formation of the material. Also, addressing the effects of organic and inorganic components present in scrubber sludge is necessary. It is difficult and cost-prohibitive to separate these organic and inorganic components from sludge. Therefore, strategies have to be achieved to develop FGD scrubber sludge materials that are not affected by these impurities of organic and inorganic components. The project goal is to develop a technology to convert sulfate-rich scrubber sludge into countertops, tiles, and siding.

4.1.2 Significance
Producing countertops and tiles capable of competing with current high-end products that capture only 5% of the market would translate into 5 million tons of sludge utilization annually [Malhotra & Chugh 2005]. Such a potential market would generate an estimated 600 jobs, or 3% of the current workforce, and generate approximately $250 million annually. Specifically, FGD scrubber sludge-comprised countertops and tiles could have the following benefits:

– Reduce disposal costs of scrubber sludge
– Generate new markets for FGD structural materials
– Generate jobs and revenue
– Give value to coal combustion by-products

4.1.3 Objectives
1. Design high pressure, high temperature molding dies to fabricate composites up to 8 inches in size using FGD scrubber sludge.
2. Develop protocols and engineering methods to develop and fabricate materials from FGD scrubber sludge.
3. Achieve mechanical strength of FGD scrubber sludge materials produced comparable with commercially available materials.
4. Optimize the use of fibers in produced materials to enhance durability and textural appearance.
5. Achieve a surface treatment to prevent scratching of materials produced.
6. Fabricate materials in different colors and patterns.
7. Explore the feasibility of forming siding materials using FGD scrubber sludge.

4.1.4 Results
The researchers designed and built a vacuum die to form countertop composites under mild vacuum. Using this die, they formed countertop composites in which they varied the concentration of FGD scrubber sludge. The concentration of the sludge in the composite was varied between 50 and 75 weight %. Researchers used up to 65-weight % scrubber sludge in material composites and were still able to obtain comparable flexural strength to that of their commercial counterparts. In fact, the flexural strength of the scrubber sludge material composites was comparable or even better than the flexural strength of commercial products.

Researchers explored whether the countertop composite’s resistance to scratching could be further enhanced by forming the composites from block copolymers. In this approach, they incorporated an additional polymer to the one that was used to form countertops. The results indicated that a second polymer could be added to improve scratch resistance without degrading the strength of the product. In fact, 5 weight % of the second polymer could accomplish this without reducing the scrubber sludge crystallites in the materials.

Flexural strength measurements on the decorative tiles indicated that the particle size of the polymer had a crucial effect on the strength of the material. The smaller the particle sizes of the polymer, the larger the flexural strength of the composite. Experiments suggested that 2 weight % decorative granules could be incorporated in the tile composites without compromising the strength of the material.

Researchers examined whether aging affected the strength of the countertops and tiles. After aging the products for one year, the strength was not adversely affected. Researchers also tested the stability of tiles. They immersed tiles in water for more than a month and yet did not observe any disintegration or swelling.

Researchers fabricated 64 decorative tiles from scrubber sludge. (See Figure 1.) Four were mounted on a commercial backing board using commercial adhesive. Results suggest that FGD sludge-derived tiles can be mounted on currently existing commercial backing boards. (See Figure 2.) Researchers also fabricated countertop composites in 11 different colors and patterns.

An economic analysis performed indicated the countertop product to be nearly 10 times less expensive.
than Corian® brand countertops. Decorative tile cost was estimated to be approximately $0.85 per tile [Malhotra & Chugh 2005].

4.2 Fiber-fly-ash-based wall-panel development
John Hunt, AeRock, Inc., Principal Investigator

4.2.1 Background
By incorporating existing and emerging technologies, a superior construction panel or siding product that uses fly ash as the primary raw material might be achieved. Such products would be lightweight and still exhibit the engineering properties of high strength, thermal insulation, fire retardant, and water resistance necessary to be viable building materials.

4.2.2 Significance
The development of such building products would potentially result in the beneficial use of an increasingly available, recycled resource.

4.2.3 Objectives
- To bring together existing and emerging technologies to produce a lightweight cementitious wall panel or building siding panel of superior construction using fly ash as the primary raw material.
- To use as high a content of ash as possible while still achieving a quick-setting mix.

4.2.4 Results
Three different technologies were explored to determine which was most effective in producing panels containing ash. These technologies included a 3-D structural design with hexagonal or square ribs; a rib space fill of aerated fly ash or some other insulation; and a high-strength, fast-setting, fiberous fly ash cement for cellular structure.

Researchers focused on combining fly ash cement with various recycled polymer and natural fibers. The cement used was derived from coal ash from the Powder River Basin in the western United States. Fibers ranged from recycled polypropylene to old newspapers. To determine strength of the mixes, compressive strength tests were performed on square cubes two inches in size. Fiber content ranged between 0.6 to 6.0% weight for cellulosic fibers and between 0.2–2.0% weight for polymer. Different cellulosic fibers were evaluated including newspaper, office waste paper, corrugated boxes, eucalyptus, kenaf, and a Douglas fir/hemlock blend. Acetylation and calcium carbonate loading were two fiber treatment methods evaluated. Recycled polypropylene and high density polyethylene fibers were evaluated and both yielded compressive strengths similar to that of sand and cement.

Flexural strength of various mixes and reformulating the cement to better perform with cellulosic fibers was then tested. Researchers used ASTM C-348, Standard Test Method for Flexural Strength of Hydraulic Cement Mortars. Dimensional and weight measurements were taken for each specimen which allowed for researchers to calculate the moment of inertia. This value along with stress versus strain curves allowed for the calculation of modulus of elasticity and modulus of rupture for each specimen.

Several full-thickness prototype panels were produced. (See Figure 3.) The panels are resistant to fire, rot, and pests, and are durable yet lighter in weight than concrete. In fact, the resultant wall panel is approximately half the weight of conventional wall panels and siding. The wall panels also exhibit high strength, good thermal insulation properties, and resistance to water. They are also capable of being used as structural components and can retain screws. The process to manufacture these panels is very economic and ready for commercialization [AeRock & EERC 2001].
4.3 High performance masonry units from 100% fly ash: synergistic approach

Hwai-Chung Wu and Peijiang Sun, Wayne State University, Principal Investigators

4.3.1 Background
Among the current uses of fly ash, one use is to partially replace sand or cement in structural concrete or in the production of masonry blocks. The emphasis has been to maximize the amount of fly ash without causing a deleterious effect. Using 20% fly ash was typically considered appropriate to provide a similar or slightly improved performance compared to using all concrete. Any higher quantity of fly ash to the concrete mixture was found to have a detrimental effect. There is also an upper limit as to how much impurity, such as unburned carbon content, can be tolerated.

4.3.2 Significance
If masonry units are produced in the vicinity of power plants where the fly ash is produced, storage and transportation costs associated with the fly ash can be greatly reduced. Also, excess heat generated from the power plant can be utilized during the processing of the fly ash into masonry units. This will result in an overall reduction in the manufacture of the fly ash masonry units. Finally, using the fly ash material for masonry units is a potentially large beneficial use of this by-product.

4.3.3 Objectives
- Produce high performance masonry units using 100% fly ash or nearly 100% fly ash with a small amount of chemical additives.
- Develop an innovative process integrated from mixing, hydrothermal hot-pressing, carbonation, and fiber-reinforcement to produce high performance masonry products.
- Identify a potential market and strategy for commercialization.

4.3.4 Results
Researchers began with a mixture of 3 parts fly ash (Class C or F), 1 part water, and a small amount of sodium hydroxide (NaOH). They mixed for five minutes and poured the mixture into a mould cylinder chamber with MTS machine pressure applied for 20 minutes to reach 20 MPa. The mould was removed and baked for several hours, cooled, and the sample removed from the mould.

Pressure, temperature, heating time, and hydroxide ratio proved to be the primary factors affecting the splitting tensile strength of the fly ash masonry units. Researchers effectively employed NaOH as an effective activator of Class C fly ash resulting in a very high tensile strength of the masonry units. In fact, the strength increases with NaOH and heating temperatures. The more hydroxide added [up to 5%], the more the strength significantly increased than when hydroxide was only 2% or not added at all.

Material strength increased with higher heating temperatures. The material achieved higher strength when a pressure of 20 MPa was maintained through the process. Higher pressure resulted in more compacted products with less flaws. As heating duration increased, strength increased. Slower cooling rates resulted in higher strength. Increased aging times also increased strength. A five day aged product had more strength than a two day aged product.

Adding PVC or Kevlar fiber increased the ductility of the masonry units. (See Figure 4.) However, these fibers decomposed when temperatures exceeded 200 degrees C. Researchers found an optimum fiber volume and optimum heating duration with NaOH activator in terms of masonry composite workability, performance, and cost. Also, the density and modulus were virtually the same for composites with either NaOH and fiber or NaOH and no fiber. Class C ash led to better performance in the manufacture of the masonry units than did Class F ash. Overall, the fly ash masonry units produced in this project show very positive performance as building materials (See Figure 5.) [Wu 2003].

Figure 4. Different failure modes of Class C samples during testing.

Figure 5. Two halves of a split sample.
4.4 Manufacturing fired bricks with Class F fly ash from Illinois Basin coals

Mei-In Melissa Chou, Illinois Geological Survey, Principal Investigator

4.4.1 Background
Researchers at the Illinois State Geological Survey and the University of Illinois are working with brick manufacturers to develop high-quality, marketable brick products using large volumes of Class F fly ash. The fly ash is generated from power plants burning Illinois coals. In this brick-making process, fly ash is used as a raw material to substitute for part of the clay and shale, which are the two main raw materials of a conventional brick. Test bricks produced so far have met or exceeded ASTM commercial specifications.

More than six million tons of Class F fly ash are generated from burning about 100 million tons of Illinois Basin coal each year. Most of this fly ash is ponded or land filled, but could be readily available for making fired bricks. Nevertheless, until the brick industry gains more confidence in using fly ash as a raw material for brick production, evaluation and testing will be needed on a case-by-case basis.

In this project, researchers determined if the Class F fly ash produced by Cinergy PSI's Cayuga (CPSIC) Power Generation Station, which burns Illinois Basin coals from Illinois and Indiana, is a viable raw material for brick production at Colonial Brick Company, a brick plant in Indiana near the Illinois border.

4.4.2 Significance
The number of bricks produced in the U.S. has steadily increased each year. In 2001, nationwide production was estimated at 8.3 billion SBE (standard brick equivalents). By the year 2003, it had increased to 8.6 billion. In 2004, it reached 9.3 billion, which would weigh 23.25 million tons at five pounds per brick; [Chou 2006]. The amount of ash that could be consumed, used as a substitute raw material, will depend on the brick plant's production rate and the amount of ash that can be successfully incorporated into the brick body.

At the current brick plant production rate of 16 million bricks per year, utilizing 40% by weight of fly ash per brick, an annual consumption of approximately 14,000 tons of fly ash could be achieved. Successful commercial manufacture of bricks containing fly ash could provide a growing and profitable market for Illinois Basin coal ashes generated. It could also encourage electric power generation companies to continue to use Illinois Basin coals, and will help provide a reliable and inexpensive new source of raw materials for fired brick manufacturing.

4.4.3 Objectives
- Assess the technical, economic, and environmental suitability of fly ash for commercial production of fired bricks
- Conduct a public outreach campaign to promote the use of similar fly ash from other adequate sources by brick producers

4.4.4 Results
To prepare for the commercial-scale production test runs, precursor tests were conducted at the Illinois State Geological Survey (ISGS) bench-scale facility and more than 80 commercial-size test bricks of various formulations were made. In addition to the paving bricks containing fly ash at 10, 20, 30, 40 and 50% of volume balanced with clay and shale material, researchers at the ISGS also made three-hole building bricks containing fly ash up to 60% of volume (about 56% of weight).

Researchers fired a set of these mold-pressed green bricks using the ISGS kiln. (See Figure 6.) They also fired another set of these mold-pressed green bricks at the brick plant as part of a commercial firing. Both firings produced high-quality, attractive, strong paving and building bricks. (See Figure 7.)

The brick plant conducted two commercial-scale production test runs of paving bricks (2,000 bricks per run including extrusion and firing; see Figure 8). Run I produced paving bricks with a raw material formulation containing fly ash at 20% of volume (about 14% of weight) balanced with shale material at 80% of volume. Run II was composed of a mix of fly ash at 20% of volume, shale at 60% of volume, and clay at 20% of volume. These runs produced high-quality paving bricks with a yield of 75 and 100% for Runs I and II, respectively.

The engineering properties of these bricks either met or exceeded ASTM standards for commercial application. For example, their compressive strength

Figure 6. Mold-pressed paving bricks before firing.
was three times greater than the minimum allowable strength.

The brick plant also conducted four commercial-scale production test runs of three-hole building bricks (2,000 bricks per run, including extrusion and firing). The bricks contained fly ash levels of 0, 20, 30, and 40% of volume (about 37% of weight). The run with zero % fly ash was used as a control run to mimic the standard production formulation for the brick plant. Each run produced strong and attractive bricks with a commercially acceptable yield of greater than 95%. The engineering properties of these bricks either met or exceeded ASTM standards for commercial application. In fact, bricks containing Class F fly ash were comparable to bricks containing no fly ash. [Chou 2003].

An evaluation indicated that it would be economically feasible for the participating brick plant to use CPSIC’s fly ash as a raw material in commercial brick production. An environmental feasibility leaching study showed that, similar to the regular commercial brick, the fly ash containing bricks are environmentally safe construction products. [Chou 2006].

4.5 Development and demonstration of coal combustion by-products-filled composite materials for utility pole fabrication

Yoginder P. Chugh, Southern Illinois University, Principal Investigator

4.5.1 Background

The utility industries (electric, telephone, cable, and municipalities) use wooden poles for installing cables, lights, and transmission lines. It is estimated that more than a million poles are used in the Midwestern region annually. These poles range in height from 15–40 feet. Although wooden poles are relatively lightweight and easy to work with, as a naturally occurring material, the engineering characteristics vary widely. Wood also absorbs moisture which can decrease its engineering properties and cause decay in time. In general, as moisture content increases, wood strength decreases. To alleviate wood decay problems, poles are chemically treated at a cost of approximately $35 per pole every 5 years. These chemical treatments are an environmental concern as they interact with soil and water in and around the poles and upon disposal.

Researchers at Southern Illinois University are working on designing utility poles made of coal combustion by-products (CCBs). Each pole would use about 400 to 600 pounds of CCBs. The CCBs are mixed into a grout that is combined with different types of polymers which are used as a binding agent. Glass or other material fiber is added to improve flexural strength.

4.5.2 Significance

Timbering can cause landslides and compromise air and water quality. Utility poles made from CCBs benefit the environment and industries that generate CCBs. In addition to saving trees and disposing or recycling problematic industrial waste, CCB-based utility poles have several advantages over wooden poles:

- They are fireproof and termite proof.
- They do not have to be treated with creosote for weatherproofing which can cause water pollution from rainwater runoff.
- They are cheaper to install than wooden poles.
- They are not as easily damaged by animals and humans.

If the Midwest region were to replace the 250,000 wooden poles it uses in one year with CCB-fabricated poles, it would save that many trees and utilize from 87,500 to 100,000 tons of CCBs [Chugh 2000].
4.5.3 Objectives
- Characterize component materials (CCBs, resin, fiberglass, polymers).
- Evaluate CCB particle size and effect of loading level on engineering properties of composite material.
- Fabricate CCB pole and characterize engineering properties (compression, tension, and shear)
- Perform mechanical analysis.
- Design and analyze a composite pole design following ANSI specifications.
- Identify and develop suitable failure criteria for CCB pole material.
- Use finite element method to analyze and evaluate stress and deformation of the pole under expected loading conditions.
- Test and demonstrate the performance of the CCB material composite poles in both laboratory and field environments.

4.5.4 Results
Researchers used two types of CCBs, Class F and FBC, for the study. The ashes were mixed with polyurethane. In general, CCBs with finer ash particles provide higher compressive strength and elastic modulus as compared to samples composed of coarser ash particles. Grout mixes containing 60–65% fly ash yielded uniaxial compressive strengths ranging from 743 to 2,265 psi, with elastic modulus of 15,613 to 74,102 psi.

CCB chemical composition also played an important role in grout development. In general, variations in silica and calcium oxide content affect how CCBs bond with different polymers. This variation in chemical composition of CCBs creates different physical properties of the composites. The chemical composition of the fly ash also determines how much polymer must be added to the grout.

Researchers developed 81 mixes of polymers and grouts. The ultimate goal was to identify the mix with the highest compressive strength and elastic modulus as compared to samples composed of coarser ash particles. Grout mixes containing 60–65% fly ash yielded uniaxial compressive strengths ranging from 743 to 2,265 psi, with elastic modulus of 15,613 to 74,102 psi.

The results of this study indicate commercial production of CCB-based poles is feasible from both technical and environmental standpoints and should be pursued. CCB-based composite poles meet or exceed ANSI standards. The preferred design is cylindrical with glass fiber-reinforced thin outer-shell without any inner core material. (See Figure 9.) Fly ash as a filler in polymer has a positive effect on outer shell material strength and stiffness. Raw fly ash material should not exceed 15%. However, if the fly ash is graded, the content can be increased to 30%. The final product is comparable or superior to its wooden counterpart. Researchers estimate the payback period for investment to industrial partners to produce fly ash composite poles to be approximately 4 years [Chugh & Jinrong 2006].

4.6 Development of CCB fill materials for use as mechanically stabilized marine structures
Kelly A. Rusch, Louisiana State University, Principal Investigator

4.6.1 Background
Our nation’s coastal regions are an invaluable resource for biological diversity and recreation. However, coastal erosion is claiming these valuable lands at an alarming rate. The reasons for increased coastal erosion are many and range from increased population growth in the coastal areas to historic decisions to straighten rivers, which has resulted in the loss of alluvial sediment transport to sensitive areas. Implications are further reaching than lost recreational lands. The wetlands and barrier islands provide protection from tropical storm and hurricane storm surges. Wetlands in these coastal areas also serve as habitat for flora and fauna.

According to the Louisiana Coastal Wetlands Conservation and Restoration Task Force and the Wetlands Conservation and Restoration Authority, coastal wetlands are being lost at a rate of one acre every 24 minutes or 33.5 square miles per year. The problems are especially acute in the southern Atlantic and Gulf coasts. Louisiana contains about 42% of the nation’s wetlands and is losing about 80% of the total area annually. The continuation of this trend could cost the United States as much as $33.6 billion over the next 50 years from lost public use value [Rusch 2002].

Engineering approaches have been used to control erosion along coastlines. Seawalls, or dikes, are traditional structures used. Made of stone, concrete, and steel, these structures are relatively expensive. Limestone is a common material used as rip-rap in dike construction.
in Louisiana. Mined and barged from Arkansas, the in-place cost of this limestone ranges from $36–$52 per ton. Tremendous amounts of limestone are needed to construct these dikes which are four feet wide at the crown with 2–3:1 back slope and 3–4:1 front slope (water side) raised 2–3 feet above the water line. Material settling is a problem due to the compression of underlying soils from the weight of the limestone. Replacing the limestone with some lightweight material could reduce the compaction and settling problem of underlying soils and possibly reduce costs. Researchers examined the possibility of using Class C fly ash and FGD sludge as a replacement material for limestone in marine structures.

4.6.2 Significance
FGD gypsum is used in wallboard manufacture and in cement additives. Therefore, it has market value. FGD sludge, however, is a solid waste with no market value that the utility industry must dispose of in holding ponds or landfills. There are at least 92 power plants in 32 states with 157 coal-fired boiler units operating wet scrubbers that produce FGD sludge. If this waste material could be used in large marine structures such as seawalls, or dikes, there could be significant reductions in the amount going to landfills.

4.6.3 Objectives
– Refine composition of mixture of waste FGD sludge, class C fly ash, and Portland type II cement to make briquettes.
– Evaluate geotechnical engineering properties of briquettes.
– Evaluate impact of salt water on briquettes.
– Evaluate environmental impact from leaching using TCLP method.

4.6.4 Results
Raw FGD sludge was obtained from a power company in Florida and oven-dried at 45–50 degrees C for 6–12 hours depending on moisture content. The dried FGD was crushed and passed through a 1.46 mm sieve and combined with dried Portland type II cement and Class C fly ash and mixed with water equivalent to 8% of the dry mixture. The mixture was poured into a steel mold and compacted. The composites were allowed to cure at room temperature and 100% humidity for over two weeks before testing. Blocks of ten different composite combinations using FGD, Class C fly ash, and Portland Type II cement were fabricated for screening purposes.

A dynamic leaching test was performed to evaluate the long-term environmental effects of the stabilized solid wastes. During the screening process, leachate from FGD blocks was collected and analyzed for calcium and sulfate, which is used as indicators of the dissolution potential of the block/briquette. During the final evaluation of the five selected briquettes, leachate from the FGD briquettes was collected and analyzed for calcium and sulfate. The leachate volume to surface area ratio was 8:1. The composites were placed in artificial seawater and the leachate completely exchanged at intervals of several days up to 28 days. TCLP tests showed concentrations of Cr, Cd, As, Pb, and Se in the extraction fluid to be far below maximum concentration limits set by the U.S. Environmental Protection Agency for declaring a waste to be hazardous.

Geotechnical tests conducted on the briquettes show that all briquettes behave similarly. The economic analysis shows that briquettes can be manufactured on a large scale basis for less than $13 per ton. These results indicate the feasibility of using light-weight stabilized FGD briquettes as conventional structural fill material in coastal protection projects [Rusch 2002].

5 NEW PROJECTS

Three new projects recently funded by the U.S. Department of Energy – National Energy Technology Laboratory Combustion Byproducts Recycling Consortium focus on using CCBs for construction materials. Preliminary results are expected in 2007. Descriptions of these projects are below.

5.1 Manufacturing building products with fly ash and advanced CCBs
Melissa Chou, Illinois State Geological Survey, Principal Investigator

5.1.1 Project description
The goal of this project is to extend brick making from using Class F fly ash to using advanced CCBs to produce high quality fired bricks and innovative autoclaved aerated concrete (AAC) blocks. Objectives include: 1) using bench-scale test results to conduct/optimize scale-up production to make AAC blocks with Class F fly ash; 2) to assess the feasibility of utilizing advanced CCBs with or in place of traditional PC boiler fly ash to produce both bricks and AAC blocks; and 3) conduct scaled-up production tests with new CCB formulated bricks.

5.2 Evaluation of the durability and commercial Potential of 100% fly ash concrete
Jerry Stephens, Montana State University, Principal Investigator

5.3 Project description
The goal of this project is to determine long term durability and possible economic benefits of using 100%
fly ash concrete in construction applications. Objectives include: 1) identify fly ashes that are suitable for producing such concretes; 2) determine durability under deleterious environmental conditions; 3) identify and quantify potential economic benefits; and 4) inform design and construction professionals of the performance opportunities of this product.

5.4 New technology based approach to advance higher volume fly ash concrete with acceptable performance

Karthik Obla, National Ready Mixed Concrete Association, Principal Investigator

5.4.1 Project description

The goals of this project are to: 1) address lower early-age strengths (less than 7 days) and 2) delayed initial setting times; both which prevent large-scale utilization of high volume fly ash (HVFA) concrete. Objectives include: 1) demonstrate that HVFA concrete in the structure has sufficient early-age strengths to allow for optimized construction scheduling; and 2) demonstrate that by using suitable chemical admixtures or additions the initial setting times of Class C fly ash concrete is not delayed.

6 CONCLUSIONS

These projects demonstrate that there are numerous potential large-volume uses of coal combustion by-products building materials and products that are feasible for usage in the construction industry. These CCB material products may become more cost-effective for and more widely accepted by building material manufacturers and the construction industry as currently used standard building materials such as concrete, lumber, and gypsum wallboard become ever more costly and in shorter supply. For more information on these and other Combustion Byproducts Recycling Consortium projects and on the program itself, log onto: http://wvwri.nrrce.wvu.edu/programs/cbrc.

REFERENCES

Properties of calcium enriched fly ash and its utilization in concrete

X. Ma & Y. Gong
Harbin Institute of Technology, Harbin, Heilongjiang, P. R. China
X. Gong
Harbin Honglin Supervision Co. Ltd. of Civil Engineering, Harbin, P. R. China

ABSTRACT: Properties of Calcium enriched fly ash (CEFA) and its utilization is introduced in this paper. CEFA is combustion residue in electric power plants burning bituminous coal. The calcium is enriched intentionally by means of grinding limestone and coal powder together. CEFA is formed in the turbofurnace of the vertical cyclone boilers at the temperature of 1450°C to 1600°C. CEFA is characterized by higher calcium oxide and free lime, lower ferric oxide, lower water requirement, and higher reactivity. Self-setting and self-hardening are the specific performance because of existence of lime, calcium silicates and calcium aluminates in it. In concrete mixture, the volume of fly ash can be as high as 20% to 70% of the binder. Concrete develops a satisfactory compressive strength when cement is partially replaced.

1 INTRODUCTION

High-calcium fly ashes (ASTM Class C) are being widely used as supplementary cementing materials for its higher reactivity and cementitious properties. Though their systematic use in concrete still presents some difficulties, it will be accepted as an important supplementary cementing material in cement mortar and concrete [Tsimas & Moutsatsou-Tsima 2005].

Almost all of the high calcium fly ashes are produced directly from lignite or subbituminous coal. Some power plants in China are trying to produce high calcium fly ash from burning bituminous coal. The excess calcium is added by means of adding ground limestone in the coal powder. The crushed limestone and bituminous are blended and ground together in the mill. The content of lime stone could be adjusted sometimes according to the calorific value of the fuel and electric power consumption. The limestone decomposes into free calcium oxide and carbon dioxide at the temperature of 1450°C ~ 1600°C in the turbofurnace. Some hydraulic minerals, such as dicalcium silicate, tricalcium aluminate and tetracalcium aluminoferrite were found in it. For this kind of cementitious fly ash, the CaO is primarily from limestone, and the additional calcium is formed in a different way from that from the coal. This kind of high calcium fly ash differs from the common high calcium fly ash in composition and properties. The authors name this kind of fly ash calcium enriched fly ash (CEFA).

The purpose of this paper is to introduce manufacturing, chemical composition, physical properties, mineralogical composition of CEFA and its utilization in concrete.

The fly ash that was used in this research is from Acheng Power Plant. There are 5 vertical cyclone boilers that are fueled with the same bituminous coal powder blended with ground limestone. The average percentage of the fuel powder retained on 45 μm sieve is 37.7%. Both cyclone dust collectors and electrostatic dust collectors are used in this electric power plant.

2 CHEMICAL COMPOSITION OF CEFA

Chemical composition of the fly ashes from the electrostatic dust collectors is shown in table 1, so as that of common high calcium fly ashes.

The typical characteristic of CEFA is high calcium content, a large part of which is from limestone in the fuel powder. The average CaO content is from 23.3 to 27.5, which is much higher than that of the so called high calcium fly ash. The free lime content is comparatively higher than that of high calcium fly ash. 107 measurements of Free CaO were carried out. The result is shown in Figure1.

The problem concerning the fluctuated free CaO remains unsolvable. Small range fluctuation does not influence the performance of fly ash. The CEFA, of
which the free CaO content is as high as 12%, was used safely in the engineering.

Higher lime and free lime contents are the typical characteristic of CEFA. The common sense is that free lime is the main cause of unsoundness. So, the soundness may be the major consideration. Cement paste or concrete, once has set, does not undergo a large change in volume. Appreciable expansion could result in disruption of hardened cement paste. The hard burnt lime in cement hydrates very slowly, and because slaked lime occupies a larger volume than the original free calcium oxide, expansion takes place. Cements which exhibit this expansion are described as unsound [Neville 2000]. If lime hydrates rapidly before the paste has set, free lime does not produce unsoundness. It should be considered that the lime in CEFA is formed in 30 to 40 seconds, and that CaO is less prone to be hard burnt for comparatively lower Fe₂O₃ and Al₂O₃. So the lime may be moderate burnt. The newly formed lime, except for that combined in the compounds, is porous. So, when the fly ash is used in cement mortar or concrete, the free lime slakes before final set of the cement paste.

3 PHYSICAL PROPERTIES OF CEFA

The physical properties of the fly ashes are as follows. Specific surface area of the fly ash is from 310 m²/kg to 390 m²/kg, which is larger than that of cement. CEFAs from electrostatic dust collectors are finer than those from cyclone dust collectors. In common sense, powder with higher specific surface area absorbs more water, which may reduce the fluidity of the paste. But it is not the case in practice [Sun et al. 2003] The results indicate that the water requirement is lower than 90%, which means that CEFA is water reducing. The reasons may be that the particles of CEFA are spherical (Figure 2).

The lubricating effect of its spherical particles plays an important role in reducing the water requirement. Meanwhile, the glass structure has a weaker ability to absorb water [Feng & Shi 1998].
The strength activity index with portland cement is from 97 to 101 when water-cementitious material ratios are the same. It is bit higher than that of high calcium fly ash.

As mentioned before, soundness is the main consideration. The material for soundness experiments is made of portland cement (70%) and CEFA (30%). In the standard soundness experiment with Le Chatelier apparatus, the distance between the indicators is incredibly large. A phenomenon that must be taken into consideration is that the distance increases rapidly in the first half hour, but it does not change in the left 2 and half hours. It means that the lime in this kind of fly ash hydrates more quickly than free lime in cement and high calcium fly ash. It may finish hydrating when cement paste is in plastic state. That is why it does not show unsound when it has been used in civil engineering. In the construction works where CEFA was used, no failure due to unsoundness has taken place. Meanwhile, the finely dispersed free lime expands when it hydrates, and compensates a part of shrinkage of cement mortar or concrete at early age, and pore structure might be optimized because of expansion of f-CaO hydration.

4 MINERALOGICAL COMPOSITION OF CEFA

Because of the different production conditions, the mineral composition of CEFA should be different from that of common fly ashes. The X ray diffraction analysis results are shown in Figure 3. Figure 3(a) is the XRD diagram of CEFA from cyclone dust collectors and Figure 3(b) is that from electrostatic dust collectors.
The results of XRD analysis indicate that some hydraulic materials, such as $C_2S$, $C_3A$, were formed in CEFA. So, except for its pozzolanic property, CEFA exhibits hydraulic property. Self-setting and self-hardening are the typical characteristics of this materials. Mortars for plastering can be made without cement.

5 INFLUENCE OF CEFA ON STRENGTH OF CONCRETE

High Calcium fly ash concrete develops satisfactory compressive strength[Vagelis 2000, Cengiz et al. 2004]. To testify the influence of CEFA on compressive strength of concrete, CEFA was admixed into concrete. Cement is common portland cement, and the amount is 405 kg/m³. The amounts of water were adjusted to keep the same flow, the slump of which is 90 ± 5 mm. Cement was replaced with CEFA by 10%, 20%, 30%, and 40% respectively. Because of the water reducing effect, the water-cementitious material ratios are from 0.56 to 0.45. The cubic compressive strength of concrete is presented in table 3.

Table 3. Cubic compressive strength of concrete.

<table>
<thead>
<tr>
<th>Cement</th>
<th>CEFA</th>
<th>Replacement rate (%)</th>
<th>W/C</th>
<th>3d</th>
<th>7d</th>
<th>28d</th>
<th>56d</th>
<th>90d</th>
</tr>
</thead>
<tbody>
<tr>
<td>405.0</td>
<td>0</td>
<td>0</td>
<td>0.56</td>
<td>18.6</td>
<td>23.5</td>
<td>28.9</td>
<td>35.4</td>
<td>37.8</td>
</tr>
<tr>
<td>364.5</td>
<td>40.5</td>
<td>10</td>
<td>0.54</td>
<td>17.1</td>
<td>20.3</td>
<td>31.6</td>
<td>38.0</td>
<td>42.6</td>
</tr>
<tr>
<td>324.0</td>
<td>81.0</td>
<td>20</td>
<td>0.51</td>
<td>14.6</td>
<td>20.4</td>
<td>33.8</td>
<td>44.5</td>
<td>49.5</td>
</tr>
<tr>
<td>283.5</td>
<td>121.5</td>
<td>30</td>
<td>0.48</td>
<td>13.7</td>
<td>22.4</td>
<td>35.9</td>
<td>46.8</td>
<td>50.4</td>
</tr>
<tr>
<td>243.0</td>
<td>162.0</td>
<td>40</td>
<td>0.45</td>
<td>10.4</td>
<td>21.6</td>
<td>34.8</td>
<td>46.8</td>
<td>49.5</td>
</tr>
</tbody>
</table>

Figure 4 shows that the compressive strengths of concrete containing CEFA were lower at 3days and 7days, but concrete develops steadily at later ages. Except for the decreased water-cement ratio, pozzolanic and hydraulic properties of CEFA must be the main cause of this phenomenon. The authors found in the experiments that cement replacement can result in lower compressive strength at any age when water-cement ratio keeps constant, but the flow of the mixtures are much higher. When CEFA is used in high strength concrete, water reducing admixtures can be saved if water-cementitious materials ratios and slumps are the same.

For lower strength concrete, the amount of fly ash can be as high as 40% to 70% of cementitious materials. E.g. 90 kg/m³ ~ 100 kg/m³ cement and 250 kg/m³ ~ 280 kg/m³ fly ash can yield compressive strength of 30 MPa at the age of 60 days. For high strength concrete, less amount of fly ash is expected.

6 CONCLUSIONS

Calcium in fly ash is enriched successfully by means of grinding limestone and bituminous together in the mixer mill. The calcium oxide is formed at the temperature of 1450°C ~ 1600°C in 30 to 40 seconds in the turbofurnace of the boiler. This kind of high calcium fly ash differs from the common high calcium fly ash in composition and properties. It is named calcium enriched fly ash (CEFA).

CEFA has larger specific surface area, which can optimize the granulometric composition of the binder when it is added into mortar or concrete. The water requirement that is lower than 90% results in water reducing effect. Its self-setting and self-hardening properties yield steady development of compressive strength.

The free lime in CEFA slakes quickly before the cement has set. Higher free lime content does not lead to unsoundness.

In the case where portland cement is replaced by CEFA, Concrete develops a satisfactory compressive strength though it is comparatively lower at early age. Maintaining constant fluidity, the final strength exceeds that of the control because of water reducing effect.
REFERENCES


1 INTRODUCTION

Worldwide lead production in 2005 was over 7½ million tonnes. The largest worldwide use of lead for the past five years at 71% is in lead-acid batteries [ILZSG 2006]. The recycling of lead-acid batteries is a remarkable success story driven by the intrinsic value of the lead within the batteries. Recycling rates of greater than 90% have been achieved in the UK [Thornton et al. 2001].

1.1 Lead-acid batteries and their recycling

Lead-acid batteries are mainly used to provide starting, lighting and ignition (SLI) power for conventional motor vehicles. Other uses include emergency uninterruptible power sources for computers, hospitals and industry, through to the power supply for electric wheelchairs, golf buggies and milk floats.

As can be seen in figure 1, a typical SLI lead-acid battery is made of a heavy-duty plastic case (usually polypropylene) containing lead alloy pasted grids. The grids are made up of an alloy of lead and antimony (0.75–5%) with minor additions of copper, arsenic, tin and selenium. These elements are added to give the desired properties such as grain refinement, fluidity and hardness of the grids to suit the job required. Such properties might be resistance to fatigue due to the chemical reactions or the vibration produced by a car engine. The holes in the grids are filled with a paste, which is mainly lead oxide.

These pasted grids (known as plates) when immersed in sulphuric acid form an electrochemical cell. The negative terminal is made of grey metallic lead and the positive terminal is made up of various lead oxides. When the electrodes are connected via a conductor, electricity is generated. The flow of electricity leads to a deposit of PbSO₄ at each electrode. The reaction will continue until the electrical connection is removed, either or both of the electrodes are spent, the H₂SO₄ is consumed, or until the build up of PbSO₄ blocks further reaction. The reactions require the presence of lead dioxide (PbO₂) and lead metal (Pb), and each cell produces approximately 2V. The reactions are reversible and thus the battery can be recharged using a direct current.

Figure 1. Cross-section showing the components of a typical SLI lead-acid battery [Exide Technologies 2004].
The properties of lead make it ideal for use in batteries because it has good electrical conductivity and is resistant to corrosion. Lead is actually a very reactive metal, but it is this reactivity itself, which makes it resistant to corrosion. In sulphuric acid, lead metal reacts to form a protective layer of lead sulphate (PbSO₄), which is tightly adherent and itself insoluble in sulphuric acid. Thus the lead underneath the lead sulphate layer is protected from any further attack.

\[
\text{RHS: } \text{PbO}_2(s) + 4\text{H}^+ + 2e^- \rightarrow \text{Pb}^{2+} + 2\text{H}_2\text{O} \quad \text{(1.68V)}
\]
\[
\text{LHS: } \text{PbSO}_4 + 2e^- \rightarrow \text{Pb}^{2+} + \text{SO}_4^{2-} \quad \text{(-0.41V)}
\]
\[
\text{Overall: } \text{Pb} + \text{PbO}_2 + 2\text{H}_2\text{SO}_4
\]
\[
\rightarrow 2\text{PbSO}_4(s) + 2\text{H}_2\text{O} \quad \text{(2.09V)}
\]

Recycling of lead-acid batteries requires the batteries to be broken up using a battery-breaking machine. The battery-breaking machine either removes only the acid, or it separates the batteries into their components (i.e. the pastes, grids, separators and broken cases are collected in different hoppers and the acid is drained off). The breaking method used depends on the furnace used e.g. an Isasmelt furnace requires complete separation of the components whereas a shaft furnace smelts the batteries whole after the acid has been removed.

The polypropylene cases can be cleaned and reprocessed to make recycled plastic to be used for example in new battery case production. Other polymeric components present (PVC, ebonite, terylene, fibreglass) are either landfilled or used as a reductant in the smelting process. The drained acid is neutralised and disposed of, some examples of which are listed below:

\[
\text{H}_2\text{SO}_4 + 2\text{NaOH} \rightarrow 2\text{H}_2\text{O} + \text{Na}_2\text{SO}_4
\]
\[
\text{H}_2\text{SO}_4 + \text{Ca(OH)}_2 \rightarrow \text{CaSO}_4 + \text{H}_2\text{O}
\]
\[
\text{H}_2\text{SO}_4 + \text{CaCO}_3 + \text{H}_2\text{O} \rightarrow \text{CaSO}_4 + \text{2H}_2\text{O} + \text{CO}_2
\]

The pastes and grids are smelted in a furnace. Traditionally this was done in a blast furnace, although they have recently gone out of favour due to the high costs of metallurgical coke and the difficulty of collecting the dust and fume. Nowadays it is more common to use a rotary furnace or an Isasmelt furnace. The rotary furnace is used as a one-stage process with the feed adjusted to give a lead with the desired properties or as a two-stage process producing a soft lead and an antimonial lead. With the two-stage process, during the first stage, the furnace conditions are kept such that all the antimony is oxidized while the lead is not. The antimony oxides are not soluble in lead so the molten lead metal can be removed leaving the lead oxide and sulphate within the antimonial slag. Stage two involves the addition of coke or anthracite fines and sodium carbonate to the furnace. This leads to the reduction of the lead and antimony oxides and the lead sulphate. An antimonial lead and a slag are produced which are refined and disposed of respectively.

The Isasmelt furnace is fed with battery paste that has been desulphurised, typically to a lead oxide paste or to lead carbonate.

\[
\text{PbSO}_4 + 2\text{NaOH} \rightarrow \text{PbO} + \text{Na}_2\text{SO}_4 + \text{H}_2\text{O}
\]

A lance is used to melt the desulphurised paste. For 36 hours the furnace is fed with wet paste and coal as a reductant. Every three hours lead of 99.9% purity is tapped off. As the process continues, the lead content of the slag falls as the impurity levels rise. The lance keeps the slag fluid by raising the temperature from 900°C to 1000°C during the process. After 36 hours, no more paste is added to the furnace and the remaining slag is reduced to produce an antimonial lead alloy. This occurs over two hours during which time the temperature is raised to 1200°C and fluxes are added. When the reduction is complete the lance is removed, the antimonial lead alloy (containing >98.5% lead and antimony) is tapped off and the remaining slag is discarded (containing <0.5% lead).

Similarly to the rotary furnace, off gases are cooled and filtered for dust particles. The battery grid metal is melted in a separate furnace.

The advantages of this process over the rotary furnace are a higher thermal efficiency and therefore lower running expenses, the omission of soda fluxes, the direct production of soft lead and antimonial lead alloy, production of waste slags having low lead concentrations, and the improved process hygiene since the process is semi-continuous. A disadvantage of the process is that when the pastes are desulphurised before smelting, a waste solution of sodium sulphate is produced which requires disposal.

Secondary lead is then refined in kettles after it has been collected from the smelter. The main impurities present that need to be removed are copper, tin, antimony and arsenic. The final lead produced will be 99.99% pure or even 99.9999% with extra treatment.

A diagram of the main processes involved and the volumes of products/waste streams for a typical conventional battery recycling plant are shown in figure 2.

1.2 The cleanlead project

As can be seen in figure 2, the secondary lead industry produces acidic sludge from the neutralised waste acid, sodium sulphate if the pastes are desulphurised, leachable toxic slags and vast amounts of exhaust gases containing SO₂ and metal fume. Due to recent environmental legislation, costs have increased substantially due to: (i) the necessity to dispose of the
waste produced and (ii) the energy required. Therefore the secondary lead industry is becoming uneconomical. It is feared that this will lead to a reduction in the recycling rates of lead-acid batteries unless alternative economical and zero waste production methods are developed.

A new zero waste process has been proposed for the secondary lead industry (see figure 3) and has been designed so that it can either optimise the current processes, or it can easily be added to existing plant to make it more cost effective for industry.

One of the proposals for the new process is that instead of neutralizing the acid to produce a waste acidic sludge, it was proposed that the acid is purified. This will take place either by solvent extraction, absorption resins, membrane techniques (e.g. reverse osmosis, nanofiltration, electrodialysis) or bio-reduction. The purified acid can then be used to refill new lead-acid

Figure 2. Typical lead-acid battery recycling plant showing volumes of wastes created.

Figure 3. Proposed Cleanlead factory showing zero waste streams.
batteries, or neutralised with Ca(OH)$_2$ or CaCO$_3$ under controlled conditions to produce high quality gypsum products for use in industry.

Rather than desulphurisation of the battery paste with sodium hydroxide producing the waste sodium sulphate solution, it was proposed that lime is added to the waste Na$_2$SO$_4$ solution. This will produce a clean gypsum product, which is saleable and also regenerate the NaOH leachant, thus creating no wastes.

\[
PbSO_4 + 2NaOH \leftrightarrow Pb(OH)_2 + Na_2SO_4
\]
\[
Na_2SO_4 + Ca(OH)_2 \leftrightarrow 2NaOH + CaSO_4
\]

Conventionally spent batteries are processed pyrometallurgically producing toxic slags that are land filled. In this new process, the pastes are fully desulphurised which means the furnaces will produce ferro-silicate based slags (and also less slag). Ferro-silicate slags are non-hazardous and are usable as an alternative to other raw materials e.g. as an aggregate in concrete. The desulphurisation of the pastes would also eliminate the generation of sulphur dioxide exhaust gases directly from the batteries.

A new lead electrolytic process has been proposed to produce 99.99% pure lead whilst using less energy (0.5 kWh/kg Pb) in an environmentally friendly alkaline media. This would replace the conventional battery paste smelting stages, which are the main waste generators and are energy intensive. The desulphurised paste (mainly PbO) is to be dissolved in alkaline media, where lead has a high solubility whereas the metallic impurities have a low solubility.

\[
PbO + 2NaOH + H_2O \rightarrow PbNa_2(OH)_4
\]

To prevent the passing of any impurities into the electrolyte and thus co-deposit with the lead, a counter-current leaching stage will be used. This is intended to reduce the amount of impurities such as antimony, copper, arsenic, and tin in the electrolyte. The purified electrolyte will electrowin deposit 99.99% pure lead on the cathode in an electrowinning cell while oxygen will be released at the anode.

\[
\text{Cathode: } PbNa_2(OH)_4 + 2e^- \rightarrow Pb^0 + 2NaOH + OH^-
\]
\[
\text{Anode: } 2OH^- + 2e^- \rightarrow \frac{1}{2}O_2 + H_2O
\]
\[
\text{Overall: } PbNa_2(OH)_4 \rightarrow Pb^0 + \frac{1}{2}O_2 + 2NaOH + H_2O
\]

“"The complete new process will use minimal energy, virtually no raw materials, generate zero waste and consequently greatly reduce the operating cost [CORDIS 2004]."

The role of the author within the Cleanlead consortium was to develop a high quality synthetic gypsum by neutralising the waste battery acid.

\[
Ca(OH)_2 + H_2SO_4 \rightarrow CaSO_4.2H_2O
\]
\[
CaCO_3 + H_2SO_4 + H_2O \rightarrow CaSO_4.2H_2O + CO_2
\]

1.3 Gypsum

Gypsum is a naturally occurring mineral used in a wide variety of industrial applications. These include in plasterboard, as a set-controlling additive in cement and as a soil conditioner in agriculture. Each industrial end use requires a different specification gypsum.

The chemical name of gypsum is calcium sulphate dihydrate. Calcium sulphate exists in a number of compounds with the general formula CaSO$_4$.$x$H$_2$O (where $x$ varies from 0 to 2). The most common forms are where $x = 0.5$ (bassanite, “$\alpha$- or $\beta$-hemihydrate”, “stucco”), and $x = 0$ (“anhydrite”).

Two forms of hemihydrate, $\alpha$ and $\beta$ [Hand 1997], have been accepted with some controversy. The $\alpha$-form is produced by dehydrating dihydrate gypsum at temperatures higher than 97°C in an atmosphere of saturated steam (autoclaving), or by exposing dihydrate gypsum to high concentration inorganic salt solutions or dilute sulphuric acid [Zurz et al. 1991, Othmer 1992].

The $\beta$-form of hemihydrate is produced, by dehydrating (calcining) dihydrate gypsum without an atmosphere of saturated steam. A device called a calciner is typically used and these vary between direct, indirect, batch and continuous processes.

Typical calciners used industrially include the kettle calciner and the flash calciner. A kettle operates by passing hot air through pipes which run across the kettle chamber. Inside, the ground gypsum is added by an inlet at the bottom. The gypsum then loses its water of crystallisation and converts into hemihydrate. The steam provides some agitation of the powders, keeping the system fluidised, and the “stucco” produced will rise to the top (the product being less dense than the starting material), where it overflows into a receptacle.

Kettle temperatures are adjusted depending on the exact hemihydrate requirements or on the actual kettle set-up, but typically are around 104°C during the fill cycle. Upon filling, the kettle is heated for an hour to 115–120°C to remove any free moisture. After this time, known as the boil or the drag, the temperature of the gypsum is increased to 150–155°C to form $\beta$-hemihydrate. Early kettle types were batch, indirect processes, but now modern varieties exist which are direct and continuous.

A flash calciner operates as a continuous direct process with rock gypsum being introduced into a ball mill and the dust produced immediately passing through a very high temperature air stream for a few seconds, which is sufficient for complete dehydration.
to stucco. Flash calciner stucco rapidly rehydrates back to the dihydrate when mixed with water (i.e. has a quick setting time) and is thus particularly suited for use in the plasterboard industry. The process has a disadvantage over kettles in that it is not suitable for synthetic gypsum.

Anhydrite occurs in three distinct forms, anhydrite I, anhydrite II, and anhydrite III [Hand 1997]. Anhydrite II is the common rock form found naturally in nature and is also formed from the high temperature (~900°C) dehydration of gypsum. It is often referred to as insoluble, natural or “deadburn” gypsum due to its inertness and thus it does not rehydrate on an industrially viable timescale. Anhydrite III is formed from the low temperature (140–200°C) dehydration of α- or β-hemihydrate; its trivial names include “soluble” and “overburn”, and it is a component of freshly prepared stucco. It has the same crystal lattice as hemihydrate and therefore readily absorbs moisture from the atmosphere to revert back to hemihydrate. It is sold as a highly potent and reactive desiccant Drierite™. Finally, anhydrite I is derived from the over-calcination of anhydrite-II. It is unstable at temperatures below 900°C, hence not much is known with certainty about this material. Above approximately 1200°C, it decomposes into CaO and SO₃.

1.4 Gypsum uses

Figure 4 shows that the three largest markets for European use of gypsum are plasters, plasterboard and cement (where plasters are defined as commercially available bagged powder gypsum products used for applications as diverse as making sculptures through to plastering walls). The formulation of plasters and their use on-site are region-specific. Often, plasters have been mined locally for a long time and the natural gypsum often has characteristic properties which are difficult to replicate with a synthetic material. This means that there is no uniform specification for plasters and they are therefore not being considered as an end use of the Cleanlead gypsum produced.

This leaves plasterboard and cement as the two main targets for end uses of Cleanlead gypsum. Other uses to be considered include use as a self-levelling floorscreed or as a soil re-conditioner in agriculture.

Gypsum is traditionally supplied from quarries or mines, but these sources are being increasingly replaced with by-product gypsum, primarily from flue-gas desulphurisation (FGD) plants at power stations [Ng (no date), EPRI (USA) 1994]. Other synthetic gypsums are manufactured such as titanogypsum, a by-product from the manufacture of anatase titanium dioxide [Albrecht (no date)], fluoroanhydrite, a by-product from the manufacture of hydrofluoric acid, phosphogypsum, a by-product from the manufacture of phosphoric acid mainly for the fertilisers industry, citrogypsum, tartarogypsum and lactogypsum, by-products from the manufacture of organic acids, and borogypsum, a by-product from the manufacture of boric acid.

It is anticipated that the major problems, which may occur with Cleanlead gypsum are similar to those encountered with FGD gypsum, titano-gypsum and other synthetic gypsums. These problems relate to impurities, particle size and morphology with respect to the conditions of manufacture [Ng (no date), EPRI (USA) 1994, Albrecht (no date), Park and Parrow (no date)].

Analytical techniques available to characterise the gypsum samples were X-ray diffraction (XRD), optical microscopy, particle size analysis, thermogravimetric analysis (TGA) and bulk property analysis (for example the apparent density of the sample). These techniques were chosen because they allow the gathering of all of the data required to check the quality and suitability of the gypsum produced for use in different industrial applications.

The modern plasterboard factory runs as a continuous process applying a layer of gypsum slurry to a bottom layer of paper before a top sheet of paper is applied (see figure 5). Initially the milled gypsum (natural mined rock gypsum, synthetic gypsum, or a blend of both) is calcined from the dihydrate phase to the hemihydrate phase by heating.

\[
\text{CaSO}_4\cdot2\text{H}_2\text{O} + \Delta \leftrightarrow \text{CaSO}_4\cdot\frac{1}{2}\text{H}_2\text{O} + 1\frac{1}{2}\text{H}_2\text{O}
\]

The hemihydrate plaster is then mixed with water and the required additives to control the setting of the plaster (such as starch, accelerators and retarders). This plaster slurry is then applied between two layers of paper lining. The hemihydrate plaster now hydrates back to the dihydrate phase with the gypsum crystals growing between the paper fibres and forming the “wet bond”. The plasterboard is then dried in ovens to remove any excess free water and the mechanical bond between the gypsum core and the paper is the...
only bond which prevents the paper from being separated in the drier. It is for this reason that stringent specifications exist on impurities which are known to disrupt this bond, the principal of these being chloride and soluble cations such as sodium and magnesium.

A simplified cross-sectional diagram of a sheet of plasterboard is shown in figure 6. Typically plasterboard is 12.5 mm thick and weighs 8.5 kg/m².

During the drying process, the starch blended into the slurry dissolves, forms a gel by absorbing approximately thirty-fold of its weight in water, and migrates to the paper interface, where the water evaporates. In this process, the starch gel forms a coating over the gypsum crystals, thereby providing protection of the gypsum core, and in particular the paper-gypsum interface against calcination and bond failure. After completion of the drying, the starch re-precipitates, and forms in conjunction with the gypsum crystals the “dry bond”, which is hydrogen-bonding between the cellulose and starch molecules.

The gypsum-paper mechanical bond is negatively affected by all attempts to improve the economy of the plasterboard production. Using less gypsum to reduce the cost of the plasterboard and hence having a lower board weight leads to there being less gypsum and more air at the gypsum/paper interface. This reduction in the amount of gypsum at the gypsum/paper interface weakens the mechanical bond because there is less gypsum crystal growth into the paper fibres to create the mechanical bond.

If the board production line speed is increased to speed up production, the gypsum hemihydrate slurry between the paper sheets does not have time to fully rehydrate to dihydrate before being dried. This leads to a poorer mechanical bond as not all of the hemihydrate rehydrates to dihydrate whose crystals grow into the paper fibres to create the mechanical bond.

Less water can be added to the gypsum slurry to improve economy; this has two effects. Firstly the board dries quicker and thus the board production line can be run faster. Secondly, it decreases the drying energy costs (and thus CO₂ emissions). Unfortunately the use of less water leads to slower and incomplete hydration of the hemihydrate, which leads to a poorer mechanical bond due to there being less dihydrate crystals growing into the paper fibres.

The mechanical bond is the most important factor in plasterboard production and the quality of the finished product. The quality of the gypsum used and in particular, the particle size has a very large effect on the mechanical bond. Figure 7 shows a simplistic consideration assuming spherical particles of the crystal packing at the gypsum/paper interface.

When the particle size distribution is spread over a wide range, there is greater intimacy between the gypsum particles and the paper i.e. there are fewer gaps and smaller. This means that more gypsum crystals can grow into the paper fibres and hence the mechanical bond is greater. A narrow distribution of particle sizes is undesirable as the space filling of the particles would be poor. This is because uniform size round particles leave gaps between themselves.

The specification for particle size is usually quoted as the median particle size (d₅₀). For use in plasterboard the optimum d₅₀ is 30–80 µm [Lafarge Plasterboard 2001]. As can be seen from figure 7, ideally the particle size distribution would be bimodal to give a good particle size spread (with a peak at around 30 µm and another at 80 µm) to leave fewer gaps
between the crystals. Particle size distribution also affects storage and handling since fine dust like powders are difficult to transport and handle on conveyor belts and hoppers. In the traditional plasterboard plant natural gypsum is supplied as rocks, which are easier for the machinery to handle, although these have to be finely milled before or during calcination.

Any traditional plasterboard factory running on a source of natural gypsum, and wishing to use synthetic gypsum such as desulphogypsum (DSG) from FGD plants, needs to either use a facility that converts the moist, paste-like synthetic gypsum into a dry rock-like material, or (as is often more economically viable) invest in a change of gypsum raw material storage and handling, calcination and milling equipment. It should be remembered that there are health implications associated with handling fine dust like powders.

Particle size also affects the water demand of the gypsum because both smaller and larger particles demand more water (i.e. more water has to be added to the hemihydrate in the slurry). Smaller particles have a greater surface area whereas larger particles have gaps between the crystals which fill with water upon hydration. Thus the drying time increases since more free water must be removed from the finished plasterboard. Acicular crystals will pack poorly because the excess free water must be removed. The poor packing of the acicular shaped crystals leads to a higher water demand in a similar way as an out of specification particle size distribution would. The higher water demand leads to higher drying costs and weaker paper gypsum bonding. Also the poorer space filling of the acicular shaped crystals (similar to that of the narrow particle size distribution in figure 7) leads to a weaker mechanical bond at the gypsum/paper interface. An acicular crystal is defined by industry as a crystal with a length three times or greater than its width [Lafarge Plasterboard 2001]. Acicular shaped crystals also have a tendency to interlock with each other which leads to difficulties with storage and processing as they block silos and hoppers.

The soluble ion content within the gypsum is important as well. Sodium and chloride ions particularly, and others also to a lesser extent, interfere with the hydration of the hemihydrate during plasterboard production. This weakens the gypsum/paper mechanical bond. The presence of some ions can also lead to aesthetic defects in the gypsum products such as staining of plasterboard.

Trace impurities and in particular heavy metals in the gypsum produced also need to be monitored. These trace impurities have no effect on the plasterboard produced but they need to be minimised so as to be within limits imposed by the EU Construction Products Directive in relation to harmful substances [1989].

In addition to the difficulties with handling dry powders, another handling problem with synthetic gypsum is the free water content of the received material. If the moisture content (MC) is greater than 10%, the gypsum is “sticky” and therefore difficult to handle due to sticking to conveyor belts and hoppers. The maximum permitted MC tolerated by the plasterboard industry is 12% [Lafarge Plasterboard 2001]. A higher %MC increases transportation costs because it is necessary to pay for the transportation of more water. Market reports suggest that 50–90% of the delivered cost to the user of the gypsum is due to transportation costs [Lafarge Plasterboard 2000]. To make matters worse, when the high %MC gypsum arrives at the plasterboard factory, more energy is required to calcine the gypsum to hemihydrate because the excess free water must be removed.

The shape of the gypsum crystals themselves has important implications on the quality of the stucco produced. The effect of crystal shape is similar to the effect of the particle size distribution. This is because acicular (needle) shaped crystals negatively affect the quality of plasterboard produced. Round shaped crystals will pack closely, whereas acicular shaped crystals pack poorly. The poor packing of the acicular shaped crystals leads to a higher water demand in a similar way as an out of specification particle size distribution would. The higher water demand leads to higher drying costs and weaker paper gypsum bonding. Also the poorer space filling of the acicular shaped crystals (similar to that of the narrow particle size distribution in figure 7) leads to a weaker mechanical bond at the gypsum/paper interface. An acicular crystal is defined by industry as a crystal with a length three times or greater than its width [Lafarge Plasterboard 2001]. Acicular shaped crystals also have a tendency to interlock with each other which leads to difficulties with storage and processing as they block silos and hoppers.

The use of optical and scanning electron microscopes is useful since they provide detail of the crystal shapes and also further insight into the particle size distribution. The bulk properties such as apparent bulk density of the gypsum samples can also provide a measure of the extent of the presence of acicular crystals. The apparent bulk density of a gypsum sample can be calculated by weighing a known volume of gypsum. Acicular crystals will pack poorly because gaps will be left between the crystals and will therefore have a lower apparent density. A sample with an apparent density greater than or equal to 1 kg/litre is deemed to be acceptable for industrial use [Lafarge Plasterboard 2001]. A high apparent density is important for the gypsum also to reduce transportation costs.

An important consideration for synthetic gypsum is the angle of repose of the gypsum in the store warehouse.
The angle of repose is the angle formed between the ground and gypsum from the stacking process in the warehouse where it is stored. The gypsum enters the warehouse on a conveyor belt at the top of the pile dropping it so that it stacks naturally. Natural gypsum has an angle of repose of 45° whereas synthetic gypsum has an angle of repose of 60°–80°.

Figure 8 shows the problem caused by the synthetic gypsum angle of repose. It can be seen that if synthetic gypsum is incorporated into a warehouse designed for natural gypsum where the roof slopes at an angle of 45°, there is wasted room and hence less capacity. This will reduce the economy of the manufacturing process.

If the synthetic gypsum is briquetted (see figure 9), the angle of repose can be improved. This is because the briquettes produced will be similar in shape to natural rock gypsum.

For use in plasterboard it is clear that the Cleanlead gypsum will have to be dihydrate gypsum. To produce pure β-hemihydrate from the Cleanlead process would be an advantage because calcining costs would be eliminated. Although if β-hemihydrate is manufactured from Cleanlead technology, it has to be at least 95% pure because the presence of any dihydrate gypsum or anhydrite will lead to a poor mechanical bond as these crystals will not “grow” into the paper fibres. The presence of dihydrate in β-hemihydrate is a particular problem due to the fact that β-hemihydrate will naturally re-hydrate back to dihydrate absorbing moisture from the atmosphere. Therefore all β-hemihydrate has a limited shelf life, which is a maximum of 6 months if stored under optimum “dry” conditions. X-ray Diffraction (XRD) is a technique used to determine which phases of gypsum are present in a gypsum sample (although XRD cannot distinguish between α and β hemihydrate [Hand 1997]). The plasterboard industry requires a minimum purity of 95% CaSO₄⋅2H₂O in its secondary gypsum. Thermogravimetric (TG) data can also be used to determine the phases present by recording the mass lost upon heating of a sample which corresponds to the calcinations of the water of crystallisation (although TG analysis cannot distinguish between α and β hemihydrate or anhydrite I, II and III).

An alternative industrial use for Cleanlead gypsum would be in the cement industry, which adds between 3% and 8% gypsum as an additive to cement when the cement clinker is being ground. Gypsum is added to cement to control the setting time for the cement. Tricalcium aluminate, one of the 4 main components of cement, reacts rapidly with water causing the cement to “flash set” within minutes. Gypsum controls the setting time of cement by hydrating with tricalcium aluminate to form an alumino-sulphate mineral (called ettringite) coating on the surface of the tricalcium aluminate preventing further hydration [Bye 1983].

The gypsum is added to the grinding mills as the dihydrate form with anything up to 70% anhydrite so as to alter the setting times of the cement due to the solubilities of the various gypsum phases depending on the manufacturer’s requirements. For example, anhydrite II is less soluble than dihydrate. It is important that there is no hemihydrate present in the cement, otherwise the hemihydrate could hydrate to dihydrate and harden on hydration of the cement causing “false set”.

Gypsum also has other effects in cement production such as it acts as a grinding aid in the ball mill [Bye 1983]. It also has an effect on the expansion of the cement and can stop cement shrinking on setting. Conversely it can cause negative effects such as excessive expansion and therefore cracking upon setting.

Cement has stringent limits on the presence of alkalis such as sodium and potassium due to the alkali silica reaction in concretes containing certain reactive silica aggregates, which leads to cracking of the concrete. Other impurities of concern are the chloride content which can attack and rust the steel in reinforced concrete leading to cracking. Therefore all gypsum samples must contain minimal levels of sodium, potassium and chloride.
Other uses include the use of gypsum as a floor screed. In the majority of modern construction, floor screeds are manufactured from a concrete product. Standard concrete floor screeds are not self-levelling and thus need to be carefully applied and skillfully levelled out. A newer approach is to use gypsum, which is self-levelling. This means that the need for expensive finishing work is no longer required and the products rapid strength means that workers are able to walk on it more quickly relative to concrete (48 hours for gypsum compared with weeks for concrete) and are thus able to carry on with their work. The gypsum binder is mixed with sand, other aggregates and water, then applied to the floor and encouraged to fill the entire room. The gypsum used is either anhydrite-II or \( \alpha \)-hemihydrate. Anhydrite-II can be delivered as a ready-mixed slurry as it is workable for 5 hours. The setting time and strength of the floor can be adjusted by changes to the formulation and is done at the request of the customer by the producer of the anhydrite binder. \( \alpha \)-hemihydrate is mixed in the same manner as anhydrite-II (i.e. with sand, aggregates and water) though the user carries out the mixing on-site as the setting time is approximately 30 minutes.

It is unlikely that Cleanlead gypsum could be used as a floor screed as anhydrite II cannot be manufactured directly from the Cleanlead process. There is a patented process which is used to calcine DSG to anhydrite II in a continuous flash process. It is anticipated that small proportions of the feedstock into this Rocal® process could be synthetic gypsum other than DSG.

The least economically viable use of the Cleanlead gypsum would be in agriculture as a soil conditioner. Gypsum is an excellent soil re-conditioner for many reasons. It is a good source of calcium and sulphur, which are the seventh and ninth most important of the 16 essential nutrients for plants [North Pacific AG Products 2004]. Gypsum slowly dissolves in the moisture present in soils, which allows plants roots to absorb calcium ions and sulphate ions. As well as being an essential nutrient, calcium helps plant roots in the uptake of other minerals. Other advantages of gypsum as a soil re-conditioner, the gypsum can be low quality with no requirements for particle size, crystal shape or morphology. The only specification for gypsum for use in agriculture is restrictions on heavy metal composition for the obvious toxicity implications of heavy metals passing into the food chain [MAFF 1998].

2 EXPERIMENTAL

2.1 Initial experiments

Initially a series of bench scale batch reactions were performed to assess the parameters which had the greatest influence on the quality of the gypsum produced. These experiments were performed using synthetic reagents i.e. sulphuric acid and calcium hydroxide or calcium carbonate.

Early experiments added \( \text{Ca(OH)}_2 \) powder to neutralise the \( \text{H}_2\text{SO}_4 \) to a target pH of 7. The rate of powder addition, filtration method and drying temperature were varied. It was noted that the reactions were difficult to control when adding the neutralising agent as a dry powder to the sulphuric acid, so for all subsequent reactions the \( \text{Ca(OH)}_2 \) or \( \text{CaCO}_3 \) was added as a slurry.

The following parameters were varied throughout the initial phase of experiments:

- \( \text{Ca(OH)}_2 \) or \( \text{CaCO}_3 \)
- \( \text{Ca(OH)}_2/\text{CaCO}_3 \) added as powder or slurry
- \( \text{H}_2\text{SO}_4 \) concentration (g/dm\(^3\)) of 108, 101 and 38.1
- \( \text{H}_2\text{SO}_4 \) analytical or doped with metals to “model acid concentration”
- Target final pH of 2, 4 and 7
- Reaction temperature of 20°C, 60°C and 80°C
- Reaction time (hours) of 1, 2 and 4
- Filtration method of vacuum or pressure
- Drying method of 21°C, 50°C, 70°C, 100°C and Isopropanol/Ethanol.

Further experimentation

These later experiments saw a shift to larger scale reactors and continuous or semi-continuous processes employing peristaltic pumps.

The following parameters were varied throughout these later experiments:

- \( \text{Ca(OH)}_2 \) or \( \text{CaCO}_3 \)
- Two different commercial \( \text{CaCO}_3 \)’s
- \( \text{Ca(OH)}_2/\text{CaCO}_3 \) slurry concentration (g/dm\(^3\)) of 75, 200, 350 and 500
– $\text{H}_2\text{SO}_4$ concentration (g/dm$^3$) of 100, 110, 114 and 120
– $\text{H}_2\text{SO}_4$ analytical, doped with metals to “model acid concentration” and waste battery acid
– Target final pH of 2, 3, 4 and 6
– Reaction temperature of 40°C and 60°C
– Retention time (hours) of 1, 2 and 3
– Reactor solids density (g/dm$^3$) 79, 126, 128, 133, 146, 153 and 164
– Addition of gypsum crystals to seed reaction
– Hydrocycloning gypsum pulp to return fines to reactor as seeds
– Mixer speed (rpm) of 1240, 1400, 1650 and 1900
– Filtration method of vacuum or pressure
– Drying method of 21°C, 40°C, 50°C, 80°C and 100°C.

As the experiments were continuous rather than batch, the times listed above are retention times in the reactor rather than total reaction time as in Section 2.1.

2.3 Pilot plant

Finally, three Cleanlead gypsums were produced separately in a large pilot plant from three different waste battery acid streams using commercial quality limestone. These experiments were carried out after over 150 different gypsum samples had been prepared in the previous experiments and subsequently analysed.

The pilot plant consisted of a 300 dm$^3$ reactor with a heating element and a high speed agitator designed to exert high shear forces on the reactants. Sulphuric acid and limestone slurry (500 g/dm$^3$) were pumped continuously into the reactor and the gypsum pulp with a reactor retention time of 2 hours was pumped out periodically. The collected gypsum pulp was passed through a hydrocyclone system twice with the first stage underflow collected as product and the second stage underflow collected and returned to the reactor to act as gypsum seeds. The pilot plant is shown in figure 10.

The three Cleanlead gypsums were produced from untreated contaminated battery acid, nanofiltration treated contaminated battery acid and diffusion dialysis treated contaminated battery acid.

3 RESULTS

All gypsums produced were analysed to determine the crystal shape, size, phase (and purity) and moisture content where possible.

All gypsum samples analysed by XRD and TG analysis were high purity (>95%) gypsums. However, the drying temperature of the samples had a significant affect on the gypsum phase produced. For all samples dried at 50°C or less, the samples were dihydrate gypsum.

Samples dried at 70°C varied between samples containing only dihydrate, samples containing mainly dihydrate with a trace of hemihydrate and finally samples that contained more hemihydrate than dihydrate. An example of the XRD spectra of some samples dried to constant mass at 70°C can be seen in figure 11. Samples dried at 80°C contained a majority of hemihydrate with some dihydrate. Samples dried at 100°C tended to contain almost exclusively hemihydrate.

It should be noted that prior to analysis, the samples were dried to constant mass at the specified temperature over a few days in temperature controlled ovens. Although, when pre-dried dihydrate samples were heated at relatively quick rates (5°C/min) in the TG balance, decomposition to hemihydrate and anhydrite would not start until at least 100°C or in some cases 120°C.

From these results it would appear that dihydrate gypsum begins to dehydrate at temperatures around 70°C if given enough time, otherwise it needs to be heated to over 100°C to dehydrate.

Analysis of the shape of the crystals showed that initially they were mainly acicular in nature although later experiments improved the shape towards the preferred more regular squat prismatic crystals. Figure 12 shows the improvement in crystal shape from an early sample to a sample produced on the pilot plant viewed under a scanning electron microscope. The negative effects of acicular crystals on storage of gypsum were readily observed by the extremely low bulk densities of the early samples with bulk densities as low as 0.2 kg/dm$^3$. The pilot plant gypsums fell short of the 1.0 kg/dm$^3$ target, but still achieved 0.8 kg/dm$^3$.

Another negative effect of the acicular crystals was the reduction in filtration efficiency associated with the poor packing of these crystals. As an example some early predominantly acicular samples had free moisture contents of over 40% whereas the final pilot
plant samples were around 10% or less. Some of this improvement can be accounted in the change from vacuum filtration to pressure filtration.

The particle size distributions (PSDs) of the pilot plant gypsums were all at the lower end of the industrial specification of $30 \mu m < d_{50} < 80 \mu m$. Some earlier samples appeared to have the best PSDs, although this was generally due to the presence of very long thin acicular crystals as shown in figure 12.

Figure 11. XRD spectra of three different gypsum samples all dried at 70°C. The top sample contains only dihydrate (DH), the middle sample contains mainly DH with some hemihydrate (HH) and the bottom sample contains mainly HH with some DH.

Figure 12. The left hand picture shows the poor quality acicular crystals which resulted in poor bulk handling properties and a high moisture content compared with a gypsum sample produced on the final pilot plant in the right hand picture.
A trend was noticed for samples from the same batch which had been dried at different temperatures to have smaller PSDs if they had begun to calcine from dihydrate to hemihydrate (confirmed by analysing the samples with XRD and TG analysis). High magnification analysis of these samples using a SEM showed that the crystals which were smooth and regular shaped were dihydrate whereas the samples containing dihydrate and hemihydrate had crystals which were fractured and rough (see figure 13). It is therefore likely that the shift in the PSDs of these samples to lower values was due to the ultrasonic dispersion mechanism in the laser particle size analyser dislodging the “flaky” surface or even breaking some of the crystals.

4 INDUSTRIAL APPLICATION

Following the optimisation of the gypsum production and the subsequent validating of the results in a pilot plant, the three gypsums produced were assessed for their suitability as feedstocks in the plasterboard and cement industries.

Samples of all three gypsums were sent to Lafarge Gypsum’s research laboratories in Avignon, France. Researchers at Lafarge analysed the quality of the gypsum produced and then produced samples of plasterboard using the Cleanlead gypsums alone and at blends of 70% natural gypsum and 30% Cleanlead gypsum.

The gypsums were shown to be high purity but there were some issues with regards to the crystal shapes and their consequential higher water demands (compared with natural gypsum). It was shown that the optimum use of Cleanlead gypsum would be as part of a blend with natural gypsum at a ratio of 30% Cleanlead gypsum to 70% natural.

The three Cleanlead gypsums were assessed for their suitability at Coventry University by intergrinding the gypsums with cement clinker supplied by Lafarge Cement. The cements prepared were assessed by recording their setting times and 28 day strengths according to European Standards [EN196-1:1995, EN196-3:1995]. The gypsums all performed acceptably when compared to reference cements and gypsums.

5 CONCLUSIONS

The Cleanlead project successfully generated gypsum that could be used within the plasterboard and cement industries.

Due to the comparatively low volumes of Cleanlead gypsum envisaged to be produced by a Cleanlead battery recycling factory, any gypsum supplied to the Plasterboard industry would have to be blended with other gypsum as otherwise there would not be enough to supply a factory annually.

The same would be true with a cement factory or though to a lesser extent as a cement factory uses less gypsum.

Therefore producing Cleanlead gypsum as a by-product of SLI battery recycling helps make the secondary lead industry more sustainable by turning a waste stream into a valuable product. Also Cleanlead gypsum can help make both the plasterboard and cement industries more sustainable by reducing their reliance on mined natural gypsum, a finite resource.
ACKNOWLEDGEMENTS

The author would like to say a special thank you to Dr. Heven Abdul-Jabbar at Coventry University. Also many thanks to the researchers at Lafarge Plasterboard UK, Tecnicas Reunidas Madrid (Spain), Instytut Metali Niezelaznych Gliwice (Poland) as well as all other Cleanlead project partners who contributed to the success of this project.

REFERENCES


NG, T.W., ONTARIO HYDRO TECHNOLOGIES, Canada. Modification of FGD Gypsum Crystal Habit by Metal Ions. Unknown conference, no date.


PARK, B., PARROW, S., MSE TECHNOLOGY APPLICATIONS INC, Butte, Montana, USA. Gypsum Production from Acidic Mine Water Treatment. Unknown conference, no date.

Use of waste gypsum as construction low strength materials

Seema Karami, Peter Claisse, Essie Ganjian, Homayoun Sadeghi Pouya
Department of Built Environment, Faculty of Engineering and Computing, Coventry University, UK

Rosemary Greaves
Department of Chemistry, Imperial College of London

ABSTRACT: Disposing of waste is an environmental issue. Using waste will help reduce land filling. The successful research and development of a new building material, or component using waste as raw material, is a very complex and multidisciplinary task having technical, environmental, financial, marketing, legal and social aspects.

In this paper it has been tried to find a low compressive strength mix with using only waste material include: Red Gypsum, Plaster board Gypsum, Basic Oxygen Slag, and Cement By Pass Dust. The amount of compressive strength and the flow of mixes had measured. The effect of water amount on compressive strength of some mixes has also measured.

1 INTRODUCTION

“There is a general movement of rural populations to the cities with the rapid industrialization in developing countries. The infrastructure to support these cities, such as buildings for housing and industry, mass transit for moving people and goods, and facilities for handling water and sewage will require large amounts of construction materials. Enhanced construction activities, shortage of conventional building materials and abundantly available industrial wastes have promoted the development of new building materials” [Kumar, 2002].

This paper is about using waste materials in construction. The aims of this paper are:

1. To develop uses for contaminated gypsum.
2. To carry out trial mixes incorporating red gypsum and/or waste plasterboard to see if the Compressive and Tensile Strength properties can be met.
3. To find the optimal percentage of each material in mixes for making bricks and other construction materials.

The following materials have been considered in the research in this paper:

1. Red Gypsum (RG)
2. Plasterboard Gypsum (PG)
3. Basic Oxygen Slag Dust (BOS)
4. Cement By Pass Dust (BPD)

Firstly, the chemical and some of physical characteristics of waste materials were examined.

Secondly, the materials were mixed in binary and ternary combinations and the compressive strength was tested and optimised.

2 LITERATURE REVIEW

2.1 Source of gypsum

There are different sources of by-product gypsum:

“Titanium Oxide Pigment Production yields 250 Kt of ‘red’ and 84 Kt of clean ‘white’ gypsum per year in the UK. Worldwide production of red gypsum is 1.25 Mt from one producer alone” [Claisse P. et al., 2004]. “This material contains approximately 40% moisture, 16% iron oxides, 0.5% of both MnO and SiO₂, 0.25 of Al and TiO₂ and many other elements. These render it unacceptable to the plaster and cement industries, principally due to its iron content which may cause staining in plaster products and adversely affects cement clinker chemistry.” [Claisse P. et al., 2004]

Waste gypsum also arises from plasterboard offcuts from construction sites and spent casting cores from foundries and very many areas of chemical manufacturing produce secondary gypsum from acid neutralisation. In order to meet demand, substantial amounts of quarried gypsum are also used in this country and the UK is a net importer of gypsum.
Flue gas desulphurisation at PowerStation is another source of by-product gypsum which is the single largest source of secondary gypsum with 600 Kt produced in the UK and 16 Mt in the EC in 2000. After a temporary rise to 1.5 Mt in 2005, UK arisings are expected to fall substantially as domestic coal supplies are replaced with low-sulphur imported coal by 2015.

Waste gypsum also arises from plasterboard offcuts from construction sites and spent casting cores from foundries and very many areas of chemical manufacturing produce secondary gypsum from acid neutralisation.

2.2 Source of slag

“Steel Slag is a by-product produced from either the conversion of iron to steel in a basic oxygen furnace, or melting of scrap to make steel in electric arc furnace.” [Caijun Shi, 2004]

“The use of iron blast furnace slag as a cementitious material has been practiced in Europe since the late 1800s [Tifefegi, 1997]. Today SSDs are well characterised and long-term experienced materials mainly used as aggregates for road construction.” [Mozt et al., 2001]

“In the United Kingdom about 1 million tonnes of Basic Oxygen Steel (BOS) slag is produced annually and about 10 million tonnes of BOS slag is held in stockpiles undergoing weathering to allow for hydration of free lime.” [Gurmel S Ghataora et al., 2004]

2.3 Source of BPD or CKD

During the manufacture of Portland cement, a large amount of dust is collected from kiln exhausted gases. While some of this cement kiln dust (CKD) is recycled, a large amount is disposed in landfills. The CKD has cementitious properties that make it an effective material for concrete.

“A typical Portland cement is manufactured by feeding materials containing appropriate proportions of lime, silica, alumina and iron into the upper end of kiln. The mix passes through the kiln at rate controlled by the slope of the kiln and speed at which the kiln rotates. Burning fuel is forced into the lower end of the kiln where it produces temperatures of 1400–1650°C, changing the raw mix to a cement clinker. During this operation a small percentage of the material in the form of dust (CKD) is collected. The physical and chemical properties of CKD can vary from plant- to- plant, depending on the raw materials used and type of collection process in the plant. However, the dust collected from the same kiln and producing the same cement type will typical have a relatively consistent composition. It is a good practice to frequently test the material to evaluate its characteristics and quality.” [Miller et al., 2000].

2.4 Use of waste materials as cement replacement

Grinding is one of the parameters that can affect material characteristics. The grinding process reduced both the particle size and the crystalline phase of the materials, thus improving binder reactivity. Vibratory grinding for 4 hours provided the most success for mechano-chemical activation among all grinding methods used. [Shah, Wang, 2001].

Ground granulated blast furnace slag has successfully been used with Portland cement to produce high performance cement blends that are more economical and environmentally friendly. The use and effectiveness of CKD as an activator for slag depends upon its physical and chemical characteristics, most importantly, the alkali and free lime content, and the amount of carbonates and sulfates. The effectiveness of the alkali activation of slag will depend on the alkalinity provided by the CKD. It is expected that the high free lime content of the CKD will improve the hydration process by accelerating hydration and forming more crystalline products of hydration. Sulfate ions provided either by alkali salts or anhydrite will expedite the hydration process and accelerate the pozzolanic reaction through the formation of ettringite. [Konsta et al., 2003]

Compressive strength of different kinds of CKD and Slag blends increased with curing time, indicating slag activation by the CKD and the formation, precipitation, and accumulation of products of hydration. [Konsta et al., 2003]

Composite cement pastes of fluorgypsum, blast-furnace slag and metakaolin developed and maintained strength even under water, showing improved properties over commercial gypsum. (Fraire-Luna, et al., 2006)

Adding gypsum to slag reduced the setting time of alkali-activated slag paste (AASP), increased the compressive strength and decreased the drying shrinkage. [Chang et al., 2005]

Investigated by Mum et al in 2006 shows by promoting hydration of GBFS by adding activator, a small quantity of commercial slacked lime, anhydrous gypsum, and limestone powder were applied as activators and filler, sharply increases early compressive strength than that without activator added. Also, it is confirmed that blast furnace slag cement of approximately the same early strength with Ordinary Portland Cement (OPC) is feasible to be manufactured.

Compressive strength increased with curing time according to Figure 1.

3 MATERIALS USED IN THIS PROJECT

3.1 Source of by product gypsum

Red Gypsum and Plaster board Gypsum were used in this research. The source of Red Gypsum was from
Titanium Dioxide manufacture at Tioxide Europe PLC and Plaster Board was from Lafarge sites.

The plasterboard gypsum used was obtained from the Lafarge plasterboard recycling plant located in Bristol. Waste plasterboard gypsum is collected from demolition sites.

3.2 Source of BPD

Typical analytical chemical composition of two different CKD samples from Rugby Cement and Castle Cement Barrington are given in Table 1. The BPD source was the same in step 1 and 2 reported here but the materials were taken at different times so they may have slightly different chemical and physical characteristics.

3.3 Source of BOS

The Basic Oxygen Slag that has been used in this project is from Tarmac UK (from Corus Scunthorpe plant).

4 CHEMICAL ANALYSIS

The chemical analysis of raw materials is shown in Table 1.

5 SAMPLE PREPARATION

This research was conducted by making samples in the lab with a low shear mixer. All samples were left to cure for 3, 7, and 28 days prior to compressive testing. Specimens were cast in 50mm cube moulds. The fresh samples were covered with plastic sheets to prevent evaporation. After 24hr or 48hr, the samples were removed from the moulds and cured in water.

The compressive strength were tested at 3, 28, and 28 days accordance with standard. Each compressive strength value of each mix was the average of two samples.

Flow of mixes was measured by flow table.

5.1 Optimising ternary mixes

BOS, BPD, RG, and PG were mixed. Each mix is a combination of three different materials. Here, raw materials have been mixed with different percentages. The groups’ material has been shown in Figure 2 and the proportion of each group are shown in Tables 2 and 3.
Table 2. Characterisation of BOS, BPD, and RG mixes.

<table>
<thead>
<tr>
<th>Materials</th>
<th>Compressive strength MPa.</th>
<th>Flow mm</th>
<th>Mixing time min.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3 days</td>
<td>7 days</td>
<td>28 days</td>
</tr>
<tr>
<td>BOS%</td>
<td>BPD%</td>
<td>RG%</td>
<td></td>
</tr>
<tr>
<td>85</td>
<td>10</td>
<td>5</td>
<td>0.35</td>
</tr>
<tr>
<td>85</td>
<td>5</td>
<td>10</td>
<td>0.28</td>
</tr>
<tr>
<td>80</td>
<td>10</td>
<td>10</td>
<td>0.45</td>
</tr>
<tr>
<td>76</td>
<td>5</td>
<td>19</td>
<td>0.41</td>
</tr>
<tr>
<td>76</td>
<td>19</td>
<td>5</td>
<td>0.20</td>
</tr>
<tr>
<td>72</td>
<td>10</td>
<td>18</td>
<td>0.45</td>
</tr>
<tr>
<td>72</td>
<td>18</td>
<td>10</td>
<td>0.48</td>
</tr>
<tr>
<td>68</td>
<td>15</td>
<td>17</td>
<td>0.20</td>
</tr>
<tr>
<td>68</td>
<td>17</td>
<td>15</td>
<td>0.32</td>
</tr>
<tr>
<td>65</td>
<td>10</td>
<td>25</td>
<td>0.40</td>
</tr>
<tr>
<td>64</td>
<td>20</td>
<td>16</td>
<td>0.26</td>
</tr>
<tr>
<td>60</td>
<td>20</td>
<td>20</td>
<td>0.40</td>
</tr>
</tbody>
</table>

Table 3. Characterisation of BOS, BPD, and PG mixes.

<table>
<thead>
<tr>
<th>Materials</th>
<th>Compressive strength MPa.</th>
<th>Flow mm</th>
<th>Mixing time min.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3 days</td>
<td>7 days</td>
<td>28 days</td>
</tr>
<tr>
<td>BOS%</td>
<td>BPD%</td>
<td>PG%</td>
<td></td>
</tr>
<tr>
<td>85</td>
<td>10</td>
<td>5</td>
<td>0.25</td>
</tr>
<tr>
<td>85</td>
<td>5</td>
<td>10</td>
<td>0.45</td>
</tr>
<tr>
<td>80</td>
<td>10</td>
<td>10</td>
<td>0.48</td>
</tr>
<tr>
<td>76</td>
<td>19</td>
<td>5</td>
<td>0.31</td>
</tr>
<tr>
<td>76</td>
<td>5</td>
<td>19</td>
<td>0.45</td>
</tr>
<tr>
<td>72</td>
<td>18</td>
<td>10</td>
<td>0.45</td>
</tr>
<tr>
<td>72</td>
<td>10</td>
<td>18</td>
<td>0.20</td>
</tr>
<tr>
<td>68</td>
<td>17</td>
<td>15</td>
<td>0.30</td>
</tr>
<tr>
<td>68</td>
<td>15</td>
<td>17</td>
<td>0.20</td>
</tr>
<tr>
<td>65</td>
<td>10</td>
<td>25</td>
<td>0.34</td>
</tr>
<tr>
<td>64</td>
<td>20</td>
<td>16</td>
<td>0.24</td>
</tr>
<tr>
<td>60</td>
<td>20</td>
<td>20</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Figure 3 (A and B). BOS+BPD+PG strength (MPa) after 7 and 28 days.
6 RESULTS AND DISCUSSION

Materials in this step still were BOS, BPD, PG, and RG. Different combinations have been used for mixes and the result of compressive strength after 3, 7, and 28 days are shown in Tables 2 and 3.

The results are used to draw graph as contours. The contours in Figures 3 to 4 are in fact models to predict the approximate compressive strength of mixes have not been made.

Tables 2 and 3 demonstrate the compressive strength and flow of different mixes with BOS, BPD, RG and PG. In every mixes the compressive strength has been increased by curing time. PG mixes have got more compressive strength compare to RG in mixes with BOS and BPD with the same percentage. There was no significant increase in compressive strength by increasing of BOS.

7 CONCLUSIONS

- Flow decreases by increasing the amount of RG or PG.
- The compressive strength increases by time. It means the 28 days old samples have higher compressive strength than 7 and 3 days old samples.
- Since the chemical and physical characteristics of BPD is different in any batches it is recommended to uses BPD in mixes as less as possible to have better conclusion on mixes.
- BPD can do as a good activator and help mixes to get more compressive strength in 3, 7, and 28 days.
- In both mixes graphs show, the mixes could get higher compressive strength after 7 days could get higher compressive strength after 28 days.
- It would be possible to use wastes for low construction products.
- Mixes contained PG can get higher compressive strength after 28 days compare the same mixes which have RG.

REFERENCES

Chang Jiang Jhy, Weichung Yeih, Chi Che Hung, Effects of gypsum and phosphoric acid on the properties of sodium silicate-based alkali-activated slag pastes, Cement & Concrete Composites, 2005, 27 85–91

Development of novel cementitious binders using plasterboard waste and pozzolanic materials for road bases

H. Sadeghi Pouya, E. Ganjian, P.A. Claisse & S. Karami
Department of the Built Environment, Faculty of Engineering & Computing, Sir John Laing Building, Coventry University, Coventry, UK

ABSTRACT: This paper explores the potential benefits of utilising combination blends of plasterboard waste gypsum and pozzolanic materials in developing novel cementitious pastes with lower environmental impact in comparison with conventional Portland cement. Ternary blend systems based on waste plasterboard from construction demolishing sites (PG), basic oxygen slag from steel factories (BOS) and cement kiln dust (CKD)/by past dust (BPD) from cement industries were investigated. PG, up to 30%, BOS and CKD, up to 90% were incorporated as total binder replacements for preparation of various combinations of binary and ternary blended systems to achieve the highest compressive strength. A water-binder ratio of 0.3 was used for the main group of paste mixes. Cube compressive strength was determined at the age of 3, 7 and 28 days. Based on the obtained results, the effect of PG and CKD on activation of slag with respect to compressive strength was investigated. It was found that the relation between slag content and compressive strength is sensitive to the amount and chemical composition of PG and CKD/BPD used in ternary system. In ternary system of PG-BPD-BOS, incorporation of 10% PG together with 36% BPD resulted in the highest compressive strength while in CKD-PG-BOS system; the highest compressive strength is achieved using 20% CKD and 16% PG. A road foundation site trial was successfully conducted in summer 2006 using this developed binder for road base in a car park area at Lowdham Grange prison, Mansfield construction site.

1 INTRODUCTION

The environmental challenge in terms of reducing global carbon dioxide emissions, conservation of energy, reducing reliance on prime resources and reuse of by-product and waste materials are driving the search for new cementitious materials and the development of more environmentally friendly binders to be used in construction works. The reuse of construction and demolition wastes together with various sources of pozzolanic waste materials is an attractive alternative to produce cementitious binders with a lower strength than ordinary Portland cement. These low strength binders can be used in many applications such as structural fills, insulation and isolation fills, pavement bases and sub-bases, conduit bedding, erosion control, void filling, nuclear facilities and bridge reclamation [ACI 1999].

Plasterboard gypsum (PG) offcut waste arising from construction and demolition sites is one of the challenging waste materials produced in the UK after it was reclassified by EU Landfill Directive as hazardous non-inert wastes in July 2005. The majority of plasterboard gypsum waste used to be traditionally landfilled and co-disposed of with other wastes. [WRAP 2005]. Detailed statistics on waste plasterboard arisings are currently scarce, but it is estimated that some 300,000 tonnes of waste plasterboard are generated each year from new construction activity (largely as offcuts). The amount of plasterboard waste arising from demolition projects is more difficult to quantify, but maybe in the range 500,000 tonnes to more than 1 million tonne per annual in the UK [WRAP 2006].

This investigation is aimed at developing novel-low environmental impact cementitious binder utilising blends of plasterboard gypsum waste and a range of pozzolanic industrial waste materials such as basic oxygen slag, cement kiln dust and cement by pass dust. The above materials were used in binary and ternary systems with the aim of exploiting the potential synergy between these materials to form a cementitious matrix. A site trial was also conducted using selected proportions of blended materials to make a roller compacted concrete as a sub-base for a car park.
2 EXPERIMENTAL PROGRAMME

2.1 Materials

Crushed plasterboard gypsum (PG) used throughout the investigations was obtained from the Lafarge plasterboard recycling plant located in Bristol. The plasterboard was dried, ground and passed through 600 µm sieve in order to increase the reactivity of the particles and also to remove paper pieces mixed with gypsum. The basic oxygen slag (BOS) was obtained from Tarmac UK (Corus Scunthorpe plant). The slag has been ground using laboratory ball mill and sieved through a 600 micron sieve. The cement kiln dust CKD and cement by pass dust (BPD) were supplied by Castle Cement (Heidelberg cement group) both in the form of powder. The main difference between CKD and BPD is related to the temperature at which these materials are produced. CKD is taken out of the kiln during where the temperature is about 300°C; however, BPD is from the kiln where the temperature is about 1000°C. The chemical compositions of raw materials used in this investigation are presented in Table 1.

2.2 Mix proportions

The proportioning of pastes used in this investigation is designed in order to achieve the highest compressive strength of binary and ternary mixtures. For ternary system of PG, BPD and BOS in the first stage the optimum proportion of BPD and BOS was determined by measuring the compressive strength of various binary mixtures of BPD and BOS. In the second stage, PG was added to the optimum mixture of BPD-BOS corresponding to the highest compressive strength obtained in the first stage. Table 2 shows the mix proportion of paste mixes used. The similar experimental plan was adopted for the ternary mixture of CKD, PG and BOS.

| Table 1. Chemical compositions of starting materials. |
|---------------------------------|-------|-------|-------|-------|
| Oxides | PG (%) | BOS (%) | BPD (%) | CKD (%) |
| SiO₂ | 2.43 | 11.43 | 12.86 | 9.89 |
| TiO₂ | 0.03 | 0.39 | 0.12 | 0.14 |
| Al₂O₃ | 0.81 | 1.60 | 3.50 | 3.72 |
| Fe₂O₃ | 0.36 | 28.24 | 2.12 | 1.24 |
| MnO | 0.00 | 4.35 | 0.02 | 0.02 |
| MgO | 0.40 | 8.27 | 2.46 | 0.94 |
| CaO | 37.30 | 41.29 | 58.28 | 40.42 |
| Na₂O | 0.03 | 0.02 | 1.71 | 6.36 |
| K₂O | 0.24 | 0.02 | 0.06 | 0.08 |
| P₂O₅ | 0.02 | 1.48 | 6.75 | 5.59 |
| SO₃ | 53.07 | 0.44 | 10.23 | 30.99 |
| LOI | 4.09 | 1.12 | |

| Table 2. Mix proportions for PG-BPD-BOS paste mixture. |
|---------------------------------|-------|-------|-------|-------|
| Mix code | PG (%) | BPD (%) | BOS (%) | Flow |
| BPD10/BOS90 | – | 10 | 90 | 0.3 | 87 |
| BPD20/BOS80 | – | 20 | 80 | 0.3 | 33.5 |
| BPD40/BOS60 | – | 40 | 60 | 0.3 | 38.5 |
| BPD60/BOS40 | – | 60 | 40 | 0.3 | 16.8 |
| BPD90/BOS10 | – | 90 | 10 | 0.3 | 12.8 |
| PG5/BPD38/BOS57 | 5 | 38 | 57 | 0.3 | 128.8 |
| PG10/BPD36/BOS54 | 10 | 36 | 54 | 0.3 | 125.6 |
| PG15/BPD34/BOS51 | 15 | 34 | 51 | 0.3 | 103 |
| PG20/BPD32/BOS48 | 20 | 32 | 48 | 0.3 | 101.7 |
| PG30/BPD28/BOS42 | 30 | 28 | 42 | 0.3 | 47.6 |

| Table 3. Mix proportions for CKD-PG-BOS paste mixture. |
|---------------------------------|-------|-------|-------|-------|
| Mix code | PG (%) | CKD (%) | BOS (%) | Flow |
| PG10/BOS90 | 10 | – | 90 | 0.3 | 172 |
| PG20/BOS80 | 20 | – | 80 | 3.0 | 150 |
| PG40/BOS60 | 40 | – | 60 | 0.3 | 136 |
| PG60/BOS40 | 60 | – | 40 | 0.3 | 110 |
| CKD60/PG 8/BOS32 | 8 | 60 | 32 | 0.3 | 161 |
| CKD40/PG12/BOS48 | 12 | 40 | 48 | 0.3 | 120 |
| CKD20/PG16/BOS64 | 16 | 20 | 64 | 0.3 | 92 |
| CKD10/PG18/BOS72 | 18 | 10 | 72 | 0.3 | 76 |

The mix proportions of paste incorporating CKD, PG and BOS is shown Table 3. The constant liquid to solid ratio of 0.3 was use in all mixes. The flow of all the mixes investigated were measured using the flow table according to ASTM C109.

2.3 Casting and curing

Paste specimens 50 mm cubes were cast for the determination of compressive strength. All the specimens were cast and fully compacted in three layers using a vibrating table. After casting, the samples were covered by polyethylene sheets for 24 hour. The samples were demolded the following day and then kept at 20 °C and 98% RH prior to testing at 3, 7 and 28 days.

3 RESULTS AND DISCUSSION

Compressive strength development of binary and ternary mixtures containing PG, BPD and BOS are shown in Figures 1 and 2. The strength development of paste mixes using a range of BPD and BOS showed that in the binary system the mix containing 40% of by pass dust and 60% of slag has the highest strength at 7 days, although it can be seen in Figure 1 that the
optimum amount of BPD for 3 days strength is equal to 60%, which could be because of rapid reacting components of bypass dust used. At 28 days the highest compressive strength was still achieved by the mix of BPD40/BOS60.

In the ternary system, Figures 2 shows that the highest strength at 3, 7 and 28 days corresponds to the mix containing 10% of plasterboard gypsum. It can be seen that increasing the amount of PG above 10% resulted in decrease in compressive strength. Therefore, it can be concluded that the mixture PG10/BPD36/BOS54 is the optimum mixture in this optimization order.

Figures 3 and 4 show the compressive strength development of binary and ternary mixtures of PG-BOS and PG-CKD-BOS respectively. It can be seen that in the binary system mix incorporating 20% PG and 80% BOS achieved the highest compressive strength compared to mixes with higher content of slag.

In the ternary system, the highest compressive strength was obtained from the mix with 20% CKD, 16% PG and 64% BOS. It can also be observed that as a result of increasing the amount of CKD from 40 to 60% compressive strength has been reduced by about 50% at 28 days.

In general slag is typically hydrated after mixing with Portland cement or other alkali materials such as CKD and BPD providing a source of alkalinity with which the slag reacts to form cement hydration products [Neville 2000]. The excessive amount of alkali in the system has a detrimental effect on hydration of alkali activated slag causing a delay of setting and low strength. The precise causes of this behaviour have not been clarified. The formation of monosulphate due to instability of ettringite at high pH has been suggested to be the main reason [Ottemann et al. 1951].

Comparison of mixtures containing CKD, PG and BOS revealed that the highest strength achieved from optimization experiments was related to chemical composition of alkali source and alkali content in the mixture. It was observed that two sources of alkali in this investigation resulted in different ternary proportions corresponding to the highest compressive strength. The ternary mix PG16/CKD20/BOS64 showed higher strength than mix PG10/BPD36/BOS54. It implies that activating effect of CKD on BOS was more remarkable.
than BPD in the ternary systems. This might be attributed to amount of alkalis released from these two sources of waste alkali materials.

With respect to the effect of gypsum on compressive strength, as stated by Matschei et al. [2005] the early compressive strength of supersulphated cements was enhanced by sulphate activation of the slag in comparison with slag mixes without any addition of calcium sulphate. This phenomenon can be observed in results obtained from PG, BPD and BOS mixtures shown in Figures 1 and 2.

4 SITE TRIAL

As part of the site trials conducted in this project, a 6 by 17 m car park area at Lowdham Granage prison construction site at Nottingham was constructed with Roller Compacted Concrete (RCC) as a sub-base layer using the novel developed binder. Based on numerous laboratory mixes, the final optimised proportions of the ternary mixture of PG, BPD and BOS obtained in this project were chosen for the site trials. Considering the volume of materials need for site trials, over 100 T of blended powder was prepared. The novel binder consisted of BOS, ground PG and supplied as blended powder. Figure 5 shows the bagged and shrink wrapped blended powder delivered to the ready mix plant for roller compacted concrete mix.

The roller compacted concrete designed through laboratory experiments was deemed appoint for this. The concrete mix design used is presented in Table 4. The area was stripped of the existing hardcore to expose the sub grade, which was hard clay. The roller compacted concrete mix was prepared at a Lafarge ready mix concrete plant at Lockington, Leicestershire.

A layer of 160 mm of concrete was placed over the sub-grade layer using the truck mixer chute and spread and levelled manually. As the concrete was delivered in three truck loads; placing and compaction of RCC layer was carried out in three segments of the allocated area. Figures 6 and 7 show the placing and levelling of the roller compacted concrete.

The placed concrete layer was then compacted using a 3-Tonne vibrating roller in accordance with the compaction requirements of the Specification for Highways Works to form a 100mm roller compacted concrete.

Table 4. Mix proportions of concrete mix used in Lowdham Grange site trial.

<table>
<thead>
<tr>
<th>Mix code</th>
<th>Blended novel binder</th>
<th>Water</th>
<th>Recycled Agg.</th>
<th>W/B</th>
<th>Slump mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>RA-RCC</td>
<td>400</td>
<td>100</td>
<td>1900</td>
<td>0.3</td>
<td>0</td>
</tr>
</tbody>
</table>

Figure 5. Blended powder delivered to ready mix plant in 20-kg bags.

Figure 6. Placing concrete using truck mixer chute.

Figure 7. Spreading the concrete using a mechanical excavator.
5 CONCLUSIONS

Crushed plasterboard gypsum can be used as a source of sulphate together with basic oxygen slag and cement kiln dust/by pass dust to form a sulphate activated pozzolan.

The optimum usage amount of plasterboard waste gypsum depends on the chemical properties of basic oxygen slag and cement kiln dust/by pass dust. Laboratory experiments showed that in ternary combination of PG-CKD-BOS using 16% crushed plasterboard gypsum resulted in the highest compressive strength.

Increasing by pass dust content in ternary combinations of PG, BOS and CKD/BPD resulted in less compressive strength. This can be due the location and orientation of ettringite formed around slag particles.

Roller compacted concrete (RCC) made with optimum proportions of BOS, PG and BPD with W/B ratio of 0.25 was successfully used as the sub-base layer in the car park site trail.

ACKNOWLEDGEMENTS

This work was supported by The Waste and Resources Action Programme (WRAP) under grant No. PBDS-002. The authors are especially grateful to Skanska UK for their great assistance in providing ground and equipments for site trails. We would like to thank Lafarge, Plasterboard for the generous supply of waste plasterboard gypsum.

REFERENCES


Investigation into the use of cement stabilised gypsum waste as a backfill material

Waliur Rahman, Gurmel Ghataora & D.N. Chapman
Department of Civil Engineering, The University of Birmingham.

ABSTRACT: Conventional backfill for utility trenches comprises of granular crashed rock (type I subbase), which is made of high quality materials. The increasing cut of this material and need to reduce gypsum waste led to the investigation which aimed at principally exploring the use of gypsum (both red gypsum and waste plaster-board for utility trench backfill. Backfills developed were to be cementitious and self levelling. Unconfined compressive strength (UCS), indirect tensile strength (ITS) and flowability test of different materials mixed with gypsum were conducted in the laboratory. UCS of ordinary Portland cement and red gypsum at a ratio of 1 and 5 was more than 2 MPa (28-day), which is the specified strength for cementitious backfill materials. To increase the flowability of the mix, a strength accelerator sodium Meta silicate (Na$_2$SiO$_3$, 5H$_2$O) was added with more water in the mix, 1.5% Na$_2$SiO$_3$, 5H$_2$O proved to be adequate for obtaining an acceptable flowable backfill. Quarry fines in cement-gypsum mix also showed encouraging results. Cement, red gypsum and quarry fines mixed at a ratio of 1, 4 and 15 also complied with the Specifications. The same mix with plaster-board gypsum in place of red gypsum and with slightly higher cement (1.3:4:15) also complied with the Specifications. The above materials could be ideal replacements of granular backfill materials.

1 INTRODUCTION
The New Roads and Street Works Act [1991] in England and Wales place the duty of all aspects of reinstatements, including pavement resurfacing with the Utilities. The Act also encourages reinstating the openings in roads in accordance with the Specifications for Openings in Highways developed by the Highways and Utility Committee. Therefore, any backfill technique has to comply fully with the Specifications. Detailed specifications are thoroughly described in HAUC proceedings.

In general Type I granular subbase is used for backfilling utility trenches. This subbase material comprises of high quality crushed rock and it requires compaction when placed. However, it is not always possible to maintain proper moisture content and apply adequate compaction in the confines of a trench and consequently material may lack adequate compaction loosely compacted; therefore, settlement of backfill may occur.

According to Department of Transport [1991], cemented material can be used as a backfill material. The material may be produced by a concrete batching plant or mixed on site. The mix of the material is restricted only by the need to comply with the 90 day compressive strengths from Appendix 9. That is minimum 4 MPa for Types 1 and 2 roads and minimum 2 MPa for Types 3 and 4 roads. The maximum permitted strength for all mixes is 10 MPa. Care should be taken not to exceed the maximum values for two reasons, firstly above 10 MPa it starts to behave more like structural concrete and may crack and produce reflective cracking on the road surface, secondly re-excavation would be difficult [Department of Transport, 1991]. The roads are classified according to their traffic volume. Type 1 roads carry 10 to 30 million standard axles (msa), Type 2 roads carry 2.5 to 10 msa, Type 3 roads carry 0.5 to 2.5 msa and Type 4 roads carry up to 0.5 msa [Department of Transport 1991]. According to British Cement Association [1991], the minimum specified strength is 2 MPa at 28 days.

Appendix 9 of HAUC Specifications [Department of Transport, 1991] permits the use of alternative reinstatement materials (ARMs). It states that ARMs may offer significant environmental or practical advantages and/or cost benefits, compared to conventional materials which may include reduced usage of virgin materials by including a range of factors such as recycled and secondary materials, self levelling, to avoid or reduce the need for compaction and may be placed in fewer lifts. Source of the research on the use of secondary materials that require no compaction has been reported by Ghataora and Alobaidi [2000], Ghataora et al. [2006a] and Alobaidi and Ghataora [2006b].
The materials considered by the above authors were sourced from utility excavations. However, it may also be possible to use other source of materials for backfill. This paper presents some of the findings of the research on the potential use of gypsum for trench backfill.

Gypsum has been used in construction purposes as an additive or a filler material for about five decades in different parts of the world. Gypsum does not have good bonding properties itself; therefore it has been used as a secondary cementing material in the mixes with cement or lime. In this study waste gypsum along with other additives and quarry fines materials were investigated for use as flowable fill for backfilling utility trenches. For this purpose, a number of compositions of materials were tested for unconfined compressive strength, durability, tensile strength and flowability. The results of the tests were compared with existing specifications. As flowable fills are similar to stabilised soils in respect to their use in the construction purposes the standard laboratory tests normally adopted for stabilised soils have been considered here. These tests have been used with consistent success for a wide range of materials, climate and subgrade conditions [Norling, 1963].

2 MATERIALS AND METHODOLOGY

Materials investigated during this study include waste gypsum, Portland cement, basic oxygen steel slag and limestone quarry fines. These materials are discussed briefly below.

2.1 Gypsum wastes

Titanium oxide pigment production yields 250 Kt of ‘Red’ and 84 Kt of clean ‘White’ gypsum per year in the UK. World wide production of ‘Red gypsum’ is around 1.25 Mt from Huntsman Tioxide alone. The Huntsman Tioxide plant at Grimsby has reduced the amount of Gypsum sent to land-fill by nearly 60% between 1997 and 2001 [Mini Waste Faraday Research Proposal, 2004]. Re-using this co-product not only saves waste but also helps to reduce the depletion of natural gypsum through mining.

Waste gypsum also arises from plasterboard off-cuts from construction sites and spent casting cores from foundries and very many areas of chemical manufacturing produce secondary gypsum from acid neutralization. In order to meet the demand, substantial amounts of quarried gypsum are also used in this country. The UK is a net importer of gypsum. Clean gypsum such as flue gas gypsum and white by-product titano-gypsum has a ready market in cement and plasterboard manufacture. Changes in building regulations requiring thicker plasterboard are creating increased demand. Limited amount of plaster off-cuts can be used in new board manufacture but the paper content restricts the proportions and transport cost can be prohibitive. Lafarge’s estimated quantity of surplus waste board is 30 Kt per year.

The amount of waste gypsum produced is a growing concern. This was one of the drivers for the research undertaken for this investigation. The aim of this part of the research is to explore the possibility of using gypsum in trench backfill. Both red gypsum from the Huntsman Tioxide plant and plaster-board waste supplied by Lafarge were used in this study.

2.2 Cement

The process of manufacturing of cement consist essentially of grinding the raw materials, (such as calcareous materials like, limestone or chalk, alumina and silica found as clay or shale, Marl, a mixture of calcareous and argillaceous materials) mixing them intimately in certain proportions and burning in a large rotary kiln at a temperature of about 1550°C The clinker formed is cooled and ground to a fine powder, with some gypsum added (~2–3% wt.) and the resulting product is called ordinary Portland cement [Neville 1995]. Portland cement used in this study complied with specification and was supplied by Lafarge.

2.3 Basic oxygen steel (BOS) slag

Steel slag is a byproduct produced during the conversion of iron ore or scrap iron to steel. The mineralogical composition of steel slag changes with its chemical composition. Olivine, merwinite, 3CaO–SiO₂ (C₃S), 2CaO–SiO₂ (C₂S), 4CaO–Al₂O₃–Fe₂O₃–SiO₂ (C₄AF), 2CaO–Fe₂O₃–SiO₂ (C₂F), CaO–FeO–MnO–MgO in solid solution (RO phase) and free CaO are common mineral in steel slag [Shih, Wu and Chiang, 2004]. Particle size distribution of the fines is shown in Figure 1.

2.4 Quarry fines

Limestone fines composed of 5 mm to silt sized fraction resulting form excavation of limestone [Quarry Product Association 2003]. It was supplied by Tarmac. Particle size distribution of the fines is shown in Figure 1.

3 MIX DESIGN

The following mixes were investigated:

- Red gypsum and steel slag mixes in different proportion.
- Red gypsum, with or without steel slag and hydrated or quick lime mixes in a range of ratios.
Red Gypsum and cement in different ratios and water content with and without sodium meta silicate (Na$_2$SiO$_3$, 5H$_2$O).

- Red gypsum mixed with cement and quarry fines.

4 LABORATORY RESULTS AND DISCUSSION

4.1 Compressive strength and specifications

The unconfined compressive strength (UCS) tests of various mixes consisting of red gypsum, steel slag, hydrated lime, quick lime and cement have been carried out in the laboratory. 64 mm diameter and 128 mm high cylindrical samples as well as 75 mm cube sample were tested to determine the UCS. The results shown in this study are based on the data collected from cylindrical sample; however, tests on cube specimens made from selected mixes showed that the UCS results of cube samples were about 1.25 times higher than the UCS values determined from cylindrical specimens. The HAUC Specifications require UCS on cube specimens. Cylindrical specimens were used for bulk of this study as it required lesser quantity of material and stiffness characteristics of the materials could be measured (reported elsewhere). A hand held food mixer was used to mix the materials. The strength of red gypsum (RG) and steel slag (SS) mix was (mix ratios, 1:1, 1:1.5 and 1:2) shown in Figure 2.

The strength gain of these mixes was very slow and the strengths were below the requirements of the HAUC Specifications. Therefore, hydrated lime (HL) was added to gypsum-slag mixes. With hydrated lime (HL), gypsum (RG) and steel slag (SS) mixes of

![Figure 1. Particle size distribution of BOS slag and quarry fines.](image1)

![Figure 2. UCS of RG-SS mixes.](image2)
1:8:12, 1:6:9 and 1:4:6 ratios gave 7-day strength of 0.18, 0.273 and 0.36 MPa respectively as shown in Figure 3. These strengths were only slightly higher than those gypsum-slag mixes. Therefore, quicklime (QL) and Portland Cement (PC) were added to the gypsum-slag mixes. The same ratios were used as the ratios used for HL-RG-SS mix. Seven-day strengths of 1:8:12; QL-RG-SS mix increased to 0.21 MPa and the 7-day strength of 1:5; QL:RG mix was only 0.11 MPa. None of the mixes investigated reached the required strength at 90 days [Department of Transport, 1991]. However, some of the mixes with lime had exceeded 1 MPa at 180 days (Figure 3). The water solid ratios of the above mixes were varied depending on the visual flowability of the mixes with an aim of keeping consistent flow.

The strengths of the above mixes did not comply with the standard Specification for backfill material. The targeted strength was 2 MPa, 28-days cured [British Cement Association, 1991]; however, according to HAUC the minimum requirement of unconfined compressive strength is 2 MPa, 90 days cured. High early strength of the materials is considered essential to allow for rapid permanent reinstatement. That means the material has to be strong enough to support the compaction of surfacing layer(s) of the road. As the mixes with lime did not attain the required strength Therefore, PC was used and the PC:RG mix at a ratio of 1:5 (with 0.3% water/solid ratio) generated strength of more than 3 MPa, 90 days cured, which complied with the HAUC Specification. However, early strength was still not considered to be high enough for permanent reinstatement to be carried out. Strength of 1 MPa was exceeded at 7 days. Therefore, alkaline strength accelerator sodium meta silicate (Na$_2$SiO$_3$, 5H$_2$O) was used in the OPC-RG mixes. 1.5, 3 and 4.5% Na$_2$SiO$_3$, 5H$_2$O was added to the OPC-RG, 1:5 mix. Interestingly the lesser the amount of Na$_2$SiO$_3$, 5H$_2$O in the mixes the higher was the UCS except for strength of specimens cured for 1-day. This might happen because of influence of alkali in the mixes. According to Neville [1995] increase in strength beyond the age of 28 days is strongly affected by the alkali content. The poor gain in strength between 3 to 28 days can be attributed more specifically to water soluble K$_2$O present in the cement but in the total absence of alkalis the early strength of cement paste can be abnormally low. Accelerated strength tests showed that, up to 0.4% of Na$_2$O strength increase with the alkali content. Strength of all of the mixes complied with the HAUC Specification; results of these tests are shown in Figure 4. Indirect tensile strength (ITS) test of the mixes OPC:RG = 1:5 and OPC:RG = 1:5, 1.5% Na$_2$SiO$_3$, 5H$_2$O were performed. The results show that the ITS in these cases was about one-tenth of the UCS, which complies with the specification [Technical Recommendation of Highways-13, 1986].

The best mixture was OPC:RG = 1:5, with 1.5% Na$_2$SiO$_3$, 5H$_2$O, which gave strengths of about 2 times the Specified values. This mix had low flowability, therefore more water was added to provide flow considered to be more suitable (flow table spread of 550 mm after 15 drops). This resulted drop of strength to 3.5 MPa (at 90 days). This mix still complied with the HAUC Specification. Cement used in the mix was quite high (16%), therefore, to minimise the amount of cement, a well-graded filler waste material ‘quarry fines’ was added to the cement-gypsum mixes.

### 4.1.1 Mixes with quarry fines

Mixes with quarry fines, red and plasterboard gypsum and Portland cement were investigated. Red gypsum is a sticky and lumpy material which proved difficult
to mix. However, ‘quarry fines’ is a granular material which when used with mixes containing gypsum improved mixability. Hobart bowl mixer was used to prepare the mixes. A range of mixes were tested for unconfined compressive strength and indirect tensile strength with a range of cement content (1.5 to 9.5%). Water-solid ratios of the mixes were 0.3 to 0.45. The compressive strength of all the mixes increased with the increase in cement content as expected. The mixes with quarry fines, red gypsum and cement with the ratio of 1:4:15 or with higher cement content complied with the HAUC Specification (see Figure 5).

The mixes were cured by double wrapping with cling film and stored at room temperature and humidity. Two mixes were cured at 90% humidity to observe the difference in compressive strength. The strength of the mixes cured at high humidity and at 20°C was only slightly lower than the mixes cured at room temperature and humidity (see Figure 6).

Plaster-board gypsum off-cuts were replaced by red gypsum in the mixes with cement and ‘quarry fines’. As plaster-board gypsum (PG) waste have stiffer lumps than red gypsum waste, mixing was difficult in its as delivered state, and therefore, grinding of plaster board gypsum waste was necessary to allow proper mixing of materials. Plaster-board gypsum waste was also available with and without paper. ‘Without paper’ performed better than ‘with paper’ one as was expected, because the binding of the materials was obstructed by the paper present in the mix (see Figure 7). The strength of the OPC:PG:QF = 1:4:15 mix was slightly below that required to comply with the Specification.
The strength in this case was lower than the mix with red gypsum probably due to the smaller particle size of plaster-board gypsum increases the cement demand in the mix. Slightly higher cement (OPC:PG:QF/H11005 1.3:4:15) in the mix made it acceptable as a backfill material (Figure 7). However, the acceptability of the mixes can not be ensured unless the products are durable and can sustain adverse weathering in the field condition.

4.1.2 Durability of the potential mixes
Two types of durability test were considered in this investigation: soaked UCS and freeze and thaw resistance. Soaked durability test was performed according to BS 1924 [British Standard, 2002] and as described in section 4 above. Four mixes were investigated. Gypsum-cement mixes with ‘quarry fines’ and Na$_2$SiO$_3$, 5H$_2$O showed less than 10% strength reduction after which was acceptable. Results are shown in Figure 8.

Freeze and thaw tests of the mixes with cement, red or plaster-board gypsum and ‘quarry fines’ at a ratio of 1.3:4:15 were performed. The mixes with cement, red gypsum and Na$_2$SiO$_3$, 5H$_2$O were not considered because the UCS values were much higher than the values of the mixes with quarry fines, therefore, they are expected to have good freeze and thaw resistance if the weaker mixes show adequate resistance. Results in Figure 9 show the expansion of standard 225 mm long prism for 50 cycles according to BS 5075 [British Standard, 1998]. Two specimens of the same mixes were investigated. The maximum expansion of
the specimens with red gypsum was higher (about 3 mm) than the maximum expansion with plaster-board gypsum (about 1.5 mm). This happened probably due to the grinding of the plaster-board gypsum, which, reduces the void ratio and thereby produces discrete air cavities in the material consequently reduces overall frost susceptibility.

During and after freeze and thaw cycles the expansion of both of the mixes was considerably to be low. According to Department of Transport [1991], frost susceptible material with a mean heave greater than 15 mm when tested in accordance with BS812: Part 124, however, this test is for compacted granular materials. In this study the freeze and thaw test undertaken (according to BS5075) was more suitable because this test is normally carried out for concrete. The tests showed maximum 3 mm expansion only and the two mixes examined did not disintegrate after 50 cycles. Therefore, both of the materials are expected to perform well under traffic as well as adverse weathering in field conditions.

6 CONCLUSION

The following conclusions are drawn from this study: OPC:RG = 1:5, 1.5% Na2SiO3, 5H2O and OPC: RG:QF = 1(or more):4:15 mix proved to be effective
in the respect to UCS, ITS, durability and flowability test results. However, the former mix includes high amount of cement (about 16%) than the latter mix (5% cement). The advantage of the former mix is that according to Department of Transport [1991], this material can be used in Type 1 or 2 roads. However, the latter mix can only be a backfill material of Type 3 or 4 roads.

Flowable backfill materials investigated in this study can be used in the roads. However, full scale field trials with the new materials should be undertaken and their performance should be measured for a long enough period before that.

REFERENCES


The importance of mixture proportioning in sustainable construction

N.G. Maldonado
National Technological University, Regional Mendoza College, CEREDETEC, Ciudad, Mendoza, Argentina

P. Helene
Escola Politécnica da Universidade de São Paulo, Brazil

ABSTRACT: The trend in concrete technology is to include admixtures and natural or artificial additions in the mixture proportions of concrete materials. These materials generate changes in the original base mixture and they have a strong influence on concrete. The challenge to concrete technologists is to select the best combination among different admixtures and additions which are being promoted to be used in concretes with special performance. A research program has been carried out to develop high performance concretes in Mendoza, Argentine, with local materials, as well as to obtain a better method of mixture proportions to attain strengths in compliance with national codes. The use of silica fume and a milled residue of a local industry in the mixture proportioning of concretes is evaluated. This paper emphasizes the need to evaluate industry residues by performance tests. The results show different performance in fresh state as well as in durability tests.

1 INTRODUCTION

1.1 Review of proportioning methods and codes

The methods of proportioning of structural concretes regained importance in the construction industry in the last decade due to the introduction of a third generation of additives such as superplasticizers, set-retarding or set-controlling admixtures which make possible the use of mineral additions (silica fume, fly ash, metakaolin, etc.).

Also, the concepts of durability, service life and sustainable environment have been introduced in a new generation of codes and they have had an impact in concrete technology. Simultaneously, attention is being paid to manufacturing industries residues.

The questions asked when such residues are incorporated into a material extensively used such a concrete are: can it be incorporate into plain concrete or in reinforced concrete, in what extent they have an impact in the environment due to their removal as solid residues, and how or how much they could be incorporated in the mixture proportions.

There are many proportioning methods like Bolomey, Fuller, Joisel, ACI, ICPA and others [Maldonado & Helene 2005] but the use of mixture proportions nomogram [Helene & Monteiro 1993] allows a correct and relevant comparison among the different concrete families, adopting the same ascending reference in the concrete compressive strength axis, instead of the usual poor comparison of individual results obtained with the other methods.

The fundamental contribution of the new national code is the incorporation of the environmental conditions as loads on the structure, that must be identified to establish the criteria for its protection, as well as the requirement of a concrete design with a service life of fifty years. The environmental exposure conditions are classified in general and/or specific conditions. General exposure conditions relate to degradation of the structure by corrosion processes and the specific exposure conditions relate to freezing and thawing, as well as effects of chemicals or salts contained in soils and water contacting the structure [CIRSOC 201 2002].

1.2 Research significance

In the Mendoza Regional Centre of Technological Developments in Building and Seismic Engineering (CEREDETEC), a research program has been carried out to develop high performance concretes. One of the main aims has been the study of local materials, as well as the selection of a better method for mixture proportions to obtain strength required by national codes [CIRSOC 201 2002].

The specified concrete strength in the region is under 30 MPa. During the first part of the research the
strength obtained was 60 MPa without additions [Maldonado & Helene 2002]. This study includes the use of two additions: silica fume and a milled mineral residue of a local industry.

2 METHOD OF MIXTURE PROPORTIONS OF CONCRETES

2.1 Fundamentals

The IBRACON mixture proportions method used is based on trial batches to obtain the best workability and rheological behaviour of fresh concrete, as the better way to select the best mixture proportions of materials [Levy & Helene 2004].

This method assumes the laws of Duff Abrams [1918], Inge Lyse [1932] and Molinari [Helene & Terzian 1993], which are a guideline for the interaction between water-cement ratio (W/C), strength (fc), dry aggregates/cementitious materials ratio (m) and content of cementitious materials (Cm).

The response of method procedures gives a family of concretes and it indicates how to obtain the mixture proportions of the nomogram which allows changes in mixture proportions easily without new laboratory tests.

2.2 Materials used

The portland cement used is a cement available in the region, portland-pozzolan cement CPP40 (ASTM type IP) and its properties comply with the Argentine Standard [IRAM 50001 2000].

The coarse aggregates used in this research program are extracted from alluvium bed of mountain rivers near the Andes Mountains in the West of Argentine, where there is predominant rounded gravel from basaltic and granite rock. In this case, crushed round gravel was used. The coarse aggregates have a maximum size of 19.5 mm and a fineness modulus of 6.16.

The fine aggregate used was a washed rounded river sand with a bulk density of 2.57 kg/m³, water absorption 0.81% and a fineness modulus of 3.1.

To achieve workable mixtures a superplasticizer, in the form of an aqueous solution of an acrylic polymer base with 32% solid content, was used. The compatibility between cement, additions, additives and admixtures was tested by the Kantro’s Modified Cone Method [Kantro 1980].

The milled local industry residue was generated during the production of calcium silicium by ignition of silica and calcium in an open kiln. The residues of production are kept in open piles without purpose of use. This situation generates questions about the compatibility with the environment and the probable use in concretes. The first steps were their use in high percentage (20% or more) with serious problems in fresh state and a poor performance on mechanical properties. Then, the owner of kiln requested a technical evaluation for use in concretes.

Table 1 shows the main characteristics of portland-pozzolan cement, silica fume and a finely milled local industry residue used in this work. The values of chemical composition do not allow to identify the material neither as granulate blast-furnace slag according to national standard [IRAM 1667 1990] nor as silica fume as stated by international standard [ACI 234R 2000].

The best percentage of additions was established by the test of compatibility of additions, additives

<table>
<thead>
<tr>
<th>Item</th>
<th>Portland-pozzolan cement</th>
<th>Silica fume</th>
<th>Milled local industry residue</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard</td>
<td>IRAM 50000-50001</td>
<td>CAN/CSAA 23.5-M86</td>
<td>None</td>
</tr>
<tr>
<td>Specific area, m²/kg</td>
<td>310</td>
<td>19000</td>
<td></td>
</tr>
<tr>
<td>Bulk density</td>
<td>2.9</td>
<td>2.2</td>
<td></td>
</tr>
<tr>
<td>Specific gravity, kg/m³</td>
<td>450–500</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SiO₂, %</td>
<td>28.4</td>
<td>96.5</td>
<td>59.5</td>
</tr>
<tr>
<td>Fe₂O₃, %</td>
<td>4.90</td>
<td>2.0</td>
<td>0.74</td>
</tr>
<tr>
<td>Al₂O₃, %</td>
<td>4.23</td>
<td>0.5</td>
<td>–</td>
</tr>
<tr>
<td>CaO, %</td>
<td>54.6</td>
<td>0.8</td>
<td>21.4</td>
</tr>
<tr>
<td>MgO, %</td>
<td>1.04</td>
<td>0.9</td>
<td>4.97</td>
</tr>
<tr>
<td>Na₂O, %</td>
<td>0.48</td>
<td>0.4</td>
<td>1.37</td>
</tr>
<tr>
<td>K₂O, %</td>
<td>0.59</td>
<td>2.0</td>
<td>2.55</td>
</tr>
<tr>
<td>SO₃, %</td>
<td>3.5</td>
<td>–</td>
<td>40</td>
</tr>
<tr>
<td>Pozzolanic coefficient</td>
<td>0.68</td>
<td>8.0</td>
<td>None</td>
</tr>
<tr>
<td>Lost of ignition, %</td>
<td>1.53</td>
<td>3.5</td>
<td>6.9</td>
</tr>
</tbody>
</table>

224
and admixtures, looking for the most economical combination in lab conditions. The percentage of superplasticizer used was 1.5 for (R) mixtures and 2.0 for (S) and (H) mixtures in agreement with manufacturer recommendations.

3 EXPERIMENTAL PROGRAM

3.1 Experimental mixtures

In this work the results of three different mixtures are compared: the reference mixture with portland-pozzolan cement (R), the mixture with silica fume replacing part of the cement (S) and the mixture with the industrial milled residue replacing part of the cement (H). The replaced percentage was 7.5 in both mixtures.

Three different ratios of dry aggregates/cementitious materials by mass (m: 1) were used for each concrete family: 3/1, 4/1 and 5/1.

All mixtures had the same fresh workability as measured by the slump test (70 ± 20 mm). There are 9 different concrete mixture proportions for the 3 concrete families.

The applied method requires adjustment only for the moisture content of coarse and fine aggregate. The mixture proportions studied are shown in Table 2.

3.2 Mixing method and test specimens

All materials were stored in the laboratory environment at 20°C and 50% relative humidity. Then, they were mixed in a pan type paddle mixer under the same laboratory condition.

The order of incorporation of the materials in the mixer was: 80% of the water, coarse aggregate, cement, part of the superplasticizer, fine aggregate and finally the rest of the water and the superplasticizer for mixture (R), and for mixtures (S) and (H) the additions were mixed with the fine aggregate. After all the materials were inside the mixer, the mixing began and the mixing time was between seven and ten minutes.

Each batch included sufficient concrete for slump tests [ASTM C 143], fresh air content [ASTM C 231], unit weight test [ASTM C 29] and twenty 100 mm diameter by 200 mm height cylinders for mechanical strength tests [ASTM C 39] and for absorption tests.

The cylindrical specimens were fabricated in accordance to ASTM C 31. The samples were covered with plastic film and left in moulds for 22 h, thereafter, they were removed from the moulds and immersed in curing tanks up to the time of testing. All cylindrical specimens were capped with special high resistant capping material [ACI 363-2R 1998].

The height of the disks for capillary absorption by suction test was 50 mm, which were previously sawed to exclude the first 50 mm of the cylindrical specimens.

3.3 Laboratory tests

Fresh concrete was evaluated by slump test because it is useful test to represent the properties of the concrete in the plastic state.

The tests performed on hardened concrete were: compressive strength at 28, 91 and 180 days, water absorption and total pores volume by ASTM C 642 and capillary absorption by suction test at 28 days applying a new national standard [IRAM 1871 2004].

<table>
<thead>
<tr>
<th>1:m cement: aggregates</th>
<th>Type of mixture</th>
<th>Portland-pozzolan cement:</th>
<th>Addition</th>
<th>Fine aggregate</th>
<th>Coarse aggregate</th>
<th>W/Cm</th>
<th>Cementitious materials content kg/m³</th>
<th>Slump mm</th>
<th>Density kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:3</td>
<td>R 1</td>
<td>0</td>
<td>1.24</td>
<td>1.76</td>
<td>0.34</td>
<td>547</td>
<td>50</td>
<td>2371</td>
<td></td>
</tr>
<tr>
<td>1:3</td>
<td>S 0.925</td>
<td>0.075</td>
<td>1.24</td>
<td>1.76</td>
<td>0.36</td>
<td>545</td>
<td>70</td>
<td>2377</td>
<td></td>
</tr>
<tr>
<td>1:3</td>
<td>H 0.925</td>
<td>0.075</td>
<td>1.24</td>
<td>1.76</td>
<td>0.36</td>
<td>544</td>
<td>80</td>
<td>2370</td>
<td></td>
</tr>
<tr>
<td>1:4</td>
<td>R 1</td>
<td>0</td>
<td>1.80</td>
<td>2.20</td>
<td>0.42</td>
<td>437</td>
<td>50</td>
<td>2369</td>
<td></td>
</tr>
<tr>
<td>1:4</td>
<td>S 0.925</td>
<td>0.075</td>
<td>1.80</td>
<td>2.20</td>
<td>0.46</td>
<td>432</td>
<td>70</td>
<td>2359</td>
<td></td>
</tr>
<tr>
<td>1:4</td>
<td>H 0.925</td>
<td>0.075</td>
<td>1.80</td>
<td>2.20</td>
<td>0.46</td>
<td>430</td>
<td>90</td>
<td>2350</td>
<td></td>
</tr>
<tr>
<td>1:5</td>
<td>R 1</td>
<td>0</td>
<td>2.36</td>
<td>2.64</td>
<td>0.50</td>
<td>367</td>
<td>50</td>
<td>2385</td>
<td></td>
</tr>
<tr>
<td>1:5</td>
<td>S 0.925</td>
<td>0.075</td>
<td>2.36</td>
<td>2.64</td>
<td>0.52</td>
<td>358</td>
<td>80</td>
<td>2336</td>
<td></td>
</tr>
<tr>
<td>1:5</td>
<td>H 0.925</td>
<td>0.075</td>
<td>2.36</td>
<td>2.64</td>
<td>0.52</td>
<td>357</td>
<td>90</td>
<td>2330</td>
<td></td>
</tr>
</tbody>
</table>
The results of hardened concrete properties are presented in Table 3. The value of compressive strength represents the average strength of three, 100 × 200 mm cylinders.

The mixture proportions nomogram in Figure 1 shows the results of compressive strength at 28 days for the different mixtures studied, taking into account the inclusion of the industrial milled residue as an addition.

The regression analysis for compressive strength confirms Abrams’s Law for each family of concretes [Maldonado & Helene 2002].

Figure 2 shows the increase of compressive strength with time for the studied mixes. Figure 2 also

<table>
<thead>
<tr>
<th>L:m cement : Type of aggregates mixture</th>
<th>Mixture</th>
<th>Compressive strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>28 days</td>
</tr>
<tr>
<td>1:3 R M3</td>
<td></td>
<td>41.3</td>
</tr>
<tr>
<td>1:3 S M5</td>
<td></td>
<td>52.9</td>
</tr>
<tr>
<td>1:3 H M9</td>
<td></td>
<td>38.5</td>
</tr>
<tr>
<td>1:4 R M1</td>
<td></td>
<td>35.7</td>
</tr>
<tr>
<td>1:4 S M4</td>
<td></td>
<td>45.9</td>
</tr>
<tr>
<td>1:4 H M7</td>
<td></td>
<td>28.5</td>
</tr>
<tr>
<td>1:5 R M2</td>
<td></td>
<td>29.4</td>
</tr>
<tr>
<td>1:5 S M6</td>
<td></td>
<td>40.5</td>
</tr>
<tr>
<td>1:5 H M8</td>
<td></td>
<td>23.7</td>
</tr>
</tbody>
</table>

Figure 1. Mixture proportions nomogram for 28 days.
shows that (S) mixes always attain the maximum value of compressive strength, while the values of mixes (R) and (H) are similar to each other.

The values of compressive strength comply with the requirements of national codes, but the mixes with industrial milled residue can not be distinguished by the strength values. On the other hand, the addition of silica fume always showed the highest compressive strength (Fig. 2).

When studying the workability of fresh concretes, it was observed that the main difference between (H) mixes and (R) and (S) mixes was due to the fact that the milled residue performs as fine aggregate (Fig. 1).

The suction test allows to identify effectively the use of additions due to the increment of mass of specimen. Figures 3–5 show the variation of the increment of mass with square root of time when the increase due to capillary absorption of water becomes stabilized for the different mixtures. The speed of suction in mixtures with milled local industry residue is greater than in the other mixtures.

This test corroborates the behaviour of this addition as a fine aggregate and also ratifies the fact that the addition of silica fume makes the concretes more water-tight.

Table 4 shows the results of suction tests by capillary absorption. The main difficulty with this test is to obtain the condition of hardened concrete and the time required to do the tests due to uneven distribution of

![Figure 2. Results of compressive strength for studied mixtures at different ages.](image)

![Figure 3. Results of tests of capillary suction for mixtures (R).](image)

![Figure 4. Results of tests of capillary suction for mixtures (S).](image)
moisture in the specimen [Maldonado & Helene 2005]. With regard to non aggressive conditions (low permeability), the new national code establishes a limit on the speed of suction of water by capillary absorption to less than 4 g/m²√s for concrete specimens dried at 50°C [CIRSOC 2012 2002].

Table 5 shows the results of water absorption and total pores volume at 28 days.

The best performance corresponds to mixtures (R). The incorporation of industrial milled residue in mixtures (H) implies an additional amount in admixtures, and if a greater percentage is incorporated, the compressive strength will decrease because it does not have pozzolanic properties effects.

5 CONCLUSIONS

The use of the mixture proportions nomogram allows a correct and relevant comparison between the different concrete families, adopting the level of strength required by codes. It shows the results of a family of concretes and in situ the changes can be rapidly made accurately.

The studied industrial milled residue modifies the fresh concrete properties because it acts as fine aggregate. It improves the workability but not the mechanical properties.

The studied industrial milled residue can only be incorporated on concretes that are exposed to conditions without water, moisture and reinforcement because the content of SO₃ exceeds the values of national standards.

The use of silica fume is compatible with local aggregates and the mix proportions 1:3 and 1:4 give an impervious concrete in agreement with a new national standard.

The evaluation of speed of suction shows the differences in performance between the various proportions of concretes.

The incorporation of industrial milled residue does not give better performance in relation with costs.

The incorporation of silica fume reached the best compressive strength for concretes with high strength but it is a misspent with lower strength.

ACKNOWLEDGMENTS

This work has been part of an experimental program on high performance concrete supported by the Technological National University of Argentine. The authors want to thank the University staff: Marcelo Guzmán, Marcos Oropel, Claudio Dagne, Sergio Acosta and the enterprises that they had made the development of the research program possible.

REFERENCES


INTRODUCTION

The life expectancy of reinforced concrete structures is significantly influenced by the corrosion factor of concrete and particularly of the reinforcement. Highly developed concrete corrosion can cause concrete structural failure.

Concrete can be eroded by corrosive gases and vapours, corrosive water and solutions, some solid substances, sometimes by the impact of biological factors. Concrete protection can be divided into two parts: primary and secondary protection. Primary means concrete quality improvement. By secondary protection we mean measures, which are taken after the pouring of concrete. For example: painting, use of repair materials, which will cut down the impact of corrosive media to a structure. It is possible to implement these measures at a later stage of construction or several years after finishing the construction.

By far the most significant of stages in successful concrete structure repair is a thorough survey of the structure and then the subsequent choice of optimally designed rehabilitation technology including surface treatment. For surveys of damaged concrete structures a methodology has been developed. In cases of atmospheric corrosion, especially for CO₂ attack, a procedure for damage classification and common suggestions for minimum measures for concrete rehabilitation construction has been developed by our institute.

The aim of using repair materials is to repair the surface of reinforced concrete structural elements to their original shape, respectively to reinstate or to make the covering reinforcement layer thicker. Repair materials serve above all to maintain the longevity of reinforced concrete elements and the reinstatement of its original appearance.

For secondary concrete protection, polymer based materials are mostly used, namely epoxy resins. For cost effective repairs the use of waste materials as fillers for polymers should be considered. An advantage of the usage of waste materials as fillers for repair materials is the use of industrial waste and therefore its subsequent liquidation. At the same time mineral resources are conserved. Mining and processing negatively affects the environment.

The required property of epoxy concrete material is adhesiveness to the base material and the provision of a firm hold. Good bonding can be achieved only when the original surface is properly prepared and the correct preparation of the base is made for achieving the best possible adhesion – minimally 1,5 N/mm².

Prepared surfaces must be solid, homogenous, well secured to a load bearing part of the construction, dust free, grease free and free of all other impurities, which might act as separators, and also without bigger pores. In addition it must be without rising damp. It would be advantageous to have a dry base with maximum residual dampness to 4 %. If necessary,
special materials suitable for damp bases must be used. An example of such a material, the epoxy resin LENA P102, made by Lena Chemical Ltd., Czech Republic can be used.

This type of epoxy resin can be applied to a damp base and can even be used for underwater jobs. In many cases, after a concrete structure repair is done and the original shape of the structure is achieved, it is necessary to apply a protective coating, the purpose of which is mainly to protect the structure from further penetration by corrosive materials.

The choice of the protective coating ought to be made according to certain criteria. One of the most important of those criteria is the compatibility with reconstruction materials and resistance to the corrosive influence to which the particular structure is exposed.

Together with reconstruction materials based on polymer bonding agents, coatings have also been developed, based on polymers, with fillers made from waste materials.

2 MATERIALS USED FOR THE PREPARATION OF NEW REPAIR MATERIALS

2.1 Waste materials – fillers

**Granulated blast furnace slag**: Slag is created by the rapid cooling down of melted liquid slag, which is a by-product of iron production in blast furnaces. The quicker the process of cooling down is, the more glassy substance appears in the slag and the slag is power richer, which means more reactive. The slag is characterized by a high ratio of glassy and crystalline substances, of chemical and mineral composition.

For the, Granulated blast furnace milled slag has been chosen to be used as a filler, produced by the company Trinecke Zelezarny, and modified by a company Kotouc Stramberk Ltd., Czech Republic. (Bulk density) Specific weight 2850 kg/m³, specific surface 388.7 m²/kg, dampness 0.02%.

**Foundry sand**: Foundry siliceous sand is a foundry by-product, where for better strength, clean siliceous sand is mixed with, for example bentonite, water glass, or with a mix of these substances or with others substances and then filled into moulds. The substance is pressed and a mandrel is put in place. After casting, the product is left to cool down. The cast is moved onto a grate, where the mould breaks up. The sand is sorted out in a hopper, which is placed below the grate, in addition the sand is placed into storage chambers.

Foundry sand from the gray foundry UXA Ltd., Brno, Czech Republic has been used. Specific weight 2580 kg/m³, dampness 0.13%.

**Raw Material – Fillers**

**Siliceous sand**: Siliceous sand has been used as filler to the referential mixture. Commercial standard sand has been supplied by a company GEBRUDER DORFNER GmgH, ISG mbH, Scharhof 1, D 922 42 Hirschau, Germany. Siliceous sand DORSILIT contains more than 98% SiO₂.

The sand is washed several times and properly separated to obtain more than twelve varieties. The sand is free of impurities and its’ particles are round in shape.

The Mixture ISG A1 used has grading of 0.06 – 1.5 mm. Fields of its’ application – filler for polymer concrete, for the production of plaster mixtures, filler for self levelling floor mixtures, for glue production, for pointing mixtures, for topping mixtures.

Physical chemical property – chemical characteristic: silica of white- gray colour, dampness 0.01%, specific weight 2 650 kg/m³.

2.2 Bonding agent – repair material

**Epoxy Resin (ER):** Tested epoxy resins used were made by the company Lena Chemical Ltd., Czech Republic. Resin P 130 is made for usage in the manufacture of polymer concretes.

The mixture is of low viscosity, consists of two parts, and is no solvent mixture. Scope of usage – for mixtures used for polymer-concrete flooring in areas of heavy traffic in industrial halls, repair shops, warehouses and so on.

Specification: workability at 20°C about 20 minutes, crushing strength 90 N/mm², tensile strength 53 N/mm², bending strength 93 N/mm².

2.3 Bonding agent – coating material

**Epoxy Resin (ER):** Epoxy resin is used for coating is the same kind of polymer but modified by the producer to fit little bit different requirements for coating. (Longer workability time, lower viscosity, etc.)

Specification: workability at 20°C cca 40 minutes, compressive strength 80 N/mm², tensile strength 55 N/mm², bending strength 95 N/mm².

3 WORK SEQUENCE

The first part of research was the design of repair materials (mortars) with proposed use for the repair of damaged surfaces of concrete structures. The second part of the research was devoted to the development of coating materials which should be used as the final treatment of the repaired structure, or at least the most endangered parts.

Repair mortar mixtures have been developed with increasing quantities of waste material by 5% of filling. In this paper two selected materials are described with two different waste material fillers and one referential material.

Tested materials are marked with numbers corresponding to the type of bonding material and the
amount of filler, the letter m stands for the purpose of material (m - repair mortar). An Analoguesystem was used for the development of coatings. Referential and newly designed mixtures have been examined using basic quality tests – adhesion to substrate, compressive strength, bending strength. Based on these tests, most suitable mixtures have been selected for further testing. In additional supplementary examination of properties has been carried out – surface water absorption, abrasiveness, frost resistance, module of elasticity in bend, UV radiation resistance and carbon dioxide permeability.

Not all types of tests and quality requirements for concrete construction rehabilitation are mentioned in European standards. The Czech Concrete Construction Repair Association issued a publication specifying the type of tests, methodology of tests and minimum requirements for materials used in the field of concrete construction rehabilitation.

This publication “Technical Conditions for Concrete Construction Rehabilitation” [Drochytka et al. 2003] substitutes specifications given by European standards. Repair materials with waste material fillers must fulfil minimum requirements – see Table 1.

4 SELECTED RESULTS OF TESTING

4.1 Determination of adhesion – pull off test
The aim of the adhesion test is to determine the force needed for the separation of repair mortar from a defined area of the base using a strictly perpendicular force.

According to Technical Conditions for Concrete Construction Rehabilitation [Drochytka et al. 2003] the required value of adhesion of repair materials is \( >1.5 \text{ N/mm}^2 \) and average \( >1.7 \text{ N/mm}^2 \), to what all tested mixtures complied.

4.2 Bending Strength EN ISO 178 and compressive strength EN ISO 604
According to the Technical Conditions for Concrete Construction Rehabilitation [Drochytka et al. 2003] the required value of bending strength for repair materials is \( >5.5 \text{ N/mm}^2 \) and compressive strength test the required value is \( >25 \text{ N/mm}^2 \) and \( <50 \text{ N/mm}^2 \). All examined mixtures complied with the minimum value. As far as the maximum value of 50 N/mm² is concerned, this value is needed for silicate materials due to the reason that the repair material should not have a greater module of elasticity than the base material. Polymeric composites have generally lower modules of elasticity and tested mixtures have reached values of elasticity modules below 5 GPa see Table 7. Therefore it is impossible to take the compressive strength test of 50 N/mm² as being the maximum value.

4.3 Determination of the absorption of water EN 1170–6
Absorption of water is defined as the quantity of water which is absorbed by the cavities and pores of a dry sample when submerged in water.

It is perceptible, from the figures gained, that the minimum, (virtually – zero) absorption, are shown in

---

Table 1. Required characteristics of repair materials according to the publication “Technical Conditions for Concrete Construction Rehabilitation” [Drochytka et al. 2003].

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Conclusive test</th>
<th>Control test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength</td>
<td>( &gt;25 \text{ N/mm}^2 )</td>
<td>( &gt;25 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td>Bending strength</td>
<td>( &gt;5.5 \text{ N/mm}^2 )</td>
<td>( &gt;5.5 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td>Cohesion with base (without adhesive bridge)</td>
<td>average ( &gt;1.7 \text{ N/mm}^2 )</td>
<td>average ( &gt;1.1 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>( &lt;0.5 \text{ 0/00} )</td>
<td>–</td>
</tr>
<tr>
<td>Crack inclination</td>
<td>1 crack of width to 0.1 mm</td>
<td>1 crack of width to 0.1 mm</td>
</tr>
<tr>
<td>Frost resistance</td>
<td>T 100</td>
<td>–</td>
</tr>
<tr>
<td>Coefficient of thermal expansion</td>
<td>( &lt;14 \times 10^{-6} )</td>
<td>–</td>
</tr>
<tr>
<td>Statistical module of elasticity</td>
<td>( &lt;30 \text{ GPa} )</td>
<td>–</td>
</tr>
</tbody>
</table>

---

Figure 1. Images of test beams, 7 – m slag filler, 2 – m foundry sand filler, 102 – m siliceous sand filler.
mixtures containing blast furnace slag. Mixtures containing siliceous sand ISG A1 and foundry sand have values 20x higher. It is caused by open pores and cavities, which occur on the surface of tested samples and are caused by the imperfect coating of material grains of filling by epoxy resin on the surface of the sample in contact with the mould.

When the absorption test is performed, water gets into pores and cavities, therefore these mixtures show some percentage increase in water content. Mixtures made from blast furnace slag do not show open pores and cavities.

4.4 Frost resistance, specification Czech standard CSN 72 2452

Frost resistant mortars are tested by the alternate freezing and defrosting of water saturated test samples.

### Table 2. Mixture contents and results of adhesion-pull off test.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Content [%]</th>
<th>Filler content [%]</th>
<th>Tear off place</th>
<th>Adhesion [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>130-m</td>
<td>14.3</td>
<td>85.7 s. sand</td>
<td></td>
<td>4.23</td>
</tr>
<tr>
<td>10-m</td>
<td>20.0</td>
<td>80.0 f. sand</td>
<td></td>
<td>3.17</td>
</tr>
<tr>
<td>11-m</td>
<td>22.5</td>
<td>77.5 f. sand</td>
<td></td>
<td>4.27</td>
</tr>
<tr>
<td>12-m</td>
<td>25.0</td>
<td>75.0 f. sand</td>
<td></td>
<td>3.13</td>
</tr>
<tr>
<td>16-m</td>
<td>27.5</td>
<td>72.5 slag</td>
<td></td>
<td>3.77</td>
</tr>
<tr>
<td>17-m</td>
<td>30.0</td>
<td>70.0 slag</td>
<td></td>
<td>3.53</td>
</tr>
<tr>
<td>18-m</td>
<td>32.5</td>
<td>67.5 slag</td>
<td></td>
<td>3.13</td>
</tr>
</tbody>
</table>

### Table 3. Contents of mixtures and results gained from bending strength, compressive strength tests.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Content ER [%]</th>
<th>Content and filler type [%]</th>
<th>Bending strength [N/mm²]</th>
<th>Compressive strength [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>130-m</td>
<td>14.3</td>
<td>85.7 s. sand</td>
<td>33.00</td>
<td>59.70</td>
</tr>
<tr>
<td>10-m</td>
<td>20.0</td>
<td>80.0 f. sand</td>
<td>13.65</td>
<td>28.95</td>
</tr>
<tr>
<td>11-m</td>
<td>22.5</td>
<td>77.5 f. sand</td>
<td>17.55</td>
<td>33.13</td>
</tr>
<tr>
<td>12-m</td>
<td>25.0</td>
<td>75.0 f. sand</td>
<td>18.75</td>
<td>34.30</td>
</tr>
<tr>
<td>16-m</td>
<td>27.5</td>
<td>72.5 slag</td>
<td>39.30</td>
<td>69.68</td>
</tr>
<tr>
<td>17-m</td>
<td>30.0</td>
<td>70.0 slag</td>
<td>39.60</td>
<td>72.34</td>
</tr>
<tr>
<td>18-m</td>
<td>32.5</td>
<td>67.5 slag</td>
<td>40.50</td>
<td>75.93</td>
</tr>
</tbody>
</table>

### Table 4. Quantity and type of filler influencing absorption.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Content ER [%]</th>
<th>Content and filler type [%]</th>
<th>Water absorption [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>130-m</td>
<td>14.3</td>
<td>85.7 siliceous sand</td>
<td>0.20</td>
</tr>
<tr>
<td>11-m</td>
<td>22.5</td>
<td>77.5 foundry sand</td>
<td>0.20</td>
</tr>
<tr>
<td>16-m</td>
<td>27.5</td>
<td>72.5 slag</td>
<td>0.01</td>
</tr>
</tbody>
</table>

### Table 5. Mixture contents and frost resistance tests results (after 100 freezer cycles) of compressive strength.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Content ER [%]</th>
<th>Content and filler type [%]</th>
<th>Compressive strength [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>130-m</td>
<td>14.3</td>
<td>85.7 siliceous sand</td>
<td>59.70</td>
</tr>
<tr>
<td>11-m</td>
<td>22.5</td>
<td>77.5 foundry sand</td>
<td>33.13</td>
</tr>
<tr>
<td>16-m</td>
<td>27.5</td>
<td>72.5 slag</td>
<td>69.68</td>
</tr>
</tbody>
</table>

### Table 6. Mixture contents and frost resistance tests results (after 100 freezer cycles) of bending strength.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Content ER [%]</th>
<th>Content and filler type [%]</th>
<th>Bending strength [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>130-m</td>
<td>14.3</td>
<td>85.7 siliceous sand</td>
<td>33.00</td>
</tr>
<tr>
<td>11-m</td>
<td>22.5</td>
<td>77.5 foundry sand</td>
<td>17.55</td>
</tr>
<tr>
<td>16-m</td>
<td>27.5</td>
<td>72.5 slag</td>
<td>39.30</td>
</tr>
</tbody>
</table>

One freezing cycle consists of at least four hours of freezing at temperatures of −20 ± 3°C and at least two hours of defrosting in a water bath at a temperature of about 20 ± 3°C. Tested samples were subjected to 100 freezing cycles. Then bending strength tests and compressive strength tests were been carried out.

According to “Technical Condition for Concrete Construction Rehabilitation” [Drochytka et al. 2003],
repair material must comply with 100 freezing cycles. This requirement has been fulfilled by all test mixtures.

4.5 *Module of elasticity in bend, EN ISO 178*

According to “Technical Conditions for Concrete Construction Rehabilitation” [Drochytka et al. 2003]...
The possibility of the usage of waste material as fillers for epoxy repair materials in the area of concrete construction rehabilitation has been shown. Usage of these materials is financially beneficial in the production of epoxy concrete repair plasters.

The most important aspect however is the ecological standpoint – it saves non-renewable sources of high quality siliceous sands which are used e.g. for the production of Czech glass. Estimated deposits of high quality glass sands in the Czech Republic are estimated to last only for another 15 years maximum. A significantly positive element is also the usage of industrial waste, which continues to rise in quantity every year.

The development of the above mentioned mixtures containing waste materials as fillers is still in progress. Materials meet the requirements of the main guidelines in the Czech Republic: “Technical Conditions for Concrete Construction Rehabilitation”, and research for other tests and their implementation, as required by EU standards is also in progress.

REFERENCES


ACKNOWLEDGEMENT

This paper was prepared with financial support from grant of the Czech Grant Agency 103/05/P262, entitled: “Thin layer protection systems for concrete exposed to special environment” and from the research project CEZ – MSM 0021630511, entitled: “Progressive Building Materials with Utilization of Secondary Raw Materials and their Impact on Structures Durability”. And in framework of project FRVŠ 1560/2007/G1
Development of an optimal high-performance concrete mixture for Tennessee bridge decks

R.M. Salem, E.G. Burdette & J.H. Deatherage
Department of Civil and Environmental Engineering, University of Tennessee, Knoxville, TN, USA

ABSTRACT: This paper describes a laboratory investigation of the development of an optimal high-performance concrete (HPC) mixture for Tennessee cast-in-place concrete bridge decks. Four modified concrete mixtures were developed by reducing the cementitious content of Tennessee Department of Transportation (TDOT) Class D mix and using a denser graded aggregate combination, while keeping the water-cementitious material ratio constant. Fly ash, slag, and silica fume were used as cementitious replacement materials to evaluate and develop the optimal HPC mix. Various parameters relevant to bridge deck performance were evaluated, including compressive strength, drying shrinkage, freeze-thaw durability, and chloride ion permeability. All four modified mixes exhibited higher compressive strength, lower drying shrinkage, and lower chloride ion permeability than the Class D control mix. Based on performance and potential economic savings, two of the modified mixes that contained fly ash and slag as cement replacements can be recommended as potential replacements for TDOT Class D concrete.

1 INTRODUCTION

In the United States, many concrete pavements and bridge decks fail long before reaching their expected service life. A survey of the data reported by [Stidger 2003] showed that about 27% of the 584,000 bridges in the U.S. are rated substandard and have major problems associated with crack deterioration in their decks. However, these problems may be overcome in future construction projects by using high-performance concrete (HPC) mixes that provide an improved overall durability performance and thus reduce the life-cycle costs of structures [Moore 1999]. Studies to identify various concrete mixture proportions to attain the required performance characteristics are abundant [Cabrera et al. 1997; Hou et al. 2001; Kovler et al. 2000; Ozyildirim et al. 1994]. In fact, the American Association of State Highway and Transportation Officials (AASHTO) Task Force on Strategic Highway Research Program (SHRP) Implementation, in conjunction with the Federal Highway Administration (FHWA) and the Transportation Research Board, established an HPC lead state team in 1996 to coordinate the efforts to promote HPC use. Due to these efforts, HPC was successfully used for bridge deck construction in a number of states [Beacham 1999; Edwards 2000; Ozyildirim 2002; Ralls 1999; Ramakrishnan et al. 2001; Streeter 1996; Waszczuk et al. 1999].

The concrete production industry typically uses only two aggregate combinations, coarse and fine, which usually results in gap-gradation mixes with a high volume of internal voids. As a result, these mixes require more cement paste to fill these voids and glue the aggregates together. This cement paste is prone to cracking due to its high thermal and drying shrinkage strains and permeability. Substantial increases of the paste above the optimal content will, therefore, result in major reductions in the long-term durability of concrete mixes. Thus, any improvements in the aggregate gradation from a gap to a more dense gradation to limit the use of cement paste would not only improve the long-term durability of the mixes but would also result in considerable savings in material costs.

This study focused on developing appropriate mix proportions using materials available in the state of Tennessee through adjustment of TDOT Class D gap-graded mix to a more densely graded mix and by incorporating fly ash, slag, and silica fume as cement replacements. As a result, an optimum mix that exceeds the TDOT specifications in term of durability is being recommended to serve, potentially, as the standard TDOT bridge deck mix in the future.

2 EXPERIMENTAL PROGRAM

2.1 Materials

Type I Portland cement, Class C fly ash, ground-granulated blast-furnace slag, and silica fume were
used in this study with a specific gravity of 3.15, 2.30, 2.90, and 2.20, respectively. Two chemical admixtures: high-range water-reducing agent (HRWRA) conforming to ASTM C 494 type A and F and air-entraining agent (AEA) conforming to ASTM C 260 were also used. The aggregates used consisted of limestone #57, limestone #7 and natural sand; their properties are provided in Table 1.

2.2 Mixture development and proportion

A total of five mixtures were prepared in this study with Class D mix being the control mix. Class D mix, which doesn’t contain mineral admixtures as cement replacement, has been used as the standard bridge deck mix in the state of Tennessee. The aggregate gradation used in Class D mix is typically a gap-graded aggregate using limestone and natural sand as the coarse and fine aggregate, respectively. The other four modified mixes were developed by reducing the cementitious content of Class D mix by 20 percent, modifying the gap-gradation to a more densely graded aggregate, and by incorporating fly ash, slag, and silica fume as cement replacements as given in Table 2. Mix FA refers to 25 percent of cement replacement by fly ash. Mix FASF refers to 20 and 5 percent replacements of cement by fly ash and silica fume, respectively. Mix S refers to a 35 percent of cement replacement by slag. Mix SSF refers to 35 and 5 percent replacements of cement by slag and silica fume respectively.

Table 2 provides the mix proportions for the five mixes, while Figure 1 shows the aggregate gradation used in the mixes. Intermediate-size limestone coarse aggregate designated as #7 was used to change Class D gap-graded aggregate to a more densely graded aggregate. As shown in Figure 1, for the control mix, about 32% of the aggregates were retained on the 12.5 mm sieve, while only 13% and 9% were retained on the 9.50 mm and the 4.75 mm sieves. By incorporating a third aggregate (#7), the aggregates retained on these three sieves were 21%, 16%, and 18% respectively, resulting in a more uniform particle distribution. This third aggregate served as an intermediate size aggregate for filing the major voids within the lager aggregate size (#57).

2.3 Mixing and curing of concrete

Concrete mixtures were prepared using a mechanical mixture and applying standard rodding for compaction. The specimens were cured in a standard curing chamber until the day of testing.

2.4 Test procedures

The following procedures were used to evaluate the various characteristics of the HPC mixes:

- The compressive strength of the cylinder specimens was measured in accordance with ASTM C 39 and tests were conducted on the specimens at 28 days.
- The change in length (drying shrinkage) of the concrete specimens was measured in accordance with ASTM C 157. After 7 days of water curing, for each mixture, three specimens were tested at 1, 2, 3, 4, 5, 6, 7, 14, 21, 28, 56, and 112 days.
- The concrete’s resistance to freezing and thawing was determined by testing 76 × 102 × 406 mm concrete prisms in accordance with ASTM C 666, Procedure A.
- The chloride ion permeability test was conducted in accordance with ASTM C 1202 and AASHTO T 277. An electric current of 60 volts DC was applied across the properly cured specimens at 56 days, and the total charge passed during a 6-hour period provided the chloride ion permeability of the concrete.

Table 1. Physical properties of coarse and fine aggregates.

<table>
<thead>
<tr>
<th>Material</th>
<th>Limestone #57</th>
<th>Limestone #7</th>
<th>Natural sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal size</td>
<td>25.4</td>
<td>19.0</td>
<td>9.5</td>
</tr>
<tr>
<td>Absorption (%)</td>
<td>0.49</td>
<td>0.35</td>
<td>0.62</td>
</tr>
<tr>
<td>SSD specific gravity</td>
<td>2.79</td>
<td>2.84</td>
<td>2.60</td>
</tr>
</tbody>
</table>

Table 2. Laboratory mix proportions.

<table>
<thead>
<tr>
<th>Materials (kg/m³)</th>
<th>Class D</th>
<th>FA</th>
<th>FASF</th>
<th>S</th>
<th>SSF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>368</td>
<td>223</td>
<td>223</td>
<td>193</td>
<td>178</td>
</tr>
<tr>
<td>Slag</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>104</td>
<td>104</td>
</tr>
<tr>
<td>Silica fume</td>
<td>—</td>
<td>—</td>
<td>15</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Fly ash</td>
<td>—</td>
<td>74</td>
<td>59</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Limestone, #57</td>
<td>1157</td>
<td>703</td>
<td>702</td>
<td>709</td>
<td>707</td>
</tr>
<tr>
<td>Limestone, #7</td>
<td>—</td>
<td>511</td>
<td>511</td>
<td>515</td>
<td>514</td>
</tr>
<tr>
<td>Sand</td>
<td>682</td>
<td>748</td>
<td>748</td>
<td>755</td>
<td>753</td>
</tr>
<tr>
<td>Water</td>
<td>147</td>
<td>119</td>
<td>119</td>
<td>119</td>
<td>119</td>
</tr>
<tr>
<td>w/cm</td>
<td>0.40</td>
<td>0.40</td>
<td>0.40</td>
<td>0.40</td>
<td>0.40</td>
</tr>
</tbody>
</table>

Figure 1. Gradation chart for the control and modified mixes.
3 DISCUSSION OF RESULTS

3.1 Compressive strength

The laboratory results for the compressive strength of the design mixes at 28 days are shown in Figure 2. The results show that all of the design mixes exceeded the design compressive strength of TDOT Class D concrete (27.5 MPa).

The compressive strength of all four modified mixes was higher than that of the control mix (Class D mix), even though the cementitious content of the modified mixes was lower than that of the control mix. There may be two main reasons for this. First, since the w/cm ratio was kept constant for all of the mixes, the mixing water was reduced for the four modified mixes as the cement content was reduced. Lower cement and water content in these mixes produced less cement paste and, consequently, higher aggregate contents per unit volume. In addition to the higher strength provided by the higher aggregate content in these mixes, additional strength may also have been provided by the improved aggregate interlocking in the cement paste due to the aggregate particles being closer to each other. Also, less cement and water content in these modified mixes would have allowed less bleeding in these mixes than in the control mix. This could have improved the interface between the cement paste and the coarse aggregate surface by providing a stronger transition zone and thus increasing the overall compressive strength. Therefore, designing a mix with a rich cement paste (high cement content) beyond what is needed for a workable mix would appear to have negative effects on the compressive strength. In addition, using a more densely graded aggregate than that used in the control mix is a viable technique for reducing the original cementitious content by as much as 20 percent while still providing the required compressive strength. This technique not only produces economical mixes, but also ensures improvements in their performance properties.

3.2 Resistance to freezing and thawing

Resistance of concrete to freezing and thawing is measured in terms of the durability factor (DF) which is defined as

\[ DF = \frac{P_N}{300} \]

Where \( P \) is the relative dynamic modulus of elasticity at \( N \) cycles (as a percent of the initial value, at 0 cycles) and \( N \) is the number of cycles at which \( P \) falls below 60% (\( N \) is taken to be 300 when \( P \) remains higher than 60% after the completion of 300 cycles of freezing and thawing).

The durability factors (DFs) for the mixes are shown in Figure 3. As shown in the figure, some of the mixes had lower values of DFs but these values are about 80 percent and thus were still high enough to classify these mixes as clearly acceptable [Goodspeed et al. 199; Zia et al. 1991].

3.3 Drying shrinkage

The length change of the concrete specimens was measured at 1, 2, 3, 4, 5, 6, 7, 14, 21, 28, 56 and 112 days. The results for the mixes are shown in Figure 5 as logarithmic trend-lines to indicate development of shrinkage strain over time. As shown in the figure, the control mix had the highest shrinkage compared to the other four modified mixes, which may be attributed to its higher cement paste content. The mix with slag only had the second highest. Decreasing the cementitious content in the four modified mixes lowered their shrinkage, which is believed to be one of the main causes of cracking in bridge decks. Figure 4 also shows that drying shrinkage rate increased rapidly up to 28 days, and then became quite low, with almost a flat slope.
3.4 Chloride ion permeability

One of the major tasks of this study was to develop HPC mixture proportions that allow minimal penetration of chloride ions so that the corrosion potential of the reinforcing steel can be reduced and, therefore, the overall bridge deck durability enhanced. This task was accomplished by comparing the permeability of the four modified mixes developed in this study to the permeability of TDOT Class D standard bridge deck mix.

According to ASTM C 1202, the permeability of concrete mixes is ranked as follows: 100–1,000 Coulomb, very low permeability; 1,000–2,000 Coulomb, low permeability; 2,000–4,000 Coulomb, medium permeability; and above 4,000 Coulomb, high permeability. As shown in Figure 5, the four modified HPC mixes had significantly lower permeability values than the control mix. This may have been due to the use of more densely graded aggregate in the four mixes, which resulted in a decreased cement paste content. A lower cement paste content produced less porous material in a given volume of concrete, resulting in more resistance to chloride penetration. Therefore, it can be concluded that using more densely graded aggregate in these HPC mixes and reducing their cementitious content resulted in mixes with lower permeability and higher durability. Like other standard highway specifications, current TDOT specifications require at least a minimum cementitious content and specify a maximum w/cm ratio for Class D bridge decks. These specification requirements traditionally may have been prescribed to ensure adequate workability, finishibility, strength, and durability. However, with the advent of many chemical admixtures and cementitious replacement materials, the current prescriptive method of a minimum cementitious content and a maximum w/cm ratio may no longer be the most effective approach for addressing the durability performance of concrete. For example, the chloride ion permeability for standard Class D bridge deck concrete was found to be 2800 Coulombs, which is about three to six times higher than that of the four modified mixes. Therefore, replacing Class D concrete mix by one of the four modified mixes would not only achieve economical savings but would also improve the major performance properties of the bridge decks in the state of Tennessee.

Based on the previous discussion, all four modified mixes had comparable strength, drying shrinkage, freeze-thaw durability, and chloride ion penetration; however, the mixes with fly ash and slag (mixes FA and S) would be the most cost-effective mixes based on material cost only (Class D: $40/m³, FA: $34/m³, S: $36/m³, FASF $39/m³ and SSF: $40/m³), and could, therefore, be a potential replacement for TDOT Class D concrete.

4 CONCLUSIONS

Based on the findings of this study, the following conclusions can be made:

- Using a densely graded aggregate in HPC mix design is a viable technique for reducing the original cementitious content.
- Lowering the cementitious content of a mix, while keeping the w/cm ratio constant, has positive effects on its compressive strength, chloride ion permeability, and drying shrinkage.
- Changing the gap-graded TDOT Class D concrete bridge deck mix to a more densely graded mix and using 25 percent of fly ash or 35 percent of slag as a cement replacements would reduce the chloride ion permeability by 70 percent and the drying shrinkage by 20 percent, resulting in a more economical and durable high-performance concrete mix. Therefore, either FA or S mix could be a potential replacement for TDOT Class D concrete.

REFERENCES


Establishing optimum mixture proportions for concrete durability using recyclable by-products

D.M. Vruno
Minnesota Concrete Council, St. Paul, Minnesota, USA

ABSTRACT: Recyclable by-products have been used as cementitious material for the past four decades in our area. Originally, the use of lignite fly ash as a partial replacement for Portland cement was a result of finding a use for a waste product. The turn of the century has coincided with the emergence of a new philosophy with regard to the use of cementitious materials. This philosophy, based on the purposeful reuse of waste stream materials, embraces incorporating various combinations of cement, granulated slag, fly ash, and micro silica in the production of concrete. In light of environmental controls on the production of cement, shortages of suitable source materials, and an ever-growing worldwide demand for concrete, Minnesota Concrete Council (MCC) believes this new approach is here to stay. The intent of this study is to understand the effect each of these cementitious materials has on concrete durability. The Establishing Optimum Mixture Proportions for Concrete Durability has been a result of understanding the enhancements provided by the use of recyclable by-products.

1 INTRODUCTION

In the 19070's there was a “new material neurosis” with regard to adding lignite fly ash to concrete. The majority of our original research was to overcome this fear. The beneficial effects of fly ash with regard to its pozzolanic activity was known yet not fully embraced.

The decade of the 1980’s resulted in acceptance of recycled by-products for cementitious material in our area. Design engineers were specifying compressive strengths four times greater than what the industries had been producing. The use of fly ash and silica fume allowed compressive strengths to exceed f’c of 110 MPa by the end of the decade. Of the twenty tallest skyscrapers in the Twin Cities of St. Paul and Minneapolis, Minnesota, eighteen of them were constructed between 1985 and 1995. The concrete structural elements were designed to take up the least amount of usable space, yet they needed to support increased loads. This combination of requirements resulted in high compressive strength concrete. It was the recyclable by-products that allowed us to meet the higher compressive strengths.

Our focus readjusted at the turn of the century. No longer was compressive strength of concrete the important parameter. The decade of the skyscrapers had past. The new focus became the rehabilitation of our infrastructure. Our interstate freeways were half a century old and in need of replacement. Concrete durability, not compressive strength, became our new focus. Once again recyclable by-products were utilized to achieve what was required of concrete.

2 THIRTY YEAR CONCRETE DURABILITY STUDY

One advantage of being old is that you have the opportunity to analyze results of long-term durability studies that you were involved in since the beginning. The truest test of a concrete's durability is its service life. In 1973, the North Dakota Highway Department and the Federal Highway Administration determined that sufficient data had been presented to allow a 15% replacement of cement by lignite fly ash in concrete paving of Interstate 29 south of Fargo, North Dakota. Construction of a freeway ramp in the fall of 1973, and a 10-mile section of Interstate 29 in the late summer of 1974 was completed, utilizing lignite fly ash. A field laboratory test program was conducted by future members of the Minnesota Concrete Council in conjunction with the North Dakota State Highway Department to verify the suitability of lignite fly ash use in actual pavement construction. The purpose of the test program was to determine the strength and durability properties of lignite fly ash concrete under field construction conditions. Extensive field testing was conducted to verify the properties of the plastic concrete. Concrete cylinders and beams were cast on the project for a compressive and flexural strength test.
tests, rapid freeze-thaw testing and air void system analysis.

For thirty-three years the Interstate 29 south of Fargo, North Dakota has been subjected to traffic, deicers, plowing, and temperature extremes which ranged from $-33^\circ F$ to $106^\circ F$. Interstate 29 provided an excellent opportunity to observe the durability of Portland cement – lignite fly ash concrete. The majority of the pavement was placed with a 15% replacement of Portland cement with a lignite fly ash. A small percentage of the placement was placed without fly ash, thus providing a direct comparison of concrete with fly ash and concrete without fly ash.

Condition surveys were performed by the authors on the subject Interstate during July 1988 and July 2000. There existed no visual difference between the concrete with fly ash and the concrete pavement without fly ash. Four specific locations were chosen for extraction of samples for laboratory testing.

A review of the data (see Tables 1 and 2) indicates that the use of lignite fly ash as a partial replacement for Portland cement in concrete produced a strong, durable concrete in the paving of Interstate 29 south of Fargo, North Dakota. The concrete containing lignite fly ash as a 15% replacement attained higher strengths and greater resistance to deicer migration.

### 3 GLOBAL WARMING

One ton of CO₂ and other greenhouse gasses are released to the atmosphere per every ton of Portland cement clinker produced in the USA. It is a complicated relationship, but the most important concept to understand is the more CO₂ in the atmosphere the more the sun’s heat is trapped resulting in Global warming.

The most promising aspect of utilizing recyclable by-products as cementitious material is that not only will we be responsible stewards of this planet, we will also be producing a more durable concrete.

### 4 WHAT IS CONCRETE DURABILITY?

Durability of hydraulic-cement concrete is defined as its ability to resist weathering action, chemical attack abrasion, or any other process of deterioration. Durable concrete will retain its original form, quality and serviceability, when exposed to its environment [ACI 1991]. Concrete durability is largely dependent on the ease with which either or both liquid and gases enter into and move through the concrete. This is commonly referred to as the permeability of concrete.

#### 4.1 Principles responsible for concrete deterioration

The principal causes of deterioration of concrete structures in our area are the corrosion of reinforcing steel, exposure to cycles of freezing and thawing, alkali silica reaction and sulfate attack. In each of these four cases of concrete deterioration, water is implicated in the mechanisms of expansion and cracking. Also, water is the primary vehicle for the diffusion of aggressive ions into the interior of the concrete. Once the water tightness of concrete is lost, the interior of concrete can become saturated. Consequently, water and ions which play an active role in the process of deterioration can now be transported readily into the interior. If the
hydroxyl ions in the cement paste are being leached away and replaced by chloride or sulfate ions, the calcium silicate hydrate will suffer a loss of adhesion and strength.

4.2 *Enhanced durability through use of recyclable by-products*

Increasing the resistance of concrete to penetration of chloride-ion is the “first line of defense” in increasing the service life of concrete structures. The use of fly ash through its combination with calcium, potassium and sodium hydroxides to produce calcium silicate hydrates increases the resistance of concrete to penetration of chloride-ions. Fly ash reduces permeability, thereby reducing access by aggressive chemicals, oxygen, and moisture [Kreck 1997].

When both fly ash and silica fume are used, the synergetic effect of both materials on chloride-ion penetrability of the concrete, in which the silica fume decrease the chloride-ion penetrability at an early age and the fly ash decreases it at later age, results in a concrete with very low chloride-ion penetrability at both 28 and 120 days.

Concrete containing both ground granulated blast furnace slag and silica fume offer particular good resistance to chloride-ions. Silica fume reduces the permeability of the transition zone around the aggregate to particles as well as the permeability of the bulk cement paste [Neville 1997].

One of the most positive effects of slag, fly ash, and silica fume on the chloride-ion penetrability of concrete is mainly due to the fact that the incorporation of such materials in concrete results in finer pores in the hydrated cement paste.

5 STUDY STRATEGY

Research has shown that increasing the cement content of a mix increases the potential for both drying and thermal shrinkage of hardened concrete, leaving the concrete susceptible to equal or greater water ingress. Contrary to conventional thinking, cement content of a mix does not control its durability. Durability depends largely on the properties of the hydrated cement paste and therefore, the cement content of the paste itself is relevant. Keeping the paste content as low as possible and the aggregate content as high as possible was the strategy chosen to reduce both the drying and thermal shrinkage as well as related cracking, while optimizing durability.

5.1 *Gary Brenno*

The late Gary Brenno was the first Minnesota Concrete Council member to understand this strategy. He would have us batch concrete by placing all the well graded aggregate in the mixer first and then we would mix a cementitious slurry separately at a prescribed W/cm. We would then add as much slurry as needed to the mixer to obtain a workable mix. Our goal was to determine our total cementitious materials needed to have a workable mix; thus limiting our total paste yet have the paste used have the most cementitious material as possible per unit volume.

6 EVALUATION PARAMETERS

Given the above considerations, the establishment of the various mixes included in the program, supplying of materials and the batching and testing has required a cooperative effort of our members as well as financial and technical assistance from ARM (Aggregate Ready Mix of Minnesota).

The performance of the chosen mixes being evaluated is based on testing the following properties:

- Initial Time of Set and Drying Shrinkage (ASTM: C403).
- Compressive Strength (ASTM:C39).
- Scaling Resistance (ASTM:C672).
- Finishability Rating
- MCC Permeability Test Method (alternate method).

6.1 *Study controls*

The study is complex considering the number of mix variations. However, to keep the scope of the program as controlled as possible, specific parameters were essentially fixed to best illustrate paste performance with variations in cementitious constituents. For example, the following controls were incorporated into the program:

- All mixes were designed to achieve compressive strength of 21 MPa in 30 hours and 42 MPa in 28 days.
- Recyclable by-product replacement would not exceed 35% fly ash, 35% slag, and 4% silica fume.
- Total cementitious contents would not exceed 298.5 Kg (7 bag mix).
- Maximum W/cm of 0.42
- Aggregate gradations, types, and proportions were the same for all mixes.
- Entrained air content fell within a range of 6% ± 1% W/cm of 0.42.
- Water reducing admixtures were restricted to polycarboxilates.
- Average drying shrinkage in 6 months should be less than 0.05%.
 Rapid chloride permeability shall be less or equal to 1000 coulombs in 6 months.
 Scaling of blended mixes shall perform as well or better than control mixes of Portland cement.

The study was designed to eliminate the normal variations associated with using different operators during the proportioning and batching processes. The same technical staff of one laboratory was used to pre-weigh, containerize the various ingredients, and batch all 19 mixes. This approach continued through the casting of required test specimens as well as recording the time related data of the plastic properties. The mixes used in the program are shown in Table 3 below.

To allow the reader to better track the performance of a specific mix when reviewing the testing results, a key was assigned to the mix identification to portray the type and percentages of cementitious constituents. For example, Mix 17 has been designated as 30S, 1MS, which indicates the mix contains 30% slag, 1% microsilica, and 0% fly ash. Mixes 17-3MS, 20S, 20CA, which indicates the mix contains 3% silica fume, 20% slag, and 20% Type C fly ash, Mixes 1-PC and 8-PC are both for control (at two different cementitious contents) and contain only Portland cement as the cementitious material. The symbol FA indicates Type F fly ash as a cementitious replacement (see Table 3 for a total list of proportions).

### RESULTS

The results listed below (Tables 4–6) are from testing performed on the laboratory batched concrete. These mixture proportions were also placed in two 38.2 cubic meters placements in the fall of 2006 for long term durability analysis.

#### 7.1 Plastic properties

A review of data below indicates the following:

- Slump ranged from 11.4 to 20.3 centimeters, the highest slump occurring in Mix 19-30CA.
- Entrained air content ranged from 5.0 to 7.2% generally within the study control of 6%.

*Batch 18 is the same as Batch 10, except no corrosion inhibitor.
**Batch 19 is the same as Batch 9, except the water cementitious ratio is 0.50.

### Table 3. Mix variations and proportions.

<table>
<thead>
<tr>
<th>Batch no.</th>
<th>% cement</th>
<th>% micro silica</th>
<th>% slag</th>
<th>% fly ash</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. PC</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2. 30S</td>
<td>70</td>
<td>0</td>
<td>30</td>
<td>0</td>
</tr>
<tr>
<td>3. 30FA</td>
<td>70</td>
<td>0</td>
<td>0</td>
<td>30 (F)</td>
</tr>
<tr>
<td>4. 30S, 1MS</td>
<td>69</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5. 30FA, 1MS</td>
<td>69</td>
<td>1</td>
<td>30</td>
<td>30 (F)</td>
</tr>
<tr>
<td>6. 30S, 3MS</td>
<td>67</td>
<td>0</td>
<td>3</td>
<td>0</td>
</tr>
<tr>
<td>7. 30FA, 3MS</td>
<td>67</td>
<td>3</td>
<td>0</td>
<td>30(F)</td>
</tr>
<tr>
<td>8. PC</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9. 30CA</td>
<td>70</td>
<td>0</td>
<td>0</td>
<td>30(C)</td>
</tr>
<tr>
<td>10. 30CA, 1MS</td>
<td>69</td>
<td>1</td>
<td>0</td>
<td>30(C)</td>
</tr>
<tr>
<td>11. 30CA, 3MS</td>
<td>67</td>
<td>3</td>
<td>20</td>
<td>30(C)</td>
</tr>
<tr>
<td>12. 20S, 20FA</td>
<td>60</td>
<td>0</td>
<td>0</td>
<td>30(F)</td>
</tr>
<tr>
<td>13. 20S, 20CA</td>
<td>60</td>
<td>0</td>
<td>0</td>
<td>30(C)</td>
</tr>
<tr>
<td>14. 1MS, 20S, 20FA</td>
<td>59</td>
<td>1</td>
<td>20</td>
<td>30(F)</td>
</tr>
<tr>
<td>15. 1MS, 20S, 20CA</td>
<td>59</td>
<td>1</td>
<td>20</td>
<td>30(C)</td>
</tr>
<tr>
<td>16. 3MS, 20S, 20FA</td>
<td>57</td>
<td>3</td>
<td>20</td>
<td>30(F)</td>
</tr>
<tr>
<td>17. 3MS, 20S, 20CA</td>
<td>57</td>
<td>3</td>
<td>20</td>
<td>30(C)</td>
</tr>
<tr>
<td>18. *1MS, 30CA</td>
<td>69</td>
<td>1</td>
<td>0</td>
<td>30(C)</td>
</tr>
<tr>
<td>19. **30CA</td>
<td>70</td>
<td>0</td>
<td>0</td>
<td>30(C)</td>
</tr>
</tbody>
</table>

*Batch 18 is the same as Batch 10, except no corrosion inhibitor.
**Batch 19 is the same as Batch 9, except the water cementitious ratio is 0.50.

<table>
<thead>
<tr>
<th>Batch no.</th>
<th>Test results</th>
<th>1-PC</th>
<th>2-30S</th>
<th>3-30FA</th>
<th>4-30S, 1MS</th>
<th>5-30FA, 1MS</th>
<th>6-30S, 3MS</th>
<th>7-30FA, 3MS</th>
<th>8-PC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slump (cm)</td>
<td>12.7</td>
<td>16.5</td>
<td>19.1</td>
<td>17.8</td>
<td>17.8</td>
<td>12.7</td>
<td>15.2</td>
<td>17.8</td>
<td></td>
</tr>
<tr>
<td>Air content (%)</td>
<td>6.2</td>
<td>5.0</td>
<td>5.1</td>
<td>6.0</td>
<td>7.2</td>
<td>5.4</td>
<td>7.2</td>
<td>6.4</td>
<td></td>
</tr>
<tr>
<td>Unit wt (Kg/m³)</td>
<td>2,351</td>
<td>2,358</td>
<td>2,351</td>
<td>2,340</td>
<td>2,337</td>
<td>2,355</td>
<td>2,345</td>
<td>2,338</td>
<td></td>
</tr>
<tr>
<td>Initial set (hr:min)</td>
<td>3:33</td>
<td>4:30</td>
<td>4:45</td>
<td>4:50</td>
<td>4:47</td>
<td>4:50</td>
<td>5:15</td>
<td>4:00</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Batch no.</th>
<th>Test results</th>
<th>9-30CA</th>
<th>10-30CA, 1MS</th>
<th>11-30CA, 3MS</th>
<th>12-20S, 20FA</th>
<th>13-20S, 20CA</th>
<th>14-1MS, 20S, 20FA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slump (cm)</td>
<td>16.5</td>
<td>16.5</td>
<td>12.7</td>
<td>19.1</td>
<td>17.8</td>
<td>17.8</td>
<td>17.8</td>
</tr>
<tr>
<td>Air content (%)</td>
<td>5.0</td>
<td>6.2</td>
<td>5.0</td>
<td>5.0</td>
<td>6.5</td>
<td>5.1</td>
<td>5.4</td>
</tr>
<tr>
<td>Unit wt (Kg/m³)</td>
<td>2,359</td>
<td>2,351</td>
<td>2,363</td>
<td>2,338</td>
<td>2,355</td>
<td>2,355</td>
<td>2,358</td>
</tr>
<tr>
<td>Initial set (hr:min)</td>
<td>4:20</td>
<td>4:30</td>
<td>4:40</td>
<td>4:40</td>
<td>4:37</td>
<td>4:05</td>
<td>4:41</td>
</tr>
</tbody>
</table>
Given control of the aggregate volume and the cementitious content, the unit weights were found to be relatively consistent varying from 2336 to 2367 Kg/m³.

Table 6. Plastic properties.

<table>
<thead>
<tr>
<th>Test results</th>
<th>15-1MS, 20S, 20CA</th>
<th>16-3MS, 20S, 20FA</th>
<th>17-3MS, 20S, 20CA</th>
<th>18-1MS, 30CA</th>
<th>19-30CA**</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slump (cm)</td>
<td>12.7</td>
<td>17.8</td>
<td>12.7</td>
<td>11.4</td>
<td>20.3</td>
</tr>
<tr>
<td>Air content (%)</td>
<td>6.0</td>
<td>6.2</td>
<td>7.1</td>
<td>5.0</td>
<td>5.5</td>
</tr>
<tr>
<td>Unit wt (Kg/m³)</td>
<td>2,345</td>
<td>2,351</td>
<td>2,340</td>
<td>2,359</td>
<td>2,351</td>
</tr>
<tr>
<td>Initial set (hr:min)</td>
<td>4:10</td>
<td>4:45</td>
<td>4:14</td>
<td>5:01</td>
<td>4:49</td>
</tr>
</tbody>
</table>

Figure 1. Time of set.

Figure 2. Time of set.

7.2 Time of set

A review of data (Figures 1 and 2) below indicates the addition of recyclable by-products replacing Portland cement extends the setting time. The two mix propor-
tions that had the quickest initial set times were the controls that were 100% Portland cement. Generally, the higher percentages of Portland cement replacement resulted in the longest setting times.

7.3 Compressive strength

The one day compressive strength data is similar to the initial set time data (see Figures 3 and 4). The control mixes with 100% Portland cement had the highest one-day strength. The mixes with the highest cement replacement had the lowest one-day compressive strength. The inverse is true at 28 days with the control mixes having the lowest compressive strengths. The mixes with the highest cement replacement had the highest 28-day compressive strength.

7.4 Rapid chloride permeability

The rapid chloride permeability test (RCP) was created in the early 1980’s and was adopted by both AASHTO
T 277 and ASTM:C1202. In the RCP test, saturated 5.08 cm thick by 10.16 cm diameter concrete discs are prepared and subjected for six hours to direct electric current having a potential difference of 60 volts. Based on experimental evidence, a low passage of charge in this test, on the whole, indicates low chloride permeability, and a high charge passage indicates high chloride permeability.

A review of the RCP test results reveals that the mixes with the highest cement replacement had the lowest passage of charge which indicates low chloride permeability (see Table 7).

7.4.1 Altered electrical properties effect on RCP test results

Some concretes can have altered electrical properties and can give false readings. When we compared Batch 18 to Batch 10 in our study, Batch 10 passed twice the charge as Batch 18. The tests suggest the difference in the RCP test values results from the inclusion of a corrosion inhibitor in Batch 18. The additional ions contributed by the corrosion inhibitor increases the passage of electrical charges during the test.

Another example of altered electrical properties which can give false readings is the use of pozzolans that tie up normally free ions within the concrete pours. The reduced ion concentration results in less charge being passed. This is not to say that the concrete with pozzolans have lower chloride permeability, but may not be as low as the rapid test indicates.

7.5 Drying shrinkage

Among the more important factors influencing the drying shrinkage of concrete are the content of cement paste and its quality (that is, W/cm ratio and degree of hydration). A review of Figures 5–8 reveals that the mixes with the highest cement replacement had the least amount of shrinkage between four months and nine months. The pozzolanic materials increase the degree of hydration in the long term. This can be seen by comparing the control mixes Set #1, Figure 13 and Set #8, Figure 14 to the remainder of the sets. The controls continue to shrink at approximately the same rate whereas the mixes with pozzolans flatten out.

7.6 Scaling resistance

This test method covers the determination of the resistance to scaling of a horizontal concrete surface exposed to freeze-thaw cycles in the presence of deicing chemicals (ASTM:C672). Essentially, the test calls for casting samples, then placing a one-inch high dike around the perimeter of the specimen in order to pond a .64 cm deep solution of calcium chloride and water. The specimens are then subjected

Table 7. RCP testing.

<table>
<thead>
<tr>
<th>Batch no.</th>
<th>% cement</th>
<th>% micro silica</th>
<th>% slag</th>
<th>% fly ash</th>
<th>56 days</th>
<th>6 months</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. PC</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1281</td>
<td></td>
</tr>
<tr>
<td>2. 30S</td>
<td>70</td>
<td>0</td>
<td>30</td>
<td>0</td>
<td>1066</td>
<td></td>
</tr>
<tr>
<td>3. 30FA</td>
<td>70</td>
<td>0</td>
<td>30</td>
<td>0</td>
<td>366</td>
<td></td>
</tr>
<tr>
<td>4. 30S, 1MS</td>
<td>69</td>
<td>1</td>
<td>0</td>
<td>30 (F)</td>
<td>606</td>
<td></td>
</tr>
<tr>
<td>5. 30FA, 1MS</td>
<td>69</td>
<td>1</td>
<td>0</td>
<td>30 (F)</td>
<td>269</td>
<td></td>
</tr>
<tr>
<td>6. 30S, 3MS</td>
<td>67</td>
<td>3</td>
<td>0</td>
<td>30 (F)</td>
<td>620</td>
<td></td>
</tr>
<tr>
<td>7. 30FA, 3MS</td>
<td>67</td>
<td>3</td>
<td>0</td>
<td>30 (F)</td>
<td>305</td>
<td></td>
</tr>
<tr>
<td>8. PC</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1427</td>
<td></td>
</tr>
<tr>
<td>9. 30CA</td>
<td>70</td>
<td>0</td>
<td>0</td>
<td>30 (C)</td>
<td>1869</td>
<td>590</td>
</tr>
<tr>
<td>10. 30CA, 1MS</td>
<td>69</td>
<td>1</td>
<td>0</td>
<td>30 (C)</td>
<td>2519</td>
<td>955</td>
</tr>
<tr>
<td>11. 30CA, 3MS</td>
<td>67</td>
<td>3</td>
<td>0</td>
<td>30 (C)</td>
<td>1952</td>
<td>876</td>
</tr>
<tr>
<td>12. 20S, 20FA</td>
<td>60</td>
<td>0</td>
<td>20</td>
<td>20 (F)</td>
<td>1034</td>
<td>386</td>
</tr>
<tr>
<td>13. 20S, 20CA</td>
<td>60</td>
<td>0</td>
<td>20</td>
<td>20 (C)</td>
<td>986</td>
<td>347</td>
</tr>
<tr>
<td>14. 1MS, 20S, 20FA</td>
<td>59</td>
<td>1</td>
<td>20</td>
<td>20 (F)</td>
<td>707</td>
<td>292</td>
</tr>
<tr>
<td>15. 1MS, 20S, 20CA</td>
<td>59</td>
<td>1</td>
<td>20</td>
<td>20 (C)</td>
<td>1046</td>
<td>445</td>
</tr>
<tr>
<td>16. 3MS, 20S, 20FA</td>
<td>57</td>
<td>3</td>
<td>20</td>
<td>20 (F)</td>
<td>717</td>
<td>263</td>
</tr>
<tr>
<td>17. 3MS, 20S, 20CA</td>
<td>57</td>
<td>3</td>
<td>20</td>
<td>20 (C)</td>
<td>732</td>
<td>313</td>
</tr>
<tr>
<td>18. *1MS, 30CA</td>
<td>69</td>
<td>1</td>
<td>0</td>
<td>30 (C)</td>
<td>1433</td>
<td>538</td>
</tr>
<tr>
<td>19. **30CA</td>
<td>70</td>
<td>0</td>
<td>0</td>
<td>30 (C)</td>
<td>2624</td>
<td>893</td>
</tr>
</tbody>
</table>

* Batch 18 is the same as Batch 10, except no corrosion inhibitor.
** Batch 19 is the same as Batch 8, except the water cementitious ratio is 0.50.
Figure 5. ASTM:C157 drying shrinkage, sets 1–5.

Figure 6. ASTM:C157 drying shrinkage, sets 6–10.
Figure 7. ASTM:C157 drying shrinkage, sets 11–15.

Figure 8. ASTM:C157 drying shrinkage, sets 16–19.
to 50 freeze-thaw cycles. The rating system is as follows:

0 No scaling.
1 Very light scaling (.32 cm depth maximum, no coarse aggregate visible).
2 Slight to moderate scaling.
3 Moderate scaling (some coarse aggregate visible).
4 Moderate to severe scaling.
5 Severe scaling (coarse aggregate visible).

A review of the scaling data in Table 9 reveals that the mixes with the highest cement replacement were most resistant to scaling. The only panels with no scaling were Batches 14 and 15 which contained 1% silica fume, 20% slag, 20% fly ash, and 59% Portland cement.

8 CONCLUSIONS

Our understanding of using recyclable by-products has grown since the days of “new material neurosis.” Originally finding a place for waste products was the driving force. The high compressive strength requirement at the end of last century and the current interest in rehabilitation our infrastructure with durable concrete is the new impetus for using recyclable by-products. Silica fume, fly ash, and slag enhance both the compressive strength and durability characteristics of concrete.

The most promising aspect of utilizing recyclable by-products as cementitious material is that not only will we be responsible stewards of this planet, we will also be producing a more durable concrete.

REFERENCES

Kreck, R., Riggs, E. 1997 Specifying Fly Ash for Durable Concrete. Concrete International – April 1997, Vol. 19, No. 4
Effects of cement and mineral admixture on migration of chloride ions and generation of hydration heat in concrete

Jaekang Yoo, Sangjoon Park & Bohwan Oh
Daewoo Institute of Construction Technology, Suwon, Gyeonggi-do, South Korea

ABSTRACT: This paper describes chloride migration coefficient and hydration heat generation properties of concrete composed of various types of cement and mineral admixture with the water-cementitious materials ratio ranging from 0.30 to 0.40. Slag cement and low heat cement were effective in reducing the hydration heat compared with normal portland cement but the low heat cement had negative influence on the chloride migration property. As for the mineral admixtures, fly ash was effective in reducing the hydration heat, and both silica fume and meta kaolin were able to increase the compressive strength and reduce the chloride migration coefficient significantly. The experimental results showed that the concrete composed of three kinds of binder (normal portland cement + 20% fly ash + 10% silica fume) led to the reduction of the maximum temperature from 37 ~ 43°C to 33 ~ 35°C and the chloride migration coefficient from $9.1 \times 10^{-12}$ m²/s to $2.0 \sim 3.8 \times 10^{-12}$ m²/s as compared with the plain concrete (W/Cm 0.30 ~ 0.40).

1 INTRODUCTION

The chloride ions penetrate and/or diffuse into concrete matrix under salt laden environment. The ingress of chloride ions into concrete induces corrosion of reinforcing bar and the corrosion causes the premature deterioration of reinforced concrete structures. The transport property of aggressive substances is determined by the tightness of concrete formed by mix condition, material, and construction. Durable structure is made from dense concrete by increasing the resistance against chloride invasion and restricting harmful cracks through material and constructional measures [Boulfiza et al. 2003].

Figure 1 demonstrates the methods of enhancing durability against chloride attack in reinforced concrete structures [Yoo et al. 2003]. The methods can be classified in two categories. The first is the restriction method against chloride invasion and the second is the protection method of reinforcing bar against corrosion. Many regulations suggest the guideline of mix condition (water-cement ratio or unit cement content) and the minimum compressive strength for the concrete exposed to chloride attack environments [ACI 318 1995, KCSS 2003]. This method can be classified as the restriction method against chloride invasion by enhancing the density of concrete. In addition, the use of some pozzolans has a positive influence on the durability by decreasing the permeability of concrete as low as 1/10th or 1/100th of that of comparable concrete of equal strength made without pozzolans [ACI 201 1997]. Choosing the optimum cementing material system is a key question when designing a concrete structure to last for a long time in a chloride environment.

This paper investigates the effects of cement type and various compositions of cementitious materials (cements and mineral admixtures) on the property of hydration heat generation, compressive strength, and chloride migration coefficient of concrete. It can be used as a basic data to reduce the hydration heat and enhance the resistance properties of massive coastal reinforced concrete structures against chloride invasion.

Coat the concrete surface
Densify the cover concrete
Anti-corrosion agent
Coat the steel re-bar
Sacrificial anode
Non-steel material reinforcing

Figure 1. Methods of enhancing durability against chloride attack in reinforced concrete structures.
2 EXPERIMENTAL PROGRAM

2.1 Experimental plan

As shown in Table 1, the experiment plan of this study includes three series.

Three types of cement (normal portland cement (NPC), low heat cement (LHC), and Ground granulated blast-furnace slag cement (SLC)) were used with the water-cement ratio (W/C) ranging from 0.30 to 0.40 in the series I.

Series II comprises 54 mix conditions with the three types of cement used in series I and three kinds of mineral admixtures (fly ash (FA), silica fume (SF), and meta kaolin (MK)). FA was replaced with the ratio of 0% (plain), 10%, and 20%, and SF and MK were replaced with 0%, 5%, and, 10%, respectively.

Series III includes 36 mix conditions composed on the basis of NPC and three kinds of mineral admixture. For each mixture, two kinds of mineral admixture were replaced with NPC with the composition of NPC + admixture 1 + admixture 2.

Target slump flow of the concrete mixed with water-cementitious materials ratio (W/Cm) of 0.30 and 0.35 was 500 ± 50 mm, and target slump of the concrete mixed with W/Cm of 0.40 was 180 ± 25 mm. Target air content of all mixtures was maintained as 4.5 ± 1.5%.

2.2 Materials

The physical properties of cement used in this study are listed in Table 2. SLC was made from 55% of NPC and 45% of ground granulated blast-furnace slag powder.

Table 3 shows physical and chemical properties of the mineral admixtures used in this study and Figure 2 shows the chemical composition of FA, SF, and MK. The main component of all mineral admixtures is SiO₂, which holds up to 90% of SF composition. MK has similar chemical composition as in FA, which mainly consists of SiO₂ and Al₂O₃, and the blaine is approximately equivalent to 11,000 cm²/g.

Table 4 shows the physical properties of sand, gravel, and super-plasticizer used in this study.
2.3 Measurements

2.3.1 Hydration heat

To measure hydration heat, insulated box (foamed polystyrene box) was filled with fresh concrete and thermocouple was set up in the center of specimen. The temperature was measured with data logger for a week in a laboratory of constant temperature and humidity (20°C, RH 50%). (Figure 3)

2.3.2 Electrically accelerated chloride migration test [NT-Build 492 1992]

The specimen for electrically accelerated chloride migration test has a thickness of 50 ± 2 mm and a diameter of 100 mm (Figure 4). The specimen is sliced from a usual cylindrical specimen after curing and is set on a silicon rubber sleeve with a clamp.

After setting on the chloride migration test set-up (Figure 5), cathode side of the specimen is in contact with 10% NaCl solution and anode side is filled with 0.3 N NaOH solution. Stainless steel plate is set on cathode side and stainless steel mesh is set on anode side of specimen.

Table 3. Physical and chemical properties of mineral admixtures.

<table>
<thead>
<tr>
<th>Mineral admixture</th>
<th>Mark</th>
<th>Kind</th>
<th>Specific gravity (cm²/g)</th>
<th>Blaine (g/cm²)</th>
<th>Ignition loss (%)</th>
<th>Chemical composition (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FA</td>
<td>Fly ash</td>
<td>2.27</td>
<td>3157</td>
<td></td>
<td>3.57</td>
<td>SiO₂ 56.4 Fe₂O₃ 9.0 CaO 2.5 MgO 1.3 SO₃ 1.1</td>
</tr>
<tr>
<td>SF</td>
<td>Silica fume</td>
<td>2.24</td>
<td>228,000</td>
<td></td>
<td>0.27</td>
<td>Al₂O₃ 23.7 Fe₂O₃ 9.0 CaO 0.7 MgO 0.3 SO₃ 2.1</td>
</tr>
<tr>
<td>MK</td>
<td>Meta kaolin</td>
<td>2.55</td>
<td>11,000</td>
<td></td>
<td>2.10</td>
<td>SiO₂ 52.0 Al₂O₃ 40.0 CaO 1.2 MgO 0.5 –</td>
</tr>
</tbody>
</table>

* Test was performed at an accredited testing center and the Blaine of MK is an average value.

Table 4. Physical properties of materials.

<table>
<thead>
<tr>
<th>Material</th>
<th>Kind</th>
<th>Specific gravity</th>
<th>Fineness modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>Washed sand</td>
<td>2.60</td>
<td>2.60</td>
</tr>
<tr>
<td>Gravel</td>
<td>Crushed stone (25 mm)</td>
<td>2.63</td>
<td>6.96</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>Poly carboxylic acid</td>
<td>1.20 ± 0.02</td>
<td>–</td>
</tr>
</tbody>
</table>

Figure 2. Chemical compositions of mineral admixtures.

Figure 3. Simplified test for measuring hydration temperature with insulated box and data logger.

Figure 6 shows the test set-up corresponding to the test method.

Because the migration property of chloride ion depends on the density of concrete, the test method suggests the readjustment of applying voltage and the determination of test duration based on the initial current value under the application of 30 V.

Non-steady-state chloride migration coefficient can be calculated from the average temperature of anolyte solution, the penetration depth of chloride ion which appears by 0.1 M silver nitrate solution spraying, and the test duration (Equation 1).
\[ D = \frac{RT}{zFE} \cdot \frac{x_d - a \sqrt{x_d}}{t} \]

Where, \[ E = \frac{U - 2}{L} \], \[ a = 2 \sqrt{\frac{RT}{zFE} \cdot erf^{-1} \left( 1 - \frac{2c_d}{c_0} \right)} \]

- \( D \): Non-steady-state migration coefficient (\( m^2/s \))
- \( z \): Absolute value of ion valance, for chloride, \( z = 1 \)
- \( F \): Faraday constant, \( F = 96,481.04 \text{ J/V} \cdot \text{mol} \)
- \( U \): Absolute value of the applied voltage (V)
- \( R \): Gas constant, \( R = 8.314 \text{ J/K} \cdot \text{mol} \)
- \( T \): Average temperature in the anolyte solution (K)
- \( L \): Thickness of the specimen (m)
- \( x_d \): Average value of the penetration depths (m)
- \( t \): Test duration (sec)
- \( erf \): Error function
- \( c_d \): Cl concentration at the color changes (\( c_d \approx 0.07 \text{ N} \))
- \( c_0 \): Cl concentration in the catholyte solution (\( c_0 \approx 2 \text{ N} \))

### TEST RESULTS

#### 3.1 Effects of types of cement (Series I)

##### 3.1.1 Hydration heat

Figure 7 demonstrates hydration heat curves of the mixtures with various types of cement. The temperature rising velocities and the maximum temperatures are different for each type of cement.

In case of NPC, the temperature increased rapidly after 7 hours and reached 40.8°C after 18 hours. After 20 hours, the temperature decreased rapidly. However, the temperature of LHC and SLC increased slowly.
compared with NPC, and the maximum temperature of LHC was 33.6°C after 21 hours and that of SLC was 32.8°C after 24 hours. It is noted that the temperature of SLC decreased slowly compared with NPC and LHC.

It seems that the tendency of the hydration heat generation of each type of cement was caused by the chemical composition. Compared with NPC, LHC is usually manufactured in order to restrict the hydration heat by decreasing the quantity of C3S (3CaO·SiO2) and increasing the quantity of C2S (2CaO·SiO2). Therefore, the increased C2S content and the decreased C3S content may have caused the reduction of the maximum temperature and the retardation of the temperature rising velocity.

Because the SLC was manufactured from 55% of NPC and 45% of ground granulated blast-furnace slag powder, it seems that the reduction mechanism of hydration heat was similar to the case of LHC.

Thus, it can be assumed that the use of LHC or SLC is more effective in reducing the hydration heat than the use of NPC for massive structures.

### 3.1.2 Compressive strength

Figure 8 demonstrates the compressive strength for different types of cement.

The compressive strength of LHC was lower than that of NPC for all W/C. For the same W/C conditions, the compressive strength of LHC was 50 ~ 93% of that of NPC. It seems that the trend of compressive strength was caused by the chemical composition of LHC.

Besides, the compressive strength of the SLC with 0.30 W/C was lower than that of NPC at 3 days but it exceeded that of NPC at 7 days. In case of the mixture with 0.35 W/C, the compressive strength of SLC was similar to NPC, and the mixture with 0.40 W/C has shown lower compressive strength than that of NPC at all ages.

It is clear that the strength development property of concrete mixed with SLC depends on the W/C and curing ages.

### 3.1.3 Chloride migration coefficient

The chloride migration coefficient of concrete mixed with various types of cement is shown in Figure 9.

As the W/C increased, the chloride migration coefficient of LHC increased rapidly compared with those of NPC and SLC. The chloride migration coefficient of LHC was $27.0 \sim 51.5 \times 10^{-12} \text{ m}^2/\text{s}$ at 7 days and $15.7 \sim 27.5 \times 10^{-12} \text{ m}^2/\text{s}$ at 28 days but those of NPC and SLC were $9.1 \sim 15.0 \times 10^{-12} \text{ m}^2/\text{s}$ and $2.3 \sim 8.9 \times 10^{-12} \text{ m}^2/\text{s}$, respectively.

SLC shows 26 ~ 66% of chloride migration coefficient compared with that of NPC and 12 ~ 22% of that of LHC. This result is similar to the report of prior study: “Concrete containing ground granulated blast-furnace slag powder shows absorption of chloride ions, and the absorption causes the increase of resistance against chloride intrusion” [Sakurata & Maruyama 1999; Maruya 2002; Takegami 2002].

Thus, it is assumed that the cement containing ground granulated blast-furnace slag is effective in reducing the chloride migration coefficient and enhancing the resistance against chloride intrusion.

### 3.2 Effects of kinds of mineral admixture (Series II)

To evaluate the effect of mineral admixtures and their replacement ratio on the properties of hydration heat generation and chloride migration coefficient, fly ash, silica fume, and meta kaolin were replaced with NPC, LHC, and SLC, respectively.

Fly ash was replaced in the range from 0% to 20%, silica fume and meta kaolin were replaced with NPC, LHC, and SLC, respectively.

Silica fume and meta kaolin were replaced in the range from 0% to 20%, silica fume and meta kaolin were replaced in the range from 0 to 10%.
Target slump flow and air content of fresh concrete, the test items were the same as series I.

### 3.2.1 Hydration heat
Figure 10 shows the maximum temperature of the concrete replaced with mineral admixtures.

The maximum temperature decreased when W/Cm decreased and replacement ratio of mineral admixture increased.

By replacing 20% of cement content with fly ash, the maximum temperature decreased 10% compared with plain mixture without mineral admixtures.

It is clear that the maximum temperature was influenced by mix conditions such as W/Cm, type of cement, kind of mineral admixture and their replacement ratio.

### 3.2.2 Compressive strength
Figure 11 demonstrates the compressive strength of concrete replaced with mineral admixtures.

In case of the concrete replaced with fly ash, the compressive strength decreased as the replacement ratio increased. However, the compressive strength of the concrete replaced with silica fume or meta kaolin increased with the replacement ratio.

The compressive strength of concrete mixed with NPC was more influenced by the use of mineral admixture than those of concrete mixed with LHC and SLC, respectively.

### 3.2.3 Chloride migration coefficient
Figure 12 shows the chloride migration coefficient of concrete replaced with mineral admixtures.

The chloride migration coefficient of concrete shows a great difference according to the W/Cm, types of cement, kinds of mineral admixture and their replacement ratios.

The chloride migration coefficient of NPC was efficiently reduced from $9.1 \sim 11.7 \times 10^{-12} \text{m}^2/\text{s}$ to $2.0 \sim 3.0 \times 10^{-12} \text{m}^2/\text{s}$ by replacing 10% of cement...
content with silica fume. In case of LHC, the chloride migration coefficient was \(1.7 \sim 5.8\) times and \(5.4 \sim 11.0\) times larger than those of NPC and SLC, respectively. The chloride migration coefficient of the LHC replaced with 5\% of silica fume showed similar as that of the plain mixture of NPC.

Besides, all mixtures of SLC showed the chloride migration coefficient below \(3.5 \times 10^{-12}\) m\(^2\)/s at 28 days irrespective of kinds of mineral admixture and their replacement ratios.

It is clear that the chloride migration coefficient of the concrete mixed with SLC and mineral admixture was more influenced by the use of SLC than the replacement of mineral admixture.

3.3 Effects of ternary mix composition (Series III)

The ternary mix composition was manufactured by replacing NPC with two kinds of mineral admixture among fly ash, silica fume, and meta kaolin.

Because the LHC was not effective in reducing chloride migration coefficient, LHC was excluded in series III.

SLC was also excluded in series III because it is not necessary for the mixture of SLC to reduce hydration heat and chloride migration coefficient by replacing the cement content with mineral admixtures.

For the cement type, NPC was used in series III.

3.3.1 Hydration heat

Figure 13 demonstrates the maximum temperatures of the mixtures composed of three kinds of binder, where the maximum temperature decreased as W/Cm and replacement ratio of mineral admixture increased.

Compared with the maximum temperature of the plain concrete in series II, the temperature can be reduced \(4.3 \sim 8.3\)°C by replacing the cement content with two kinds of mineral admixture.

The maximum temperatures were clearly related with the unit cement content excluding the contents of mineral admixtures (Figure 14).

It will be possible to restrict the generation of hydration heat by reducing unit cement content of concrete for coastal reinforced concrete structures in the boundary of the designed compressive strength and the target chloride migration coefficient.

3.3.2 Compressive strength

Figure 15 demonstrates the compressive strength of concrete composed NPC and two kinds of mineral admixture at 28 days.

The compressive strength decreased as the fly ash replacement ratio increased. The compressive strength of the concrete replaced 20\% of cement content with fly ash showed \(89 \sim 95\%\) of that of the concrete replaced 10\% of cement content with fly ash.

However, in case of silica fume and meta kaolin, the compressive strength has shown increasing tendency with the replacement ratio. For the same W/Cm and fly ash replacement ratio condition, the compressive strength of the concrete mixed with silica fume was 10\% higher than that of the concrete mixed with meta kaolin.

3.3.3 Chloride migration coefficient

Figure 16 shows the chloride migration coefficient of the mixture composed of three kinds of binder.

The concrete containing fly ash and meta kaolin has shown high degrees of chloride migration coefficient compared with the concrete containing silica fume. By replacing the cement content with silica fume, the chloride migration coefficient of the present study could be reduced below \(6.0 \times 10^{-12}\) m\(^2\)/s.
The difference of chloride migration coefficient according to the W/Cm was remarkable in the mixture without silica fume but the difference decreased as the replacement ratio of silica fume increased.

It seems that the chloride migration coefficient of the mixture composed of three kinds of binder was influenced by W/Cm, kinds of mineral admixture, and their replacement ratios.

3.4 Relationship between compressive strength and chloride migration coefficient

The relationship between compressive strength and chloride migration coefficient of this study is shown in Figure 17. The chloride migration coefficient decreased as the compressive strength increased.
The chloride migration coefficients of the concrete for the compressive strength 40～60 MPa differ greatly according to the types of cement. The magnitude of chloride migration coefficient according to the types of cement has shown in the order of LHC > NPC > SLC.

Besides, it seems that the deviation of NPC was caused by the replacing the cement content with mineral admixtures used in the series II and series III.

4 CONCLUSIONS

The following conclusions can be made based on the test results of this study:

1. The LHC and SLC were effective in reducing the maximum temperature and the temperature rising velocity of concrete compared with the NPC. The chloride migration coefficient of SLC has shown 26～66% and 12～22% of those of NPC and LHC, respectively.

2. The maximum temperature decreased as the replacement ratio of mineral admixture increased and it was influenced by the mix condition such as W/Cm, types of cement, kinds of mineral admixture, and their replacement ratios. By replacing the NPC with 10% of silica fume, the chloride migration coefficient decreased from 9.1～11.7 × 10^{-12} m^2/s to 2.0～3.8 × 10^{-12} m^2/s compared with the plain.

3. LHC was effective in reducing the generation of hydration heat, whereas the chloride migration coefficient was 1.7～5.8 times larger than NPC and 5.4～11.0 times larger than SLC despite of replacing LHC with mineral admixtures.

4. The maximum temperature of the concrete composed three kinds of binder was related with the unit cement content excluding the mineral admixture.

5. The concrete composed of three kinds of binder (NPC + 20% of fly ash + 10% of silica fume) shows the reduction of the maximum temperature from 37～43°C to 33～35°C and the chloride migration coefficient from 9.1～11.7 × 10^{-12} m^2/s to 2.0～3.8 × 10^{-12} m^2/s as compared with the plain concrete (W/Cm 0.30～0.40).

6. The chloride migration coefficient and the compressive strength have a negative relationship, and the tendency was different for each type of cement.

ACKNOWLEDGMENTS

The work presented in this paper was funded by Center for Concrete Corea (05-CCT-D11), supported by Korea Institute of Construction and Transportation Technology Evaluation and Planning (KICTTEP) under Ministry of Construction and Transportation (MOCT).

REFERENCES

ACI 318 1995. Requirements for structural concrete and commentary (Part 3 – Construction requirements, durability requirements), American Concrete Institute: 35–37.


Boulfiza, M., Sakai, K., Banthia, N., & Yoshida, H. 2003. Prediction of chloride ions ingress in uncracked and


NT-BUILD 492 1999. Concrete, mortar and cement-based repair materials – chloride migration coefficient from non-steady-state migration experiments, NORDTEST.


Hemp as a filler of cement-bonded particleboards

J. Bydžovský & F. Khestl
Brno University of Technology, Faculty of Civil Engineering, Institute of Technology of Building Materials and Components, Veveří 331/95, 602 00, Brno, Czech Republic

ABSTRACT: This paper deals with possibilities of partial supplying of primary components in original prescription of wood-cement boards by secondary raw materials. That will achieve a decrease in production loads, modification of building board characteristics and, last but not least, a decrease in ecological damage. Unfortunately, not every fast renewable or secondary raw material could be used as filler in cement bonded building materials. Alternate aggregates must not noticeably affect adversely interaction with cement or any other used component at hydration. One of the potential alternatives is hemp.

1 INTRODUCTION

The cement-bonded particleboards can be applied in system buildings of all kind, they are ideal for dry method of construction, for buildings in demanding climatic conditions and in areas, where the other advantageous properties of this building material can be used. The boards are composed from wooden material, cement, water and hydration admixtures.

Today, we have a lot of experiences with production of these cement-bonded particleboards; nevertheless, like for every material and product, there are also higher and specific requirements. Besides, environmental strain becomes to be more and more noticeable, the load of wastes begins to be unacceptable and the world society is (at last) becoming conscious of this fact. Considering these trends is only natural that also building industry begins with using secondary raw materials in a higher degree. Usage of secondary raw and fast renewable materials in building materials and products happens not only to decrease environmental strain, but largely for economic merit as well as for some secondary raw material characteristics, which are favorable to some material (product) resulting characteristics.

The wooden material for cement bonded wood composites is composed of spruce, pine or fir timbers, which are decorticated. These wood types are preferred for the better interconnection with bonding material and qualities influencing hydration cementing compound. Sugars present in the wood are working as cement hydration inhibitor, which can lead to incomplete hydration thereby deteriorate final product quality.

For better characteristics chemicals are used, such as CaCl₂ for accelerating cement hydration. Wood must be inhibited against pests and decomposition, and also its absorptivity must be reduced; it can be reached for e.g. by using water-glass. As a binder there is most often used Portland cement CEM I 42.5. The major component, which forms filler in cement bonded wood composites is wood. Wooden particles are gained from timbers, which means that it is necessary to produce them first. Application of sundry alternative fillers may partly or even fully exclude this part of production. This will achieve lower costs and at the same time it contributes to – by smaller consumption of wooden particles – to decrease logging; wood is also a renewable raw material, but with lower growth rate.

Unfortunately, not every fast renewable or secondary raw material could be used as filler in cement bonded building materials. Alternate aggregates must not noticeably affect adversely interaction with cement or any other used component at hydration. One of alternatives that might meet price and technological requirements is substitution of wooden particles by hemp. Hemp (Cannabis sativa) is used as a full or partial substitute for wood particles. Fundamental contribution of this application is renewable raw materials utilization, at contemporary achievement of specific material characteristics and their economic preferences.

As it mentioned above, basic and the most important aim in developing new materials is reaching the lowest production costs at keeping the same or better material characteristics.

Cement-bonded particleboards can be improved by several technological ways. We can divide the whole components of boards into: filler – wooden material, bonding component – cement, water and hydration admixtures. On the basis of this listing we can say, that
we have only three possibilities to improve board characteristics. We can either modify bonding components, filler or both of them. In this paper there are described some problems with partial supplying of primary components in original prescription of wood-cement boards by fast renewable material – hemp (as a filler).

2 HEMP

2.1 Hemp introduction

*Cannabis sativa*, also known as hemp is a dioecious herb of the Hemp family – Cannabaceae – that grows up to 3.7 m in height. The leaves are palmately divided into 3 – 9 narrow, toothed segments, most about 7–15 cm long. The stems are rough and scabrous and the inner bark is fibrous. Hemp is cultivated in almost everywhere but native from Asia. Hemp has become notorious during recent years as an illicit source of narcotic drugs. In fact, the species is versatile and have the wide variety of possible uses. Its seeds, stalks, flowers and oils can be used in wide range of industry, like agriculture, automotive industry, cosmetics, building industry, furniture, paper, textile, food, recycling, etc.

It is excellent alternative to wooden material for building material; moreover, in some aspects its qualities even surpass the wood characteristics.

Expressive development of hemp cultivation for technical purposes in the Czech Republic began the period between 1999 and 2000. Because of this reason this raw material has been only sporadically and unsystematically exploited, nevertheless external experiences demonstrate its perspectivity. Clear and evident benefits can be seen in fast renewability; withal in comparison with other similar organic materials attain interesting mechanical qualities, usable in building materials.

3 HEMP UTILIZATION IN CEMENT BONDED PARTICLEBOARDS

For the first time a producer delivered grounded raw hemp. Hemp in this conditions couldn’t be used, for further application to modified particleboards it had to be processed and sifted to separate fractions of hemp.

3.1 Sieve analysis of treated hemp particles

3.1.1 Hemp utilization in cement matrix

For hemp utilization as a filler to cement bonded particleboards interaction with bonding component
(which is cement in this case) is very important. The hemp may not impact hydration of cement components which secure compactness of this composite. There is no unified method to prescribe hemp usability as a filler to cement bonded boards. There was created our own method in this purpose.

Leach was made from wood particles and then from hemp for experiments. For comparison there was used common water from water-supply system. To obtain the leach we left to infuse wood or hemp particles in water in ratio 1:10 weight parts. The leaches had been left in laboratory environment for 24 hours. Then we removed from the leach any solids by filtration through the filter paper. So we got net leaches from both, wood and hemp particles.

The experiment itself was executed using a comparative method. We tested and compared the initial and final setting times and final strengths of spot samples. We compared three standard samples 40 × 40 × 160 mm from each test mixture. Each mixture was made according to standard (ČSN EN 196-1: Methods of testing cement Part 1: Determination of strength), with different mixing water (leaches). Results are written in table No. 3.

### Table 1. One day hemp and wood leach and their comparison.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Mix water requirements, ČSN EN 1008</th>
<th>Leach</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Wood</td>
<td>Hemp</td>
<td></td>
</tr>
<tr>
<td>CHSK Cr mg/l</td>
<td>–</td>
<td>897</td>
<td>2640</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sulphates mg/l</td>
<td>Max. = 2000</td>
<td>75</td>
<td>159</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SO₄²⁻</td>
<td></td>
<td>9.93</td>
<td>50.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Humic substances mg/l</td>
<td>Qualitative setting, brightly yellow Max. 100</td>
<td>&lt;50</td>
<td>&lt;50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reducing sugars mg/l</td>
<td>Max. 100</td>
<td>&lt;50</td>
<td>&lt;50</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 2. Tested parameters.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Bulk density [kg/m³]</th>
<th>R₀ [N/mm²]</th>
<th>Rₜ [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood particles leach</td>
<td>2050</td>
<td>8.0</td>
<td>43.0</td>
</tr>
<tr>
<td>Hemp leach</td>
<td>2030</td>
<td>7.8</td>
<td>42.5</td>
</tr>
<tr>
<td>Common water</td>
<td>2070</td>
<td>7.9</td>
<td>43.2</td>
</tr>
</tbody>
</table>

4 HEMP BASED CEMENT BONDED PARTICLEBOARDS

#### 4.1 Treated hemp

In this year full cement bonded particleboards with treated hemp as a filler were mixed and pressed in the cooperation with CIDEM Hranice, a. s., division CETRIS. The slabs were conditioned as well as common cement bonded particleboards in drier and then sawed up to testing samples. Their physical and mechanical properties were determined in the labs.

#### 4.2 Composition

Hemp based cement bonded particleboards which were made are from treated hemp particles, cement, water and hydration admixtures. Fine fraction lay on
both sides of coarse fraction of slab mixture. This is the reason of fine surface of final products.

5 CONCLUSIONS

At present the project of modification of cement-bonded particleboards is in the beginning; therefore the paper describes results and knowledge from the first time period of modification and testing. There are some positive hints that show a promising solution way of developing new progressive material on base of secondary raw and (or) fast renewable materials.

The results of executed experiments show that substances contained in hemp did not have noticeably affected bonding interactions between hemp and bonding material – cement. The leach from hemp showed higher value of parameters (table no.2), but they do not even reach determined limits. From this point of view tested hemp leach complies with the values determined by norm. First of all it concerns the presence of substances that can cause decrease of material durability by their aggressive actions in cement matrix (e.g. sulphates) or substances that retard cement hydration, so by this way they may have
bad influence on cement matrix formation (organic compounds, e.g. carbohydrates, phosphates, etc.). All three samples, mixture from hemp or wood particles leach or common water, were correctly and smoothly hardened. In these samples there will be determined other physical, mechanical and chemical characteristics (DTA, RTG, etc.) to final elimination of any possible harmful reactions of hemp and parts of cement matrix.

Potential harmful changes might have a harmful influence on hemp based cement bonded particleboards durability. Therefore another part of our testing is focused on hemp filler behavior in cement matrix and potential changes observed in time. In the cooperation with CIDEM Hranice, a.s., division CETRIS full cement bonded particleboards were mixed and pressed with treated hemp as a filler. The slabs were conditioned as well as common cement bonded particleboards in drier and then sawed up to testing samples. In these samples their physical and mechanical properties were determined in the labs; they show very high values comparable with properties of common cement bonded particleboards. For now it seems that hemp has no influence on cement matrix behavior and also with high properties of final hemp based cement bonded particleboards they can be used as a full substitution for cement bonded particleboards.

AKNOWLEDGEMENTS

The paper contains results obtained during the solution of the Project GA 103/05/H044 “Science Advancement Stimulation of Doctoral Students of Building Material Engineering Branch” and with the support of MSM 0021630511 “Progressive Building Materials with Utilization of Secondary Raw Materials and their Impact on Structures Durability”.

REFERENCES

Chemical and physical characterization of coarse bauxite residue (red sand) for concrete making

Majid Ghiafeh Davoodi, Hamid Nikraz
Curtin University of Technology, Department of Civil Engineering, Perth, WA, Australia

Evan Jamieson
Alcoa World Alumina Australia, Technology Delivery Group, Kwinana, WA, Australia

ABSTRACT: This paper describes the works carried out in laboratory to investigate the possibility of using coarse bauxite residue (red sand >100 µm) and its derivatives as a fine aggregate in concrete mixture designs. Alcoa World Alumina (Alcoa) has three refineries in Western Australia producing approximately 20,000 tonnes of mud and a similar level of red sand each day. Much research has been undertaken to establish a sustainable use for this residue. The main purpose of this research was to establish a scientific solution for the utilization of red sand and its derivatives as a replacement of natural fine aggregate (yellow sand) in concrete mixtures. The comparative analysis between normal mix proportions and the mix proportions utilizing red sand was achieved. For this project, the coarse bauxite residue obtained in the Bayer process was neutralized (carbonated) and washed to a by-product standard. Further processing was conducted using a magnetic separator to form different fractions such as High Iron Red Sand and High Silica Red Sand. Physical, chemical and mineralogical properties of these materials have been tested and compared with that of natural sand. Several mixtures of concrete were then designed in order to verify their effects on strength and other properties. From the results obtained, it can be deduced that this material can be used in a wide range of concrete applications in Civil Engineering and Construction.

1 INTRODUCTION

Bauxite residue is a by-product of the Bayer process. In general terms, production of 1 tonne of alumina is accompanied by the creation of about 1-2.5 tonnes of this residue depending on the bauxite composition. The large scale of bauxite mining in Australia and around the world inevitably generates an enormous amount of bauxite residue which must be disposed of economically and in an environmentally acceptable manner.

Some problems associated with this residue in Western Australia are:
- Environment related issues
  - Highly caustic
- Economic issues
  - High cost of managing and maintaining of residue storage facilities
  - Land occupation for its storage (growing demand for land in WA due to urban sprawl)
- Sustainability issues

Many technologies have been designed and efforts have been made to use bauxite residue beneficially, but to date few economically viable solutions have been found so its utilization has been limited. For example, within the last decades many technologies have been proposed for the recovery of metal contained in this waste material [Piga et al. 1995]. Some technologies have been developed to use bauxite residue as a raw material for the production of tiles or red bricks [Knight et al. 1986]. Some products of this by-product also have been used as levee core, road base or as fertilizer filters and synthetic soils [Vsévolod et al. 2003].

Nowadays many by-products and waste materials such as fly ash and furnace slag are widely used in the concrete industry [Shehata et al. 1996, Yunping Xi et al. 2004]. Many researchers have investigated usage of different types of such waste materials in concrete mixtures as aggregates and the results have revealed the possibility of using this material in concrete effectively [Maslehuddin et al. 2003]. World concrete consumption is estimated at more than 12 billion tonnes each year and since 24-28% of concrete ingredients is fine aggregate this industry has a good potential for using bauxite residue and its derivatives.

Over the last decades, Alcoa has investigated many alternatives for the sustainable use of this residue [Jamieson et al. 2005].
In this regard, Alcoa has been working to develop red sand by a process of separation from the bulk residue, neutralisation by carbonation and washing low in salt. [Jamieson et al. 2006].

Bauxite residue from Bayer process has high alkalinity (pH > 11) and consists of gibbsite Al(OH)₃, boehmite (γ-AlOOH), illite, quartz (SiO₂), goethite FeO(OH), hematite (Fe₂O₃), sodalite (Na₄Al₃Si₃O₁₂Cl), calcite (CaCO₃) and calcium aluminosilicate. This residue is separated into mud, the so-called red mud, which is a fine fraction of the residue, and sand fraction with the nominal particle size bigger than 100 micron. This raw red sand requires neutralisation and washing to become a product. (Hereafter referred to as ‘Washed and Carbonated Red Sand’ -WCRS). Washed and carbonated red sand subjected to a magnetic separator is rewashed and splits into 3 fractions which are “high in silica red sand”, “middle”, and “high in iron red sand”.

Alcoa World Alumina in Western Australia currently has three alumina refineries located at Kwinana, Pinjarra and Wagerup producing approximately 20,000 tonnes of red mud and a similar level of red sand each day.

In this study, the physical and chemical characteristics of red sand were investigated. Furthermore, the possibility of using this residue and its derivatives as aggregate fine sand substitution in concrete mixtures to produce concrete composites was investigated.

This paper presents laboratory results that confirm the viability of using red sand as a basic construction material for concrete making. These new fine aggregates can be used in concrete making in a vast range of applications. In addition to the economic benefits accruing from their use, they can also displace the mining of virgin natural sand the associated environmental impacts.

2 METHODOLOGY

2.1 Materials

2.1.1 Fine aggregate

The red sand and red mud used in the present study as a fine aggregate were obtained from Wagerup refinery, located in Western Australia by permission of the Alcoa World Alumina. Natural sand from local suppliers was used in control mixes as a reference material. Figure 1 shows the appearance of the fine aggregates used in this research underlining red-brownish classic colour of different types of red sands; their rough surface texture and angular shape in comparison to the rounded shape of natural yellow sand. It can be concluded that the angular shape of red sand will lead to higher water demand in concrete mixture. In an effort to negate this impact, red mud (fine fraction) was also added to improve rheology.

All fine aggregates used in this study were prepared in such ways that were at saturated surface dry (SSD) condition.

The typical elemental composition of the red mud, washed and carbonated red sand, high silica-rich red sand and middle fraction which has been used in this research plus iron-rich red sand which can possibly be processed and used in the steel industry is provided in Table 1.

2.1.2. Coarse aggregate

Coarse aggregate used in this research was crushed granite with nominal size of 20 mm.

2.1.3 Cement

A normal portland cement (PC) conforming to relevant Australian standard was used to manufacture concrete mixtures.

<table>
<thead>
<tr>
<th>Chemical composition</th>
<th>Red Mud</th>
<th>Washed and carbonated RS</th>
<th>Red sand</th>
<th>High silica fraction</th>
<th>Middle sand</th>
<th>High iron red sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂ (%)</td>
<td>40.7</td>
<td>19–44</td>
<td>87.7</td>
<td>39.9</td>
<td>41.6</td>
<td></td>
</tr>
<tr>
<td>Fe₂O₃ (%)</td>
<td>30.8</td>
<td>11–45</td>
<td>4.27</td>
<td>35.7</td>
<td>39.9</td>
<td></td>
</tr>
<tr>
<td>Al₂O₃ (%)</td>
<td>19.5</td>
<td>11–26</td>
<td>3.51</td>
<td>13.4</td>
<td>10.91</td>
<td></td>
</tr>
<tr>
<td>TiO₂ (%)</td>
<td>3.0</td>
<td>1–4</td>
<td>0.23</td>
<td>1.82</td>
<td>1.54</td>
<td></td>
</tr>
<tr>
<td>CaO (%)</td>
<td>4.4</td>
<td>2–8</td>
<td>0.03</td>
<td>0.12</td>
<td>0.22</td>
<td></td>
</tr>
<tr>
<td>Na₂O (%)</td>
<td>3</td>
<td>1–6</td>
<td>0.01</td>
<td>0.11</td>
<td>0.18</td>
<td></td>
</tr>
</tbody>
</table>

Table 1. Typical composition of fine aggregates [Jamieson et al. 2005].
Table 2. Mixture proportion (Kg/m³).

<table>
<thead>
<tr>
<th>Mix no</th>
<th>Fine aggregate type</th>
<th>W/C</th>
<th>Agg./C ratio</th>
<th>Water</th>
<th>PC</th>
<th>Fine aggregate</th>
<th>Coarse aggregate</th>
<th>Slump (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Natural sand</td>
<td>0.35</td>
<td>2.69</td>
<td>205</td>
<td>586</td>
<td>425</td>
<td>1149</td>
<td>135</td>
</tr>
<tr>
<td>2</td>
<td>High silica red sand</td>
<td>0.35</td>
<td>2.35</td>
<td>225</td>
<td>643</td>
<td>393</td>
<td>1119</td>
<td>160</td>
</tr>
<tr>
<td>3</td>
<td>Washed and carbonated red sand</td>
<td>0.35</td>
<td>2.35</td>
<td>225</td>
<td>643</td>
<td>408</td>
<td>1104</td>
<td>105</td>
</tr>
<tr>
<td>4</td>
<td>HSRS 1:1 WCRS</td>
<td>0.35</td>
<td>2.35</td>
<td>225+</td>
<td>643</td>
<td>401</td>
<td>1111</td>
<td>130</td>
</tr>
<tr>
<td>5</td>
<td>Red Mud (fine)</td>
<td>0.35</td>
<td>2.35</td>
<td>225+</td>
<td>643</td>
<td>378</td>
<td>1134</td>
<td>60</td>
</tr>
<tr>
<td>6</td>
<td>HSRS 1:1 RM</td>
<td>0.35</td>
<td>2.35</td>
<td>225+</td>
<td>643</td>
<td>385</td>
<td>1127</td>
<td>70</td>
</tr>
<tr>
<td>7</td>
<td>WCRS 1:1 RM</td>
<td>0.35</td>
<td>2.35</td>
<td>225+</td>
<td>643</td>
<td>393</td>
<td>1119</td>
<td>No Slump</td>
</tr>
<tr>
<td>8</td>
<td>HSRS 0.9:0.1 RM</td>
<td>0.35</td>
<td>2.35</td>
<td>225</td>
<td>643</td>
<td>393</td>
<td>1119</td>
<td>70</td>
</tr>
<tr>
<td>9</td>
<td>WCRS 0.9:0.1 RM</td>
<td>0.35</td>
<td>2.35</td>
<td>225</td>
<td>643</td>
<td>408</td>
<td>1104</td>
<td>30</td>
</tr>
</tbody>
</table>

* The real W/C after adding extra water.
** The amount of extra water added to obtain desire workability.

Table 3. Physical and chemical characteristics of fine aggregate [ * Sri Wahyuni et al. 2006].

<table>
<thead>
<tr>
<th>Property</th>
<th>Natural sand</th>
<th>Red mud</th>
<th>Washed and carbonated red sand (WCRS)</th>
<th>High silica red sand (HSRS)</th>
<th>Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity *</td>
<td>2.64</td>
<td>-</td>
<td>2.81</td>
<td>2.60</td>
<td>AS1141.5</td>
</tr>
<tr>
<td>Fineness Modulus *</td>
<td>2.47</td>
<td>-</td>
<td>1.99</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Water Absorption (%)</td>
<td>0.242</td>
<td>3.057</td>
<td>2.048</td>
<td>0.305</td>
<td>AS1141.5</td>
</tr>
<tr>
<td>Surface Area (m²/gr)</td>
<td>0.66</td>
<td>20.44</td>
<td>5.38</td>
<td>0.98</td>
<td>BET</td>
</tr>
<tr>
<td>Chloride, Cl (mg/kg)</td>
<td>80</td>
<td>230</td>
<td>&lt;50</td>
<td>80</td>
<td>AS1012.20</td>
</tr>
<tr>
<td>Sulphate as SO₃ (mg/kg)</td>
<td>&lt;100</td>
<td>24,000</td>
<td>&lt;100</td>
<td>&lt;100</td>
<td>AS1012.20</td>
</tr>
<tr>
<td>Sugar presence</td>
<td>Absent</td>
<td>Absent</td>
<td>Absent</td>
<td>Absent</td>
<td>AS1141,Sec35</td>
</tr>
</tbody>
</table>

2.2 Method

2.2.1 Mixture proportion
Seven different mixtures were employed to examine the influence of incorporating red sand as a fine aggregate on concrete properties at first stage; however, due to weak strength of red mud in mixtures 5–7 another two mixtures were added with a different percentage of red mud inclusion. Mixture proportion for a medium strength concrete (f’c₂₈= 50 Mpa) is presented in Table 2. The water-cement ratio was kept constant for all mixtures at 0.35 to obtain comparable strengths at different ages.

The concrete mixtures were as follows: 1-Control Mix (Natural Yellow Sand), 2- Processed High Silica Red Sand (HSRS), 3- Washed and Carbonated Red Sand (WCRS), 4- HSRS 1:1 WCRS, 5- Red Mud (RM-fine), 6- HSRS 1:1 RM, 7- WCRS 1:1 RM, 8-HSRS 0.9:0.1 RM, 9- WCRS 0.9:0.1 RM

Different specimens for testing in the hardened state for the purpose of compressive strength, flexural and splitting strength, shrinkage and apparent volume of permeable voids (AVPV) were cast by vibration to ensure proper consolidation and after 24 hours they were demoulded and cured in water according to relevant Australian standards until test day.

3 EXPERIMENTAL RESULTS AND DISCUSSION

3.1 Basic physical and chemical characteristic of red sand
Some physical and chemical properties of fine aggregates used in this research are summarized in Table 3. The similarity of natural sand properties with high silica red sand are noteworthy. Due to higher percent of fines in HSRS its water absorption and surface area are greater than natural sand.

The sieve analysis results (AS1141.11-1996) of washed and carbonated and high silica red sand are plotted in Figure 2 and Figure 3 respectively.

It can be observed from the results that both the washed and carbonated and high silica red sand are reasonably well graded and meet the grading requirement.
of the Australian Standard AS 2758.1-1998 for concrete aggregate. However, both types of red sands have higher levels of fines (<150 µm) compared to natural sand.

3.2 Workability of fresh concrete

In general, the desirable slump for all designed mixes were specified 80–120 mm (for a reinforced wall); however, in practice as presented in Table 2, except WCRS, other mixtures’ slumps were higher or lower value and gave inconsistent results. In spite of this fact except mixture 7 and mixture 9 which had a low slump all the other mixtures were within the standard deviation limit.

The declines in workability in mixtures incorporating WCRS and red mud were due to higher water absorption capacity and in particular, higher surface area of these fine aggregates. Angular particle shape of WCRS and red mud also helped to decrease workability.

3.3 Compressive strength of the concrete

In the hardened state, 100 mm cylindrical specimens with the high of 200 mm were tested for compressive strength in accordance with AS1012.9-1999. Compressive strength was determined at 3, 7, 14, 28, 56, 90 days curing and the values are an average of 3–5 determination.

Figure 4 shows the curve for obtained compressive strength values over a period of 90 days for all mixtures. In general, this Figure reveals that the values of compressive strength for all mixtures except mixtures 5–7 are higher than the average specified 28-days compressive strength (50 MPa).

The compressive strength for HSRS shows an average decline of 4% for 28 days. This decrease is higher for early strength; however, it fulfils the proposed objective of 50 Mpa after 28 days.

Usage of WCRS shows an average of almost 10% rise in all ages.

Compared with the control mix the change of strength for 50–50 HSRS and WCRS is not significant;
however it is negative for early ages and positive for 28 days. The mixtures with the addition of 50–100% red mud as a fine aggregate has given the lowest strength of around 20 MPa. This reveals that red mud significantly reduces the compressive strength of concrete and in regards to workability results it shows that it is neither acceptable nor applicable for industry uses at such high application rates. The decrease in strength may be partly due to the amount of water added to maintain the consistency of mortar to the specified flow.

Finally, the lower application rate of 10% RM shows up to 15% strength increase in the early ages and 5–10% higher for 28 days or more.

In general the compressive strength is satisfactory in mixtures incorporated HSRs, WCRS and in mixtures with inclusion of 10% red mud. Except HSRs the evaluation over the period of 3–90 days is positive with improvement of 7–16% for mixtures 3 and 4 and around 12% for mixtures 8 and 9, and is promising with respect to the durability of red sand concrete.

In regards to HSRs, although its strength declines an average of 6% for ages 3–90 days, it is acceptable and higher than target strength (50 Mpa).

3.4 Tensile strength

3.4.1 Indirect tensile strength (‘Brazil’ or splitting test)

Large cylinders 150 mm in diameter and 300 mm in height were used in this test. Indirect tensile strength was determined after 7, 14, 28 days of curing in accordance with AS1012.10-2000. Based on the values depicted in Figures 5 and 6 replacement of natural sand with red sand has a negative effect on indirect tensile strength. This shows an average reduction for 28 days’ age up to 11% and 4% for HSRs and WCRS respectively. This may due to several factors such as the properties of the fine aggregate as well as the grading of the aggregates [Neville 1993]. Compared with HSRs and WCRs, this decrease for mixtures with usage of 10% red mud is higher with the average of 25%, hence it appears red mud has caused negative effects on HSRs and WCRs tensile strength.

In comparison with control mix overall changes over the period of 7–28 days are negative 7%, 3% and 12% for mixtures 2–4 respectively.

The reduction in tensile strength can be a result of the porous structure of red sand. The porous material lowers the tensile strength.

3.4.2 Flexural strength

Beam specimens, 100 × 100 × 400 mm, were cast to measure the flexural strength according to AS1012.11-2000. As Figure 7 shows in comparison with natural sand, flexural strength has risen significantly in all
mixtures. This increase is 70% and 45% for HSRS and WCRS respectively. The inclusion of 10% RM caused 60% growth in flexural strength. Use of 10% RM shows a negative effect on flexural strength for HSRS but positive for WCRS. (Almost – 6% and 10% respectively)

This increase is due to the angular shape of red mud and red sand which causes a better bond between cement and fine aggregate. The flexural strength will be more important in such applications as road, pavement where the resistance to bending is highly in demand.

3.5 Drying shrinkage

Beams measuring 75 × 75 × 280 mm were tested for drying shrinkage evaluation according to AS1012.13-1992. Test specimens were cured in lime saturated water for 7 days and then removed and placed in a drying room. The change in length of these specimens was measured at periodical intervals using the instrumentation specified in the above mentioned Australian standard. The measurement from day one is referred to as the initial length measurement.

Figure 8 illustrates the shrinkage results over a period of 1–56 days. The control mix and HSRS exhibited the same profile especially during the first 28 days; however the shrinkage value for HSRS decreases up to 25% at 56 days age.

3.6 Apparent volume of permeable voids (AVPV)

The AVPV test was conducted based on AS1012.21–1999 which basically gives the measure of how much water is absorbed by voids in the concrete. A higher percentage value for AVPV means that the concrete has poor durability characteristics so that it is easier for the water borne ions to infiltrate into the concrete and initiate corrosion of the reinforcing steel.

From the results in Figure 9 it can be seen WCRS has the lowest AVPV with 14.8% followed by the control mix with the 15.5%. The inclusion of red mud had a negative effect on AVPV values with the 10% and 18% rise for mixtures 8 and 9 respectively. This is due to these mixtures having a low slump resulting in more pores in the concrete samples and the pores in turn enabling easy ingress of water to the concrete samples.

The values in this study, compared with the durability requirements for this test stated in the publication by the Concrete Institute of Australia [Concrete Institute of Australia 2001] were shown the marginal limit except the mixtures which incorporated red
The durability classification indicator for these two mixtures was in the unacceptable range.

4 CONCLUSIONS

According to these research findings the complete replacement of natural sand with coarse washed and carbonated red sand and high silica red sand as fine aggregate had no significant negative impact upon the concrete compressive strength. Physical and chemical properties of high silica red sand indicate it has similar characteristics to natural sand. So in general, it can be concluded that HSRS and WCRS can be used in medium strength concrete mixtures as both showed sufficient strength.

The Inclusion of 10% red mud has an adverse effect on properties of concrete except the strength and it reduces the workability of concrete dramatically. It is clear that natural sand can not totally be replaced by red mud as it reduces strength sharply.

The results obtained in this work reveal that all mixtures except mixtures 5–7 having target strength of 50 MPa were found adequate in terms of strength, including compressive strength, indirect tensile strength and flexure strength; they also had good length stability in terms of shrinkage. The mixtures incorporating 10% red mud did not demonstrate a good result for AVPV which is a primary indicator of durability; however other mixtures’ durability also is in marginal limit. It is difficult, though to draw conclusions from only AVPV test results.

The outcomes of this research are encouraging in view of the possibility of using coarse bauxite residue for construction purposes in concrete mixtures.

5 RECOMMENDATION

The findings of this work suggest that further work should be conducted to examine the durability of concrete especially water permeability and chloride ingress tests. Usage of other pozzolans such as fly ash or slag also would be other options to reduce the cement content in mixture designs from the economic standpoints and also to avoid the heat of hydration.

Usage of red mud in other applications (lower target strength than 50 Mpa) also needs to be tested with more mixtures.

ACKNOWLEDGMENT

The authors wish to thank the Centre for Sustainable Resource Processing and Alcoa World Alumina Australia, for the supply of samples and support.

REFERENCES

Concrete Institute of Australia 2001, Performance Criteria for Concrete in Marine Environments, Concrete Institute of Australia.
Shehata, I. & Varzavand, S. & ElSawy, A. & Fahmy, M. May 1996, The use of solid waste material as fine aggregate substitutes in cementitious concrete composites, in Semisequicentennial Transportation, Iowa State University, Iowa State University, Ames, Iowa, USA.
Effect of ground perlite incorporation on the performance of blended cements

T.K. Erdem, Ç. Meral, M. Tokyay, T.Y. Erdoğan
Department of Civil Engineering, Middle East Technical University, Ankara, Turkey

ABSTRACT: Perlite is a volcanic rock that contains relatively high amounts of SiO₂ and Al₂O₃. Due to its proper chemical composition and glassy structure, it can be used as a pozzolanic addition in blended cements. In this study, ground perlite was used as a cement replacement material in blended cements. Several mortar mixes were prepared to investigate the performance of those cements. The results showed that perlite incorporation caused early age strength losses when compared to the control mortars containing only portland cement; however, the difference between them decreased in time due to the pozzolanic reactions. The strengths of the blended cements were still within the limits of the EN standards. Moreover, it was observed that use of ground perlite increased the durability of portland cement mixes.

1 INTRODUCTION

Pozzolans are defined in ASTM C618 (ASTM 2002a) as “siliceous or siliceous and aluminous materials which in themselves possess little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties”. Volcanic tuffs, volcanic ashes and diatomaceous earths can be listed as examples of natural pozzolans; fly ashes, silica fumes, and granulated blast furnace slags may be counted as artificial materials which show pozzolanic properties.

The effects of pozzolans vary according to their type and amount; however, generally speaking, they can improve workability, strength, and durability, and reduce the cost of the cement-based materials such as mortars and concretes. They can be regarded as very important materials also for sustainable development: the increased durability reduces the repair costs and material consumption; the lower clinker production reduces environmental hazards of the cement production through lower CO₂ emission and through lower energy requirement during calcining and grinding the materials.

Perlite is a form of altered obsidian which has some combined water (2–6%) in its structure. This glassy volcanic rock contains approximately 70–75% SiO₂ and 12–18% Al₂O₃. When it is heated to high temperatures like 900°C, the chemically combined water in the structure causes it to expand and become a cellular material which can be used very efficiently in constructions as light-weight aggregate, insulation material, etc. The bulk density of expanded perlite is generally 60–120 kg/m³.

There are very large perlite reserves in the world (around 6700 million tons) and approximately two thirds of those are in Turkey (TSPO 2001). Due to its proper chemical composition (relatively high amounts of SiO₂ and Al₂O₃) and glassy structure, perlite can be used as a pozzolanic addition in blended cements (Erdem et al. 2007). The grindability of perlite was also found suitable for cement production (Erdem et al. 2007). However, despite the large perlite reserves in the world and the potential benefits of the pozzolans discussed above, studies on this subject are very limited. Therefore, this study aims to extent our knowledge on the use of perlites in blended cement. For this purpose, two natural perlites from different locations of Turkey were, first, ground and then mixed with Portland cement in different amounts. The investigated properties include water requirement, setting time, soundness, compressive strength, rapid chloride permeability and sulphate resistance.

2 EXPERIMENTAL PROGRAM

Portland cement (PC) used in this study was produced by grinding a normal portland cement clinker together with 4% gypsum in the laboratory to a Blaine fineness of 320 ± 5 m²/kg. The chemical composition of the PC is given in Table 1.
Two different perlites were brought to the laboratory from two cities of Turkey, which are Izmir and Erzincan. The chemical compositions of Izmir perlite (IP) and Erzincan perlite (EP) are given in Table 1.

Both of the perlites were ground in the laboratory by a grinding mill until reaching Blaine finenesses of 320 ± 5 and 370 ± 5 m²/kg. A summary of the particle size distribution of the perlites are given in Table 2. Then, each perlite was mixed with PC to obtain blended cements so that the perlite amount is 5, 20 or 30% of the cement. The cements produced in this study and their compositions are summarized in Table 3. (The cements with 5% perlite were used only for sulphate resistance and rapid chloride permeability tests.)

The cements were tested to determine their water requirement, setting time, soundness, compressive strength, rapid chloride permeability and sulphate resistance according to the relevant ASTM specifications (ASTM 2002a, b).

ASTM C187 was followed to determine the water requirement of the cement to have normal consistency. The same cement pastes were used also to measure the setting time and soundness (autoclave expansion) according to ASTM C191 and C151, respectively.

For compressive strength tests, performed in accordance with ASTM C109, PC mortars were prepared with 0.485 water-cement ratio (w/c). The blended cements mortars were proportioned by adjusting the water content in such a way that they yield a flow of 110 ± 5. Three 5-cm cubes were tested at 2, 3, 7, 28, 56 and 91 days.

Rapid chloride permeability tests (RCPT) were performed according to ASTM C 1202. For each cement type, mortars (containing 500 g cement, 1375 g standard RILEM sand and 242 ml water) were prepared several times to produce three 10 cm × 20 cm cylindrical specimens for testing one specimen at each testing age (28 days, 3 months and 6 months). At each age, three slices with 10 cm diameter and 5 cm height were cut from a specimen by using a saw. After the slices were placed in a vacuum desiccator, the vacuum pump was run in dry condition for 3 hours and in wet condition for 1 hour. Following the soaking under water for 18 hours, each slice was placed between two testing cells and sealed with silicon. One of the cells was filled with 3.0% NaCl solution while the other was filled with 0.3 N NaOH solution. After making the electrical connections, the test was started. The current was automatically recorded by the testing machine at every 30 minutes. The test was terminated after 6 hours.

The sulphate resistance of the mortars was determined according to ASTM C1012. For each cement type, mortar mixes were prepared to produce 15 cubes (5 cm × 5 cm × 5 cm) and 6 bars (25 mm × 25 mm × 285 mm). 2 cubes were tested under compression at several ages to check whether the strength reached 20 MPa or not. During this period, the bars and the remaining cubes were stored in lime-saturated water at 23°C. When the compressive strength of the cubes reached 20 MPa, the lengths of the mortar bars were measured, and then, they were immersed in sulphate solution (pH = 7 ± 1) containing 352 moles of Na₂SO₄ per m³ (50 g/L). The length changes of the mortar bars were determined at 1, 2, 3, 4, 8, 13, 17, 26 and 39 weeks after the bars were placed in the sulphate solution. The measurements were taken by using a comparator with 0.0025 mm precision. The solutions were changed with a fresh solution of pH = 7 ± 1 at each testing age.

Table 1. Chemical composition of PC, IP and EP.

<table>
<thead>
<tr>
<th>Oxides (%)</th>
<th>PC</th>
<th>IP</th>
<th>EP</th>
</tr>
</thead>
<tbody>
<tr>
<td>CaO</td>
<td>63.58</td>
<td>0.51</td>
<td>0.60</td>
</tr>
<tr>
<td>SiO₂</td>
<td>21.00</td>
<td>76.57</td>
<td>75.30</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>4.98</td>
<td>9.99</td>
<td>9.35</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>3.57</td>
<td>0.96</td>
<td>1.36</td>
</tr>
<tr>
<td>MgO</td>
<td>1.86</td>
<td>0.03</td>
<td>0.05</td>
</tr>
<tr>
<td>K₂O</td>
<td>0.74</td>
<td>5.58</td>
<td>4.82</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.14</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Table 2. Particle size distribution of the perlites.

<table>
<thead>
<tr>
<th>Percent of material larger than</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perlite 93μm</td>
</tr>
<tr>
<td>IP-320</td>
</tr>
<tr>
<td>EP-320</td>
</tr>
<tr>
<td>IP-370</td>
</tr>
<tr>
<td>EP-370</td>
</tr>
</tbody>
</table>

Table 3. Labels and compositions of the cements produced.

<table>
<thead>
<tr>
<th>Cement label</th>
<th>Perlite type</th>
<th>Perlite fineness (m²/kg)</th>
<th>Perlite amount (%)</th>
<th>PC fineness (m²/kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC</td>
<td>–</td>
<td>–</td>
<td>0</td>
<td>320</td>
</tr>
<tr>
<td>IP-370-5%</td>
<td>IP</td>
<td>370</td>
<td>5</td>
<td>320</td>
</tr>
<tr>
<td>EP-370-5%</td>
<td>EP</td>
<td>370</td>
<td>5</td>
<td>320</td>
</tr>
<tr>
<td>IP-320-20%</td>
<td>IP</td>
<td>320</td>
<td>20</td>
<td>320</td>
</tr>
<tr>
<td>EP-320-20%</td>
<td>EP</td>
<td>320</td>
<td>20</td>
<td>320</td>
</tr>
<tr>
<td>IP-320-30%</td>
<td>IP</td>
<td>320</td>
<td>30</td>
<td>320</td>
</tr>
<tr>
<td>EP-320-30%</td>
<td>EP</td>
<td>320</td>
<td>30</td>
<td>320</td>
</tr>
<tr>
<td>IP-370-20%</td>
<td>IP</td>
<td>370</td>
<td>20</td>
<td>320</td>
</tr>
<tr>
<td>EP-370-20%</td>
<td>EP</td>
<td>370</td>
<td>20</td>
<td>320</td>
</tr>
<tr>
<td>IP-370-30%</td>
<td>IP</td>
<td>370</td>
<td>30</td>
<td>320</td>
</tr>
<tr>
<td>EP-370-30%</td>
<td>EP</td>
<td>370</td>
<td>30</td>
<td>320</td>
</tr>
</tbody>
</table>
3 RESULTS AND DISCUSSIONS

3.1 Water requirement

Water requirement of the cements were determined by the amount of water necessary to bring the cement pastes to a condition of normal consistency defined according to ASTM C187. The results were expressed as w/c in Table 4.

As can be seen from Table 4, w/c of the PC paste was 0.234 whereas w/c was minimum 0.242 for the other cements. Therefore, it can be concluded that the blended cements required a higher amount of water than PC for the same consistency. Moreover, when the cements having the same perlite type and fineness are compared, it is observed that the w/c increased as the perlite amount in the pastes increased. For example, in the case of EP-370-20% and EP-370-30% types of cements, w/c was 0.244 and 0.250, respectively. In the same way, the effects of perlite type and perlite fineness can be observed by comparing the cements which have the same parameters except the parameter under question. Such kind of comparisons from Table 4 shows that the cements containing perlites with 370 m²/kg fineness have slightly higher than those with 320 m²/kg fineness. This is an expected result since a finer material has higher amount of surface area to be wetted. When comparisons are made for the same Blaine fineness and perlite amount, it is seen that the water requirements of the cements with EP were generally higher when compared to those with IP. This can be attributed to the fineness again: EP has a finer particle size distribution than IP for a given Blaine fineness (see Table 2) and required more water to cover the higher surface area of the particles.

3.2 Soundness

It is essential that a hardened cement paste does not undergo large volume changes causing cracks. Some compounds in the cement, namely free lime, magnesia, and calcium sulphate may cause such expansions, and therefore the cement should be tested for its volume stability, i.e. soundness. The results for the autoclave tests for determining the soundness are shown in Table 4.

All of the blended cements showed lower expansion values than PC except the paste containing EP-320-20% cement type, which was equal to PC. Moreover, the expansions decreased as the perlite amount in the pastes increased when the cements having the same perlite type and fineness are compared. The results also showed that the expansions were lower for the pastes prepared with IP than for those containing EP. Therefore, the lowest expansions were observed for the cements containing 30% IP, that is, IP-320-30% and IP-370-30%.

Finally as seen from Table 4, fineness of the perlite present in the paste did not change the results significantly.

3.3 Setting time

When the setting times are considered, it is seen that PC has the shortest setting times. Moreover, the higher the perlite amounts, the longer the setting times. This delay in the setting time for the pastes with higher amounts of perlites is due to their lower clinker amounts. Another reason for this may be the higher w/c of the pastes containing more perlite. Initial setting times were similar for the EP and IP pastes; however, the effect of perlite type was more significant for the final setting times of the pastes with finer perlites: the pastes with EP resulted in higher values. The setting time values were generally shorter when the fineness of the perlite increased.

3.4 Compressive strength

As stated in the Experimental Program section, PC mortars were prepared with 0.485 w/c while the water content of the blended cements mortars were adjusted for a flow of 110 ± 5. Therefore, w/c varied according to the cement type. The w/c and compressive strengths of the cements are given in Table 5.

As seen from Table 5, the strengths of blended cements were lower than those of PC. However, the differences became smaller for the later ages. For example, the strengths of IP-320-20% cement at 2, 28 and 91 days are 77.8%, 84.3% and 93.7% of the PC, respectively. The reason why the strengths get closer in time is the ongoing pozzolanic reactions of perlite in the blended cements. As known, the main hydration products of cements are calcium silicate hydrates (C-S-H), which provides the binding property, and calcium hydroxide. Pozzolans react with the calcium hydroxide which is released from the hydration of the

---

Table 4. Results of the tests for water requirement, soundness and setting time.

<table>
<thead>
<tr>
<th></th>
<th>w/c</th>
<th>Autoclave expansion (%)</th>
<th>Setting time (min)</th>
<th>Initial</th>
<th>Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC</td>
<td>0.234</td>
<td>0.07</td>
<td>124</td>
<td>168</td>
<td></td>
</tr>
<tr>
<td>IP-320-20%</td>
<td>0.242</td>
<td>0.05</td>
<td>139</td>
<td>190</td>
<td></td>
</tr>
<tr>
<td>EP-320-20%</td>
<td>0.243</td>
<td>0.07</td>
<td>148</td>
<td>190</td>
<td></td>
</tr>
<tr>
<td>IP-320-30%</td>
<td>0.242</td>
<td>0.02</td>
<td>174</td>
<td>209</td>
<td></td>
</tr>
<tr>
<td>EP-320-30%</td>
<td>0.247</td>
<td>0.06</td>
<td>175</td>
<td>216</td>
<td></td>
</tr>
<tr>
<td>IP-370-20%</td>
<td>0.240</td>
<td>0.05</td>
<td>142</td>
<td>177</td>
<td></td>
</tr>
<tr>
<td>EP-370-20%</td>
<td>0.244</td>
<td>0.06</td>
<td>146</td>
<td>195</td>
<td></td>
</tr>
<tr>
<td>IP-370-30%</td>
<td>0.245</td>
<td>0.02</td>
<td>153</td>
<td>179</td>
<td></td>
</tr>
<tr>
<td>EP-370-30%</td>
<td>0.250</td>
<td>0.05</td>
<td>150</td>
<td>195</td>
<td></td>
</tr>
</tbody>
</table>
calcium silicates in the cement and produce extra C-S-H gels. Therefore, the pozzolanic reactions proceed with the calcium hydroxide production from the cement hydration. This may result in strength losses at early ages; however, secondary C-S-H produced as a result of pozzolanic reaction starts to contribute to the strength in time.

In order to observe the effect of perlite amount, the mortars containing the cements with the same perlite fineness and same perlite type were compared. For example, IP-320-20% and IP-320-30% can be considered for this purpose. It is seen from Table 5 that 20% replacement produced higher strengths than 30% replacement. The lower strengths of the mortars with the cements containing 30% perlite can be explained by their lower PC contents (especially for the earlier ages during which pozzolanic reactions were insignificant).

The comparison of the cements with the same perlite types and perlite amounts but different perlite finenesses (for example, IP-320-20% and IP-320-30%) yields that an increase in the perlite fineness results in higher strengths. This can be explained by the higher rate of pozzolanic reactions for finer pozzolans.

Further investigation of the Table 5 revealed that the compressive strengths of EP mortars were sometimes higher, but sometimes lower when compared to IP mortars.

### 3.5 Rapid chloride permeability

Rapid chloride permeability test (RCPT) determines the electrical conductance of concrete to provide a rapid indication of its resistance to the penetration of chloride ions. The chloride ion resistance gives an indirect measure for permeability and internal pore structure, as more current passes through a more permeable structure. The results of this test can be used to assess the durability (Poon et al. 2001).

### Table 5. Compressive strengths of the cements.

<table>
<thead>
<tr>
<th>Cement label</th>
<th>w/c</th>
<th>2 days</th>
<th>3 days</th>
<th>7 days</th>
<th>28 days</th>
<th>56 days</th>
<th>91 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC</td>
<td>0.48</td>
<td>22.5</td>
<td>24.4</td>
<td>35.5</td>
<td>49.6</td>
<td>51.5</td>
<td>53.6</td>
</tr>
<tr>
<td>IP-320-20%</td>
<td>0.50</td>
<td>17.5</td>
<td>21.7</td>
<td>26.1</td>
<td>41.8</td>
<td>47.2</td>
<td>50.2</td>
</tr>
<tr>
<td>EP-320-20%</td>
<td>0.50</td>
<td>17.9</td>
<td>21.8</td>
<td>26.6</td>
<td>38.1</td>
<td>47.6</td>
<td>50.4</td>
</tr>
<tr>
<td>IP-320-30%</td>
<td>0.51</td>
<td>13.7</td>
<td>16.6</td>
<td>27.6</td>
<td>34.7</td>
<td>40.1</td>
<td>42.6</td>
</tr>
<tr>
<td>EP-320-30%</td>
<td>0.51</td>
<td>12.6</td>
<td>17.5</td>
<td>27.0</td>
<td>35.3</td>
<td>42.4</td>
<td>45.3</td>
</tr>
<tr>
<td>IP-370-20%</td>
<td>0.51</td>
<td>17.9</td>
<td>21.8</td>
<td>29.3</td>
<td>42.7</td>
<td>48.4</td>
<td>52.0</td>
</tr>
<tr>
<td>EP-370-20%</td>
<td>0.51</td>
<td>19.4</td>
<td>21.2</td>
<td>28.3</td>
<td>40.4</td>
<td>49.2</td>
<td>51.7</td>
</tr>
<tr>
<td>IP-370-30%</td>
<td>0.51</td>
<td>15.7</td>
<td>17.1</td>
<td>27.7</td>
<td>39.8</td>
<td>44.2</td>
<td>47.4</td>
</tr>
<tr>
<td>EP-370-30%</td>
<td>0.51</td>
<td>14.2</td>
<td>17.9</td>
<td>27.2</td>
<td>37.6</td>
<td>42.8</td>
<td>46.6</td>
</tr>
</tbody>
</table>

### Table 6. Rapid chloride permeability test results.

<table>
<thead>
<tr>
<th>Total Charge Passed in 6 hours (coulombs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>28 days</td>
</tr>
<tr>
<td>---------------</td>
</tr>
<tr>
<td>PC</td>
</tr>
<tr>
<td>IP-370-5%</td>
</tr>
<tr>
<td>IP-370-20%</td>
</tr>
<tr>
<td>IP-370-30%</td>
</tr>
<tr>
<td>EP-370-5%</td>
</tr>
<tr>
<td>EP-370-20%</td>
</tr>
<tr>
<td>EP-370-30%</td>
</tr>
</tbody>
</table>

RCPT involves the application of a voltage between two sides of a concrete specimen with solutions of sodium hydroxide and sodium chloride on opposite sides. (In the present study, mortar specimens were instead of concrete specimens). The total charge passed during a six-hour period provides a measure of the permeability (Poon et al. 2001, Mindess et al. 2003, Mohr et al. 2000, Houssam 1999). It was stated that for concretes with w/c ratios between 0.4 and 0.75, RCPT results correlate well with both of the more conventional pressure methods used to measure permeability and total porosity (Mindess et al. 2003).

RCPT results at 28 days, 3 months and 6 months are shown in Table 6. From the previous tests, it was observed that the finer perlite resulted in higher strengths. Therefore, for RCPT and sulphate resistance tests, only the perlite with 370 m²/kg fineness was used; however, a new incorporation amount, 5%, was included in the test program.

As it can be seen from Table 6, the total charge passed in 6 hours decrease in time. In other words, chloride permeability of the specimens gets lower for the later ages, as expected. The decreased chloride permeability of PC specimens with time can be explained by the higher degree of hydration of portland cement while the improved permeability of the
specimens containing perlite was not only due to the PC hydration but also due to the higher amount of pozzolanic reactions of perlite. The lower permeability obtained at later ages can also be observed from Table 7. (In this Table, the total charge values of each cement type at 6 months were expressed as percentages of the total charge values of that cement type at 28 days.) However, Table 7 was prepared to show that the improvements in the specimens containing perlite were higher than those in the PC specimens. Such a result indicates that pozzolanic reactions can decrease the permeability in a greater extent than hydration of PC can. As seen from Table 7, the higher benefit provided by the perlite incorporation was more significant as the perlite amount in the specimens increased.

As stated above, the increase in the perlite amount increases the difference between the permeability at 28 days and 6 months. Table 6 shows that as the perlite amount increases, permeability decreases at a given age, too.

It can also be observed from Table 6 that for the same replacement levels, while the values for IP are slightly lower than those for EP, they are generally close to each other. In other words, IP and EP displayed similar behaviors in improving the permeability.

### 3.6 Resistance to sulphate attack

As stated in Section 3.5, the sulphate resistance tests were performed by using the cements containing the perlites with 370 m²/kg fineness only. The replacement amounts were 5, 20 and 30%. The results of the tests performed up to 39 weeks (or 9 months) are given in Figures 1 and 2.

It is worth to note that the expansions of different mixes in the first four weeks are very small and very close to each other. Therefore, it is more appropriate to discuss the results starting from 8 weeks.

<table>
<thead>
<tr>
<th>Total Charge Passed as % of the 28-day values</th>
<th>Improvement (Difference)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC</td>
<td>100</td>
</tr>
<tr>
<td>IP-370-5%</td>
<td>100</td>
</tr>
<tr>
<td>IP-370-20%</td>
<td>100</td>
</tr>
<tr>
<td>IP-370-30%</td>
<td>100</td>
</tr>
<tr>
<td>EP-370-5%</td>
<td>100</td>
</tr>
<tr>
<td>EP-370-20%</td>
<td>100</td>
</tr>
<tr>
<td>EP-370-30%</td>
<td>100</td>
</tr>
</tbody>
</table>

![Figure 1](image1.png)

Figure 1. Expansions of Mortar Bars Containing PC and Blended Cements with IP due to Sulphate Attack.

![Figure 2](image2.png)

Figure 2. Expansions of Mortar Bars Containing PC and Blended Cements with EP due to Sulphate Attack.

Figures 1 and 2 show that PC mortar displayed higher expansions than the perlite mortars. Moreover, the expansions in the mortars with 5% replacement were higher when compared to those with 20% and 30% replacements. There was not so much difference in the expansions of the mortars with 20% and 30% perlite.

As known, the hydration products of portland cement are calcium silicate hydrates (C-S-H gels), calcium hydroxide and calcium sulfoaluminates. When a cementitious material is exposed to sulfates (such as sodium sulfate or magnesium sulfate), calcium hydroxide in the hardened cement paste reacts with the sulfates to produce gypsum. Then, some metastable calcium sulfoaluminates in the hydrated cement paste reacts with gypsum, produced from the sulfate attack, causing the formation of ettringite. Formation of gypsum causes some expansion in the hardened cement paste; however, expansion resulting from ettringite formation is greater than that caused by gypsum formation. The destructive consequences of sulfate attack include not only disruptive expansion and cracking, but also loss of strength due to the loss of cohesion in the hydrated cement paste and of adhesion between the paste and aggregates.

283
Moreover, sulfate attack can also cause disintegration and loss of stiffness (Mehta 1983).

In the line of the above mechanism of sulfate attack, the benefit of perlite incorporation instead of using only portland cement can be explained as follows: When a portion of PC is replaced with a mineral admixture (perlite, in this case), the total C₃A content in the mix will be reduced, and, as known, the expansions due to ettringite formation will be lower with the mixes having a lower C₃A content. Another explanation for the lower expansions of the perlite mortars is the pozzolanic reactions of the perlite. Since calcium hydroxide is consumed during the pozzolanic reactions, the amount of calcium hydroxide available for the ettringite formation is reduced resulting in lower expansions. Moreover, the incorporation of the portland cement with perlite reduces the amount of C₃S and C₂S which are the compounds responsible for the calcium hydroxide production during the hydration process.

If the RCPT results are accepted to be an indication of the permeability of the mortars, Table 6 can also be used to explain the results of the sulfate attack tests. As already discussed in Section 3.5, the chloride permeability of the PC mixtures was higher than that of the blended cement mixes and it decreased as the perlite amount increased. Therefore, the lower permeability provided by the use of perlite prevented the ingress of sulfate ions into the mortar bars and reduced the deleterious effects of sulfate exposure.

3.7 Evaluation of the blended cements

In the previous sections, the effects of perlite on several properties of the blended cements were discussed. However, it is essential that these cements satisfy the requirements given in standard specifications. For this purpose, the cements produced in this study were tested to check their conformity with the three most widely used cement specifications given in ASTM and EN standards (ASTM C109, EN 196-1, respectively) are not exactly the same; however, it can be stated that the blended cements also conform to EN 197-1 since w/c of the mortars were the same with or similar to what is required in EN 197-1 (i.e. 0.5).

The standard specification ASTM C1157 does not suggest any value for the sulphate resistance of type GU cements. However, in ASTM C595, there is an optional requirement for sulphate resistance of the type IP cements, and according to this requirement, the expansion at 180 days (or 26 weeks) must be less than 0.10%. As seen from Figures 1 and 2, all of the cements satisfied this statement.

4 CONCLUSIONS

The following conclusions can be drawn from this study:

When compared to the PC, the cements containing perlite required a higher amount of water for the same consistency, showed lower autoclave expansion values, had longer setting times, had lower chloride permeability and were more resistant to sulphate attack. The perlite mortars showed also lower compressive strengths than PC; however, due to the pozzolanic reactions the differences became smaller for the later ages.

A higher perlite amount in the blended cements resulted in higher water demand for the same consistency, longer setting times and higher strengths. On the other hand, chloride permeability and autoclave expansions decreased as the percentage of the perlites increased.

For a given perlite type and amount, increasing the fineness of the perlite caused slightly higher water requirement and shorter setting times. The soundness was not affected significantly from the fineness. Higher
strengths were noted for the cements containing finer perlites.

In general, IP and EP showed comparable behaviours.

The comparison of the test results with the standard specifications for blended cements showed that the measured properties conform to the limitations given in ASTM C595, ASTM C1157 and EN 197-1.

REFERENCES


TSPO 2001. Eight five years development program on use of industrial materials such as pumice, perlite, vermiculate phosphogypsum and expanding clays. Turkish State Planning Organization (TSPO) Ankara, [in Turkish].
Use of Magnesium Oxide-cement binders for the production of blocks with lightweight aggregates

N. Vlasopoulos & C.R. Cheeseman
Centre for Environmental Control and Waste Management, Imperial College, London, UK

ABSTRACT: Magnesium oxide (MgO)-cement (PC) systems are claimed to provide a sustainable alternative to PC, as MgO is produced by low-temperature (~750°C) calcination of naturally occurring minerals such as magnesite (MgCO₃) or brucite (Mg(OH)₂) precipitated from seawater and brines. In porous or semi porous materials such as blocks and pavers, magnesium oxide forms carbonates thus reabsorbing the carbon dioxide produced during calcining. This study has assessed the effect of reactive MgO and MgO-PC binders on the physical and mechanical properties of blocks made with lightweight aggregates. The blocks have been cured under either 98% relative humidity or wet-dry cycling to assess the effect of curing conditions on binder carbonation. MgO has been found to hydrate extensively to form Mg(OH)₂, which was partially carbonated to hydromagnesite (4MgCO₃.Mg(OH)₂.4H₂O). After 90 days curing, MgO samples achieved similar compressive strengths to those prepared using PC.

1. INTRODUCTION

Concrete blocks are widely used throughout the construction industry due to their ease of use, durability, thermal properties, sound insulation properties and economy. However, the cement fraction accounts for over 90% of the embodied energy in standard concrete. The manufacture of portland cement (PC) is an energy intensive process, responsible for up to 10% of anthropogenic CO₂ emissions worldwide [Pearce 1997] and there is a need for innovative, cost effective and sustainable alternatives.

MgO cements have recently received significant publicity [Pearce 1997, 2002]. These are blends of a normal hydraulic cement, such as PC, and reactive magnesium oxide, which sometimes also include a pozzolan such as fly ash. The MgO hydration has a particularly bad reputation amongst cement scientists as it is believed to give rise to unsoundness in PC due to the associated increase in volume [Hewlett 1998]. However, MgO will only cause cracking when its hydration occurs much later than the main hydration reactions, with its hydration rate depending strongly on the calcination temperature [Maryška & Blaha 1997]. Reactive MgO is produced by low-temperature processing of naturally occurring minerals such as magnesite (MgCO₃, <750°C) [Glasson 1963, Bláha 1995, Birchal et al. 2001] and Mg(OH)₂ (<450°C) [Shand 2006], or precipitated magnesium hydroxide (<450°C) utilising seawater or natural brines as the main source of Mg²⁺ ions [Bhatti et al. 1984, Bocanegra-Bernal 2002, Martinac et al. 2004].

The objective of this research was to critically assess the effects of reactive MgO addition on the properties and microstructure of MgO-PC-lightweight aggregates blocks.

2. EXPERIMENTAL PROGRAM

2.1 Materials

A reactive MgO supplied by Causmag International (grade XLM) was used. The reactivity was determined using the citric acid test in which the time taken for the MgO sample to neutralise a citric acid solution was determined [Van der Merwe et al. 2004]. The XLM sample gave a value of 16s and times of less than 60s are normally regarded as indicating highly reactive MgO.

A normal hardening PC (CEM I 42.5 according to European Standard EN197-1, Blue Circle portland cement) was also used. The lightweight aggregates (LWA) used were sintered fly ash aggregates (Lytag aggregates) supplied by Cemex Lytag. The aggregates were supplied in two fractions; fine and coarse.

The particle size distribution of LWA aggregates was determined by sieve analysis (Fig. 1). The chemical composition of the raw materials was determined by inductively coupled plasma atomic emission
spectroscopy (ICP-AES, ARL Fisons 3580 B). The surface areas of PC and MgO were determined by N2 absorption (Coulter Omnisorp 100) using the Brunauer-Emmett-Teller (BET) method. Table 1 shows the chemical composition, and surface area values of the raw materials. It is clear that MgO has several times higher surface area than portland cement.

Finally, the moisture content of both fractions of the Lytag aggregates was monitored for over a month and found to be consistent at 27.5 %wt for the fine fraction and 24.2 %wt for the coarse fraction.

2.2 Sample preparation

The samples prepared were cylinders of 50 mm diameter and 100 mm length. All batches were made at an aggregate-binder ratio of 9:1. To compensate for the moisture of the Lytag aggregates, an extra amount of aggregates was added on batching, so that the dry weight of aggregates would still be in a 9:1 ratio with the binder. The binder used included PC, MgO and a 50:50% blend of these two materials. Apart from the water added by the aggregates, an extra amount of water was added equal to 4 % by weight of total dry solids (aggregates plus the binder). Various fine-coarse ratios were investigated to determine their effect on sample strength. Table 2 summarises the range of different MgO-PC-LWA mixes prepared.

The raw materials and water were mixed according to the recommendations for using lightweight aggregates [CEB/FIP 1977]:

1. The fine and coarse aggregates were first added to the mixer and mixed for 1min to homogenise;
2. Then the binder and 2/3 of the extra water were added and the sample mixed for another two minutes;
3. Afterwards the mixer was stopped to remove any material on the sides of the pan and the rest of the extra water was added. The sample was then mixed for a further two minutes.

The samples were prepared by adding a predetermined amount (315 g) of the mixture to a cylindrical mould (150 mm length/50 mm diameter) followed by compaction to 100 mm length/50 mm diameter. After compaction the samples were carefully demoulded and stored (25°C) in two different curing conditions: i) 98% RH (relative humidity) and ii) wet-dry cycling. During wet-dry cycling, samples were stored for 24hr in 98%RH followed by 60hr in ambient conditions (60% RH, 25°C). Wet-dry cycling was used to simulate the effect of outdoor sample exposure.

2.3 Test methods

Unconfined compressive strength of the samples was measured at 28 and 90 days. The samples were immersed in water for 24 hr prior to the testing date. The samples were then removed from the water bath,
surface dried and tested. To ensure a uniform bearing, each sample was placed between 4mm plywood sheets to take up surface irregularities. The load was applied at a rate of 300 kN/sec.

Samples from the compressive strength tests at 28 and 90 days were immersed in acetone to inhibit further reactions. The dried samples were crushed gently in a pestle and mortar in order to separate the reacted binder from the aggregates and then sieved through a 0.15 mm sieve to recover the binder fraction. The recovered binder fraction contained all aggregate fines smaller than 0.15 mm, and was stored in sealed plastic bags prior to subsequent analysis. Major crystalline phases present in the recovered binder were determined by X-Ray diffraction using Cu Ka radiation at an acceleration voltage of 40 kV and a current of 40 mA (Philips PW 1700).

The extent of magnesium carbonate formation was determined on 17 mg sub-samples of the recovered binder by thermogravimetric analysis (TGA, Polymer Laboratories PL-STA) using a heating rate of 20°C min⁻¹ in a N₂ atmosphere. Microstructural analysis of both internal and external parts of the fractured samples was carried out using scanning electron microscopy (JEOL JSM 5610).

The curing conditions influenced the development of compressive strength. As expected, PC-containing samples (sample A and B) achieved higher strengths in moist conditions (98%RH). It is observed that the compressive strength of the MgO-PC sample (sample B) is lower when cured in wet-dry conditions in comparison to the same sample cured in 98%RH. Replacement of PC by MgO lowers sample pH with increasing curing time [Vlasopoulos & Cheeseman, in prep. a]. Therefore, it is suggested that the lower pH values associated with the wet-dry conditions increased the carbonation rate of C-S-H thus reducing the rate of strength gain.

Figure 2 presents the compressive strength of samples A-C (100% fine Lytag aggregates) with increasing curing time at 98% RH or wet-dry conditions. It is clear that samples strength decreased with increasing replacement of PC by MgO up to 28 days curing. However, after 90 days, MgO-only sample (sample C) achieved comparable strengths to that of PC sample (sample A). Samples containing only PC (sample A) achieved their maximum strength at 28 days, with no strength increase with further curing. Increased curing times were critical for the strength development of MgO-containing samples (sample B and C), with MgO-only samples (sample C) achieving a 50% strength gain between 28 and 90 days.

The curing conditions influenced the development of compressive strength. As expected, PC-containing samples (sample A and B) achieved higher strengths in moist conditions (98%RH). It is observed that the compressive strength of the MgO-PC sample (sample B) is lower when cured in wet-dry conditions in comparison to the same sample cured in 98%RH.

Replacement of PC by MgO lowers sample pH with increasing curing time [Vlasopoulos & Cheeseman, in prep. a]. Therefore, it is suggested that the lower pH values associated with the wet-dry conditions increased the carbonation rate of C-S-H thus reducing the rate of strength gain.

Figure 3 presents the compressive strength of MgO-PC samples with percentage of coarse aggregates.
strengths were achieved when samples contained between 20–40% coarse aggregates irrespective of the curing conditions. However, it is clear that wet-dry conditions resulted in similar or marginally higher sample strengths in comparison to those achieved at 98%RH.

3.2 XRD analysis

Figure 4 illustrates the XRD spectra of samples A-C (100% fine LWA) cured for 28 days at 98%RH. PC containing samples (samples A and B) appear to be substantially carbonated with high levels of calcite (CaCO₃) and no presence of Ca(OH)₂. In MgO-only samples (sample C), small quantities of unreacted MgO are still detected. However most MgO has been hydrated to brucite (Mg(OH)₂). The XRD spectra of sample C shows no sign of formation of any of the various types of magnesium carbonates [Davies & Bubela 1973, Sawada et al. 1978, Königberger et al. 1999, Botha & Strydom 2001, 2003]. When PC was intermixed with MgO (sample B), brucite was not detected. The absence of brucite in this sample can be explained by the incorporation of Mg²⁺ ions in the C-S-H gel [Brew & Glasser 2005, Fernandez et al. 2005]. The smaller ionic diameter of Mg²⁺ ion (0.65Å) than that of Ca²⁺ ion (0.99 Å) means that Mg²⁺ can replace Ca²⁺ in various phases, as evidenced in the transformation of calcite to dolomite (CaCO₃,MgCO₃) and finally magnesite (MgCO₃) [Shand et al. 2006].

Figure 5 presents the XRD spectra of sample E (40% coarse aggregates/MgO binder) cured in wet-dry conditions up to 90 days. No significant difference is detected between the XRD spectra of sample E at 28 and 90 days. The intensity of Mg(OH)₂ peaks seems to remain constant with increasing curing time up to 90 days, although it is expected that more Mg(OH)₂ will be formed by the hydration of the remaining unreacted MgO. It is suggested that MgO-containing systems gain strength through carbonation of Mg(OH)₂ to hydrated magnesium carbonates, such as nesquehonite (MgCO₃.3H₂O), lansfordite (MgCO₃.5H₂O) and/or possibly amorphous carbonates [Harrison, 2005]. However, XRD analysis could not prove that the strength gain of sample E with increasing curing time (see Fig. 3) was due to the formation of crystalline magnesium carbonates.

3.3 SEM analysis

Figures 6 and 7 present the SEM micrographs of both the external and internal parts of sample E (40% coarse aggregates/MgO binder) cured in wet-dry conditions up to 90 days. At 28 days, the formation of sub-micron Mg(OH)₂ crystals in the internal parts of sample E is observed, which seem to develop outwardly from original MgO particles. The microstructure of the external surface of sample E seems to be covered by small blocky crystals, similar to those of hydromagnesite. Hydromagnesite has been reported to have either a i) rosette-like morphology [Dheilly et al. 1998, Mindat 2006] or a ii) prismatic/acicular (thinner than prismatic)/blocky (less elongated than prismatic) morphology [Mindat 2006, Mineral galleries 2006, Webmineral 2006].

The microstructure of sample E changes dramatically with increasing curing time (Fig. 7). At 90 days,
Figure 5. XRD spectra of MgO sample with 40% coarse aggregates cured at wet-dry conditions.

Figure 6. SEM micrograph of the external (left) and internal (right) part of sample E (40% coarse aggregates/MgO binder) cured in wet-dry conditions for 28 days.

Figure 7. SEM micrograph of the external (left) and internal (right) part of sample E (40% coarse aggregates/MgO binder) cured in wet-dry conditions for 90 days.
the internal microstructure comprises of platy-crystals showing a rosette-like morphology similar to that of hydromagnesite [Vlasopoulos & Cheeseman, in prep. B].

However, it is not sure whether the identified mineral phases are totally hydromagnesite as the XRD analysis (Fig. 5) still shows significant amounts of brucite present. The microstructure of the external surface of sample E at 90 days is different than that at 28 days. It now consists of prismatic hydromagnesite crystals and white rounded aggregates of dypingite.

When PC is intermixed with MgO (sample B) a different sample microstructure is observed. Figure 8 presents the SEM micrographs of both the external and internal parts of sample B (100% fine aggregate/MgO-PC binder) cured in wet-dry conditions up to 90 days. The internal microstructure is now dominated by the presence of acicular crystals.

The absence of Mg(OH)₂ from the XRD spectra of sample B (Fig. 4) even after 28 days, suggests that these acicular crystals are hydromagnesite. However, Beruto et al. [2005] reported that in Ca(OH)₂-Mg(OH)₂ putty-limes, Mg(OH)₂ is present in an acicular form.

3.4 Thermogravimetric analysis

Figure 9 presents the thermogravimetric analysis (TGA) and differential thermogravimetry (DTG) of sample E (40% coarse aggregates/MgO binder) cured in wet-dry conditions up to 90 days. Up to 28 days curing, a single weight loss is observed between 300–450°C, which corresponds to the decomposition of Mg(OH)₂ to MgO. However, after 90 days curing a second peak is observed in the DTG curve at around 440°C, while the associated weight loss also seems to occur in two phases.

This behaviour is believed to be associated with the decomposition of either hydromagnesite (4MgCO₃·Mg(OH)₂·4H₂O) or dypingite (4MgCO₃·Mg(OH)₂·5H₂O) formed when Mg(OH)₂ powder is subjected to 98%RH curing (Fig. 10) [Vlasopoulos & Cheeseman, 2020].
in prep. B]. The absence of a peak in the DTG curve at around 550°C (associated with the decomposition of MgCO₃), may be due to masking effects caused by the continuous weight loss of the fly ash particles included in the sample analysed. Therefore, the TGA data presented indicate ongoing Mg(OH)₂ carbonation, which may explain the high strength gain of MgO containing samples between 28 and 90 days.

4. CONCLUSIONS

1 Replacement of PC by MgO decreased sample strength up to 28 days curing. At 90 days, MgO-LWA samples achieved comparable strengths to that of PC-LWA samples.
2 Curing conditions did not influence the strength development of MgO-LWA samples.
3 XRD analysis did not reveal formation of crystalline magnesium carbonates.
4 SEM analysis showed the formation of hydromagnesite and dypingite crystals on the outer parts of MgO-LWA samples. The internal microstructure of MgO-LWA samples changed from sub-micron Mg(OH)₂ crystals at 28 days to platy-crystals with a rosette-like morphology similar to that of hydromagnesite at 90 days.
5. TGA data indicated an ongoing Mg(OH)₂ carbonation, which may explain the high strength gain of MgO-LWA samples between 28 and 90 days.

ACKNOWLEDGEMENTS

This project is a Resource Efficiency KTN project (formerly Mini-Waste Faraday Partnership) funded by the Engineering and Physical Sciences Research Council (EPSRC). We would like to acknowledge Geoff Fowler, Kenneth Mitchell, Stefan Algar, Clark Leslie, Richard Sweeney and Nick Royall for their technical help and advice.

REFERENCES


Vlasopoulos, N., & Cheeseman, C. Effect of surface area on the natural carbonation of MgO and Mg(OH)₂ powders, In prep. b.

1 INTRODUCTION

Coal combustion by-products utilisation has already been described in many applied studies [Mehta & Monteiro 1993, Berg & Neal 1998, Malhotra & Mehta 2002, Colllepardi 2003, Jaturapitakkul & Cheerarot 2003, Katz 2004, Hurt et al. 2004]. Combustion of biomass fuel is nowadays considered a pressing environmental need because of the several aspects linked to the sustainable development. Some of them can be synthesized as follow:

1. the urgent need to increase the use of renewable energy sources (RES) instead of the traditional fossil fuels;
2. the tendency to increase the number of CHP (Combined Heat and Power) plants as a feasible solution to the energy problem;
3. the global issue of the CO2 and other greenhouse gases emission reduction; and
4. the life cycle assessment of organic wastes.

Future prospects for a sustainable growth is even more stringent in requiring improved energy performance and increasing use of renewable energy sources. Due to the wide availability of the fossil fuel, RES has only occasionally been a significant part of the installed energy system. However, during the last decades, especially since the Kyoto Protocol (1997), the desirability to use of RES has risen significantly. In many industrialised regions of the European Union, RES is becoming one of the principal energy sources, mainly in the heating district with the bio-wastes production.

Heating districts have grown significantly during the 1990s. Consequently, today, the increasing use of biomass, such as wood, municipal wastes, rotation crops, agricultural residues, and other similar biomass generates a large amount of biomass ashes and offer a very promising sector for development.

Moreover, the biomass utilisation is encouraged by the EU Green Paper on Sustainable Energy [Green Paper, 2006]. The importance to tackle the energy mixture indicates:

1. all the advantages and drawbacks of different sources of energy, such as indigenous renewable energy sources from biomass and biofuels;
2. a detailed short-, medium-, and long-term plan to stabilise and gradually reduce the EU’s dependence on imported oil. This would build on the existing Biomass Action Plan2 [Comm. 2005] and the Strategy for Biofuels3 [Comm. 2006].

These strategy requires to seriously consider the life cycle assessment of bio-products at the end of its useful life (after the energy production process).

Traditionally cement manufacturing industries have offered valid solutions to solve the problem of coal
fly ash by means of their use in structural concrete [Helmuth 1987, Collepardi 1991, Malhotra & Ramezanianpour 1994, Malhotra 2001, Naik at al. 2003, Borsoi et al. 2006]. Currently, similar destination is envisioned for biomass ash. However, the practical feasibility needs to be carefully investigated even if, national regulations, as well as EU directives, allow the reuse of biomass ash in the building sector through simplified procedures [D.Lgs. 1997].

Within this context, this study aims to contribute to this important issue and reports on the leaching behaviour and environmental impact of concretes manufactured with biomass ashes. Their mechanical behaviour has already been considered by Corinaldesi & Moriconi [2004], Sani et al. [2003] and Fava & Monosi [1999].

2 MATERIAL CHARACTERIZATION

Seven different biomass ashes from CHP (Combined Heat and Power) plants were utilised as a supplementary material in structural concrete specimen. Extended chemical and physical characterisation of the biomass ash used have been reported in Sforza et al. [2005]. Ashes were characterized for density [Antocico 2002], specific surface by Blaine test, and fineness by wet sieving under standard conditions [UNI EN 451-2:1996; UNI EN 196-6:1991; UNI EN 1097-7:2000]. The total surface areas obtained by Blaine test resulted in a scattered data set because of the nature of the ashes. The residual combustible matter was determined on as-received ash samples using a loss on ignition (LOI) measurement [UNI EN 196-2:2005]. Values are presented in Table 1.

The LOI results are high for the ashes 3, 6, and 8 with values exceeding 10% [ASTM C618-92a 1994, Dockter & Eylands 2003].

Ashes suitability as a concrete mineral additive has been studied. Wang & Baxter [2007], Ahn et al. [1999], and Lucas et al. [2002] showed the negative effects on mechanical properties and on concrete paste integrity that resulted in a greater tendency to leaching behaviour.

Some corrective actions have been reported by properly designing paste mixture proportions, as suggested by Moriconi et al. [2004]. In other cases mineral ashes have been used as fine aggregate or in partial substitution of cement [Fava & Monosi 1999].

Morphology properties were obtained by SEM-EDS Analysis, because of their relevance in concrete performances [Malhotra & Mehta 2002]. In general, for the seven ashes, microscopic examination revealed that the inorganic portion of the ash samples consisted predominantly of glassy spheres, mainly aluminium and silicon, rarely iron. The smaller particles contained calcium and potassium, sometimes magnesium, sodium, and chloride also. Most of the particles are highly macro-porous and heterogeneous in shape and texture. Many appearing in the form of distinct, fused fragments, nearly spherical or rectangular as char particles. In particular ashes 1, 2, and 7 have strong analogies with carbon particles; 7 is a bottom ash with a high fraction of coarse particles. Ashes 3, 4, 6, and 8 exhibited a greater heterogeneity due to the biomass source combination (other details are in Sforza et al. [2005]).

A cement type CEM-II/A-L 32.5 R (according to UNI EN 197-1:2001) and natural aggregates were utilised for concrete specimen manufacturing. Natural aggregates employed had standard physical properties as reported in Table 2 and Figure 1 [UNI EN 1097-6:2002, UNI EN 933-1:1999].

Table 1. Chemical and physical properties of biomass ashes.

<table>
<thead>
<tr>
<th>ASH</th>
<th>Passing %</th>
<th>Fineness %</th>
<th>Specific surface m²/g</th>
<th>Bulk density g/cm³</th>
<th>LOI %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>74</td>
<td>26</td>
<td>0.628</td>
<td>2.405</td>
<td>6.2</td>
</tr>
<tr>
<td>2</td>
<td>16.5</td>
<td>83.5</td>
<td>0.135</td>
<td>2.044</td>
<td>2.9</td>
</tr>
<tr>
<td>3</td>
<td>59</td>
<td>41</td>
<td>0.294</td>
<td>2.554</td>
<td>17.9</td>
</tr>
<tr>
<td>4</td>
<td>23.5</td>
<td>76.5</td>
<td>0.135</td>
<td>2.460</td>
<td>9.8</td>
</tr>
<tr>
<td>6</td>
<td>55.5</td>
<td>45.5</td>
<td>0.464</td>
<td>2.227</td>
<td>17.0</td>
</tr>
<tr>
<td>7</td>
<td>5</td>
<td>95</td>
<td>–</td>
<td>2.468</td>
<td>8.6</td>
</tr>
<tr>
<td>8</td>
<td>33.5</td>
<td>66.5</td>
<td>0.212</td>
<td>2.565</td>
<td>11.8</td>
</tr>
</tbody>
</table>

Table 2. Particle density, water absorption, and fineness modulus of aggregates.

<table>
<thead>
<tr>
<th></th>
<th>Bulk specific gravity g/cm³</th>
<th>SSD %</th>
<th>Fineness modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>sand 0/2</td>
<td>2.65</td>
<td>0.7</td>
<td>1.86</td>
</tr>
<tr>
<td>sand 3/6</td>
<td>2.72</td>
<td>1.4</td>
<td>3.35</td>
</tr>
<tr>
<td>gravel 7/15</td>
<td>2.74</td>
<td>1.3</td>
<td>6.60</td>
</tr>
<tr>
<td>gravel 15/30</td>
<td>2.73</td>
<td>1.2</td>
<td>7.80</td>
</tr>
</tbody>
</table>

Figure 1. Natural aggregates: particle size distribution.
3 EXPERIMENTAL METHODOLOGY

Concrete specimens were manufactured as described in Sforza & Moriconi [2006]; and the mixtures proportions are reported in Table 3.

Test samples prepared were concrete specimens of standard dimension (15 × 15 × 15 cm), cured for 28 days in a room with 100% relative humidity.

They were manufactured with biomass ashes, natural aggregate, and normal portland cement (CEM-II/A-L 32.5 R).

The heavy metal character of the ash composition (As, Ba, Be, Cd, Co, Cr, Cu, Ni, Pb, V, Zn) were analysed using ICP-AES (Perkin Elmer Optima 3200XL).

Dynamic leaching tests were conducted using a simplified procedure. This procedure follows Italian regulations for reuse of no-toxic waste materials as by-products [D.M.A. 1998]. Examples of similar international standard test include ISO 6961:82 and ASTM C1220:92.

According to the extraction protocol of the Italian regulations, a sample is placed in contact with a precise amount of fluid for predetermined extent of time. Sample is placed in a test vessel, with enough space around, and a leachant is added and allowed to wet the sample all around. The solid to liquid ratio, expressed as the ratio of volume of solid to volume of the leachant, is 1:5.

The leachant solution is renewed to drive the leaching process. At each renewing sequence, the fluid is collected for analysis.

The renewing sequence is: 2, 8, 24, 48, 72, 102, 168, and 384 hours.

The leachant used was fresh distilled water with pH ranging around 6 and electrical conductivity around 3 mS/cm, at 20°C ± 4°C.

Because the physical integrity of the sample matrix is maintained during the test, the concrete-specimen/leachant properties affect how much material can be leached out as a function of time. In particular, the surface area reactivity of the test sample, more than the extraction force of the leachant, provides leached contaminants in the water, and the kinetic information about the dissolution process.

Metal released cumulated values were then compared with the Italian code requirements [D.M.A. 1998] in order to verify the Italian environmental standards. Additionally, in order to follow the release mechanisms, pH, Electrical Conductivity, and temperature of the leachant solution were recorded. Total alkali cumulated curves are plotted as a function of time and related to both specific surface area and bulk density of the test specimens. Curves are also compared with the pores distribution of specimens.

Additional test to determine the average pore size distribution were performed by Mercury Intrusion Porosimetry (MIP) [ASTM D4404-84], using a small amount of sample after leaching test (Table 4).

The total porosity is split in two components [Mehta & Monteiro 1993]: the micro- and mesopores and the macro-pores. Both components were used in the evaluation of the parameters driving the leaching process. The assumption used to split the total porosity fully deal with the specific surface values. A slight difference comes out in Sample B manufactured without biomass ash. The bulk density parameter seems not very sensitive to differentiate sample’s microstructure (see for example ashes 2 and 3) while it is likely that solid content influenced leaching behaviour less than microstructure.

4 RESULTS

Heavy metal leachates result fully conform to the Italian legal standards. Concrete specimens manufactured with or without Biomass ashes show comparable releases, shown in Table 5 as cumulative values. The morphology properties of the ashes (as determined and never changed during the experimental period) and the mechanical properties of the matrix, guarantee a satisfactory encapsulation of the heavy metals contained in the ashes [Côté et al. 1987], over

<table>
<thead>
<tr>
<th>Sample</th>
<th>B</th>
<th>ASH 1</th>
<th>ASH 2</th>
<th>ASH 3</th>
<th>ASH 4</th>
<th>ASH 6</th>
<th>ASH 7</th>
<th>ASH 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>sand 0/2 mm</td>
<td>270</td>
<td>242</td>
<td>237</td>
<td>234</td>
<td>241</td>
<td>247</td>
<td>247</td>
<td>241</td>
</tr>
<tr>
<td>sand 0/6 mm</td>
<td>690</td>
<td>530</td>
<td>520</td>
<td>513</td>
<td>527</td>
<td>541</td>
<td>540</td>
<td>529</td>
</tr>
<tr>
<td>coarse 7/15 mm</td>
<td>613</td>
<td>551</td>
<td>541</td>
<td>533</td>
<td>547</td>
<td>562</td>
<td>561</td>
<td>550</td>
</tr>
<tr>
<td>coarse 15/30 mm</td>
<td>370</td>
<td>333</td>
<td>327</td>
<td>322</td>
<td>332</td>
<td>380</td>
<td>339</td>
<td>332</td>
</tr>
<tr>
<td>cement</td>
<td>400</td>
<td>400</td>
<td>400</td>
<td>400</td>
<td>400</td>
<td>400</td>
<td>400</td>
<td>400</td>
</tr>
<tr>
<td>ash</td>
<td>–</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>water</td>
<td>186</td>
<td>220</td>
<td>210</td>
<td>223</td>
<td>205</td>
<td>190</td>
<td>190</td>
<td>205</td>
</tr>
<tr>
<td>additive</td>
<td>1</td>
<td>1,1</td>
<td>1,2</td>
<td>2,2</td>
<td>1,5</td>
<td>1,8</td>
<td>1,3</td>
<td>1,2</td>
</tr>
<tr>
<td>w/c (→)</td>
<td>0,47</td>
<td>0,55</td>
<td>0,53</td>
<td>0,56</td>
<td>0,51</td>
<td>0,48</td>
<td>0,48</td>
<td>0,51</td>
</tr>
<tr>
<td>w/(c+ash) (→)</td>
<td>–</td>
<td>0,44</td>
<td>0,42</td>
<td>0,45</td>
<td>0,41</td>
<td>0,38</td>
<td>0,38</td>
<td>0,41</td>
</tr>
</tbody>
</table>
the investigation time. The leachates, alkaline throughout the testing period with pH values greater than 10, indicate that the interstitial pore fluid in contact with hydrated cementitious materials is buffered by the presence of alkaline ions. So far as the local condition of the leachant remain unchanged, the heavy metals leaching remains very low, as found in previous works [Sani et al. 2004, Carrescia et al. 2004, Naik et al. 2001].

The behaviour of the concrete manufactured with biomass ashes may be observed with the electrical conductivity of the leachate, given in cumulated concentration (Figure 2) versus time. Graphs reproducing the release curves are split in two parts: release with electrical conductivity greater than the reference sample (B, without ash) and lower than the reference sample. Ashes 1, 2, and 8 (Figure 2b) show comparable cumulative conductivity, while the remaining ashes provided releases in some cases double than the reference specimen (Figure 2a). Because of their morphology such results were expected for ashes 1 and 2, but not for ash 8. Similar considerations are feasible for ashes 6 and 7, which were quite heterogeneous in shape and composition.

Curve shapes point to a diffusion-like process as the prevailing leaching mechanism, except for the first hours of the leaching test, in which the release was primarily by a surface washing, though not substantially influencing the final process.

In terms of factors that can be related to the diffusion-like process, micro-pore ratio seems more likely than bulk density, especially for those specimens that have a micro-pores ratio higher than the reference sample.

This aspect becomes more evident by expressing the ion concentration (meq/l) in terms of total mass released, per unit of mass (µeq/g) and per unit of total pore area (µeq/m²). Sample volume mass does not seem to really specify the amount of release. Nor clear trends or relationships appear when expressed in terms of particle bulk density (Figure 3a). Both remarks can probably apply that a greater number of solid particle can lead to a greater release, as well as a greater uniform matrix can lead to a lower release.

On the contrary, cumulated fractional release expressed as a function of the specific surface, shows a more promising relationship (Figure 3b). High specific surface deals with high micro-pores content (see Table 4), and, therefore, the leaching process seems uncontrolled by macro-porous structures of the concrete samples. Even though this may appear incongruous, this observation is in accord with the structural properties of the mixtures since a well defined concrete matrix implies good micro-pores distribution (not necessary low percentage). Consequently a difficulty for the leachant fluid to penetrate the matrix is understandable.

The environmental load of biomass containing concretes may be expressed by a total ions release from the specimen. Leachability can then be empirically represented by two parameters calculated from the following equation 1:

\[
\frac{C}{C_0} = b t^n
\]

where the relative concentration, \(C/C_0\) is expressed as the final versus the initial concentration, \(b\) is the value
of $C/C_0$ at time $t = 1\, \text{h}$, and $n$ is the time exponent. As demonstrated by the leaching kinetics, the ion(s) which could leach from the solid phase can be shown to follow the Fick’s law, according to a diffusion-controlled mechanism, when $n$ is equal to 0.5, under the condition of a semi-infinite medium. The results obtained with the leaching tests are presented in Figure 4 and Table 6.

The resistance to the leaching is the most important factor for the evaluation of the immobilised matrices, because water would be the primary liquid available for the potential dispersion for the heavy-metal ions. For this reason an attempt was made to estimate the leaching parameters $b$ and $n$ through regression analysis based on the mechanical performance. As shown in Figure 4 a correlation was found between the compressive strength and $b$ times $n$. A linear model is proposed as given below $1/\text{MPa} = 0.017^*b^*n + 0.020$ ($R^2 = 0.977$).

The model adopted agrees to estimate $b\cdot n$ for biomass concrete specimen cured for 28 days. Another biomass ash property, the loss on ignition (LOI %), was found to be well related to the leaching parameter $b$. Figure 5 shows the relation and using a least square technique, the following multiplicative relationship was found: LOI = $8.68^*b^0.64$ ($p<0.05$).
5 CONCLUSION

A rational approach is necessary for relating the maximum permissible replacement of traditional materials with construction-quality secondary materials in buildings, with the need to consider, besides the mechanical properties also the environmental requirements.

The Italian waste control regulation recognizes the concrete as a technology for the safe disposal of wastes providing that leachates satisfy the enforced standards. The concrete specimen manufactured with biomass ashes containing heavy metals did not manifest a release higher than the Italian standards and the pH leachates remained alkaline throughout the testing period. The methodology employed for establishing the leaching in concrete is simple, inexpensive, and is amenable to evaluate a complete leachability. Plots of total ion concentrations versus time were used to establish the leachability through two fitting parameters. The total ions environmental load results related to the unconfined compressive strength and to the loss on ignition of the biomass used.

Finally it can be postulated the possibility to describe the environmental load through the compressive strength of the specimen via the empirical relationship with the $b$ and $n$ calculated parameters and the LOI of the biomass ashes.

REFERENCES


Table 6. Leachability parameters.

<table>
<thead>
<tr>
<th>ASH</th>
<th>b [µg/g h]</th>
<th>n</th>
<th>$R^2$ (1)</th>
<th>LOI [%]</th>
<th>Unconfined compressing strength [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>0.39</td>
<td>-</td>
<td>99.96</td>
<td>4.0</td>
<td>41.2</td>
</tr>
<tr>
<td>ASH 1</td>
<td>0.30</td>
<td>0.67</td>
<td>99.96</td>
<td>6.2</td>
<td>40.0</td>
</tr>
<tr>
<td>ASH 2</td>
<td>0.44</td>
<td>0.79</td>
<td>99.93</td>
<td>2.9</td>
<td>40.2</td>
</tr>
<tr>
<td>ASH 3</td>
<td>2.87</td>
<td>0.55</td>
<td>99.85</td>
<td>17.9</td>
<td>21.0</td>
</tr>
<tr>
<td>ASH 4</td>
<td>2.23</td>
<td>0.59</td>
<td>99.74</td>
<td>9.8</td>
<td>23.7</td>
</tr>
<tr>
<td>ASH 6</td>
<td>1.61</td>
<td>0.60</td>
<td>99.70</td>
<td>17.0</td>
<td>29.2</td>
</tr>
<tr>
<td>ASH 7</td>
<td>1.06</td>
<td>0.60</td>
<td>99.74</td>
<td>8.6</td>
<td>31.8</td>
</tr>
<tr>
<td>ASH 8</td>
<td>0.97</td>
<td>0.70</td>
<td>99.96</td>
<td>11.8</td>
<td>33.9</td>
</tr>
</tbody>
</table>

(1) Obtained with 8 experimental data points.
D.M.A. 5 febbraio 1998. Individuazione dei rifiuti non peri-
doe.gov/technologies/coalpower/cwr/coal_utilization_by
products/pdf/40907FPdf>. (July 24, 2006)
doe.gov/technologies/coalpower/cwr/coal_utilization_by
products/pdf/40907FPdf>. (July 24, 2006)
ISO 6961:1982 Long-term leach testing of solidified radioactive waste forms
Malhotra, V.M. 2001. Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete. ACI SP-199, American Concrete Institute, Farmington Hills, Michigan, U.S.A.
UNI EN 197-1:2001 Cement – Composition, specifications and conformity criteria for common cements.
Wang, S. & Baxter, L. 2007 Quantitative kinetics of pozzolanic reaction and strength build-up in the mortar of Biomass/Coal Fly ash and Ca(OH)2. Fly ash, silica fume, slag and natural pozzolans in concrete; Proc. 9th CANMET/ACI, intern. conf., Warsaw, Poland. Farmington Hills, MI, USA: American Concrete Institute.
The application of paper sludge ash to extremely stiff consistency concrete product

H. Fujiwara & M. Maruoka
Department of Architecture and Civil Engineering, Faculty of Engineering, Utsunomiya University, Japan

K. Koibuchi
DC CO., LTD, Japan

K. Fujita
Department of Architecture and Civil Engineering, Graduate School of Engineering, Utsunomiya University, Japan

ABSTRACT: The main ingredients in paper sludge ash (PS ash), which is discharged by the paper industry, are SiO₂ and Al₂O₃. Therefore, the pozzolan reaction can be expected when it is added to concrete. However, its water absorptivity is so high that the plasticity of the resulting concrete is reduced. In this paper, the applicability of PS ash to extremely stiff concrete is evaluated by adding PS ash to cement and fine aggregate up to 300 kg/m³. A compressive strength of up to two times that of the original concrete was obtained.

1 INTRODUCTION

Most of the waste from the paper industry is known as paper sludge (hereafter, PS). PS is produced from minute fiber, wastepaper foreign material and fillings such as talc and kaolin that could not be converted to paper. PS is burnt and becomes PS ash. This is used as a soil improvement material and cement raw material or an iron manufacture heat insulator. The purpose of this study is technical development which can make good use of PS ash in large quantities in the field of concrete. Therefore, this study examined a strength characteristic of concrete when PS ash was substituted step-by-step for fine aggregate or cement in mortar and concrete. In addition, its water absorptivity is so high that the plasticity of the resulting concrete is reduced. Therefore this study applied PS ash to extremely stiff concrete by applying it to ILB.

2 EXPERIMENT: MORTAR EXPERIMENT

2.1 Experiment outline

This chapter discusses the strength characteristic when PS ash was mixed with mortar step-by-step: (1) strength characteristic when PS ash was substituted for cement; (2) strength characteristic when PS ash was substituted for fine aggregate with a fixed cement amount; and (3) strength characteristic when PS ash was substituted for both cement and fine aggregate.

This chapter also discusses the possibility of using a large amount of PS ash in a mortar level by referring to these experiments.

2.2 Materials

The materials used are shown in Table 1 (coarse aggregate). In addition, the chemical ingredients of the PS ash used in this study are shown in Table 2. The particle distribution of PS ash is shown in Fig. 1. In the PS ash used for this study, about 60% of the ingredients were silica (SiO₂) and alumina (Al₂O₃). Therefore, PS ash can be expected to undergo a pozzolan reaction with calcium hydroxide due to hydration of cement. A SEM photograph of PS ash is shown in Fig. 2. It is seen from this photograph that the particles of PS ash do not include spherical particles and have a rough shape.

2.3 Mixing proportions

The mixing proportions are shown in Table 3. All experiments maintained a water pulverulent ratio (W/P) of 30% and pulverulent fine aggregate ratio (C:S) of 1:1. The percentage of PS ash in the cement

characteristic when PS ash was substituted for cement; (2) strength characteristic when PS ash was substituted for fine aggregate with a fixed cement amount; and (3) strength characteristic when PS ash was substituted for both cement and fine aggregate.

This chapter also discusses the possibility of using a large amount of PS ash in a mortar level by referring to these experiments.
A (PS/C) mixture by volume was at five levels of substitution, i.e., 0, 20, 40, 60 and 80% (Series 1). The percentage of PS ash in the fine aggregate (PS/S) mixture by volume was at three levels of substitution, i.e., 10, 20 and 30% (Series 2). In the cement and fine aggregate substitution series (Series 3), the percentage of PS ash in the cement (PS/C) mixture by volume was at three levels, i.e., 30, 40, 50%. Also, the percentage of PS ash in the fine aggregate (PS/S) mixture by volume was at five levels, i.e., 0, 10, 20, 30 and 40%. In total, 15 types of mortar with different mixing proportions were tested. In addition, the PS ash was treated as fine aggregate when PS ash was substituted for fine aggregate.

For these experiments, the flow value of the test cement was specified within 130 ± 20 mm and the air content was within 2.0 ± 1.0%.

### Table 1. Materials.

<table>
<thead>
<tr>
<th>Materials</th>
<th>Symbol</th>
<th>Type of materials</th>
<th>Density (g/cm³)</th>
<th>Water absorption(%)</th>
<th>F.M.</th>
<th>Specific surface area(cm²/g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>C</td>
<td>Normal portland cement</td>
<td>3.15</td>
<td>–</td>
<td>–</td>
<td>3,260</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>RS</td>
<td>River sand</td>
<td>2.56</td>
<td>3.01</td>
<td>2.93</td>
<td>–</td>
</tr>
<tr>
<td>Coarse aggregate</td>
<td>G5</td>
<td>Crushed stone No.5</td>
<td>2.64</td>
<td>0.38</td>
<td>6.45</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>G6</td>
<td>Crushed stone No.6</td>
<td>2.66</td>
<td>0.49</td>
<td>7.07</td>
<td>–</td>
</tr>
<tr>
<td>Admixture</td>
<td>PS</td>
<td>Paper sludge Ash</td>
<td>2.27</td>
<td>20.63</td>
<td>–</td>
<td>1,360</td>
</tr>
<tr>
<td>Chemical admixture</td>
<td>SP</td>
<td>High-range water-reducing admixture</td>
<td>1.05</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

### Table 2. Chemical ingredients.

<table>
<thead>
<tr>
<th>PS ig-loss</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>CaO</th>
<th>MgO</th>
<th>Na₂O</th>
<th>K₂O</th>
<th>TiO₂</th>
<th>P₂O₅</th>
<th>MnO</th>
<th>TS</th>
<th>R₂O (%)</th>
<th>Cl (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.6</td>
<td>38.1</td>
<td>21.4</td>
<td>1.1</td>
<td>28.9</td>
<td>0.39</td>
<td>0.15</td>
<td>1.03</td>
<td>0.30</td>
<td>0.02</td>
<td>0.5</td>
<td>99.1</td>
<td>0.49</td>
<td>0.178</td>
</tr>
</tbody>
</table>

### Figure 1. Particle size distribution.

(PS/C) mixture by volume was at five levels of substitution, i.e., 0, 20, 40, 60 and 80% (Series 1). The percentage of PS ash in the fine aggregate (PS/S) mixture by volume was at three levels of substitution, i.e., 10, 20 and 30% (Series 2). In the cement and fine aggregate substitution series (Series 3), the percentage of PS ash in the cement (PS/C) mixture by volume was at three levels, i.e., 30, 40, 50%. Also, the percentage of PS ash in the fine aggregate (PS/S) mixture by volume was at five levels, i.e., 0, 10, 20, 30 and 40%. In total, 15 types of mortar with different mixing proportions were tested. In addition, the PS ash was treated as fine aggregate when PS ash was substituted for fine aggregate. For these experiments, the flow value of the test cement was specified within 130 ± 20 mm and the air content was within 2.0 ± 1.0%.

### 2.4 Test items

The following tests were carried out for all concrete.

Flow test for mortar (JIS R5201)
Air content test (JIS A1116)
Compressive strength test (JIS A 1108)

### 2.5 Test results and discussion

#### 2.5.1 SP addition amount

The relationship between the substitution amount and relative addition amount rate are shown in Fig. 3, Fig. 4 and Fig. 5. In both substitution methods, the SP addition amount was increased to obtain a predetermined mortar flow value with an increase in PS ash amount.

Even if the SP addition amount was increased to a cement substitution of more than 60% in the cement substitution series, an effect was not confirmed.
Therefore mortar of a predetermined fluidity could not be obtained.

2.5.2 Compressive strength test results

a) Series 1

The relationship between compressive strength and % of cement substitution is shown in Fig. 6. For mortar with a PS substitution amount of 20%, mortar having a compressive strength equal to that with no PS ash was obtained, as shown in Fig. 6. However, for mortar with a PS substitution amount greater than 20%, it was confirmed that the compressive strength deteriorated linearly with an increase in PS substitution amount. This is because calcium hydroxide, which is necessary for the pozzolan reaction, was not generated sufficiently due to the fact that the cement amount decreases with increase in PS. It is thought...
that it is due to the fact that the pozzolan reaction did not occur to a sufficient extent.

b) Series 2
The relationship between compressive strength and % of fine aggregate substitution is shown in Fig. 7. For mortar with a substitution amount of 30%, it was confirmed that the compressive strength increased linearly with the substitution amount of PS ash. One of the reasons is that the pozzolan reaction was caused by SiO2 and Al2O3, which are chief ingredients of PS ash and calcium hydroxide, which is produced by hydration of cement. Another reason is the fineness of the structure due to the increase in the amount of pulverized material.

c) Series 3
The relationship between compressive strength and % of fine aggregate substitution is shown in Fig. 8. In addition, the relationship between compressive strength and PS ash amount is shown in Fig. 9. It is seen from Fig. 9 that when the substitution amount of fine aggregate was changed for the same cement amount, there was a peak in compressive strength. For both peaks, the PS ash amount was 462 kg/m³, and both showed a value that was close to the compressive strength 61 N/mm² for mortar with no PS ash. It appears that strength decreased because poor filling occurred, so workability deteriorates slowly when the PS ash amount exceeds 462 kg/m³. From these results, it appeared that there was an upper limit value to the use of PS ash.

3 EXPERIMENT: EXTREMELY STIFF CONCRETE EXPERIMENT

3.1 Experimental outline
When PS ash is applied to concrete with a slump, the fluidity of the concrete deteriorates because the water
absorption rate is high. Therefore, for concrete with PS ash, the strength characteristic was examined when PS ash was partly substituted for cement and fine aggregates in low fluidity, extremely stiff concrete.

### 3.2 Materials
The materials used are shown to Table 1.

### 3.3 Mixing Proportions

The mixing proportion is shown in Table 4, and the mixing proportion condition is shown in Table 5. Based on instant stripping concrete, which is generally used, the percentage of PS ash in the cement (PS/C) mixture by volume was at six levels, i.e., 0, 10, 20, 30, 40 and 50%. Also, the percentage of PS ash in the fine aggregate (PS/S) mixture by volume was at five levels,
i.e., 0, 10, 20, 30, 40 and 50%. In total, 36 types of mortar with different mixing proportions were tested. The molding method was vibration pressurization with molding. The SP addition amount was adjusted so that molding and instant stripping could both be performed. The upper limit was taken to be 5%. The case where molding was not possible even if 5% was added was not examined. Because molding of concrete was impossible at a PS ash mixing proportion of more than 300 kg/m³, it appeared that there is a limit to the PS ash addition amount which is determined by formability.

In addition, a difference occurred to the limit of the PS ash addition amount when compared with mortar. However, when some ingredients of the mortar were removed, the compressive strength was 489 kg/m³, which almost agrees with the result of mortar.

### 3.4 Test items

The following tests were carried out for all concrete.

- **Compressive strength test (JIS A 1108)**
- **Compatibility test method for extremely dry concrete (JSCE-F 508)**

The following coefficients are provided from a compactibility test. $C_i$ is the initial coefficient of deformation (%), $C_f$ is the maximum achievable coefficient of deformation (%), $E_{98}$ is the compaction energy that is necessary to reach a filling rate of 98% (J/l), $C_e$ is the increase ratio of filling rate per unit energy in the early stage of compaction which gave 1 J/l. Curing involved steam curing. The condition of steam curing was the same as in Experiment 1. The compressive strength was measured just after steam curing (material age 1 day).

### 3.5 Test results and discussion

#### 3.5.1 Fresh concrete test result

The compactibility test result is shown in Table 7. The relationship between $E_{98}$ and PS ash substitution amount is shown in Fig. 10. It is seen that $E_{98}$ increased as the PS ash substitution amount increased for the same mixing proportion from Fig. 10. The increase is not linear because the SP addition amount is not constant.

<table>
<thead>
<tr>
<th>Coarse aggregate greatest dimension (mm)</th>
<th>Air (%)</th>
<th>W (kg)</th>
<th>W/C(%)</th>
<th>s/a(%)</th>
<th>Unit content (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>1</td>
<td>169</td>
<td>46.9</td>
<td>45</td>
<td>169 360 783 1023</td>
</tr>
</tbody>
</table>

Table 6. Formability of concrete.

<table>
<thead>
<tr>
<th>$V_{ps}/(V_c + V_{ps})$ (%)</th>
<th>0</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_{ps}/V_{fa}$ (%)</td>
<td>0</td>
<td>10</td>
<td>20</td>
<td>30</td>
<td>40</td>
<td>50</td>
</tr>
<tr>
<td>0</td>
<td>•</td>
<td>•</td>
<td>•</td>
<td>•</td>
<td>•</td>
<td>•</td>
</tr>
<tr>
<td>10</td>
<td>•</td>
<td>•</td>
<td>•</td>
<td>•</td>
<td>•</td>
<td>•</td>
</tr>
<tr>
<td>20</td>
<td>•</td>
<td>•</td>
<td>•</td>
<td>•</td>
<td>•</td>
<td>•</td>
</tr>
<tr>
<td>30</td>
<td>•</td>
<td>•</td>
<td>•</td>
<td>•</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>40</td>
<td>•</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>50</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
</tbody>
</table>

However, it was confirmed that compactibility was poor because the PS ash substitution amount increased. In addition, $E_{98}$ suddenly increased for a mixing proportion where the PS ash amount was almost 300 kg/m³. From the results shown in Table 6, it is thought that a PS ash amount of 300 kg/m³ is the limit of PS ash addition. However, in a general instant stripping product, $C_f$ does not exceed 98%. Therefore it appears that this can be applied to an instant stripping product in the combination of Experiment 2.

#### 3.5.2 Compressive strength test results

The relationship between fine aggregate substitution amount, cement substitution amount and compressive strength is shown in Fig. 11. Looking at the level of compressive strength in Fig. 11, a compressive strength of less than 30.79 N/mm² for a mixture with no PS ash was a combination with a cement amount of less than 200 kg/m³ or a PS ash amount of less than 100 kg/m³. The compressive strength increased for a mixture with no PS ash. When it was equal to or less than a cement amount of 200 kg/m³, little calcium hydroxide was generated in the case of a hydration reaction. Therefore, it appears that the pozzolan reaction did not occur to a sufficient extent due to the fact that the compressive strength did not increase. When PS ash was mixed with concrete, for a cement amount of 200 kg/m³ in the range of PS ash from 100 kg/m³ to 300 kg/m³, it appears that concrete with a higher compressive strength than for concrete with no PS ash is obtained. In particular, concrete with a compressive strength of more than 1.5 times that of concrete with no PS ash was obtained for a cement amount of 300 ± 20 kg/m³ and a PS ash amount of 200 ± 40 kg/m³.
Table 7. Test results.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Unit cemento content (kg/m³)</th>
<th>Unit PSash content (kg/m³)</th>
<th>SP (%)</th>
<th>Air (%)</th>
<th>Ci (%)</th>
<th>Cf(%)</th>
<th>E98</th>
<th>Ce</th>
<th>Compressive strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>C0-S0</td>
<td>360</td>
<td>0</td>
<td>0.7</td>
<td>1.92</td>
<td>100.0</td>
<td>103.2</td>
<td>0.00</td>
<td>0.00</td>
<td>30.79</td>
</tr>
<tr>
<td>C0-S10</td>
<td>68</td>
<td>0.8</td>
<td>2.00</td>
<td>94.6</td>
<td>99.99</td>
<td>103.5</td>
<td>0.00</td>
<td>0.09</td>
<td>41.93</td>
</tr>
<tr>
<td>C0-S20</td>
<td>136</td>
<td>1.1</td>
<td>1.14</td>
<td>84.79</td>
<td>99.99</td>
<td>13.05</td>
<td>2.51</td>
<td>61.07</td>
<td>15.56</td>
</tr>
<tr>
<td>C0-S30</td>
<td>204</td>
<td>3.5</td>
<td>2.87</td>
<td>79.46</td>
<td>99.83</td>
<td>13.05</td>
<td>2.51</td>
<td>61.07</td>
<td>15.56</td>
</tr>
<tr>
<td>C0-S40</td>
<td>272</td>
<td>4.0</td>
<td>3.74</td>
<td>80.64</td>
<td>99.94</td>
<td>91.06</td>
<td>1.98</td>
<td>67.82</td>
<td>15.56</td>
</tr>
<tr>
<td>C0-S50</td>
<td>340</td>
<td>5.0</td>
<td>14.86</td>
<td>72.04</td>
<td>98.63</td>
<td>–</td>
<td>0.87</td>
<td>24.27</td>
<td>15.56</td>
</tr>
<tr>
<td>C10-S0</td>
<td>324</td>
<td>32</td>
<td>0.6</td>
<td>2.32</td>
<td>100.0</td>
<td>103.6</td>
<td>0.00</td>
<td>0.00</td>
<td>28.40</td>
</tr>
<tr>
<td>C10-S10</td>
<td>94</td>
<td>1.0</td>
<td>2.10</td>
<td>100.0</td>
<td>101.7</td>
<td>0.00</td>
<td>0.09</td>
<td>40.29</td>
<td>15.56</td>
</tr>
<tr>
<td>C10-S20</td>
<td>162</td>
<td>1.5</td>
<td>1.73</td>
<td>79.82</td>
<td>99.99</td>
<td>10.47</td>
<td>2.72</td>
<td>53.17</td>
<td>15.56</td>
</tr>
<tr>
<td>C10-S30</td>
<td>230</td>
<td>3.5</td>
<td>3.24</td>
<td>79.3</td>
<td>98.56</td>
<td>80.94</td>
<td>1.67</td>
<td>53.17</td>
<td>15.56</td>
</tr>
<tr>
<td>C10-S40</td>
<td>298</td>
<td>5.0</td>
<td>1.84</td>
<td>79.81</td>
<td>98.63</td>
<td>585.92</td>
<td>1.46</td>
<td>58.47</td>
<td>15.56</td>
</tr>
<tr>
<td>C10-S50</td>
<td>366</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>15.56</td>
</tr>
<tr>
<td>C20-S0</td>
<td>288</td>
<td>52</td>
<td>0.7</td>
<td>1.66</td>
<td>96.1</td>
<td>122.8</td>
<td>23.13</td>
<td>0.19</td>
<td>29.60</td>
</tr>
<tr>
<td>C20-S10</td>
<td>120</td>
<td>1.0</td>
<td>0.94</td>
<td>88.6</td>
<td>99.9</td>
<td>12.50</td>
<td>0.01</td>
<td>39.13</td>
<td>15.56</td>
</tr>
<tr>
<td>C20-S20</td>
<td>188</td>
<td>3.0</td>
<td>1.62</td>
<td>80.43</td>
<td>99.87</td>
<td>61.13</td>
<td>2.26</td>
<td>53.17</td>
<td>15.56</td>
</tr>
<tr>
<td>C20-S30</td>
<td>256</td>
<td>5.0</td>
<td>0.30</td>
<td>79.91</td>
<td>100.04</td>
<td>585.92</td>
<td>1.46</td>
<td>58.47</td>
<td>15.56</td>
</tr>
<tr>
<td>C20-S40</td>
<td>324</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>15.56</td>
</tr>
<tr>
<td>C20-S50</td>
<td>366</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>15.56</td>
</tr>
<tr>
<td>C30-S0</td>
<td>252</td>
<td>78</td>
<td>0.8</td>
<td>1.18</td>
<td>96.5</td>
<td>100.0</td>
<td>4.16</td>
<td>0.03</td>
<td>25.35</td>
</tr>
<tr>
<td>C30-S10</td>
<td>146</td>
<td>1.2</td>
<td>2.23</td>
<td>86.0</td>
<td>100.0</td>
<td>5.23</td>
<td>0.29</td>
<td>33.11</td>
<td>15.56</td>
</tr>
<tr>
<td>C30-S20</td>
<td>214</td>
<td>4.0</td>
<td>1.45</td>
<td>86.1</td>
<td>99.92</td>
<td>6.71</td>
<td>0.04</td>
<td>24.28</td>
<td>15.56</td>
</tr>
<tr>
<td>C30-S30</td>
<td>282</td>
<td>5.0</td>
<td>5.26</td>
<td>75.25</td>
<td>98.94</td>
<td>59.73</td>
<td>1.59</td>
<td>52.71</td>
<td>15.56</td>
</tr>
<tr>
<td>C30-S40</td>
<td>350</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>15.56</td>
</tr>
<tr>
<td>C30-S50</td>
<td>392</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>15.56</td>
</tr>
<tr>
<td>C40-S0</td>
<td>216</td>
<td>104</td>
<td>1.0</td>
<td>2.20</td>
<td>91.7</td>
<td>100.0</td>
<td>4.20</td>
<td>0.04</td>
<td>21.98</td>
</tr>
<tr>
<td>C40-S10</td>
<td>172</td>
<td>1.4</td>
<td>1.70</td>
<td>84.9</td>
<td>100.0</td>
<td>7.68</td>
<td>1.61</td>
<td>33.23</td>
<td>15.56</td>
</tr>
<tr>
<td>C40-S20</td>
<td>240</td>
<td>4.0</td>
<td>1.34</td>
<td>81.74</td>
<td>99.58</td>
<td>21.9</td>
<td>1.75</td>
<td>46.76</td>
<td>15.56</td>
</tr>
<tr>
<td>C40-S30</td>
<td>308</td>
<td>5.0</td>
<td>3.60</td>
<td>77.5</td>
<td>96.1</td>
<td>–</td>
<td>1.37</td>
<td>49.16</td>
<td>15.56</td>
</tr>
<tr>
<td>C40-S40</td>
<td>350</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>15.56</td>
</tr>
<tr>
<td>C40-S50</td>
<td>418</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>15.56</td>
</tr>
<tr>
<td>C50-S0</td>
<td>180</td>
<td>130</td>
<td>2.0</td>
<td>3.90</td>
<td>91.4</td>
<td>100.0</td>
<td>0.00</td>
<td>0.00</td>
<td>18.09</td>
</tr>
<tr>
<td>C50-S10</td>
<td>198</td>
<td>3.0</td>
<td>2.75</td>
<td>86.3</td>
<td>98.8</td>
<td>27.44</td>
<td>1.72</td>
<td>33.78</td>
<td>15.56</td>
</tr>
<tr>
<td>C50-S20</td>
<td>266</td>
<td>5.0</td>
<td>1.70</td>
<td>84.27</td>
<td>99.96</td>
<td>98.78</td>
<td>1.50</td>
<td>24.44</td>
<td>15.56</td>
</tr>
<tr>
<td>C50-S30</td>
<td>334</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>15.56</td>
</tr>
<tr>
<td>C50-S40</td>
<td>402</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>15.56</td>
</tr>
<tr>
<td>C50-S50</td>
<td>470</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>15.56</td>
</tr>
</tbody>
</table>

Figure 10. Relationship between E98 and fine aggregate substitution.

Figure 11. Compressive strength test result.
4. EXPERIMENT: EXTREMELY STIFF CONCRETE PRODUCT FROM PS ASH

4.1 Experimental outline

A water-holding ILB usually has more voids to increase water absorptivity/water retentivity, and the block retains water therein. However, this causes the strength of the block to decrease. It is thought that water retentivity/water absorptivity can be increased while maintaining strength by adding PS ash when water absorption is high. Therefore, in this experiment, a water-holding ILB was used as one of the applications for the extremely stiff concrete product of PS ash.

4.2 Materials

The materials used are shown in Table 8. In this experiment, recycled materials are used as well as PS ash. Sewage melted slag (MS) was used instead of sand as fine aggregate since there have been many reports of its use as fine aggregate. In addition, Perlite waste did not foam when perlite was generated. However, high water absorptivity was confirmed for perlite waste. Therefore, it was used to increase water retention ability. A reproduction aggregate is an aggregate produced when a defective product is crushed. In addition, pumice has a high water absorption and is porous. Therefore it was used to increase water retention.

4.3 Mixing proportions

PS ash, perlite waste and pumice were used as water retention materials in this experiment. The mixing proportions are shown in Table 9. As regards missing proportions, a two-mix proportion was examined wherein water retention materials were changed based on the mixing proportions of a normal interlocking block. Mixing proportion III is the mixing proportion of a normal ILB. This was compared with a water-holding ILB. In addition, the water amount was changed to ensure formability of the block. Therefore, the mixing proportion list is the mixing proportion for the revised water amount.

4.4 Test items

The following examinations were performed based on the “concrete quality standard for a water retention pavement”. In addition, the standard value determined by JIPEA is shown in Table 10.

Table 8. Materials

<table>
<thead>
<tr>
<th>Materials</th>
<th>Symbol</th>
<th>Kinds</th>
<th>Density (g/cm³)</th>
<th>Water absorption(%)</th>
<th>F.M.</th>
<th>Specific surface area(cm²/g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>C</td>
<td>Ordinary portland cement</td>
<td>3.15</td>
<td>–</td>
<td>–</td>
<td>3,260</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>RS</td>
<td>River sand</td>
<td>2.56</td>
<td>3.01</td>
<td>2.93</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>P</td>
<td>Pumice stone</td>
<td>1.69</td>
<td>44.76</td>
<td>3.43</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>MS</td>
<td>Sewage sludge melting slag</td>
<td>2.64</td>
<td>0.18</td>
<td>3.61</td>
<td>–</td>
</tr>
<tr>
<td>Coarse aggregate</td>
<td>RA</td>
<td>Recycled aggregate</td>
<td>2.46</td>
<td>4.02</td>
<td>7.07</td>
<td>–</td>
</tr>
<tr>
<td>Admixture</td>
<td>PS</td>
<td>Paper sludge Ash</td>
<td>2.27</td>
<td>20.63</td>
<td>–</td>
<td>1,360</td>
</tr>
<tr>
<td></td>
<td>PW</td>
<td>Perlite waste</td>
<td>2.35</td>
<td>12.8</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Chemical admixture</td>
<td>SP</td>
<td>High-range water-reducing admixture</td>
<td>1.05</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

Table 9. Mixture proportions.

<table>
<thead>
<tr>
<th>No.</th>
<th>Unit content (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>W</td>
</tr>
<tr>
<td>I</td>
<td>400</td>
</tr>
<tr>
<td>II</td>
<td>313</td>
</tr>
<tr>
<td>III</td>
<td>240</td>
</tr>
</tbody>
</table>

Table 10. Quality standard.

<table>
<thead>
<tr>
<th>Test items</th>
<th>Quality standard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quantity of water retention (g/cm³)</td>
<td>≥0.15 g/cm³</td>
</tr>
<tr>
<td>Suction height (%)</td>
<td>≥70%</td>
</tr>
<tr>
<td>Flexural strength (N/mm²)</td>
<td>≥3.0 N/mm²</td>
</tr>
</tbody>
</table>
Quantity of water retention (g/cm$^3$) = (mt-md)/V  

– Wet mass: The test piece absorbs water at 15–25°C for 24 hours. It is placed in a sealed plastic container and water is then drained off at a room temperature of 15–30°C for 30 minutes. After wiping off visible water, the mass is measured promptly as wet mass.  
– Dry mass: The mass becomes dry mass when it is cooled to normal temperature, after drying until the test body reached a constant mass in a dryer at a temperature 105 ± 5°C.  

Suction height examination
The water absorptivity examination requires a water suction height for 30 minutes.  
(1) The block is cooled to normal temperature after drying until it reached a constant mass in a dryer at a temperature 105 ± 5°C.  
(2) The block is installed in a water absorptivity examination device. The water level then reaches a height of 5 mm from the block base. The water is assumed to be spring water at 15–25°C. Material that return water to the block base such as wire netting is used for the top surface of the test piece stand, or a water-absorbent sponge can be inserted.  
(3) The block is taken out after 30 minutes, water is drained off to the degree that water does not drip, and visible drops of water are wiped off with a dishcloth. This mass becomes the suction mass (m30) after 30 minutes.  
(4) The suction height is found from the following calculation:

\[
\text{Suction height (\%)} = \frac{(m30-md)}{(mt-md)}
\]

**Table 11. Test result.**

<table>
<thead>
<tr>
<th>Test items</th>
<th>I</th>
<th>II</th>
<th>III</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quantity of water retention (g/cm$^3$)</td>
<td>0.250</td>
<td>0.280</td>
<td>0.150</td>
</tr>
<tr>
<td>Suction height (%)</td>
<td>80.00</td>
<td>86.00</td>
<td>22.40</td>
</tr>
<tr>
<td>Flexural strength (N/mm$^2$)</td>
<td>4.30</td>
<td>3.60</td>
<td>7.50</td>
</tr>
</tbody>
</table>

**4.5 Test results and discussion**
The test results are shown in Table 11. From the test results, both blocks produced experimentally satisfied the standard value for a water-holding ILB. In addition, the water-holding ability (water retention and suction height) largely increased when compared with an ordinary ILB. It appears there is a case for using a large amount of material with a high water absorption rate, such as PS ash, as a water retention material. The water retention showed a value of 1.6 times - 1.8 times the standard value, and the suction height showed a large value of more than 10% the standard value. From these results, it was found that production of a water-holding ILB with superior performance was made possible by using PS ash.

**5 CONCLUSIONS**
From the result of Experiment 1, it was seen that there was a limit to the PS ash amount to be used. In addition, from Experiment 2, it was found that using PS ash increased the strength of extremely stiff concrete. When PS ash was mixed with concrete for a cement amount of 200 kg/m$^3$ and the range of PS ash addition was 100 kg/m$^3$ to 300 kg/m$^3$, it appeared that concrete with a compressive strength higher than that of concrete with no PS ash could be produced. In addition, PS ash has a high water absorption. Therefore, production of a water-holding ILB with superior performance was made possible by using PS ash.  

**REFERENCES**
2) H. Fujiwara etc, 2001, Fundamental study for improving properties of concrete containing a large quantity of sewage sludge melting slag in aggregate, Utsunomiya, Tochigi, Japan: Japan cement association.
313

Alkali-activation of natural pozzolan for geopolymer cement production

D. Bondar & C.J. Lynsdale
Dept. of Civil and Structural Engineering, University of Sheffield, Sheffield, UK

A.A. Ramezanianpour
Dept. of Civil Engineering, Amir Kabir University, Tehran, Iran

N.B. Milestone
Dept. of Engineering Materials, University of Sheffield, Sheffield, UK

ABSTRACT: The challenge for the civil engineering community in the near future will be to realize structures in harmony with the concept of sustainable development, through the use of high performance materials of low environmental impact that are produced at reasonable cost. Geopolymer materials could provide a route towards this objective. The main benefit of geopolymer cement is their reduction in environmental impact as they use lesser amounts of calcium-based raw materials, have lower manufacturing temperature and use lower amounts of fuel, resulting in reduced carbon dioxide emissions for geopolymer cement manufacture by up to 80%–90%, in comparison with Portland cement. It is possible to use alkali-activated natural pozzolans to synthesize environmentally friendly and sound cementitious construction materials. This paper presents preliminary results of geopolymer using an activated Iranian natural pozzolan, namely Taftan. The formation of gel phase and the factors affecting this have been investigated using leaching tests, ICP-AES, FTIR, XRD and mechanical tests.

1 INTRODUCTION

Geopolymer materials are inorganic polymers based on alumina and silica units and are synthesized from a wide range of dehydroxylated alumina-silicate powders including natural pozzolans condensed with sodium silicate in a highly alkaline environment [Xu & Deventer 2000, 2003]. The formation of geopolymers requires reactive precursor materials and reagent species, especially OH\(^-\) [Shi 2001]. Highly alkaline solutions are used to induce the silicon and aluminium atoms in the source materials to dissolve and form the geopolymer paste. The three main steps in the process are dissolution, with the formation of mobile precursors; partial orientation of mobile precursors; and re-precipitation where the whole system hardens to form an inorganic polymeric structure.

In this research, a natural pozzolan is used as the raw material, which is alkali activated to produce the geopolymer. Pozzolans are widely used as cement replacements in Portland cement concrete because of their advantageous properties/effects, which include reduction in heat evolution, decreased permeability, alkali aggregate expansion control, increased chemical resistance, reduced concrete drying shrinkage, improvements in the properties of fresh concrete and cost reduction. On the other hand, they impart longer setting time, which may or may not be advantageous, and result in lower early strength compared with unblended Portland cement. Due to their dilution effect, curing is paramount to achieve full benefit from these materials, especially when they are used at high cement replacement levels. Different techniques have been tried to increase the reactivity of natural pozzolans to overcome the above disadvantages. A comparison based on a strength-cost relationship, indicated that chemical activation is the most effective and the cheapest method for achieving this [Shi & Day 2001].

In alkali-activation, the nature of the starting materials (i.e. aluminosilicate minerals) and the type and concentration of alkali in solution affects the formation and setting of the geopolymer gel phase. It has been established that in activation with KOH solutions, more geopolymer is present, resulting in better setting and higher compressive strength geopolymers than in comparison with other alkaline activators [Xu & Deventer 2000]. This is due to the fact that the presence of K\(^+\) favours the formation of large silicate oligomers to which Al (OH)\(_4^-\) prefers to bind. The more long-chain
silicate oligomers there are, the more readily the geopolymer precursor forms [Xu & Deventer 2000]. This is why the addition of extra Na$_2$SiO$_3$ is essential as it supplies a source of readily soluble silicate. There are many factors that affect the formation of the complex geopolymer system. Reference [Xu & Deventer 2000] gives guidelines on the effect of the various parameters on strength, for example the %CaO, the molar Si/Al ratio in the original mineral, the use of KOH, the extent of dissolution of Si and the molar Si/Al ratio in solution all have positive effects on the final compressive strength of the material, whereas %K$_2$O in the original mineral and the use of NaOH can result in a negative effect on strength.

Results of an investigation into the microstructure and strength of alkali-activated Taftan pozzolan systems is studied with the aim of establishing the optimum concentration of alkali solution for producing geopolymer cement from natural pozzolan. This work is part of a larger investigation studying various types of natural pozzolans.

2 EXPERIMENTAL TECHNIQUES

2.1 Materials

The natural pozzolan used throughout this work was Taftan pozzolan obtained from south eastern Iran. This pozzolan is used by Khash Cement works (Iran) for the production of Portland pozzolan cement. The Taftan pozzolan used has particle size finer than 75 µm. The chemical composition of Taftan pozzolan, as determined by X-ray Fluorescence (XRF), using a Philips PW 1480 instrument, is listed in Table1. X-ray diffraction (XRD) recorded on a Philips PW1800 machine is presented in Figure 1, showing its crystallinity and the major and minor minerals it contains. It can be seen that the loss on ignition of this pozzolan is low, and the %K$_2$O is lower than %Na$_2$O, which should correlate negatively with strength according to [Xu & Deventer 2000]. Taftan pozzolan contains a comparatively high CaO content, which should affect the strength of the geopolymer gel produced.

2.2 Sample preparation and methods

In order to determine the compressive strength and determine the optimum concentration of alkali solution for activating Taftan pozzolan, alkali activated samples were prepared by mixing 600 g Taftan pozzolan with KOH solutions with different concentrations 2.5, 5, 7.5, and 10 M, (MERK KOH pellets) and made up with sufficient distilled water to produce initial water to solid ratio (w/s) of 0.31. The ratio of alkaline hydroxides (ml)/ alkali silicates Na$_2$SiO$_3$ (ml) and total dry mix (g)/total solution (ml) were 7.7 and 3.2, respectively. The mixture was blended using a Hobart Canada N-50-1425 rpm blender. The resulting paste was cast in $50 \times 50 \times 50$ mm polyvinyl chloride (PVC) moulds and kept at room temperature for 24 hr. After removing from the mould, some of samples were autoclaved at 2.5 MPa pressure for 3 hr, and the rest were

<table>
<thead>
<tr>
<th>Material</th>
<th>LOI</th>
<th>SiO$_2$</th>
<th>Al$_2$O$_3$</th>
<th>Fe$_2$O$_3$</th>
<th>CaO</th>
<th>MgO</th>
<th>TiO$_2$</th>
<th>K$_2$O</th>
<th>Na$_2$O</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constituent, %</td>
<td>1.85</td>
<td>61.67</td>
<td>15.90</td>
<td>4.32</td>
<td>7.99</td>
<td>2.04</td>
<td>0.438</td>
<td>2.12</td>
<td>3.21</td>
</tr>
</tbody>
</table>

Table 1. Chemical composition of Taftan pozzolan.
wrapped and sealed for further setting and hardening in an oven for 27 days at temperatures of 40 and 60°C. The compressive strength of the specimens was tested according to ASTM C39. The setting characteristics of paste were also monitored and it was found that setting is generally fast, on average it took about 15 minutes from start to end of setting.

Leaching tests were carried out to investigate activator dosage corresponding to optimum dissolution rate and to contrast this with results found for compressive strength. Using a magnetic stirrer, 2.5 g mass of natural pozzolan was mixed with 25cc of KOH solution in a polypropylene beaker at room temperature for 5 hours. KOH solution concentrations investigated were 2.5, 5, 7.5, and 10 M. The solution and cake were separated by centrifuging for 25 minutes at 6000 rpm. After centrifuging, the clear liquid part of the solution was diluted and neutralized to pH < 1 (the concentration was reduced to 0.2 N and it then was diluted with 36% HCl). After two days, these samples were analyzed by inductively coupled plasma with atomic emission spectroscopy ICP-AES (JY -124 Sequential Jobin-Yvon ICP-AES) to assess the concentration of SiO₂, Al₂O₃, and Ca.

In order to investigate the extent of alkali-activation of solid, the resultant cake after leaching was filtered for 1.5hr and washed with 200 ml water in two stages and scattered over a watch glass and dried in an oven for 48 hrs at 60°C. A sample KBr pellet was prepared (0.5 mg of dried solid residue ground to fine powder with 150 mg of KBr) and dried in an oven at 60°C overnight before pressing and scanning. The powder was analyzed using a Bruker Equinox55FTIR spectrometer with an aperture selected at 8 cm⁻¹, with a total of 64 scans per analysis.

3 RESULTS AND DISCUSSION

Taftan pozzolan has been activated with KOH at four concentrations, 2.5, 5, 7.5, and 10 M. Figure 2 shows the compressive strength of the geopolymer cements synthesized using the above alkali concentrations. KOH concentrations in the range 5–7.5 M were found to generate geopolymers with higher compressive strengths. Leaching tests were carried out to find the optimum concentration of KOH for activating Taftan pozzolan. Using ICP-AES, Ca, Si and Al ion concentrations were measured in a solution resulting from leaching (Table 2). It can be concluded that the optimum dissolution of Si and Al, of 274.4 and 68.22 ppm respectively, occurred at the KOH concentration of 7.5 M. Increasing the alkali concentration to 10 M does not have a significant effect on the levels of Si and Al leached. Geopolymers prepared with a 7.5 M KOH solution, also corresponded to a higher strength (Figure 2).

It is believed that at low activator concentrations (i.e. less than 5 M KOH) there is significantly lower dissolution of natural pozzolan (see Table 2) resulting in the formed gel phase possessing lower binding strength. However, the higher viscosity of the alkaline hydroxide solution at concentration greater than 7.5 M means that the resultant geopolymer pastes need a longer time and/or a higher temperature for the excess water to evaporate before full strength is gained [Xu & Deventer 2003].

Increasing the activator concentration beyond a certain alkali content (depending on mineral, activator and curing conditions), may not result in further increase in strength, and detrimental effects such as efflorescence and brittleness resulting from the effects of high free alkali in the product have been reported [Xu & Deventer 2000]. Therefore, trying to increase the strength by increasing the alkali dosage is not recommended from both economic and performance points of view.

With respect to the curing conditions as observed from the results in Figure 2, Taftan pozzolan shows higher strength at 60°C. Statistical analysis by Shi and Day [2001] indicated that the strength development rate constant increased exponentially with curing temperature from 23 to 65°C, but the ultimate strength of these pastes decreased linearly with curing temperature [Shi 2001]. Shi and Day [2001] found that maximum strength was achieved at 40°C when activating natural pozzolan. In this current study, it was found that maximum strength for Taftan pozzolan was achieved at 60°C or when autoclaved at 2.5 MPa for 3 hr.

Table 2. ICP-AES results for Taftan pozzolan leaching.

<table>
<thead>
<tr>
<th>[Si] ppm</th>
<th>[Al] ppm</th>
<th>[Ca] ppm</th>
<th>KOH (molar)</th>
</tr>
</thead>
<tbody>
<tr>
<td>29.75</td>
<td>not detectable</td>
<td>28</td>
<td>2.5</td>
</tr>
<tr>
<td>147.99</td>
<td>not detectable</td>
<td>98.25</td>
<td>5</td>
</tr>
<tr>
<td>274.40</td>
<td>68.22</td>
<td>14.98</td>
<td>7.5</td>
</tr>
<tr>
<td>235.20</td>
<td>69.36</td>
<td>51.43</td>
<td>10</td>
</tr>
</tbody>
</table>

Figure 2. Effect of activator concentration and curing condition on gel compressive strength.
X-ray diffraction of Taftan pozzolan alkali-activated and cured under autoclave pressure indicates that the only peaks that remain unchanged are those related to albite, a major feldspar phase at $2\theta = 20 \leq 30^\circ$, which are the same as those for untreated pozzolan. The peaks related to quartz ($2\theta = 26.6^\circ$) and calcite are no longer present and the peaks for the crystalline phase related to hornblende have decreased so this mineral appeared as a minor phase only. This indicates that at elevated temperatures a different kind of reaction occurs which can utilise phases that are not available at lower temperatures.

Some additional tests were carried out to understand the gel conformation and its structure and elemental analysis of treated Taftan pozzolan. Figure 3 depicts the PAS-FT-IR (Photo acoustic FTIR) spectra recorded for Taftan powders before and after leaching in 2.5, 5.0, 7.5 M KOH. Taftan’s spectrum before treatment contained two main peaks at 1032 and 1089 cm$^{-1}$ with a number of smaller peaks. It can be noted that after leaching, the results indicate that with increased concentration of alkali, the vibration band with its maximum at 1032 cm$^{-1}$ is shifted towards a lower wave number. The extent of the shift has also been found to be approximately linear to the alkali content. Therefore, the original silicate and/or alumina-silicate structures in the natural pozzolan have been significantly depolymerised as they are dissolved.

According to Lee and Deventer [2003] during alkali activation; every bridging oxygen atom (BO) on the surface of the original alumino-silicate is replaced by two negatively charged non-bridging oxygen atoms (NBO), which are charge compensated by alkali. As a result, the infrared (IR) band attributable to the T-O-Si asymmetric stretching vibration of the TO$_4$ tetrahedral of an (alumina) silicate (glass) has been found to shift to the lower energy end with increasing alkali content. The shift observed would suggest that on leaching all the polymerised alumino-silicate has dissolved leaving layered silicates with a number of siloxyl groups.

Generally, the mid IR contains a number of main peaks from 800 to 1200 and the matrices display a substantially higher degree of the Al-Si disorder with bands becoming broader. This phenomenon suggests that the disordering of the primary structure of alumino-silicate and a reforming of polymers during the treatment could result in some kind of Al-Si precipitates forming on the surfaces of unreactive Taftan pozzolan particles.

All samples display an absorbance in the region of 1404–1471 cm$^{-1}$ although absorbance increases after leaching. The absorbance in this region could be generated by a vibration of carbonate salts formed on the surface of the residues [Lee & Deventer 2003]. Thus, the new bands at 1404–1471 cm$^{-1}$ might imply that some kind of hydrated carbonates and K$^+$ related aluminates are precipitated on the surfaces of the residues produced during leaching.
4 CONCLUSIONS

1. Geopolymer paste sets rapidly, on average there is about 15 minutes between the beginning and end of setting.
2. The best dosage for Taftan pozzolan activation with KOH was found to be within the range of 5–7.5 molar.
3. The curing temperature for achieving the highest strength for alkali-activated Taftan pozzolan was 60°C. However, 40°C was found to be adequate for achieving strength in a range suited for structural concrete.
4. Comparable strengths can be achieved with 60°C curing and autoclaving at 2.5 MPa for three hrs.
5. The effective alkali activator dosage for achieving the highest strength was found to correlate well with that for optimum leaching.

ACKNOWLEDGEMENT

The authors wish to express their gratitude to the Natural Disasters Research Centre, Iran, for support rendered throughout this experimental research programme, which was carried out at the concrete technology laboratory of P.W.U.T., and to I.P.P.I Tehran, Iran for the PAS-FT-IR analysis.

REFERENCES

Effect of spent catalyst, obtained from the catalytic cracking of petroleum, on the compressive strength of concrete

K.O. Ampadu  
*Department of Feeder Roads, Ministry of Road Transport, PMB, Kumasi, Ghana*

K. Torri  
*Department of Civil Engineering, Kanazawa University, 20-40-20 Kodatsuno, Kanazawa, 920–8667, Japan*

J.J. Vordoagu  
*Department of Civil Engineering, Kwame Nkrumah University of Science and Technology, Kumasi, Ghana*

ABSTRACT: This study looks at the possibility of using spent catalyst (SC) a by-product of the residual fluid catalytic cracking unit of Tema Oil Refinery (TOR) in Ghana as a pozzolana in the production of concrete for structural or aesthetic use and the optimum ratio of cement/SC, which produces acceptable strengths.

The SC currently poses environmental nuisance and health hazard due to its powdery nature and its possible contamination to ground water since it contains heavy metals (Chromium and Zinc).

The use of the spent catalyst in structural concrete was investigated by designing concrete of design strength 40 N/mm² and varying the proportions of cement, SC and hydrated Lime, making up the binder, whilst keeping water to binder ratio and the proportions of coarse and fine aggregates constant. Compressive strength tests performed on test cubes of each mixture confirmed that SC can be used as a pozzolana in structural concrete as a safe disposal method.

The future use of SC as a cement replacement in the concrete industry looks bright but more work has to be done to determine the durability characteristics of concretes made with SC replacements.

1 INTRODUCTION

1.1 Background

The residual fluid catalytic cracking unit of Tema Oil Refinery (TOR) uses a catalyst consisting mainly of Aluminium Oxide (Al₂O₃), Silicon dioxide (SiO₂) and Rare Earth (RE₂O₃) to convert the residual fuel oil into gasoline and liquefied petroleum gas (LPG) in the reactor. Davison Cracking Catalysts produce this catalyst, which is a refining catalyst, and it is part of the Ultima Series of catalysts. The product as exported to TOR is branded as “FCC Ultima – 448”.

The spent catalyst (SC) is accumulating at an alarming rate and causing environmental nuisance; a use has to be found for it as a safe and environmentally friendly way of disposal since the spent catalyst contains some heavy metals.

The basic aim of this study is to find a possible use for the spent catalyst by incorporating it as part of the ingredients in concrete, since the SC contains amorphous silica, the active ingredient in pozzolanic materials. The hydration reaction of the cement and also the pozzolanic reaction of the SC is expected to immobilize the heavy metals in the spent catalyst by incorporating them into the interstices of the cement matrix and hence provide an environmentally safe disposal method.

Spent fluid catalytic cracking catalyst from petroleum refinery plays a pozzolanic role in portland cement systems, (Paya, 2003). This study also aims at providing cost effective construction means in the concrete industry since the use of spent catalyst as a pozzolanic material could reduce the cost of concrete in construction projects.

The study also looks at the effect of the addition of external source of Ca(OH)₂ to the pozzolanic reaction of the SC by adding hydrated lime to the mix as part of the binder.

1.2 Physical properties of spent catalyst

1.2.1 Colour and Texture

The spent catalyst has a light greyish colour and feels gritty.
1.2.2 Grading
Grading analysis was performed on the spent catalyst to ascertain its particle size distribution. The particle size of the spent catalyst mainly lies within 20 µm and 150 µm.

The particle size distribution chart (Figure 1.1) and Table 1.1 below shows that the spent catalyst basically falls within fine sand and silt range of soil classification.

1.3 Chemical properties of spent catalyst
The chemical constituents obtained from chemical analysis on the spent catalyst are presented in Table 1.2.

The major chemical components of spent catalyst are Aluminium oxide and silica. It also contains some heavy metal oxides such as Chromium oxide and Zinc oxide.

Aluminium oxide forms about 50.28%, silica forms 45.26% while chromium oxide, the highest heavy metal oxide forms about 0.132% of the spent catalyst.

1.4 Preliminary work on the spent catalyst
Preliminary tests conducted on the use of the spent catalyst by the authors (unpublished) revealed that the optimum cement/spent catalyst (SC) combination ratio is 70% cement and 30% spent catalyst. It was also
observed that replacement of cement by SC results in an increase in the water demand of the concrete.

2 EXPERIMENTAL PROCEDURE

2.1 Materials

In this research, the following materials were used: normal Portland cement (obtained from GHACEM), spent catalyst from Tema Oil Refinery (specific gravity 2.44), hydrated lime, fine aggregate (source: washed beach sand from Cape Coast beach, specific gravity 2.68, coarse aggregate (granitic chippings, from Consar Quarry Ltd., Kumasi, specific gravity 2.69, absorption capacity 0.60%, maximum size 20 mm), (water-binder ratio \(H11005\) 0.42).

2.1.1 Preparation of materials

The coarse and fine aggregates were, air dried and stored in containers in the lab for use in the casting of concrete cubes.

Cement, hydrated lime and the spent catalyst were also stored in cool dry places in the lab in their respective bags.

2.2 Mixture proportions

The ACI method of mix design was used to obtain the mix proportions of the various constituents based on a characteristic strength of 40 N/mm². Table 2.1 shows the proportion in kg/m³ of the constituents for the various concrete specimens that were cast.

The batching was done by weight. The weight composition of coarse aggregate and fine aggregate as well as the water-binder ratio were kept constant while the binder content was varied.

Six different types of test cube samples were prepared namely:

1. The control labelled as CON which has 100% cement content as binder.
2. SC 30 which is made up of 70% cement and 30% spent catalyst as binder.
3. SC 30 L10 which is made up of 60% cement, 30% spent catalyst and 10% hydrated lime as the binder.
4. SC 30 L15 made up of 55% cement, 30% spent catalyst and 15% hydrated lime as the binder.
5. SC 30 + L10 which is made up of 70% of cement, 30% of spent catalyst with 10% of hydrated added as an additive and
6. SC 30 + L15 made up of 70% cement, 30% spent catalyst with 15% hydrated lime as an additive.

The SC 30 L10 and SC 30 L15 test samples were prepared to determine the effect of the addition of external source of Ca(OH)₂ as part of the binder on the hydration of the spent catalyst, a pozzolana, that has been blended with cement.

The SC 30 + L10 and SC 30 + L15 test samples were also prepared to examine the effect of adding external source of Ca(OH)₂ as an additive on the hydration of the cement-spent catalyst blend.

2.3 Experimental test procedure

The various proportions of the constituents were batched by weight. The constituents were then mixed together manually using a mixing bowl and hand trowel until a homogeneous concrete mixture was obtained.

The concrete was placed in the 150 mm \(H11003\) 150 mm \(H11003\) 150 mm moulds and well compacted by rodding. The cubes were later labelled according to the test ID.

Nine (9) test cubes were prepared for each test ID using the 150 mm \(H11003\) 150 mm \(H11003\) 150 mm moulds. The cubes were removed from the moulds after 24 hours air curing and then cured in water bath maintained at room temperature of 28 °C.

Compressive strength tests were performed on the cubes after seven (7) days, fourteen (14) days and twenty-eight (28) days of curing.

3 OBSERVATIONS, RESULTS AND DISCUSSIONS

3.1 Observations

The following observations were made during the test.

1. Workability of the concrete mix for the specimen with 30% spent catalyst (SC 30) was less than that of the control specimen (CON).
2. Concrete made from addition of spent catalyst is lighter in colour than the control due to the light greyish colour of the spent catalyst.

3. Addition of lime also increases the degree of lightness in the colour of the resulting concrete and a decrease in the workability of the resulting concrete as evident in the low slump values obtained from the slump tests.

3.2 Results

Table 3.1 shows the results obtained from the compressive strength test tests carried out on the test cubes. It also includes the slump tests results on the fresh concrete before casting the cubes.

Table 3.1. Summary of compressive strength test results.

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Slump (mm)</th>
<th>7 days</th>
<th>14 days</th>
<th>28 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>CON</td>
<td>95</td>
<td>30.10</td>
<td>30.57</td>
<td>41.90</td>
</tr>
<tr>
<td>SC 30</td>
<td>27</td>
<td>23.47</td>
<td>32.95</td>
<td>39.79</td>
</tr>
<tr>
<td>SC 30 L10</td>
<td>13</td>
<td>25.30</td>
<td>31.17</td>
<td>35.89</td>
</tr>
<tr>
<td>SC 30 L15</td>
<td>5</td>
<td>23.38</td>
<td>25.57</td>
<td>33.59</td>
</tr>
<tr>
<td>SC 30 + L10</td>
<td>10</td>
<td>31.91</td>
<td>40.77</td>
<td>47.50</td>
</tr>
<tr>
<td>SC 30 + L15</td>
<td>5</td>
<td>28.92</td>
<td>40.74</td>
<td>45.30</td>
</tr>
</tbody>
</table>

3.3 Discussions

Figure 3.1 shows that the compressive strength of the concrete test cubes generally increased with age.

The 7 days strength for the control was 30.1 N/mm² (75% of the designed strength) that of SC 30 was 23.5 N/mm² (59% of the designed strength); that of SC 30 L10 was 25.3 N/mm² (63% of the designed strength) and that of SC 30 L15 was 24.0 N/mm² (60% of the designed strength). The 7 days strength for SC 30 + L10 was 31.91 N/mm² (representing 80% of the design strength) while that of SC 30 + L15 was 28.92 N/mm² (representing 72% of the design strength).

Thus, the 7 days compressive strength of SC 30 L10 and SC 30 L15 were both higher than that of SC30 by a small margin despite their lower proportion of cement compared to SC 30, indicating that there is early strength development in the specimens with hydrated lime in their mix compared to that with only the spent catalyst (SC 30).

The 7 days compressive strengths of SC30+L10 and SC30+L15 were both quite higher than that of SC 30 with the 7 days strength of SC30+L10 higher than that of the control.

Thus the presence of hydrated lime in their mix might have aided pozzolanic reaction at the early days of hydration.

The 14 days strength for the control was 30.57 N/mm² (76% of the designed strength) while...
that of SC 30 was 32.95 N/mm² (82% of the designed strength), 31.17 N/mm² (representing 78% of the designed strength) and 25.57 N/mm² (representing 64% of the designed strength) were the 14 days compressive strengths for SC30L10 and SC30L15 respectively. The 14 days strength for SC30+L10 was 40.77 N/mm² (102% of the designed strength) while that of SC30L15 was 40.74 N/mm² (102% of the designed strength).

It is significant to note that the 14 days compressive strength of SC30/H11001L10 and SC30/H11001L15 were both higher than the designed strength.

The 28 days strength for the control was 41.9 N/mm² (103% of the designed strength) while that of SC 30 was 39.79 N/mm² (99.5% of the designed strength). The 28 days strength of SC 30 L10 was 35.89 N/mm² (89.7% of the designed strength) while that of the SC 30 L15 was 33.59 N/mm² (84% of the designed strength). The 28 days strengths of SC30+L10 and SC30+L15 were 47.50 N/mm² (representing 119% of the designed strength) and 45.30 N/mm² (representing 113% of the designed strength) respectively.

The strength for SC30 L10 and SC 30 L15 appears to be on the lower side, however, if one considers that SC 30 L10 and SC 30 L15 contained 40% and 45% less cement but attained strength of 10.3% and 16% respectively lower than the designed strength then one will appreciate that the spent catalyst/hydrated lime blend performed well. Thus the presence of the hydrated lime enhanced pozzolanic reaction.

Also, the higher strengths obtained for SC30+L10 and SC30+L15 specimens than the other test specimens could be attributed to the availability of more external Ca(OH)₂ from the hydrated lime to react with the spent catalyst; a pozzolanic material. Thus the presence of hydrated lime as an additive also enhanced pozzolanic reaction in the cement-spent catalyst blend.

The above explanations took into account the high FCC pozzolanic behaviour demonstrated in previous studies, (Paya, 2004).

4 CONCLUSION AND RECOMMENDATION

4.1 Conclusions

From the above results, it can be concluded that;

1. The spent catalyst can be used as a pozzolanic material in concrete for structural use.
2. The cement-spent catalyst replacement ratio of 30% is the optimum cement replacement ratio.
3. The addition of external Ca(OH)₂ in the form of hydrated lime as a binder replacement enhances pozzolanic reaction resulting in strength development in the early stages of hydration.
4. The use of hydrated lime (Ca(OH)₂) as an additive to the concrete mix enhances pozzolanic reaction resulting in higher strength development both in the early and later stages of hydration.
5. Blending the SC with cement to produce concrete is an environmentally safe and economical way of disposing of this waste material from the refinery.

4.2 Recommendations for further research

This research was limited in scope as it deals with only results from compressive strength test on the cubes made from the various test IDs.

It is recommended that more work be done in this regard so as to find out other properties such as durability, chloride and water absorption and permeability of concrete produced using spent catalyst as a pozzolanic material.

Also chemical analysis is required to confirm the pozzolanic property of the spent catalyst as well as the contribution if any of the external source of hydrated lime on pozzolanic reaction.

In addition, more work has to be done to determine the optimum percentage of hydrated lime that should be added to the cement-spent catalyst blend to obtain optimum results.

The spent catalyst as obtained from the catalytic cracking of fuel at Tema Oil Refinery was used in its raw form, without further processing, in this research. It is recommended that the spent catalyst be milled to reduce the particle size to the fineness of cement to increase the rate of reaction.

The future use of SC as a cement replacement in the concrete industry looks bright but more work has to be done to determine the durability characteristics of concretes made with SC replacements.

The authors are committed to undertake the above research but collaboration from other researchers will enhance and validate the results.

ACKNOWLEDGEMENTS

The authors are grateful to Mr. J. A. Frimpong; Ghana Highway Authority (Materials Lab) – Kumasi Mr. Gilbert Fiazdoe; Civil Engineering Department of Kwame Nkrumah University of Science and Technology (KNUST) – Kumasi, Mr. Charles Anku and Agnes Afu Fugah; Department of Feeder Roads – Kumasi for their immense assistance in the laboratory works.

REFERENCES


1 INTRODUCTION

The energy consumption in all over the world has been given a great push by rapid population increase and economic development. This at the same time has resulted in environmental pollution and degradation in natural resources. Such an undesired by-product of economic development has brought the use of by-products to the scientific agenda. Cement is one of the important structural materials, which has contributed a lot to the economic development. However, the current manufacturing technology of cement clinker has great detrimental effects on ecology. It uses large number of natural resources and energy and releases hazardous gases such as CO$_2$, NO$_x$ and SO$_x$ into the atmosphere. To minimize these detrimental effects, various natural mineral additives, industrial wastes and by-products can replace some of cement clinkers. This process will increase the total production and save energy, which will ultimately enhance the environmental quality [Reiser 1999, Yeginobali et al. 2000] thereby raising the cement production process to an ecologically acceptable level. Such a cement production technology is based on mixing portland clinker, (limestone) and mineral additives, which are ground together. This process will improve the durability of cement-based materials and products without any decrease in standard compressive strength of hardened mortars. This improvement is achieved by using local low-cost mineral additives and industrial by products, which would otherwise harm the ecological system nearby. Besides, the energy used per ton will be lower in this case than that in normal cement production process. This is also beneficial to ecology since less contaminant will be
disposed of during both cement production and energy production. Naik reported the first case of concrete made in 1984 with 70% Class C fly ash as a replacement for cement for pavement construction in Wisconsin. [Naik & Ramme 1989]. He also reported that HVFA concrete can be proportioned using large amounts of fly ash to meet strength and durability requirements for structural-grade as well as high-strength concrete [Naik et al. 2003].

The industrial wastes that can be used in replacing portland clinker include bottom-ash, fly ash and silica fume. These by-products have such cementitious and pozzolanic properties that can allow their use in cement and concrete production [Naik & Ramme 1989, Demirbas & Aslan 1999, Targan et al. 2002, Naik et al. 2003, Sobolev & Soboleva 1996, Canpolat et al. 2004, Mahmud et al. 2004].

Industrial raw materials are the main input to the technology of the 21st century; zeolites are one of the most investigated raw materials. The investigations have revealed many explorations on its possible use areas. Zeolites have now become one of the important raw materials of current industry due to their crystal structure and chemical properties. The reason why natural and artificial zeolites have found a wide variety of use areas in industry is the compatibility of their properties to the several areas. The properties that make them so suitable to industry include high ion exchange capacity, the suitability of crystal structure for dehydration without deforming, low density and having silica content. Zeolites have been in use in history as a structural material and in agriculture without being known as zeolite due to them being light and light colored material, and having very fine crystal structure and their silica contents [Sariz & Nuhoglu 1992].

Silica fume (SF) is a by-product obtained during the production of siliceous and ferrousilicious alloys in electrical ovens. Silica fume is a very active pozzolan with its very fine particles (20 m²/g) and poses 75% amorphous silica content. These properties make it quite suitable for using as cement or concrete additive. However, the properties mentioned, at the same time, increase the water requirement in concrete or cement. This, therefore, requires the use of superplasticizer (SP) together. The joint use of SF and SP additives in concrete considerably reduces the water/cement ratio. Moreover, this yields more intense and high quality mortar internal structure by pozzolanic reactions thereby offering possibility to obtain high strength and durable concrete.

Turkey has a rich zeolite potential in western part of it, the potential has not been sufficiently developed for industrial use. Major areas of use in recent years include wastewater treatment and agricultural purposes [Erdem et al. 1999].

Up until 1970, additives in concrete had been used in concrete due to the economic and ecological reasons. Their contribution to the quality of concrete had not been explored. Later it has been found that these additives reduce the permeability of concrete and increase sulfate resistance of the concrete by reducing the C₃A ratio [Mahmud et al. 2004]. Recent studies have also shown that replacing portland cement by the mineral additives in certain rates improves the surface resistance of concrete produced by such cement [Mangat & El-Khatip 1992, Tiksalsky & Carrasquillo 1993].

Irrassar et al. [Irrassar et al. 1996] investigated sulfate effect of mineral added concrete using fly ash, natural pozzolan and volcanic ash as additives. They left some of the specimens in an atmospheric sulfate solution whereas others were buried in a high sulfate density soil. The specimens were prepared by replacing Portland cement 8.5% C₃A with mineral additives in certain rates. Sulfate effects were analyzed based on appearance, weight loss, and compressive strength and X-Ray test. They have found that mineral added concrete is more resistant to sulfate than normal concrete.

The interaction of concrete and sulfate depends on the structure of the cement used, the concrete made, renewal of solutions and their movements. In reinforced concrete structures, adding pozzolanic material to the cement used can reduce the damage caused by sulfate attack.

The quantity and the property of the reaction products that take place between concrete and the sulfate solutions are dependent on cations associated with sulfate, different sulfate mixings and the ratios between different mixings. The reaction products also vary the amount of C₃A. The impact mechanics of sulfates are rather complex. The sulfate ions in a sulfate solution react with cement hydration product CH and then gypsum is produced (CSH₂). The effect of gypsum on volume expansion softening in concrete is limited and not at all harmful. However, the gypsum together with C₃A in cement and its hydration products C₃AH₆, C₄AH₁₃, C₆ASH₁₂–₁₈ produce candlot salt (C₄AS₃H₃₂). The candlot salt is quite similar to the natural mineral ettringite and causes large volume expansions due to its 32 molecular crystal water [Cohen & Bentur 1988, Neville 1990]. The reactions products fill in the blanks and improve the mechanical properties during early ages [Kayyali 1989, Irrassar 1990]. However, the volume expansion in reaction products induces additional cracks in the structure. These cracks deteriorate the performance of the material by resulting in the system failure.

In all over the world, there is a growing trend to use mineral additives in cement. To this end, the use of fly ash as additive in cement production will save energy during clinkerisation, blending and drying, for the manufacturers. This will benefit environment for both waste disposal and energy usage points of views.

The pozzolans such as zeolite, volcanic ash, silica fume, fly ash, and bottom ash connect free lime which
is a cement hydration product by the active SiO\textsubscript{2} in their structure. They make a calciumsilicatehydrate (C-S-H) gel which is not soluble in water and prevents CH from melting. The ash fills in the cement mortar–aggregate interface and capillary pores resulting in reduction of pore sizes, discretisation of capillary pores and improvement of mechanical and physical properties of the product. Therefore, the diffusion of water and sulfate ions into the mass improves the sulfate resistance [Mehta & Gjorv 1982, Bonen 1993].

This study aims at showing the usability of natural zeolite and silica fume in cement production as a mineral additive. For this purpose, natural zeolite and silica fume provided from Manisa, Gördes, Turkey, in various rates were ground together with clinker. The resultant cement was used to produce mortars by which some physical and mechanical properties of the cement were determined. The performance of natural zeolite and silica fume were compared with both each other and the control specimens prepared with no additives. The additive rates were 5, 10 and 15% zeolite in mixtures (ZC) and 10% fixed silica fume plus 5, 10 and 15% zeolite in mixtures (ZSFC). All mixtures were added with 5% limestone. The range of mixtures rates enabled to produce cements in different composition.

2 EXPERIMENTAL

In this study, 5% sodium sulfate and 5% magnesium sulfate solutions were used. Experiments were carried out using mortar samples prepared from natural zeolite of 5, 10, and 15% of cement by weight and silica fume of 5, 10, and 15% separately and both these materials together. Experiments were undertaken to investigate the effects of 5% Na\textsubscript{2}SO\textsubscript{4} and 5% MgSO\textsubscript{4} solutions on the physical and mechanical properties of hardened mortar specimens. The materials used during the preparation of the mortar samples were obtained from different sources namely natural zeolite from Enli Mining Ltd, Gördes, Manisa, Turkey; Standard Sulfate Resistant Portland Cement (SC 32.5) and Ordinary Portland Cement (PC 42.5) from Set Cement Co. plant, Ankara, Turkey; silica fume from Etibank Electro metallurgy plant, Antalya, Turkey; Standard Rilem Cembureau Sand (TS 819) from Set Trakya Cement Co. plant Trakya, Turkey [Turkish National Standards 1985] and the water from Izmir, Turkey municipal water system. Experimental study is based on mortar samples prepared according to TS 24 [Turkish National Standards 1985]. The specimens were cured up to 28 days under standard conditions.

The specimens were taken out of water tank and placed into water and Na\textsubscript{2}SO\textsubscript{4}, MgSO\textsubscript{4} solutions with 5% concentrations. This time is accepted considered as the beginning of the aging period. When preparing the solutions, industrial type Na\textsubscript{2}SO\textsubscript{4} and MgSO\textsubscript{4} salts were used. The material quantities required for a certain concentration was specified by standard analytical experiments.

In mortar samples: a commercially available Rheobuild® 716 [Master Builder Inc. 2002] a ready-to-use liquid admixture comprising of synthetic polymers specially designed to impart rheoplastic qualities to concrete was added. It contained no chloride and is a high range water-reducing admixtures. (HR WRA). The quantity of this superplasticizer used was10% of the silica fume in mortar.

The specimens were made according to Table 2. This table shows the cement used, the rate of zeolite added and the codes given to specify the specimens. The 28th day was accepted as the beginning, namely t = 0 and the control experiments were performed at the beginning, 2nd, 7th, 28th and 90th days. For each experiment set,
tensile strength test and the compressive strength test on the cracked parts of specimens were carried out using 3 specimens with 40\(\times\)40\(\times\)160 mm in dimension. For specific gravity experiments, 3 more specimens for each group were also prepared. The solutions in which the specimens were kept were renewed at every 14 days and the weight losses at these intervals were determined.

3 DISCUSSION OF THE EXPERIMENTS

The physical characteristics of blended cements are presented in Table 3. As seen from this table, the grinding periods decrease as the quantity of additives increases. This shows that zeolite is an easily grindable material. Zeolite is softer than portland clinker and hence when it is ground together with the PC, the grinding period of the mixture decreases.

From Table 3, it can also be seen that the specific gravities of cements with additive are lower than that of portland cement without additive. The specific gravities of each three groups of cements decrease as the quantity of additives increase.

The cure periods and volume expansions of the cements are given in Table 4. The table shows that the values obtained for all types of cements are within the tolerable limits of Turkish Standards [Mehta 1986].

3.1 The Results of compressive strength test

The compressive strength tests were under using 6 specimens. The results are discussed separately below for the groups kept in normal water and in sulfate solutions.

3.1.1 The groups kept in water

The 28-day compressive strength of the portland cement concrete was used as the reference to calculate the relative compressive strength in all the figures. As seen from Figs 1, 2, 3, and 4, initially the compressive strengths of mortars with additives are higher than those of the reference group for the samples kept in water. The most efficient additive rate is 5, followed by 10 and 15%.
In mortars made of zeolite and silica fume additives (ZSFC) with 5, 10, 15% additive rates, there are significant increases in compressive strength values with respect to the reference sample the age of 2. This is caused by the existence of pozzolanic activity. The increases in strength values are slightly bigger in ZC samples. This can be attributed to the fact that the quantity of the mineral additive replacing portland clinker in ZSFC samples are more than others. In addition to the positive effect on compressive strength, pozzolonic activities of the additives improve the impermeability of the mortars since the pozzolans fill the micro pores, which make positive contribution to the durability of mortars. The strength values obtained are within the limits specified by the Turkish Standards. For the cements prepared by silica fume and zeolite the additives are proved to be used in cement production up to 15%. When the zeolite rate rises up to 15%, the strength increases with respect to without–additive samples. At later ages the strength values close to each other. In this group, the appropriate additive rate is 15% for all ages. For the first 7 days, the compressive strength value of specimens containing zeolite decreased as the quantity of zeolite increases.

3.1.2 The groups kept in sulfate solutions

The results of the compressive strength test carried out using the specimens kept in 5% sodium sulfate solutions (Fig. 4) show that there are variations in time. However, a decrease in strength values of all groups was observed on 90th day. This is also true for
the groups kept in 5% magnesium sulfate solutions. However, the rate of increase slightly varies within the group. It is expected that the compressive strength values of mortars with silica fume are lower than that of reference mortar initially. This is because the hydration heat of pozzolanic material is lower than that of portland cement. However, the heat reaches to the maximum value eventually [Mehta 1986].

Montgomery et al. [Montgomery et al. 1981] state that fly ash particles help the mechanism reducing the micro cracks by acting as kernel points. The increase in compressive strength observed up to 90th day of the mortars with and without silica fume that are subjected to sulfate effects can be attributed as the newly appeared products such as gypsum and ettringite fill in the pores [Reiser 1999]. The basic character of the sodium hydroxide released during the making of hydration products prevents sulfate effect on concrete [Irassar et al. 1996].

However, on later days, the cracks occurring at solid phase due to the volume expansions caused by gypsum and ettringite.

In this study, during the experiments it was not observed that even 5% Na$_2$SO$_4$ and 5% MgSO$_4$ solutions did not make any harm to the structure that would be made of these cements. However, it is necessary that the experiment periods should be extended and internal structural analyses should be carried out in order to determine the changes with time.

3.2 Specific gravity

The sulfate solutions in which the specimens were kept were renewed every 14 days and on the changing days the weight losses were determined. The results of these experiments are presented in Figs. 5 and 6. These figures give weight change with respect to the reference specimen on the 90th day.

As seen from the table the losses show variations with respect to cement type prepared using zeolite and silica fume in a varying rate. Generally speaking the loss of weight values are lower in specimens kept in 5% magnesium sulfate solutions than those kept in sodium sulfate solutions. The values for the case of magnesium sulfate are between 0.16 and 19% by weights of specimens.

There is no loss for ZSFC5 sample. The maximum value for the case of sodium sulfate is 0.55% in ZSFC15 specimens. The minimum value, however, is 0.19% in ZC15 specimens.

4 CONCLUSIONS

In this study, the resistance of the cements produced by adding zeolite and silica fume against sulfate exposure was investigated. In literature, these pozzolans are stated to have improving effects on cements [Neville 1990, Erdem et al. 1999, Reiser 1999, Yeginobali et al. 2000, Naik et al. 2003].

Based on the results of the experiments undertaken to test the performance of pozzolans in cement, the following conclusions can be drawn:

In specimens with zeolite, no softening, cracking, and crashing have been observed up to 90 days. The sulfate solutions used have not been observed to have any negative effects on the specimens up to 90 days. For these sulfated media and the test period, the most effective addition rates are found to be 10% for silica fume and 15% for zeolite. The durability to sulfate attack was obtained with silica fume replacement 5–10%, but strength loss of 10–15% can be expected. Mortars with SF and Z were damaged in magnesium sulfate solution. The compressive strength decreases with increasing SF and Z content. The increase in natural zeolite (Z) and zeolite plus silica fume (ZSFC) rate reduces the specific

Figure 5. Changes in weight of hardened cement mortars kept in 5% sodium sulfate solutions.

Figure 6. Changes in weight of hardened cement mortars kept in 5% MgSO$_4$ magnesium sulfate solutions.
gravity of the cement to which they were added. The chemical compositions and some physical properties of ZC and ZSFC samples satisfy the corresponding Turkish Standards (TS 10156, TS 25 and TS 640) [Turkish National Standards 1985]. When natural zeolite is solely added to clinker more than 15%, the compressive strength of the mortars reduce. The compressive strength of 20% zeolite – added cements (ZC) and 5% zeolite and 10% silica fume added cements are higher than that of control cement (PC 42.5). Although the strength and weight losses of the mortars with additive are slightly varied with respect to those of the mortars with no additives, no significant increase or decrease was observed. Therefore, the aging period should be extended beyond 90 days, which was considered in this study to see what possible negative effects may arise in longer periods.

In conclusion, the findings of this research are satisfactory in using zeolite and silica fume in obtaining sulfate resistant cement. This article shows a study on the process of deterioration and the formation of reactants by chemical reaction of mortars and pastes with and without SF and Z in sodium and magnesium sulfate solutions.

In the sodium sulfate solution, the use of SF and Z have beneficial effect on the compressive strength of blended cements. In the magnesium sulfate environment, the test results showed that the use of Z and SF in concrete exposed to magnesium sulfate environment is not recommended.

However, as mentioned previously, the longer periods should be considered to strengthen the results of this research. In addition to that, internal structure of the mortars should be investigated to verify the results of this research from that point of view.

ACKNOWLEDGMENTS

The authors would like to thank Cimentas Co. (Izmir, Turkey) and Dr. Kozanoglu, C. (Civil Eng. Dept., Celal Bayar University, Turkey) for providing the facilities needed to carry out the experiments mentioned in this paper.

REFERENCES


Rheobuild® 716 2002. High range water-reducing admixtures. (HRWRA), Master Builder Incorporation, Cleveland, Ohio.


Strength of mortar and concrete using fine powder of molten slag made from municipal waste as a cementitious material

K. Fukuzawa  
*Dept. of Urban and Civil Engineering, Faculty of Engineering, Ibaraki University, Hitachi, Japan*

D. Kwak  
*Research & Development Laboratory, Taiheiyou Materials Corporation, Sakura, Japan*

O. Abe  
*Research Center of Superplasticity, Faculty of Engineering, Ibaraki University, Hitachi, Japan*

H. Ojima & H. Yoshida  
*Ibaraki Industrial Technology Center, Ibarakimachi, Japan*

ABSTRACT: This paper describes the basic study on the technology which uses ground granulated slag made from municipal waste as alternate material of the portland cement. The granulated slag from municipal waste has the latent hydraulic property, because it is made through the quenching process, and it forms the hardening body by the stimulation of the alkali material. In this study, it was confirmed that various factors such as average particle size of the slag powder, dosage of the alkali stimulative material and curing condition influence the compressive strength of mortar and a high strength concrete is obtained using the slag.

1 INTRODUCTION

In Japan, municipal wastes are fused at a high temperature more than 1,200°C to adapt the upper limit of dioxines in the residual material regulated in the “dioxines countermeasure special action law” enforced in January 2000 and reexamined in December 2002. The fused material discharged from the furnace is cooled rapidly with water, and the granulated slag is produced. From the viewpoint of the eco design, effective utilization of the granulated slag has become a matter of concern. The research on slag utilization as a fine aggregate for concrete or as the sub-base material for road has been started by many researchers. Considering that a large amount of energy is used to produce the slag, a higher price material should be replaced by the slag. We propose that it must be used as a cementitious material, utilizing the latent hydraulic property of the slag. The characteristics of the concrete with ground granulated slag made from sewage sludge has already been reported [Nitta et al. 2000]. However, the components of ground granulated slag made from municipal waste is different from that made from sewage sludge. Therefore, the results could not be applied to the concrete with ground granulated slag made from municipal waste. In this study, influence on mortar strength of various factors such as average particle size of the slag powder, curing condition and dosage of the alkali stimulative material are discussed. Further the strength of concrete used granulated slag from municipal waste as cementitious material was examined.

2 TESTING PROCEDURES

2.1 Property of ground granulated slag made from municipal waste

Two kinds of slag which were made in two cities were used in the experiments. The granulated slag is ebony, and has smooth surface. Almost all particles of the molten slag are angular containing some needle shaped particles. Figure 1 shows the result of X-ray diffraction method on Slag A. It is proven that the granulated slag is in amorphous glassy state, as there is no protruded peak. The chemical composition of molten slag used in this study is shown in Table 1. Over 90% of the molten slag consists of SiO$_2$, Al$_2$O$_3$, and CaO. The other components are small. The basicity b is one of the index which evaluates the
hydraulic property of blast furnace slag as an admixture, and it is calculated by Equation 1.

\[
b = \frac{(\text{Al}_2\text{O}_3 + \text{CaO} + \text{MgO})}{\text{SiO}_2}
\]  

(1)

The basicity of molten slag from municipal waste is smaller than the standard value 1.60, which is the regulated value of the ground granulated blast furnace slag [JIS A 6206 1997]. It is reported that the molten slag with such low basicity shows a good hydraulic property, as it contains Fe$_2$O$_3$ or P$_2$O$_5$ which destabilise the glass structure compared with blast furnace slag [Okamoto et al. 1997].

Slag A was pulverized in the fineness of 4 levels. The particle size distribution curve was measured using the laser diffraction method. The particle size distribution curves are shown in Figure 2. The particle size at the 50% passing rate in particle size distribution curve was defined as average particle size $D_{50}$. Observation of the particles of one of the molten slag is shown in Figure 3. Slag B was pulverized until $D_{50}$ became $10 \times 10^{-6}$ m.

### Table 1. Chemical composition of slag.

<table>
<thead>
<tr>
<th>Oxides, %</th>
<th>Slag A</th>
<th>Slag B</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO$_2$</td>
<td>42.2</td>
<td>44.4</td>
</tr>
<tr>
<td>Al$_2$O$_3$</td>
<td>18.0</td>
<td>13.4</td>
</tr>
<tr>
<td>Fe$_2$O$_3$</td>
<td>2.8</td>
<td>2.4</td>
</tr>
<tr>
<td>CaO</td>
<td>28.2</td>
<td>30.8</td>
</tr>
<tr>
<td>MgO</td>
<td>3.5</td>
<td>2.7</td>
</tr>
<tr>
<td>SO$_3$</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td>Na$_2$O</td>
<td>1.4</td>
<td>2.5</td>
</tr>
<tr>
<td>TiO$_2$</td>
<td>1.7</td>
<td>1.0</td>
</tr>
<tr>
<td>Basicity, b</td>
<td>1.18</td>
<td>1.06</td>
</tr>
</tbody>
</table>

2.2 Other materials and testing procedure

Five kinds of alkali stimulative materials were compared, namely: water glass, Na$_2$SiO$_3$, Na(OH), Ca(OH)$_2$, Al$_2$(OH)$_3$. The component of this water glass is composed of sodium oxide 17.48%, anhydrous silica acid 37.03% and water. Crushed sand was used as the fine aggregate. An enforcement stirred type mortar or concrete mixer was used, and the mixing carried out for 90 seconds after 1 minute mixing of fine aggregate and slag powder before putting other materials. Cylindrical specimens whose diameter was 50 mm and height was 100 mm were produced, and they were steam-cured. Steam curing was done at the condition of 3 hours for pre soaking time, heating speed for the 15°C/hour, and the maximum temperature for 65°C and duration time for 5 hours. After the steam curing,
specimens were stripped on the next day, and they were placed in a constant temperature and humidity room of 20°C and 65% RH until 7, 14 and 28 days.

3 INFLUENCES OF VARIOUS FACTORS ON THE COMPRESSIVE STRENGTH OF MORTAR

3.1 Influence of the average particle size \( D_{50} \) on the strength of mortar

The effect of the average particle size \( D_{50} \) of molten slag powder on compressive strength of mortar was examined. In the experiment, ground granulated slag and alkali material were used in place of the portland cement of usual mortar mix. A water glass was used as alkali stimulative material. From the observation of the specimens just after steam curing, it was observed that the mortar expanded a little.

Figure 4 shows the influence of average particle size of slag powder on compressive strength of mortar. It is confirmed from the figure that the compressive strength of mortar increases as average particle size decreases. It is conceivable that the increase of mortar strength is caused by the increase of quantity of the hardening body with the increase of surface area of slag powder. In this study, the minimum average particle size was \( 5 \times 10^{-6} \) m. Figure 4 indicates that the strength increases with age by using molten slag fine powder whose average particle size is smaller than \( 5 \times 10^{-6} \) m.

3.2 Influence of the curing temperature on the strength of mortar

The maximum temperature in steam curing was varied to 45, 65 and 85°C. The molten slag powder with an average particle size of \( 5 \times 10^{-6} \) m was used in the experiment. The effect of the maximum curing temperature on the compressive strength at the age of 7, 14 and 28 days is shown in Figure 5. The compressive strength of the mortar is influenced by the maximum curing temperature as is shown in the figure. Within the curing temperature set in this study, it is confirmed that the relation between compressive strength and curing temperature is linear.

3.3 Influence of type of alkali stimulative materials on the compressive strength of mortar

Five kinds of alkali stimulative materials, namely water glass, sodium methasilicate, sodium hydroxide, calcium hydroxide and aluminium hydroxide were compared.

The influence of the kinds of the alkali stimulative material on the compressive strength of mortar at the age of 28 days is shown in Figure 6. The compressive strength is 32 MPa, when water glass is used as the alkali stimulative material. High compressive strength 53 MPa is obtained, when sodium methasilicate is used as the alkali stimulative material. Considering that the difference between sodium methasilicate and water glass is not large in the strength seems to be caused by the mixing rate.

The high mixing rate between slag powder and the sodium methasilicate acid causes a high reactivity and the high strength, because sodium methasilicate is floury, while the water glass is a liquid with high viscosity, and molten slag fine powder and alkali stimulative material are sufficiently mixed by pre-mixing. The strength of mortar using the calcium hydroxide as the alkali stimulative material was low.
The calcium hydroxide seems to have no sufficient reactivity as the alkali stimulative material to get a sufficient compressive strength.

3.4 Influence alkali materials/slag ratio on the compressive strength of mortar

Figure 7 shows the influence of alkali material/slag ratio on the compressive strength of mortar at the age of 1 day. It is shown in the figure that compressive strength of mortar increases linearly with the increase of alkali material/slag ratio until it becomes 15% and that the strength increase rate decreases over 15%. It seems to have the same relation between compressive strength at 28 days and alkali material/slag ratio. The fluidity immediately after mixing is improved by increase of the rate. However, the fluidity after 30 minutes lowers when the ratio exceeds 25%.

3.5 Influence of the water/slag ratio on the compressive strength of mortar

Figure 8 shows the relation between compressive strength of mortar at the age of one day and water/slag ratio when it is varied from 30 to 70%. This figure shows that the compressive strength of the mortar decreases linearly with the increase of the water/slag ratio. It seems to have the same relation between compressive strength at 28 days and water/slag ratio, as the compressive strength at the age of 28 days is nearly equal to that at 1 day. The consistency of mortar decreased with the increase of water/slag ratio.

4 COMPRESSIVE STRENGTH OF CONCRETE

The concrete was obtained by adding coarse aggregate to a mortar whose alkali/slag ratio was 22.5% and water/slag ratio was 42.9%. The mix proportion of the concrete is shown in Table 2. An AE agent was used for the air entraining. The concrete showed a high viscosity and fluidity. The slump flow test [JIS A 1150 2001] was applied to the measurement of the fluidity.

The slump flow of the concrete was 605 mm, and the air quantity was 5.2%. Cylindrical specimens of 100 mm diameter and 200 mm height were produced. A part of the specimens was cured in steam, and another part of specimens was cured in water or in air. Steam curing was done at the condition of 3 hours for presoaking time, heating speed for the 15°C/hour, and the maximum temperature for 65°C and duration time...
for 5 hours. Figure 9 shows the compressive strength of concrete. It was confirmed that a high strength is obtained by using ground granulated slag from municipal wastes as cementitious material.

5 CONCLUSIONS

This study examined the substitution of the portland cement with molten slag of municipal waste. The findings obtained from this study can be concluded as follows:

1. The compressive strength of mortar increased, as average particle size of molten slag fine powder became smaller. It is considered that the increase of the strength is caused by the increase of the total surface area and increase of reaction product quantity with the decrease of the average particle size.

2. Steam curing is necessary for the hardening of the molten slag fine powder with the alkali stimulative material. The compressive strength of mortar increases with the increase of curing temperature.

3. The high strength mortar is obtained, when the silicate or sodium hydrate is used as an alkali stimulative material. Especially, the compressive strength of mortar drastically increases by using powderly sodium methasilicate.

4. The compressive strength of the mortar increases with an increase in the additive rate of the alkali stimulative material up to 15%, beyond which the increase rate becomes almost nil.

5. The compressive strength of mortar decreases linearly with the increase of water/slag ratio.

6. It is confirmed that the concrete whose alkali stimulative material/slag ratio is 22.5% and water/slag ratio is 42.9% has high fluidity and high strength.

REFERENCES


Use of supplementary cementing materials for cost-effective repair of marine corrosion damage

D.V. Reddy, J.C. Bolivar, & V. Gutierrez
Center for Marine Structures and Geotechnique, Florida Atlantic University, Boca Raton, FL, USA
Z. Abou Saleh
PTE Strand, Miami, FL, USA

ABSTRACT: Corrosion damage is caused by penetration and diffusion of chloride ions into the outer layer of the concrete, causing the destruction of the gamma-ferric oxide layer on the reinforcement. The selection of materials for repairs of corrosion damage in reinforced/prestressed concrete is more difficult than for new construction. The range of materials used is broader than just cementitious materials, because many polymeric composites can be used effectively for concrete repair. Frequently, the period in which the structure can be brought back into service, the accessibility, and the position of the repair in relation to the level of the tide are also important considerations in the selection of materials and methods for repair. In the earlier analytical investigation by Reddy et al. [2004] the time for chloride concentration to reach the threshold value that initiates corrosion in the reinforcement was determined by Fick’s law, extended to 2–D and 3–D chloride diffusion, for a) uncracked concrete, and b) for cracked concrete with the Simplified Smeared Approach (SSA). The structural integrity of the concrete circular pile was compared, before and after repair, by a) finite element modeling using ANSYS software with the maximum deflection, and b) beam strength analysis to find the moment capacity for cracked and ultimate conditions. The overall analytical findings indicated the adequacy of the repair procedures. The objective of the on-going experimental investigation is the comparison with the analytical values. Six different types of repair are being carried out on sets of three specimens. All the specimens will then be exposed to simulated tidal conditions in seawater tanks to determine their durability. Ultimate strength testing will be carried out to determine the loss of structural integrity.

1 LITERATURE REVIEW

Corrosion of the reinforcement is one of the major reasons for deterioration of reinforced concrete structures. The corrosion process is very complex, and the modeling is often based on observations, or speculations, rather than a clear understanding of the physical and chemical processes.

Reinforced and prestressed concrete structures in a marine environment are under constant attack from various elements, which result in the corrosion of the reinforcing steel, thereby weakening the structure and shortening their lifespans. The longer corrosion is left to progress, the more damage it causes; therefore, early detection is essential. Corrosion of reinforcement causes a volume increase of the corroded part of the rebar material, and splitting stresses are induced in the concrete. This often determines the durability of concrete structures. Existing specifications for concrete do not appear to be effective in ensuring long-term performance in the marine environment, and many structures require repairs during their design lives.

The main cause of corrosion of steel in the marine environment is the presence of chloride in the environment. When the percentage of chloride ions at the steel interface exceeds the threshold value, assuming oxygen is present, corrosion initiates. Depending on the effectiveness of the corrosion-protection measures, the penetration of chloride ions into the concrete from seawater, or deicing salts, can result in the corrosion of the reinforcement.

The main form of attack is the penetration and diffusion of chloride ions into the outer layer of the concrete, causing the destruction of the gamma-ferric oxide layer on the reinforcement, and exposing it to corrosion. The selection of materials for concrete repairs is more difficult than for new construction. The range of materials used is broader than just cementitious materials, because many polymeric composites can be used effectively for concrete repair. Frequently,
the period in which the structure can be brought back into service, the accessibility, and the position of the repair in relation to the level of the tide are also important considerations in the selection of materials and methods for repair.

Figure 2 illustrates the marine pile zones and their corresponding corrosion susceptibility.

Depending on the size, location, and the general function of bridge components, various materials are available for repair. The following influence the selection of materials: a) material compatibility to original concrete, b) environment, including aesthetics, c) cost-effectiveness, d) service life, e) availability, and f) familiarity of the contractors with the product. Therefore, the durability of concrete repair depends both on the careful selection of the repair material, and the method of repairing.

Experience shows that many repairs fail through debonding of the repair material from stresses at the interface caused by drying shrinkage, differential thermal strain, and elastic mismatch [Mehta 1991]. Fig. 3 illustrates a severely deteriorated pile, which had previously been repaired with a fiberglass jacket. High-performance concrete may be differentiated from high strength concrete as that designed for environmental exposure. A well-engineered high-performance repair concrete will typically involve water/binder (cementitious mix) - w/b ratios of 0.40 or lower. In fact, concretes with w/b ratios as low as 0.24 have been investigated in the past, with compressive strengths as high as 140 MPa (20,000) realized, with associated higher tensile strengths.

A calcium nitrite inhibitor is readily available through W.R. Grace under the trade name “Darey Corrosion Inhibitor (DCI)” or through Degussa Construction Chemicals as “Rheocrete CNI (Calcium Nitrite Inhibitor).” The calcium nitrite serves to change the ferrous ions produced when reinforcing steel is placed in the alkaline concrete environment into a stable passive layer on the steel which blocks active corrosion.

A new type of fiber reinforced concrete, Slurry Infiltrated Fibrous Concrete (SIFCON), high performance material containing a relatively higher volume of fibrous concrete, has already been used for
construction of structures subjected to impact, blast, and dynamic loading, and also for refractory applications, overlays, and repair of structural components because of its high tensile strength and ductility.

For over twenty years, polymers in various forms have played an important part in the protection and repair of piles in the marine environment. The long-term performance and cost effectiveness of styrene-butadiene latex-modified concrete as a repair material for concrete has been evidenced by several case histories. Polymer liquid additives have been shown to increase performance in adhesion, thermal compatibility, water-repellent properties, and freezing and thawing resistance.

Composites offer potential replacements for conventional materials, such as steel and concrete, in infrastructure facilities. FRP (Fiber Reinforced Polymer Composite) is a composite in which strong fibers (glass, aramid, carbon, etc.) are embedded in a matrix of plastic (resin). The advantages are high tensile strength, low density, high-energy absorption, impact resistance, and corrosion property.

GFRP (Glass Fiber Reinforced Plastic) bars have high resistance to acids, but can deteriorate in an alkaline environment. In a recently completed study for GFRP bars in concrete applications, a particular type of GFRP with some special matrix resins showed excellent durability under high alkaline environment [ACI Committee 440, 1995].

A limited number of tests for tensile strength, moduli of elasticity, and development lengths were carried out for GFRP (E–glass) and Aramid Fiber Plastic (AFRP Kevlar) rods by Pleiman [1987]. All bars were deformed by helical wrapping with strands on the outer surface. The GFRP rods indicated lower tensile strength and modulus of elasticity, but better bond strength than the AFRP (Kevlar 49) ones.

Even though CFRP (Carbon Fiber Reinforced Plastic) bars also have high resistance to severe environment and higher longitudinal modulus of elasticity than GFRP bars, direct connection to the steel reinforcement or stirrups will cause galvanic corrosion damage on the surface of steel that can cause catastrophic failure of the structure. Splicing of CFRP bars to the steel reinforcement or tendons should be avoided unless proper insulation is provided between the two materials.

In the mid-nineteen eighties, Ohbayashi Co. and Mitsubishi Kasei Co. of Japan developed the concepts of strengthening and retrofitting existing concrete structures using carbon fiber strands and tapes. Three types of structures were targeted: building columns, bridge columns, and chimneys [ACI Committee 440, 1995]. These types of composite strengthening repairs and retrofits typically involve layers of structural epoxy and carbon fiber adhered to external surfaces of concrete structural components.

Glass fiber woven roving can be used to repair concrete structures in a similar manner to carbon fiber UD tape wrapping. Mehta [1991] suggested the use of alkali resistant glass fiber reinforcement wrapping on repaired concrete surfaces, with acrylic rubber emulsion film coating, to strengthen the structure. This would also improve the resistance to ingress of water, chloride, and oxygen for marine concrete structures.

The use of High Density Polyethylene (HDPE) corrugated exterior/smooth interior pipe jacketing, together with high performance fiber reinforced concrete, is an innovative repair method proposed by Reddy [1995]. Many repair processes similar to those reviewed in “Cost-Effective Repairs of Marine Concrete Piles,” by Reddy & Ahn [1999] have been carried out in the past. However, most laboratory and field investigations focused mainly on the elimination or deceleration of the corrosion process in repaired marine piles, without addressing durability characteristics. Of course, corrosion in prestressing and reinforcing steel directly affects the durability of the concrete structure, although little research seems to exist on the strength of actual patch material and bonding agents.

Corrosion of steel reinforcement limits the service life of concrete exposed to sea water. Because of the expansion of the steel due to corrosion, concrete piles are extremely susceptible to cracking and spalling. Although steel reinforcing bars are initially protected by the alkaline nature of the surrounding concrete,
chloride ions from sea water can slowly infiltrate through the concrete cover and into the reinforcement. All chloride contaminated concrete must be removed and replaced with low shrinkage, low modulus, high creep, high tensile strength patch material having the same thermal expansion and oxygen permeability [Vaysbund & Emmons 2000]. Such repairs can barely be carried out under wet conditions and are unrealistic for piles corroding in tidal waters [Mullin et al. 2005].

Surface treatments have been tested by Sagues [1994] at the University of South Florida to determine if the corrosion process can be reduced or eliminated. This process was tested on exposed marine piles above the water line. The two types of surface treatments used included an alkyl alkoxy silane and an alkyl alkoxy siloxane. After a three-year testing period, both the silane and siloxane were beneficial in preventing water ingress, but did not cause water to be retained to the extent that unwanted enhancement of corrosion macrocell action would have taken place. Although the Sagues [1994] test was not for an actual concrete repair procedure, it offers valuable insight on surface treatments that can aid in decelerating the corrosion process.

In the past, replacing damaged concrete piles was considered more cost-effective and efficient than repairing. However, new technologies, along with the ever increasing cost for new construction are focusing attention on different repair techniques. The Allen Creek Bridge located in tidal waters in Clearwater Florida has recently undergone repair on its prestressed piles [Mullin et al. 2005]. Carbon and glass fiber-reinforced polymer material were wrapped underwater to repair damaged concrete. The underwater wrap consisted of a water-cured polyurethane resin system that can be used with woven glass fabric, unidirectional glass fabric, woven carbon fabric, or unidirectional carbon fabric. This system eliminated the need for cofferdam construction, which is required for dewatering of a site that may adversely cause restrictions to the cross-section of the water body. Linear polarization measurements, taken before and after the wrapping, showed the corrosion in the wrapped section to be consistently lower than that of the unwrapped sections. The findings of Mullins et al. [2005] are encouraging, and suggest that underwater wrapping, without cofferdam construction, may provide a cost-effective solution for pile repair.

Sen & Mullins [2005] of the University of South Florida also conducted a research program using fiber-reinforced polymers to reduce the post-repair corrosion rate in prestressed piles. Their experiments addressed four objectives: (1) The effect of the fiber type and number of FRP layers on the corrosion rate, (2) The effect of exposure on the FRP-concrete bond, (3) The role of surface preparation on FRP performance, and (4) Strengthening the effectiveness of underwater wrap using a newly developed water-actuated resin. Both ambient and accelerated conditions results showed that the FRP wrap was successful in slowing down the rate of corrosion.

Whiting et al. [2000] simulated marine conditions to test conventional repairs, such as patches, sealers and coatings. Their study included a survey of twelve prestressed bridges with evidence of deterioration due to corrosion, as well as an extensive laboratory examination of the repairs. Tests included visual examinations, cover, survey, delamination surveys, half cell potential measurements, corrosion rate (linear polarization) measurements, chloride sampling, and petrographic analysis. All specimens were located in the United States, and had been previously exposed to either the marine environment or inland exposure areas. Materials evaluated in the laboratory repairs included conventional portland cement concrete, latex-modified fiber reinforced patching mortar, and silica fume concrete containing either organic or inorganic corrosion inhibitors. Specimens were exposed to seawater for approximately 200 weeks. The beams and slabs were subjected to wet and dry cycles, and the piles were exposed to a 24-hr immersion cycle. After testing, Whiting et al. [2000] concluded that there was more deterioration and corrosion in the steel where latex- modified mortar had been used as repair material than where conventional concrete or silica fume concretes were used. Results also showed that the inorganic corrosion inhibitors succeeded in decelerating corrosion rate more so than the organic inhibitors. Although these materials slowed the process of corrosion, they did not appear to ensure long-term protection and should only be considered for temporary repairs.

The method of choice for repairing one of the longest bridges, the Lake Pontchartrain Causeway located in New Orleans, consisted of using an all-polymer encapsulation process. A report written by Trader of the MADCON Corporation described the procedures and findings. Engineers picked the all-polymer or advanced encapsulation process (A-P-E), over other repair processes, such as epoxy paste, crack injection, and several wrap methods, because they believed the A-P-E would not only seal the cracks, but provide a composite barrier that would completely surround the affected length of the pile and eliminate any further deterioration. The basic features of the repair procedure included using translucent FRP jackets and pumping aggregate filled polymer grout into the jackets from the bottom up. In 1998, 21 piles were encapsulated for a length of ten feet in the splash zone. After service for seven years, tests were performed by taking several cores through the old encapsulations. The cores revealed that a tightly bonded composite barrier was still in place. The piles showed no evidence of further deterioration. Tensile tests were also performed on the specimens to determine the bond strength of the
encapsulated materials. The test method chosen was an in-situ direct bond test using the Modified Electrometer Method. When the specimens were tested to failure, the tensile strength exceeded that of 8500 concrete. The results of the all polymer encapsulation process were convincing enough for the engineers to want to use the same process on other damaged piles of the Lake Pontchartrain Causeway Bridge.

While the engineers of the Lake Pontchartrain Causeway claim to have positive results using jackets, Sohanajpurwala & Scannel [1994] indicate that there is no real benefit against corrosion. Two of the reasons are as follows: (1) Capillary action allows water from the submerged section of the pile to rise up the pile, and (2) High levels of chloride ions remain in un-repaired areas. Corrosion may actually be accelerated within these repair zones, because the concrete is never allowed to dry out and there is always a strong presence of oxygen. One example of jackets having a detrimental effect was found in the pilings at the Bryant Patton Bridges in Florida. The Florida Department of Transportation found that over 50% of the piles, that had been repaired previously using fiberglass jackets, were now deficient. This caused FDOT to start a new rehabilitation project that required the addition of supplementary piles for each and every pile that had been jacketed.

1.1 Analytical procedures for determining chloride diffusion in concrete and effectiveness of marine concrete pile repairs

Funahashi [1990] developed a finite difference method by which the future chloride concentration could be calculated from the present chloride ion concentrations. As an example, a detailed investigation was conducted on a selected area, where high chloride ion contents were found. To estimate the remaining life of the deck system of an existing parking garage, cored concrete powder specimens at different depths were obtained by the present chloride concentration. Using the known interior and boundary grid pattern with time (in years) and concrete depth was predicted. If it is assumed that a threshold chloride concentration to initiate corrosion exists, it is possible to calculate the time when the chloride ion concentration will be exceeded at any given depth.

Funahashi [1990] evaluated the chloride diffusion coefficient from the specimens taken from the aforementioned parking garage to be $0.54 \text{ cm}^2/\text{year}$. The threshold chloride concentration of 250 parts per million (ppm) was used and the future chloride concentrations plotted. The surface chloride concentration for the diffusion problem was assumed to be a linear function of the square root of the age of the structure. From the diffusion coefficients obtained from experiments on actual structures and the time dependent constant coefficient mentioned above, the chloride concentration and initiation time of corrosion were predicted.

The calculated chloride profiles related closely to the experimental values. Hence, it was concluded that it was safer to estimate chloride content distribution by assuming that the surface chloride content changes with time, than by assuming it to be constant.

The two following approaches have been recently developed for modeling the behavior of chloride into cracked concrete media: (1) A smeared approach presented by the Japanese Society of Concrete Engineers [1999], where diffusivity and permeability increase according to the extent of damage in the cracked area as measured by a damage variable; and (2) A discrete approach by Tsukahara & Uomoto [2000] where the crack geometry is explicitly represented by two surfaces between which flow occurs, and an exchange of moisture and chloride ions is allowed between the crack and the surrounding concrete.

Reddy et al. [2004] carried out an analytical investigation to compare the performance characteristics of marine piles corroded by chloride diffusion, and to determine their structural integrity after repair by several different methods, for both uncracked and cracked concrete. The long-term objective is the comparison of the analytical values with those from the on-going experimental evaluation. The time for chloride concentration to reach the threshold value that initiates corrosion in the reinforcement was determined by Frick’s law, extended to 2-D and 3-D chloride diffusion, for a) uncracked concrete, and b) for cracked concrete with the Simplified Smeared Approach (SSA). The structural integrity of the concrete circular pile was compared, before and after repair, by a) finite element modeling using ANSYS software with the maximum deflection, and b) beam strength analysis to find the moment capacity for cracked and ultimate conditions.

1.2 Numerical analysis of corrosion

1.2.1 Assumptions for analysis

The following assumptions are evaluated and modified for the estimation of time for chloride concentration at reinforcing bar level to reach threshold value.

1. The initial chloride concentration on the surface is assumed to be 3.5% by weight of cement i.e. $C_0 = 3.5$ considering seawater.

2. The threshold value for the corrosion of steel is taken as 0.4% chloride by weight of cement [Tutti, 223–236].
1.2.2 Diffusion constants
In view of the scatter of diffusion coefficient values in the literature reviewed, Kuhlmann [1981] used the empirical equation-based values of the Japanese Society of Civil Engineers (JSCE), together with values obtained from the Simplified Smeared Approach (SSA), and some other sources to postulate applicable values of D as functions of water-cement ratio (w/c).

1.2.3 Diffusion constants for uncracked concrete
Equation 1 (JSCE based) pertains to the case where concrete contains BFS and SF (blast furnace slag and silica fume), and Equation 2 applies to concrete mixtures that do not contain BFS or SF. In other words, Equation 2 (JSCE based) is for concrete with no additives and/or concrete with fly ash (FA), but without BFS or SF. i.e. (normal concrete).

\[
\log D = -3.0(w/c)^{2} + 5.4(w/c) - 13.7 \quad (1)
\]

\[
\log D = -3.9(w/c)^{2} + 7.2(w/c) - 14.0 \quad (2)
\]

1.2.4 Diffusion constants for cracked concrete
Equation 3 assumes that chloride ions ingress into cracked concrete can be approximated using Fick’s Second Law of diffusion in which the following average coefficient is used:

\[
D_{av} = D + (w/l) D_{cr} \quad (3)
\]

where \(D_{av}\) is the average diffusion coefficient; \(D\) is the diffusion coefficient of the uncracked concrete; \(w\) is the crack width; \(l\) is the crack spacing; and \(D_{cr}\) is the diffusion coefficient inside the crack.

The average diffusion coefficient is hence proportional to the crack width and inversely proportional to the crack spacing.

1.2.5 Uncracked concrete (Fick’s law)
1.2.5.1 One-dimensional Equation
Prior to any major crack creation, concrete is considered only a uniformly porous material. Chloride transport is classically modeled using Fick’s Law of Diffusion or one of its time dependent variants as follows:

\[
\frac{\partial C}{\partial t} = \frac{\partial}{\partial x} [D(x,t) \frac{\partial C}{\partial x}] \quad (4)
\]

where \(C\) is the concentration of chlorides at distance \(x\); and \(D\) is the diffusion coefficient.

Equation 4 can be solved analytically for different boundary conditions, depending on the exposure conditions, to estimate chloride profiles as a function of time \(t\), position \(x\), and the diffusion coefficient \(D\). Assume that the concrete to be exposed to a constant chloride concentration \(C_{(x,0)} = C_{0}\).

The amount of chloride at a given distance \(x\) and time \(t\) is given by:

\[
C_{(x,t)} = C_{0} [1 - \text{erf} \left( \frac{x}{\sqrt{4Dt}} \right)] \quad (5)
\]

where \(C_{0}\) is the equilibrium chloride concentration on the concrete surface, given, as % by weight of cement; and erf is the error function.

1.2.5.2 Two-dimensional Equation
To take the two-dimensional nature into consideration, Equation 4 has to be modified as follows:

\[
\frac{\partial C}{\partial t} = D \left( \frac{\partial^{2} C}{\partial x^{2}} + \frac{\partial^{2} C}{\partial y^{2}} \right) \quad (6)
\]

Ranade [1993], as well as Reddy & Ahn [1995], have derived the solution of the Equation 6 above in terms of the error function as follows:

\[
C_{(x,y,t)} = C_{0} \left[ 1 - \text{erf} \left( \frac{x}{\sqrt{4Dt}} \right) - \text{erf} \left( \frac{y}{\sqrt{4Dt}} \right) \right] \quad (7)
\]

1.2.5.3 Three-dimensional Equation
In order to take the three-dimensional nature into consideration, Equation 6 has to be modified as follows:

\[
\frac{\partial C}{\partial t} = D \left( \frac{\partial^{2} C}{\partial x^{2}} + \frac{\partial^{2} C}{\partial y^{2}} + \frac{\partial^{2} C}{\partial z^{2}} \right) \quad (8)
\]

The solution to this equation has been postulated as an extension of that in Equation 7.

\[
C_{(x,y,z,t)} = C_{0} [1 \cdot \text{erf} \left( \frac{x}{\sqrt{4Dt}} \right) \cdot \text{erf} \left( \frac{y}{\sqrt{4Dt}} \right) \cdot \text{erf} \left( \frac{z}{\sqrt{4Dt}} \right)] \quad (9)
\]

1.2.6 Cracked concrete (SSA approach)
Simplified Smeared Approach (SSA)-crack development has been shown to be controlled by a number of factors, including member geometry and loading, variability in concrete properties, volumetric changes in the concrete, amount and distribution of the reinforcement and bond between the concrete and the reinforcement. The distances between cracks randomly vary over a very wide range, with the maximum crack spacing being about twice the minimum crack spacing. As a result, there will be a large variation in crack spacing. Transverse reinforcing bars act as crack initiators and hence the crack spacing.

1.2.6.1 One-dimensional Equation
According to Boulfiza et al. [2000] the amount of chloride diffusion is assumed to obey Fick’s law under variable boundary conditions and at a given time by

\[
C_{(x,0)} = s \sqrt{t} \cdot \exp \left( \frac{-x^2}{4Dt} \right) \cdot 1.57x \cdot \text{erfc} \left( \frac{x}{\sqrt{4Dt}} \right) \quad (10)
\]
where \( s \) is an empirical constant, and \( \text{erfc} \) is the complementary error function.

1.2.6.2 Two-dimensional and 3-D Equations
To take the two-dimensional and three-dimensional natures into consideration, by comparison Equation (10) was modified as follows by Reddy et al. [2004]:

\[
C_{(x,y)} = s\sqrt{t} \left[ \exp \left( -\frac{x^2}{4Dt} \right) \exp \left( -\frac{y^2}{4Dt} \right) \frac{1.57x}{\sqrt{4Dt}} \right] \text{erfc} \left( \frac{x}{\sqrt{4Dt}} \right) \exp \left( -\frac{z^2}{4Dt} \right) \frac{1.57z}{\sqrt{4Dt}} \right] \text{erfc} \left( \frac{y}{\sqrt{4Dt}} \right)
\]

(11)

\[
C_{(x,y,z)} = s\sqrt{t} \left[ \exp \left( -\frac{x^2}{4Dt} \right) \exp \left( -\frac{y^2}{4Dt} \right) \exp \left( -\frac{z^2}{4Dt} \right) \frac{1.57x}{\sqrt{4Dt}} \right] \text{erfc} \left( \frac{x}{\sqrt{4Dt}} \right) \text{erfc} \left( \frac{y}{\sqrt{4Dt}} \right) \text{erfc} \left( \frac{z}{\sqrt{4Dt}} \right)
\]

(12)

1.3 Finite element analysis of strength

1.3.1 Assumption for analysis
Most common engineering materials exhibit a linear stress-strain relationship up to stress level known as the proportional limit. Beyond this limit, the stress-strain will become nonlinear not necessarily become inelastic. Plastic behavior, characterized by non-recoverable strain, begins when stresses exceed the material’s yield point. Because there is usually little difference between the yield point and the proportional limit, the ANSYS program assumes that these two points are coincident in plasticity analyses. The structural system for the pile is considered as a simply supported beam, with hinges at the ends.

1.3.2 Element type
The element SOLID65 in ANSYS was used by Abou-Saleh & Reddy [2003] for the three-dimensional modeling of solids, with or without reinforcing bars (reinforcing bars). The solid is capable of cracking in tension and crushing in compression. In concrete applications, for example, the solid capability of the element can be used to model the concrete and the reinforcing bar capability for modeling reinforcement behavior. Other cases for which the element is also applicable would be reinforced composites (such as fiberglass), and geological materials (such as rock). The element is defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions. Up to three different reinforcing bar specifications may be defined.

The most important aspect of this element is the treatment of nonlinear material properties. The concrete is capable of cracking (in three orthogonal directions), crushing, plastic deformation, and creep. The reinforcing bars are capable of tension and compression, but not shear. They are also capable of plastic deformation and creep.

1.3.3 Material properties
\( E = 3,600 \) for undamaged concrete
\( E_p \) = varies for repair specimens

1.3.4 Geometrical modeling
3D solid cylindrical shape was used as the first step for modeling undamaged concrete, and followed by a simulated patch solid repair area by using a partial solid cylinder. In order to differentiate between undamaged and repair concrete, the command Boolean was used. Using Boolean operators, the solid model can be “sculpted” using intersections, subtractions, and other Boolean operations. Booleans allow working directly with higher solid model entities to create complex shapes. Booleans-Subtract was used to take apart the materials. Also, the Booleans-Glue function found in the ANSYS Engineering Analysis User’s Manual [1998] has been used for bonding between the repair and undamaged areas. The different materials can be specified using the element as the attribute before meshing.

1.3.5 Loading
To compare the structural integrity before and after repair, point force \( P \) at midspan was calculated from the capacity of section before the repair i.e. specific plastic deformation at midspan of beam.

\[
M = \frac{l}{4} P
\]

(13)

where \( M \) is the moment due to \( P \); and \( l \) the beam span.

The plastic moment equation for circular section is given by:

\[
M_p = S f\gamma
\]

(14)

Table 1. Material properties.

<table>
<thead>
<tr>
<th>Repair material</th>
<th>Modulus of elasticity ( E_p ) (GPa)</th>
<th>Compressive strength ( f'_{pc} ) (GPa)</th>
<th>Unit weight ( \gamma_p ) (kg/m(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>SIFCON</td>
<td>42.29</td>
<td>55.15</td>
<td>4309</td>
</tr>
<tr>
<td>Styrene butadiene latex mortar</td>
<td>48.95</td>
<td>59.98</td>
<td>4806</td>
</tr>
<tr>
<td>Silica fume</td>
<td>51.71</td>
<td>61.36</td>
<td>5126</td>
</tr>
<tr>
<td>Normal concrete</td>
<td>30.44</td>
<td>41.37</td>
<td>2403</td>
</tr>
<tr>
<td>High performance concrete</td>
<td>41.79</td>
<td>55.15</td>
<td>4277</td>
</tr>
</tbody>
</table>
where

\[ f'_{y} \text{ is the yield stress} = 0.4f_{c} \]  \hspace{1cm} (15)

\[ f'_{c} = \text{Compressive strength for undamaged concrete}, \]  \hspace{1cm} (16)

\[ f_{c} = 28 \text{ Mpa (4000 psi)} \]

\[ S = \text{plastic modulus for circular section} = d^3/6 \]

\[ d = \text{diameter of beam} = 6'' \]

\[ M = M_p \]

\[ P = 0.8 f'_{c} \frac{d^3}{3I} \]  \hspace{1cm} (17)

In order to compare the strength of structure before and after repair, the maximum deflection has been calculated due to the force \( P \) before repair as follows. For a simply supported beam due to concentrated force \( P \) at mid-span

\[ \Delta = \frac{PL^3}{48EI_0} \]  \hspace{1cm} (18)

Where

\[ \Delta = \text{maximum deflection}; \]

\[ I = \text{the beam length}; \]

\[ E = \text{modulus of elasticity before repair}; \]

\[ I_0 = \text{gross moment of inertia}. \]

\[ I_0 = \frac{P\pi d^4}{64} \]  \hspace{1cm} (19)

1.4 Overview of proposed repairs

1.4.1 Ongoing experimental investigation

Six different types of repair, reviewed by Reddy & Ahn [1999] are being carried out on sets of three specimens. All the specimens are exposed to simulated tidal conditions in seawater tanks. Ultimate strength testing will be carried out to determine the loss of structural integrity. Each of six individual mixes was used to repair three piles making a total of 18 specimens. Prior to patching, the reinforcing steel in each specimen was coated with zinc-rich reinforcing bar primer to protect against corrosion during the wet and dry cycles. Forms were then attached to all the 18 concrete piles. The mixture proportions had been previously calculated; however, adjustments to the water/binder ratio were necessary to increase the lateral flow of the concrete mixtures. This was done to ensure that the volume of voids was kept as small as possible. A concrete mixer was used to mix all the concrete and all pouring was done by hand without the use of pumping or shotcrete. Tamping and vibration of the concrete was carried out during the pouring to ensure a minimal void volume. For mixes requiring surface treatments, a time period of 3–4 days after repair was required for curing times of patch material. The procedures and final mixture proportions are as follows:

1.4.1.1 Repair 1: Patch Repair with Slurry-Infiltrated Fiber Concrete, SIFCON, and spliced FRP (glass/aramid) reinforcement

After the steel reinforcement was coated, it was spliced with the FRP reinforcement, Figure 6. This was achieved by simply cutting the FRP bars to an equal length of the exposed steel reinforcement, and attaching the FRP reinforcement to the steel using wire ties. The ties were also coated with the zinc-rich reinforcing bar primer, Figure 7.

The forms were then mounted and the concrete mix poured. The mix comprised Type I and Type II portland cement, sand, fly ash, water, Florida peacock aggregate, and polyolefin fibers. The proportions were as follows: 1:2 water to binder, 1:1.25:1.25 (binder:sand:aggregate). The polyolefin fibers consisted of 5.85% by volume in the SIFCON mix. The amounts used for the repair per cylinder are presented in Table 2.
1.4.1.2 Repair 2: Styrene-butadiene latex polymer grout patching followed by prewet glass fabric wrapping

Sika 211 which consists of prepacked aggregate and cement was used in this mix along with sand, water, and lime. The proportions were as follows: 1:2.65 water to binder and 1:1.25 binder to sand, with the binder including the aggregate. Lime was added at 10% by volume of Sika 211. Fig. 9 shows the pouring process for the specimens.

After the mix was cured, surface treatments consisting of incorporating a styrene-butadiene latex mortar, Sika Latex R, and a vinylester surface coating, Sika 670W, will be used to provide effective salt-damage protection. The amounts used in the repair for each cylinder are given in Table 3.

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sika 211</td>
<td>4.55</td>
</tr>
<tr>
<td>Sand</td>
<td>5.68</td>
</tr>
<tr>
<td>Water</td>
<td>1.70</td>
</tr>
<tr>
<td>Lime</td>
<td>1.50</td>
</tr>
</tbody>
</table>

Table 3. Repair 2 – material quantities.
1.4.1.3 *Repair 3: Silica fume concrete patching followed by carbon tape and carbon fiber strand wrapping*

This mix consisted of Type I and Type II portland cement, sand, Florida Pearock Aggregate, water, and silica fume with proportions of 1:1.6 water to binder, 1:2.2:2.5 (binder:sand:aggregate). Silica fume was added as replacement of 8.5% by weight of cement, as shown in Figure 11.

The surface of the repair was then wrapped with carbon strand, and then covered with a layer of Sikadur 301 mix, in order to glue the wrapping to the specimens. The amounts used in repairs for each cylinder are shown in Table 4.

1.4.1.4 *Repair 4: Conventional normal concrete patch repair with spliced FRP (glass/aramid) reinforcement*

FRP reinforcement was spliced with the steel reinforcement with the same procedure as Repair 1 before placement of the forms. The concrete mix consisted of Type I and Type II portland cement, sand, Florida Pearock aggregate, water, and gypsum, Figure 13.

The proportions were as follows: 1:1.25 water to binder, 1:1.4:1.28 (binder:sand:aggregate). Gypsum was added at 10% by weight of cement to facilitate the build-up of thick layers. The amounts used in repairs for each cylinder are shown in Table 5.

1.4.1.5 *Repair 5: High performance fiber reinforced concrete patching with HDPE jacket*

Repair 5 involved using Type I portland cement, superplasticizers, Florida Pearock aggregate, sand,
silica fume, fly ash, water, and polyolefin fibers. The water to binder ratio was 1:2.88, with silica fume and fly ash each contributing 6.5% to the binder (cementitious mix). The remaining proportions were as follows: 1:1.22:1.09 (binder:sand:aggregate). Polyolefin fibers were added at 1.5% by total volume and the superplasticizer at 2% by weight of cement.

The patched region sets were wrapped with two semi-cylindrical pieces of corrugated HDPE pipe and sealed together with special polyurethane resin. The amounts used in repairs for each cylinder are shown in Table 6.

### 1.4.1.6 Repair 6: Modified ASANO refresh method

This mix consisted of Sika 211 with prepacked aggregate and the Sika Latex R as the styrene-butadiene latex mortar. Water, sand, and lime were also included in the mix. The water to Sika 211 ratio was 1:2.27, and

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>3.64</td>
</tr>
<tr>
<td>Water</td>
<td>1.45</td>
</tr>
<tr>
<td>Aggregate</td>
<td>4.55</td>
</tr>
<tr>
<td>Sand</td>
<td>5.09</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>0.30</td>
</tr>
<tr>
<td>Fly Ash</td>
<td>0.30</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>0.07</td>
</tr>
</tbody>
</table>

Table 6. Repair 5 – material quantities.
the Sika 211 to sand ratio 1:1.25 (binder:sand, with the binder including the aggregate). The Sika Latex R was mixed with 10% by weight of Sika 211. Lime was added at only 5% by volume of the Sika 211.

Surface coatings of the material, including salt damage prevention finish material, consisting of organic paint, and Sika 670 W (vinylester) were used for the final processing. The amounts used in repairs per cylinder are shown in Table 7.

### Table 7. Repair 6 – material quantities.

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sika 211</td>
<td>4.55</td>
</tr>
<tr>
<td>Water</td>
<td>2.00</td>
</tr>
<tr>
<td>Lime</td>
<td>0.09</td>
</tr>
<tr>
<td>Sika Latex R</td>
<td>0.45</td>
</tr>
<tr>
<td>Sand</td>
<td>5.68</td>
</tr>
</tbody>
</table>

2 CONCLUSIONS

The process of repair has so far proven to be suitable to perform additional testing under seawater exposure. The comparison of the results from upcoming testing, such as chloride diffusion rates and loss of structural integrity, with the results obtained from the previous research by Reddy et al. [2004] will allow to provide an assessment of the cost-effectiveness of each repair method, and will enable the development of appropriate repair and rehabilitation techniques for concrete marine structures.

3 ACKNOWLEDGEMENTS

The authors would like to thank the National Science Foundation for financial support of the project entitled “Cost-Effective Repair of Marine Piles”, under the U.S.A.-Mexico International Cooperative Program, Grant No. CMS-0116602, from 2001 to 2006. Gratitude is expressed to the Contract Monitor Dr. Larsen-Basse, for his interest and encouragement. The authors cannot sufficiently thank Mr. Bruce Stoerkel, of Structural Preservation Systems, Inc., Mr. Richard Ford, of Florida Waterproofing Supplies, and the Sika Corporation for their generous assistance with gifts of materials and participation in the repair work.

REFERENCES


ACI Committee 440 1995. A state-of-the-art report on Fiber Reinforced Plastic (FRP), reinforcement for concrete structures. American Concrete Institute, Detroit, MI, USA.


Trader, B., Pile restoration of the Lake Pontchartrain Causeway, world’s longest bridge. MADCON Corporation, Ellicott City, MD, USA.


Change of concrete properties due to impressed current of cathodic protection

T. Suginoue  
West Japan Railway Company, Osaka, Japan

H. Seki  
Dept. of Civil and Environmental Eng., Waseda University, Tokyo, Japan

ABSTRACT: This study has been carried out to clarify changes to the properties of concrete surrounding anode material when cathodic protections is adopted to protect corrosion damage to reinforced concrete members. An accelerated impressed current was applied between anode and cathode within a short period, so that the accumulated current density corresponded to a period of 40 years. The specimens were soaked in water to increase the concrete’s conductivity. The instant-off potential of the anode material remained higher than that obtained from NACE, but was within 4.0 V and thus satisfied NACE. After the accelerating impressed current test, a test was carried out on the adhesion test between lower concrete and overlay material. To determine changes in properties of overlay mortar, EPMA and SEM analysis were adopted for overlay mortar surrounding the anode material. Test results indicated the following:

(1) There was no degradation of adhesion between concrete and overlay after the severe accelerated test.
(2) The properties of overlay mortar surrounding the anode area were changed by localized severe impressed current. Some of concrete components, such as Ca, liquated out, so that the cement mortar probably became porous.

1 INTRODUCTION

Cathodic protection is probably effective in minimizing deterioration of RC structures caused by corrosion of reinforcing bars. A merit of this method is that it prevents corrosion of reinforcing bars without damaging the concrete. This method may prolong a structure’s service life. When using an impressed current system, it is necessary to examine its influence on the material surrounding the anode material, particularly its durability. NACE (National Association of Corrosion Engineers) suggests a method for testing the anode material, in which its is immersed in a solution to evaluate its durability. In a pilot experiment which used the anode material in a new structure as embedding electrode to the concrete beam member, the crack was generated by applying excessive impressed current on the surface of structure. A large current impression is required when the electrode’s charge area is increased. Therefore, the electric current from the anode material must be restricted. In the present study, experimental work was carried out to determine the influence of impressed current on surrounding materials, i.e., mortar material. The impressed current from the anode material was excessive.

2 TESTING METHOD

2.1 Specimens

The shapes of the test specimens are shown in Fig.1. They consisted of two layers. The lower layer was made of concrete with a PC rod embedded in it, and its upper surface was sandblasted. After the concrete hardened, a titanium mesh anode was attached to the surface, and the upper layer was then overlaid on it. The overlay material was cement mortar of a type usually used for repair of concrete members. Table 1 shows the kinds of test specimens. Experimental parameters were accumulated

![Figure 1. Shape of Specimen.](image-url)
current density, present current density, and existence of chloride in overlay mortar. The concrete mixture proportions are shown in Table 2.

### 2.2 Test Method

The test specimens were placed under water to reduce their electrical resistance, as shown Fig.2, and a direct current was applied using a constant-current power supply. The experimental parameters were electrical potential, adhesive strength between lower concrete and overlay mortar and analysis of overlay mortar around the anode.

The potentials of both anode and cathode were monitored.

In the adhesive strength test, the overlay mortar was cut to about $50 \times 50$ mm, a tension jig was attached by adhesive to the overlay material, and a tensile force was vertically applied with the jig. EPMA and SEM analyses of the overlay mortar were carried out.

### 3 CHANGE OF MATERIALS SURROUNDING ANODE

#### 3.1 Change of Potential

Examples of potential monitoring results are shown in Fig.3. The left side shows a test specimen without chloride, and the right side shows one with chloride.

![Figure 2. Method of Applying Current.](image)

![Figure 3. Test Results of Potential Monitoring.](image)
The potential of the anode material in the test specimen without chloride increased with time, but the instant-off potential was almost constant. This may have been because the organization of overlay mortar was altered by the lowering of the pH surrounding the anode material. The overlay material near the anode material became discolored.

The potential change of the anode material is shown in Fig.4. The potential did not exceed 4.0 V, and it is clear that anode material satisfied the NACE standard.

3.2 Adhesion property

Table 3 indicates the test results of adhesive strength between the lower layer concrete and the upper layer overlay mortar after the electric current was applied. There was no significant change in adhesive strength. The adhesive strength should generally be over 1.0 N/mm², and the present results satisfied this condition.

3.3 Change of properties on overlay area

3.3.1 Property surrounding anode material

The area surrounding the anode material was observed before and after electric current was applied. Fig.5 shows an example of a visual observation, where the area around the anode material became slightly white. In the figure, the upper part is the overlay mortar and the lower part is the concrete base material. This

Table 3. Test result of adhesion strength.

<table>
<thead>
<tr>
<th>No. specimen</th>
<th>Fracture plane*</th>
<th>Adhesive strength, N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Value</td>
<td>Average</td>
</tr>
<tr>
<td>0-0A 1 conc.</td>
<td>1.90</td>
<td>1.92</td>
</tr>
<tr>
<td>2 conc.</td>
<td>2.08</td>
<td></td>
</tr>
<tr>
<td>3 conc.</td>
<td>2.12</td>
<td></td>
</tr>
<tr>
<td>0-0B 1 conc.</td>
<td>2.11</td>
<td>1.77</td>
</tr>
<tr>
<td>2 conc.</td>
<td>1.52</td>
<td></td>
</tr>
<tr>
<td>3 conc.</td>
<td>2.26</td>
<td></td>
</tr>
<tr>
<td>1-1A 1 inter.</td>
<td>2.51</td>
<td>2.09</td>
</tr>
<tr>
<td>2 inter.</td>
<td>1.79</td>
<td></td>
</tr>
<tr>
<td>3 conc.</td>
<td>2.24</td>
<td></td>
</tr>
<tr>
<td>1-1B 1 inter.</td>
<td>1.49</td>
<td>1.83</td>
</tr>
<tr>
<td>2 inter.</td>
<td>1.82</td>
<td></td>
</tr>
<tr>
<td>3 conc.</td>
<td>2.07</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.81</td>
<td></td>
</tr>
</tbody>
</table>

* conc.: lower layer of concrete fails near interface.
inter.: interface zone between lower layer and upper fails.

Figure 4. Test result of NACE.

Figure 5. Visual observation of anode area.
phenomenon was remarkable when the applied current was high and chloride was added.

3.3.2 Analysis by EPMA
Plane analysis of EPMA (Electron Probe Micro Analyzer) was carried out to clarify the above phenomenon. An example of an analysis result is shown in Fig. 6.

The Ca analysis result was as follows. In the whitened area of the overlay material, the Ca concentration was extremely low. That is to say, the Ca of overlay mortar surrounding the anode material dissolved in capillary water and liquated out.

Si concentrated in whitened region surrounding the anode material.
Cl accumulated on the periphery of the whitened region, and it was confirmed that it concentrated along the ring region. The EPMA analysis was carried out after the test specimen was dried after the electric current was turned off. Thus, the Cl probably moved during the drying stage. It was confirmed that the concentration of chloride in the anode material decreased after the electric current was applied because of leaching of chloride.

3.3.3 Analysis by SEM
An enlarged view was obtained by SEM (Scanning Electron Microscope) in order to examine in detail the property changes of the overlay mortar around the anode material. Fig. 7 shows an analysis result.

In the figure, (a) and (c) denoted the overlay material at the interface of the overlay mortar and the anode material, and (b) and (d) indicate the overlay mortar in the vicinity of the anode material.
Before applying electric current, SEM photos were taken. At the interface of the overlay mortar and the anode material, crystals of acicular ettringite and layered calcium hydroxide were scattered in a non-dense state. The mortar in the vicinity of anode material was comparatively dense, and calcium hydroxide and C-S-H hydrate accumulated.

After the current was turned off, gypsum crystals were observed at the interface of the overlay mortar and the anode material. The organization of the mortar in the vicinity of anode material became porous and silica gel was observed. Therefore, the calcium hydroxide disappeared into the interface of the overlay mortar and the anode material, and Ca in the ettringite dissolved out. Gypsum was formed in these areas.

In the vicinity of the anode material, the calcium hydroxide disappeared and Ca disappeared from the C-S-H hydrate. As the result, the porous silica gel formed Si.

In the present experimental work, the applied current was excessive compared to the normal cathodic protection. The instant-off potential of cathode material may not rise up to over 0.5 v vs.CSE in the actual stage under the applying current density of 20 mA/m². When cathode material is suitably arranged and the applied current is small, these phenomena do not occur.

These phenomena are rarely produced, but present experiment verifies them by the application of partial and excessive current. In the 1-3A test-piece in which the applied current was small, there was very little discoloration of concrete around the anode material.

4 SUMMARY

Electric current was applied between anode material and PC tendons to test the assumption of cathodic protection. Change of overlay material properties was experimentally examined. During the test, the potential of the anode material did not change much, being 4 V or less. The experiment clarified the following.

(1) Although the potential of the anode material in the test specimen without chloride increases with time, the instant-off potential was almost constant.

(2) Adhesive strength between lower layer concrete and upper layer overlay mortar was over 1.0 N/mm². Excessive current did not damage the adhesion between the lower concrete and the overlay mortar.

(3) The environment around the anode material acidified when an excessive current partially flowed into the anode material. In addition, components such as Ca dissolved in the overlay material, and the organization became porous.

REFERENCES


1 INTRODUCTION

When concrete is exposed to the atmosphere over an extended period, atmospheric carbon dioxide diffuses into the capillary pores of the cement and reacts with the calcium hydroxide, cement hydrates, residual unhydrated cement, and alkalis within the pore solution of hardened cement matrix to produce calcium carbonates and bicarbonates, hydrated silica and alumina, and alkali carbonates. This process is known as carbonation. Carbonation of calcium hydroxide is normally supposed to occur at a faster rate than carbonation of other cement hydrates [St Jones & Poole 1998, Ngala & Page 1997, Gonzalez et al. 1983]. The rate and extent of carbonation by atmospheric CO2 is partly dependent on the chemical nature of the cement hydration products but is mainly influenced by the porosity and water content of the exposed cement surface [Knopf et al. 1999]. The process reduces the hydroxide concentration in the pore solution and lowers the pH to about 8.3, depending on the HCO3⁻ /CO3²⁻ equilibrium which in turn will be controlled by both the amount of carbonation and alkalis present [Page & Treadaway 1982, Ngala & Page 1997, Gonzalez et al. 1983, Van Gerven et al. 2004].

The carbonation process under atmospheric conditions is however very slow and it may take years for full carbonation to develop within the structure. This is due mainly to the low partial pressure of CO2 in the atmosphere, about 0.03–0.06% v/v, and the slow rate of diffusion of CO2 into the structure of hardened cement paste [Fernandez Bertos et al. 2004].

Passive, atmospheric carbonation of reinforced concrete is normally seen as deleterious owing to the depassivation of reinforcing steel caused by the reduction in pore solution pH. However, the effect on plain concrete is to reduce porosity and increase compressive strength. If carbonation could be effected within reasonable timescales, then active carbonation might be of interest in the production of e.g. pre-cast concrete elements. By using supercritical CO2 technology, the carbonation process can be significantly accelerated, from years to a matter of a few hours.

2 SUPERCRITICAL CARBON DIOXIDE

A supercritical fluid is defined as a substance above its critical temperature (Tc) and critical pressure (Pc). The
critical point represents the highest temperature and pressure at which the substance can exist as a vapour and liquid in equilibrium. The phenomenon can be easily explained with reference to the phase diagram for pure carbon dioxide shown in Figure 1 [Cancell & Chevalier 1999]. This shows the areas where carbon dioxide exists as a gas, liquid, solid or as a supercritical fluid. The curves represent the temperatures and pressures where two phases coexist in equilibrium (at the triple point, the three phases coexist). The gas-liquid coexistence curve is known as the boiling curve. If we move upwards along the boiling curve, increasing both temperature and pressure, then the liquid becomes less dense due to thermal expansion and the gas becomes more dense as the pressure rises. Eventually, the densities of the two phases converge and become identical, the distinction between gas and liquid disappears, and the boiling curve comes to an end at the critical point [Cancell & Chevalier 1999].

Supercritical fluids close to their critical points have densities and thus dissolving power comparable to that of liquids, are much more compressible than dilute gases, and have transport properties intermediate between gas and liquid. In the case of carbon dioxide the critical point occurs at a pressure of 73.8 bar and a temperature of 31.1°C and above these values the fluid will enter the supercritical state [Jones 2002].

2.1 Supercritical carbonation

The effect of scCO2 on the structure of cements was initially investigated by the oil industry [Onan 1984] but its use as a treatment to improve the properties of cementitious composites was first initiated by Jones in the late 1990s [Jones 1996]. The main reactions involved in the carbonation of hydrated cement result in the decalcification of portlandite and calcium silicate hydrate (C-S-H) gel, with consequent crystallisation of calcium carbonate. Like natural carbonation, the process reduces the hydroxide concentrations in the pore solution, lowers the pH and alters the mechanical properties and microstructure of the cementitious materials [Garcia-Gonzalez et al. 2006]. It has been shown that carbonation of cement based materials by the supercritical process is greatly accelerated (from years to hours) in comparison with natural carbonation. This is due to the significant rise of CO2 solubility in water with the increase in pressure [Garcia-Gonzalez et al. 2006] and to the relative ease of penetration and diffusion of the supercritical carbon dioxide in the cement paste pore network [Garcia-Gonzalez et al. 2006].

More recent studies have demonstrated that scCO2 treatment can significantly enhance the matrix strength, fibre-matrix bond, durability and dimensional stability of cement based composites, especially glass-fibre reinforced concrete [Short et al. 2001, Purnell et al. 2001, Purnell et al. 2003, Seneviratne et al. 2002].

The improvements in durability were associated with the reduction in the alkalinity of the solution, and substantial conversion of calcium hydroxide and anhydrous materials to calcium carbonate and silica gel. Improvements in mechanical properties and dimensional stability were related to significant reductions in total porosity (due to deposition within the pore network of the calcium carbonate formed by the process) and modification in micro-pore structure within the C-S-H gel phase [Garcia-Gonzalez et al. 2006, Short et al. 2001, Purnell et al. 2001, Purnell et al. 2003, Seneviratne et al. 2002 Short et al. 2004]. These investigations have also established some of the parameters that govern the efficacy of scCO2 treatment, particularly that the moisture content of components immediately prior to treatment is crucial to the degree of property enhancement, the amount of calcium hydroxide, anhydrous material and C-S-H gel remaining after carbonation, and the resultant pore structure [Short et al. 2004].

3 PROJECT OUTLINE

Although this preliminary work has been successful, many unanswered questions remain and to this end an EPSRC-funded joint programme between Leeds, Aston and Warwick Universities and industrial partners in the UK has been initiated to investigate supercritically carbonated calcareous ceramics. This project is aimed at investigating the potential for using a supercritical carbonation process in combination with other novel processing techniques such as compression moulding to fabricate a new class of ceramic composites from a variety of calcium-based precursor materials, including lime and cement.

Compression molding is being developed based on the process used for mass production of polymer components such as rubber tires and various polymer matrix composite parts for e.g. automotive applications. The technique, as used for mass-produced polymer components, is used to fabricate green forms (having ‘sand-castle’ strength and able to carry their
self weight while avoiding structural failure), which are then hardened by treatment with supercritical CO₂. The project will also investigate the suitability of more advanced forming technology such as extrusion, injection molding, and the use of established polymer composite forming techniques to incorporate fibres into the cementitious composites.

While there is some literature regarding the effect of supercritical carbonation on the cement based composites, no work appears to have been done so far on the effect of such treatment on the structure of hydrated lime precursors. Lime (calcium oxide CaO) is produced by firing calcium carbonate at a temperature of approximately 900°C, well below that required for manufacturing cement (1450°C) [Sebaibi 2004]. The lime is then treated with water to form a hydrated or slaked lime (calcium hydroxide, Ca(OH)₂). Lime components gain strength via cementation caused by carbonation alone; there is no hydration reaction as with cement composites. Carbon dioxide reacts exothermically with the moist hydrated lime to form calcium carbonate, water and heat [Moorehead 1986]. The reaction proceeds in two stages [Moorehead 1986, Van Balen & Van Gemert 1994].

1) Dissolution of carbon dioxide in water forming carbonic acid
\[ \text{CO}_2 + \text{H}_2\text{O} \leftrightarrow \text{H}_2\text{CO}_3 \]

2) Reaction of lime with carbonic acid
\[ \text{Ca(OH)}_2 + \text{H}_2\text{CO}_3 \rightarrow \text{CaCO}_3 + 2\text{H}_2\text{O} \]

The cementation process by the natural atmospheric CO₂ is very slow and it has been suggested that the total carbonation of mortar by the atmospheric CO₂ could take centuries [Cowper 1998]. Using scCO₂ technology may accelerate the reaction sufficiently to open new production routes and applications for lime-based composites; lime may also be used to partly replace cement in green forms.

The project is also investigating the use of waste materials such as waste glass, as feed stocks for green forms.

### 3.1 Sustainability aspects

The use of scCO₂ to form cementitious composites has a number of possible sustainability benefits:

- As the technique uses CO₂ and recombines it into stable solid forms, it has the potential to help mitigate the CO₂ impact of the manufacture of cements and/or lime by re-combining some of the CO₂ emitted.
- Composites produced with scCO₂, with enhanced mechanical properties, long term durability, improved dimensional stability and reduced tendency to efflorescence, when compared with existing cement based products, should have longer in-service lifetimes and reduced maintenance.

### 4 SUMMARY OF PROGRESS TO DATE

Preliminary results covering the effect of variations in mix design on the efficacy of moulding green forms, degree of carbonation and mechanical properties along with results on effect of water curing on the degree of carbonation and strength of control and carbonated samples are considered here.

### 4.1 Materials and methods

Green samples were manufactured using a compression moulding technique combined with vacuum dewatering. The tool as shown in Figure 2 is capable of producing six samples at a time, each 170 mm long having a trapezoidal cross sectional shape with 22/34 mm wide × 17 mm depth. This shape was chosen for ease of demoulding and was settled upon after many trials and alterations.

The mould was fixed to a thermoset compression moulding (DASSETT) machine and samples were subjected to 19 MPa pressure (466 kN load) from the machine for 1 minute. Excess water was squeezed out of the samples and removed by vacuum through a filter sheets and holes in the tool.

Binders used were combinations of Limbux shaft Lime [L] and Ordinary Portland cement [C]. Aggregates
[Agg] were combinations of normal silica sand [S] and crushed limestone (2.36 mm to dust fraction) [CL].

Pre-carbonation moisture content or ‘degree of drying’ (DOD, expressed as a percentage of the total weight loss achieved by drying at 105°C) was adjusted by oven drying at 25°C for approximately 12 hr, previous studies having shown that DOD influences the efficacy of supercritical carbonation. Samples were then treated with scCO₂ at 100 bar and 60°C for 24 hr using a sealed heated stainless steel vessel, as shown in Figure 3 (internal dimensions Φ100 × 500 mm) in which a measured quantity of dry ice is allowed to vaporise.

Degree of carbonation was determined semi-quantitatively by using XRD to measure depletion of calcium hydroxide (CH) and alite (C₃S). Flexural strength was measured using a fully articulated 4-point bending rig on a 100 kN Testometric test machine.

4.2 Results and discussion
4.2.1 Compression moulding development – making green forms

Initial tests concentrated on pressing relatively dry mixes of cement and/or lime with sand aggregate at a water:binder ratio (w:b) of ~0.3. It proved difficult to achieve sufficient green strength to ensure successful demoulding and storage. Surface finish and compaction were also poor. Subsequent tests used a more fluid mix with w:b ~ 0.43–0.85 and the moulding tool was redesigned to allow vacuum extraction of water expelled during pressing. Furthermore, the aggregate was changed from 100% S to a mix of 35:65 w/w CL:S. It was found that a wide range of mixes could then be used to produce adequate green forms in this manner although lime and/or C:A ratios greater than ~2 caused poor green strength and surface finish. The w:b ratio after pressing ranged from 0.3–0.42, generally increasing with aggregate:binder (i.e. A:[L + C]) ratio. Tests are continuing to establish the relationships between pressing time and pressure, mix design and green strength.

4.2.2 Supercritical CO₂ treatment

Figure 4 shows control and treated strength for various mix designs. Flexural strength increases with C:L ratio and aggregate content. Mixes with crushed limestone included in the aggregate achieve greater strengths after treatment than those without (i.e. with 100% S). Furthermore, mixes with larger quantities of crushed limestone (i.e. 65:35 w/w CL:S) achieved higher strength after carbonation.

The two bars on the far right of Figure 4 compare samples with same mix design but different aggregate combinations. Although the strengths of the control samples are the same; the treated samples with higher quantity of limestone are significantly stronger.

Figure 5 shows, for the same mixes, the degree to which CH was depleted by treatment as measured by XRD (and is thus semi-quantitative). Depletion was generally > 90%. In all these samples, C₃S was depleted by treatment to below the detection limit. This
suggests that C₃S is more susceptible to carbonation than CH. Furthermore, XRD results (Figure 5) showed that CH was depleted to trace amounts in the samples with cement, and completely depleted in those without; thus no correlation between degree of carbonation and treated strength could be established. Treated samples containing cement were much stronger than those without cement, suggesting that the binding ability of the carbonated CSH gel is far greater than that of interlocking calcium carbonate crystals although more research is required.

The effect of conventional water curing on mechanical and micro-structural properties of treated and control samples was examined. In total, 10 curing regimes were investigated, which are explained fully in Table 1. The flexural strength results for these specimens are shown in Figure 6.

In general, compared to the flexural strength achieved by carbonating samples after a ~ 12 hour pre-conditioning period immediately after demoulding, no benefit is gained from any period of water curing pre-carbonation, and only minor strength improvements were observed in samples water cured post-carbonation. Thus carbonating immediately after demoulding is the preferred industrial option. The flexural strength of samples thus treated was about 60% higher than that achieved by normal 28-day curing of uncarbonated samples.

The XRD results, (Figures 7 and 8) showed complete depletion of CH and C₃S in treated samples exposed to
Table 1. Water Curing regimes for control and treated samples produced by optimum mix design.

<table>
<thead>
<tr>
<th></th>
<th>Curing Regime Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>P→preC→scCO₂→test both control and carbonated specimens</td>
</tr>
<tr>
<td>C1-28</td>
<td>P→preC→scCO₂→C28→test both control and carbonated specimens</td>
</tr>
<tr>
<td>C3</td>
<td>P→preC→scCO₂→test both control and carbonated specimens</td>
</tr>
<tr>
<td>C3-28</td>
<td>P→C3→preC→scCO₂→C28→test both control and carbonated specimens</td>
</tr>
<tr>
<td>C7</td>
<td>P→preC→scCO₂→test both control and carbonated specimens</td>
</tr>
<tr>
<td>C7-28</td>
<td>P→C7→preC→scCO₂→C28→test both control and carbonated specimens</td>
</tr>
<tr>
<td>C3*</td>
<td>P→preC→scCO₂→C3→test both control and carbonated specimens</td>
</tr>
<tr>
<td>C3*-28</td>
<td>P→preC→scCO₂→C3→C28→test both control and carbonated specimens</td>
</tr>
<tr>
<td>C7*</td>
<td>P→preC→scCO₂→C7→test both control and carbonated specimens</td>
</tr>
<tr>
<td>C7*-28</td>
<td>P→preC→scCO₂→C7→C28→test both control and carbonated specimens</td>
</tr>
</tbody>
</table>

P: processed green forms via compression moulding.  
PreC = Pre-carbonation conditioning by oven drying at 25°C for about 12 hr.  
C = Conditioned in water for duration of 3, 7, 28 day.  
scCO₂ = Supercritical carbonation for 24 hr at 100Bar and 60°C.

Figure 6. Flexural strength vs. mix design 1.5C:0.5L:5Agg with different curing regimes. Treated = scCO₂, 60°C, 100bar, 24Hr. Aggregate = 35%S, 65%CL. Error bars = ± st. dev. See Table 1 for details of samples.

Figure 7. XRD analysis of CH depletion vs. curing regime. Samples as Fig. 6. Angles refer to 2θ peaks on XRD traces using CuKα radiation wavelength 1.5418 Å. Average of triplicate samples.
6 CONCLUSIONS

1. Sufficient green strength for robust sample handling can be obtained by using compression moulding methods combined with vacuum dewatering. These green forms will carbonate readily given simple pre-processing for moisture content. In general, higher cement:lime ratios produce higher strength carbonated samples.

2. Over the range studied, all samples were more or less fully depleted with regard to CH and C₃S content after carbonation as indicated by XRD. The exception to this was samples cured in water prior to carbonation, in which some CH and C₃S remained. ‘Wet’ (i.e. not pre-conditioned) samples were resistant to carbonation.

3. In samples containing only lime as a binder (i.e. with no cement), full carbonation of the lime was observed with little corresponding increase in strength. Thus carbonation (measured as CH depletion) is not necessarily a reliable indicator of strength development in carbonated samples and some addition of cement in the mix is necessary for achieving high strength materials.

4. C₃S depletion, over the entire range of samples studied, was more marked than CH depletion during carbonation. Thus C₃S would appear to be more susceptible to carbonation than CH.

5. No significant benefit was gained from any period of water curing either pre or post –carbonation.

The flexural strength of treated uncured samples (C1) was about 60% higher than that achieved by normal 28-day curing of uncarbonated samples. Thus carbonating immediately after demoulding/pre-conditioning is the preferred industrial option.

ACKNOWLEDGEMENTS

This project is supported by EPSRC grant GR/T01518/01 and industrial support from Buxton Lime Industries Ltd, Marshalls plc, Critical Processes Ltd and Imerys Minerals Ltd is gratefully acknowledged.

REFERENCES


Supercritical carbonation of Casamic

S.J. Shaw, C.L. Page, A.R. Brough, J.P. Forth & M.M. Page
School of Civil Engineering, University of Leeds, UK

T.R. Jones
Camborne School of Mines, University of Exeter, UK

M. Carsana
Dipartimento di Chimica, Materiali e Ingegneria Chimica “G. Natta”, Politecnico di Milano, Italy

ABSTRACT: Lime-based composite materials gain strength through atmospheric carbonation but the process is normally very slow. Using supercritical carbon dioxide to carbonate lime-based composites rapidly could potentially produce strong, chemically stable materials. Investigations were therefore carried out to assess the effectiveness of supercritical carbon dioxide treatment, applied to samples of a particular type of lime-based material (Casamic), produced by autoclaving compacted mixtures of soda-lime glass, hydrated lime and water. Increased bending strength and reduced reversible shrinkage were observed after supercritical carbonation, but the strength increase was not sustained when samples were exposed to water. Microstructural changes induced by supercritical carbonation were examined by SEM and $^{29}$Si MAS-NMR and it was found that formation of crystalline sodium bicarbonate was a factor associated with the increase in bending strength. The high water solubility of this substance appeared to account for the observed reversion of strength on wetting of supercritically carbonated Casamic. Work in progress is aimed at evaluating effects of replacing soda-lime glass powder with flyash and preliminary results have been promising.

1 INTRODUCTION

Casamic [Jones & Pascoe 2002, Jones et al. 2003] is a relatively new ceramic material made from recycled soda-lime glass powder, lime and water, using a simple energy efficient technique. It is microporous, shows very little shrinkage on curing and has a bending strength range of 22–26 MPa, similar to wall tile. This makes Casamic a possible material for use in the construction and building industry.

The bending strength achieved by Casamic has been attributed to the formation of calcium silicate hydrates produced by the pozzolanic reaction of hydrated lime with the silicates present on the surface of the glass particles. This pozzolanic reaction is accelerated by curing in steam for several hours.


This paper investigates how exposure to supercritical carbon dioxide can affect the physical and chemical properties of Casamic.

2 EXPERIMENTAL PROCEDURE

2.1 Materials

Glass powder made from recycled green soda-lime glass, was obtained from Minelco, with a particle size less than 100 µm. Analytical grade calcium hydroxide was used. Analysis and particle size distribution for both glass and hydrated lime are shown in table 1 and figure 1 respectively.

2.2 Preparation of Casamic

To prepare Casamic, 100 g of glass powder, 10 g of hydrated lime and 10 g of water where interground in
a Glen Creston gyro mill, for 3 minutes. The resulting damp powder was then brushed through a 600 micron sieve and 6.5 g portions of the sieved powder were placed into a die. The die was pressed uniaxially to a pressure of 50 MPa and held at that pressure for 30 seconds. The compacted disc was then removed and cured in an autoclave at 115°C for 3 hours, to form a hardened disc approximately 31.5 mm diameter, 4.3 mm thick.

2.3 Supercritical carbonation (SCC) of Casamic

Moisture conditions have to be optimized for supercritical carbonation to proceed efficiently, Purnell et al. [2001b] have demonstrated the need for an internal relative humidity (R.H.) of ~33% in order to achieve a fully carbonated condition rapidly in cement pastes. Therefore, the discs were placed into a container with a relative humidity of 33% for 7 days, before they were sealed in a pressure vessel. The pressure vessel was placed inside an oven, set at 38°C and liquid carbon dioxide was pumped into the pressure vessel until 80bar was reached. The supercritical carbon dioxide was maintained within the pressure vessel for 12 hours.

2.4 Shrinkage tests

Measurements of variation in disc diameter were carried out to evaluate volume changes caused by cyclic wetting and drying. This was done using a standard micrometer with a reading error of ±10 µm. Also weight measurements were performed using a digital balance with reading error of ±100 µg.

To evaluate the shrinkage behaviour these samples were exposed to seven days wetting with de-ionised water at a temperature of 20°C, alternating with seven days of drying (in a sealed box at 20°C and 35% R.H., the latter controlled by a saturated salt solution of MgCl₂).

3 RESULTS AND DISCUSSION

3.1 Shrinkage

Maximum shrinkage occurred during the first drying stage and part of this was irreversible. Subsequent drying and wetting cycles resulted in almost completely reversible shrinkage, indicating there were no further changes in internal structure. The absence of fully reversible behaviour was probably due to the introduction of additional bonds within the gel during the first drying stage, when close contact between the gel particles was initially established.

After SCC, carbonation shrinkage can be observed, which is connected to changes in the spacing between calcium-silicate-hydrate layers [Richardson et al. 1992]. Fully reversible shrinkage, after SCC, indicates no structural changes occurred due to alternating wetting/drying cycles.

Shrinkage and weight changes were reduced after carbonation by supercritical carbon dioxide (figures 2 & 3). This is probably due to a reduction in the porosity of the carbonated material.
3.2 Bending strength tests

In order to evaluate the strength of Casamic, bending strength tests were carried out by the “ball-on-ring” method using a Hounsfield CK10 instrument. Casamic discs 31.5 mm in diameter and 4.3 mm thick were supported on a ball race, 19 mm diameter, and a 5 mm diameter steel ball was pressed with increasing force into the centre of the disc, until it fractured. Bending strengths were then calculated from the force of fracture using the formula of de With [1989] and assuming a Poisson ratio of 0.3. The bending strength range for Casamic before and after SCC is shown in table 2.

Supercritical carbonation of Casamic increased the average bending strength by ~40%.

3.3 Simultaneous thermal analysis (STA) with mass spectroscopy (MS)

Casamic samples were analysed using a Stanton Redcroft Simultaneous thermal analyser with the gaseous emissions analysed using a Cirrus mass spectrometer. STA is a combination of differential thermal analysis (DTA) and thermogravimetry (TG).

STA of a cured Casamic sample (figure 4) shows a large peak between 80°C and 180°C with a corresponding weight loss due to loss of water. This is due to the dehydroxylation of calcium-silicate-hydrate (C-S-H), formed during curing and evaporation of water present within the pores. Also there is no peak between 425–550°C, this denotes the absence of calcium hydroxide within the matrix. This indicates that all the lime has reacted with the glass to form C-S-H.

STA of SCC Casamic (figure 5) shows a peak at 710–730°C, this is due to the presence of calcium carbonate; it is also confirmed by the presence of evolved CO₂ in the mass spectra. Also, it can be seen that there is a strong crystalline peak at 150°C and a corresponding increase in both carbon dioxide and

<table>
<thead>
<tr>
<th>Sample</th>
<th>Number of samples</th>
<th>Highest bending strength (MPa)</th>
<th>Lowest bending strength (MPa)</th>
<th>Mean (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cured Casamic</td>
<td>9</td>
<td>26.01</td>
<td>22.42</td>
<td>23.58</td>
</tr>
<tr>
<td>SCC Casamic</td>
<td>9</td>
<td>37.38</td>
<td>30.26</td>
<td>32.99</td>
</tr>
</tbody>
</table>

Figure 3. Variations in diameter of Casamic in both wet (de-ionized water) and dry (35% R.H.) conditions before and after SCC.

Figure 4. STA & MS plot for cured Casamic.
water. This is probably due to sodium bicarbonate forming during the carbonation reaction [Heda et al. 1995].

3.4 Mercury intrusion porosimetry (MIP)

The MIP pore size distribution (figure 6) in cured Casamic was found to be unimodal with the majority of the pores falling within the range of 0.1–0.5 microns. After SCC, the MIP pore size distribution was now bimodal with the majority of the pores either within the range 0.2–0.7 microns or 0.005–0.015 microns. The increase in the number of smaller pores is presumably due to calcium carbonate precipitating inside the larger pores. This may account for the increase in strength caused by SCC of Casamic.

3.5 $^{29}$Si Magic angle spinning nuclear magnetic resonance (MAS-NMR) spectroscopy

$^{29}$Si MAS-NMR spectroscopy was used to examine the degree of polymerisation of the silica gel formed after curing. It was also used to investigate how the structure of the silica gel altered after supercritical carbonation.

Silicate structures may be regarded as being built up of tetrahedral units with varying degree of polymerisation. These can be described in terms of a ‘Q’ notation, where Q denotes silicon bonded to four oxygen atoms. A superscript n, where n = 0 to 4 is used to indicate the number of other Q units attached to the unit in question. Thus, Q$^0$ denotes silicon bonded through oxygen to no other network forming elements, whereas Q$^4$ denotes silicon bonded through oxygen to four other silicons. Q$^4$ is the preferred co-ordination for silicon. The $^{29}$Si chemical shift becomes increasingly negative with additional Si-O-Si linkage due to increased electronic shielding of the central Si. Q$^0$ units in monosilicates have typical chemical shifts of about –65 ppm, changing in steps of about 10 ppm for each additional bonded Si tetrahedron, up to about –110 ppm for Q$^4$ units of fully polymerised silica polymorphs [MacKenzie & Smith 2002].

The samples were analysed using a Varian Infinity plus 300 MHz solid-state NMR spectrometer. $^{29}$Si spectra were acquired at 59.5 MHz with proton (H) decoupling using a 6 mm Chemagnetics probe, with MAS at 3.5 kHz. The spectra were processed with 30 Hz exponential line broadening.

The NMR spectra for cured and SCC Casamic are shown in figures 7 and 8 respectively. The intensity scales for each of these spectra vary and only qualitative considerations are possible. The spectra in figure 9 show how subtracting different percentages of the
glass signal can affect the resulting spectra for the silica present in either the gel or the hydrates. Either one of these spectra is possible and so it is not clear how much of the glass signal needs to be subtracted in order to obtain the correct spectra for the silicates present in the C-S-H or silica gel, therefore the spectra for the silicates present within C-S-H or silica gel is just an estimate based on our own judgement. The solid traces, shown in figures 7 and 8, indicate the $^{29}$Si NMR spectra of Casamic samples. The intermediate dotted traces represent $^{29}$Si NMR spectrum of glass powder. A more thinly dotted trace describes the difference between $^{29}$Si NMR spectrum of glass powder and that of Casamic sample to reveal the spectra for the silicates within either the C-S-H or silica gel. NMR analysis of the cured Casamic sample (figure 7) clearly reveals that a silicate structure which consists of silicate anions and their condensed forms was present in the Casamic samples as a result of the pozzolanic reaction. A mixture of dimer and trimer condensed silica species ($Q^2$ and $Q^3$) around $-85$ and $-95$ ppm indicates the presence of C-S-H structure.

NMR analysis on SCC Casamic shows an increasing proportion of $Q^3$ and $Q^4$ silica species indicating a greater degree of silicate polymerization occurring within the calcium silicate hydrate gel as a result of applying supercritical carbonation.

### 3.6 Strength loss after wetting/drying cycles

After Casamic was exposed to water, a loss in bending strength was observed for supercritically carbonated samples (figure 10). Further investigations by thermal analysis revealed a reduction in the amount of sodium bicarbonate within the matrix. SCC Casamic contains $\sim 3.31\%$ sodium bicarbonate, after submersion in water for 1 day, the level of sodium bicarbonate had reduced to $\sim 1.67\%$.

### 3.7 Scanning electron microscopy (SEM)

The reduction in the levels of sodium bicarbonate within the SCC Casamic matrix has been confirmed by SEM. The ‘honeycomb’ type structure seen in between the glass particles is assumed to be calcium silicate hydrate, bonding the glass together giving the sample its cured strength (figure 11).

After Casamic has undergone SCC (figure 12) the internal structure changes, the ‘honeycomb’ like structure is gone and has been replaced with silica gel and calcium carbonate. Notice the crystals shaped...
like ‘lolly sticks’, spot x-ray analyses on these crystals show them to be high in sodium and are probably sodium bicarbonate crystals.

After the sample has been exposed to water (figure 13), the ‘lolly stick’ shaped crystals have disappeared leaving only silica gel and calcium carbonate to bond the glass particles together. Crystalline sodium bicarbonate probably has an effect on the strength of the material.

3.8 Flyash/hydrated lime mixtures

The removal of sodium from soda-lime glass and the formation of water-soluble sodium bicarbonate within the matrix has given us some concern as to the long-term chemical stability of the material. Alternative materials such as flyash contain high amounts of silica with low amounts of sodium (table 3).

Samples were produced in the same way as the Casamic material, with curing and supercritical carbonation conditions the same as those shown in sections 2.2 & 2.3. Initial experiments into flyash/hydrated lime systems have shown that a 50%/50% mix of flyash/hydrated lime with a water/solid (w/s) of 0.1 and 1%(weight of solids) superplasticizer can produce, after supercritical carbonation, a relatively strong material that maintains its strength after being submerged in water for 1 day (figure 14). Further development of this material is now being undertaken.

<table>
<thead>
<tr>
<th>Oxides %</th>
<th>Glass powder</th>
<th>Flyash</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>67.96</td>
<td>52.92</td>
</tr>
<tr>
<td>TiO₂</td>
<td>0.06</td>
<td>&lt;0.01</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>2.05</td>
<td>26.85</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>0.63</td>
<td>8.64</td>
</tr>
<tr>
<td>Mn₃O₄</td>
<td>0.04</td>
<td>&lt;0.01</td>
</tr>
<tr>
<td>MgO</td>
<td>2.01</td>
<td>1.65</td>
</tr>
<tr>
<td>CaO</td>
<td>10.59</td>
<td>4.49</td>
</tr>
<tr>
<td>Na₂O</td>
<td>13.38</td>
<td>1.2</td>
</tr>
<tr>
<td>K₂O</td>
<td>0.84</td>
<td>3.26</td>
</tr>
<tr>
<td>P₂O₅</td>
<td>&lt;0.02</td>
<td>&lt;0.01</td>
</tr>
<tr>
<td>Cr₂O₃</td>
<td>0.06</td>
<td>&lt;0.01</td>
</tr>
<tr>
<td>LOI</td>
<td>0.4</td>
<td>0.36</td>
</tr>
</tbody>
</table>
4 CONCLUSIONS

Supercritical carbonation of Casamic has been shown to improve its physical properties in the following ways:

1. A reduction in reversible shrinkage occurs, which appears to be associated with the formation of a cross-linked silica gel that is more dimensionally stable on wetting and drying than the calcium silicate hydrate present in the uncarbonated material.
2. Bending strength is increased by 40% when compared to cured Casamic. This is probably associated with the redistribution of pore sizes induced by SCC and the formation of crystalline sodium bicarbonate.

Sodium bicarbonate probably has an influence on the bending strength of the material; consequently the material can only maintain the increased bending strength obtained after supercritical carbonation, when stored in a dry environment.

Initial experimental work on flyash/hydrated lime mixtures has produced high strength SCC materials that appear to be more tolerant to wet environments. Continuing research, to be reported elsewhere, is aimed at investigating the effects of supercritical carbonation on flyash/hydrated lime mixtures as well as other lime-pozzolan mixtures of varied composition.

REFERENCES


Jones, Jr. 1996. Cement treated with high-pressure CO₂, US Patent number 5,518,540


Jones, Jr. 1999. Hardened hydraulic cement, ceramic or coarse concrete aggregate treated with high pressure fluids, US Patent number 5,965,201


Investigations on the efficiency of Enhanced Porosity Concrete in Containing vehicular oil spills

B.V. Bhayani, T.M. Holsen, & N. Neithalath
Clarkson University, Potsdam, NY 13699, USA

ABSTRACT: Groundwater contamination from vehicular oil spills due to stormwater runoff is a major concern in highly impervious areas. This paper examines the potential of Enhanced Porosity Concrete (EPC) in retaining spilled oil in its pore structure. In this study EPC mixtures with porosities varying between 13 and 35% were examined. Known quantities of oil were introduced in different EPC specimens and rain events of different magnitudes were simulated. Oil recovered in the water that drained through the sample was quantified using a partition gravimetric method. The influence of (i) material properties of EPC such as porosity and pore sizes, and (ii) varying environmental conditions such as rainfall intensities and rates of oil and water addition on the quantities of oil retained by the system was quantified. It has been found that there is a certain optimal range of porosity in which the oil retention by the system is maximum. These experimental studies provide an understanding of the influence of pore structure features that contribute to retention of seeped oil in EPC.

1 INTRODUCTION

Enhanced porosity concrete (EPC, also known as Pervious Concrete) is a special type of concrete that has received considerable attention due to its multiple environmental benefits like runoff reduction, contaminant retention (ACI 522R 2006, Newman et al. 2004, Colandini et al. 1995), and noise reduction (Neithalath et al. 2005). It is a unique type of concrete as gap graded aggregates and a low water to cement ratio (w/c) are used in order to produce higher than normal porosities and pore sizes in the material. It is usually a mixture of 9.5 mm to 12.5 mm average diameter aggregates, portland cement, other cementitious admixtures, and water. The total cementitious material content is kept low so that there is just enough cement paste to coat the coarse aggregate particles without the paste overflowing and blocking the percolating pores. The highly porous nature of EPC facilitates the percolation of rainfall runoff through the pervious surface, thus reducing the runoff rates and protecting water supplies from contamination. It has also been reported that EPC can remove some pollutants from stormwater runoff (Colandini et al. 1995, Legret et al. 1999). Environmental Protection Agency (EPA) has advocated the use of EPC as a Best Management Practice (BMP) for the problem of stormwater runoff, which contributes a significant proportion of pollutants to receiving water bodies.

Various studies (Legret et al. 1999, Colandini et al. 1995) have found that porous media like EPC are beneficial for removal of metals while some others (Newman et al. 2002, Coupe et al. 2003) have reported that EPC-like layered systems can be used for the slow removal of oils by biodegradation.

Hydrocarbons are one of the major pollutants in stormwater runoff and one of the major sources of hydrocarbons is vehicular oil spill from impervious surfaces. The underlying premise of this paper is that grease and low volatility oils in parking areas, such as oil drips from vehicles, can be adsorbed in the pore walls of the EPC or get trapped in the pores of EPC, thus minimizing or eliminating its transport to groundwater or surface water.

This paper investigates the influence of material characteristics like porosity and pore sizes on the efficiency of EPC in retaining oil in its pore structure. Along with the current understanding of beneficial pore structure needed for water transport through EPC, the knowledge of material characteristics that contribute to better retention of oil will improve EPC design.

2 EXPERIMENTAL PROGRAM

2.1 Materials and mixture proportions

Type I normal portland cement and limestone aggregates were used to prepare the EPC mixtures. The aggregates were sieved into different size groups to facilitate gap grading. Two different aggregate sizes
were used to prepare the EPC specimens—aggregates having nominal maximum sizes (d_{max}) of 12.5 mm and 9.5 mm. The w/c was maintained constant at 0.3 and aggregate to cement ratio was kept constant at 5 for all the mixtures. The mixtures were prepared using a laboratory mixer in accordance with ASTM C-192-00, cast in 100 × 200 mm cylinder molds and were consolidated using tamping. After curing for 7 days, the specimens were cut to 150 mm length using a wet cutting blade. The surfaces of the specimen were finished smooth.

2.2 Measurement of permeability and porosity

To quantify the physical properties of EPC, the permeability and porosity were determined. The diameter of specimen was reduced to 95 mm by grinding. Cylindrical specimens (95 mm in diameter and 150 mm long) were immersed in water for 24 hours to saturate the pores in the matrix phase. A falling head permeability cell (Neithalath et al. 2006) was used to measure hydraulic conductivity (permeability) of the EPC specimens. The coefficient of permeability of the specimen was calculated using Darcy’s law.

The porosities of the EPC specimens were measured using a volumetric method. The 150 mm long specimens were cut into 50 mm thickness, thus producing three companion slices. The saturated specimens were then placed on a stainless steel plate and the sides and edges were sealed. Water was added to the top of the specimen until it was filled which indicated that all the interconnected pores were saturated with water. The volume of water added expressed as a percentage of the total volume of the specimen is reported as the porosity of the specimen.

Fig. 1 shows the relationship between porosity and permeability of various EPC specimens tested in this study. It can be clearly noted that permeability of the EPC specimens increases with porosity. The porosity-permeability relationship obtained from this study is fairly linear with a R^2 value of 0.93. Such a relationship is not uncommon for porous materials; however there are other pore structure features like pore sizes and pore connectivity (or tortuosity) that influence the permeability.

2.3 Experiments to study the transport behavior of oil in concrete

For laboratory oil transport studies of constructed pervious layers, an optimal oil loading rate of 2–5 mL per rainfall event has been reported. (Newman et al. 2004). On the basis of this finding, an oil-loading rate of 3 mL per rainfall event was used. To replicate rainfalls of varying intensities, 25, 51 and 102 mm of water addition per day was adopted. The volume of water required to simulate each of these rainfall intensities were calculated by multiplying the specimen surface area with design rainfall intensities. This amount of water was uniformly distributed over the entire duration of the experiment at intervals of 3 hours.

2.3.1 Simulation of oil spill and rainfall

The 50 mm thick EPC specimens were sealed on their sides to prevent seepage of oil and water. Each experimental run comprised of 3 mL (2.5 g) of oil addition and water corresponding to the chosen rainfall intensity, which was distributed over the entire duration. Both oil and water were distributed uniformly over the surface area of the EPC. In the preliminary study 102 mm of water was added over the entire surface of the specimen over 24 hrs. Further studies were carried out with 25 mm, 51 mm, and 102 mm water additions. The specimen was kept undisturbed for 3 hours, before the start of the next run, to measure oil recovery. Fig. 2 shows recovery of a mixture of oil and water, seeped
out from a specimen under consideration. The oil in oil-water mixture that seeped out was quantified to determine oil recovery using Partition gravimetric method from Standard Methods for the Examination of Water and Wastewater published by American Water Works Association (Expt. 5520-B). The experiment involved dissolving oil in a solvent (N-hexane) and separating them using a separatory funnel and distillation apparatus (Fig. 3).

3 ANALYSIS AND DISCUSSIONS

The influence of pore sizes (using aggregate sizes as surrogate measure), porosity, rainfall intensities, and rate of water and oil addition on oil retention in EPC are discussed in the following sections.

3.1 Influence of pore sizes

A preliminary study of eight specimens (four with $a_{\text{max}}$ of 9.5 mm and porosities ranging between 18% and 23.3% and another four with $a_{\text{max}}$ of 12.5 mm with porosities ranging between 30% and 34%) was carried out to study the effect of pore sizes on oil retention. Since accurate measures of pore sizes are not easily obtained, the aggregate sizes chosen for mixtures were used as surrogate indicators of pore sizes. It has been reported that the use of larger aggregates leads to larger pore sizes in EPC and the relationship between aggregate sizes and pore sizes is linear (Marolf et al. 2004). Fig. 4 shows the relationship between aggregate sizes (and thus the pore sizes), and the porosity of EPC specimens used in this study. It is noticed from the graph that the specimens made with smaller sized aggregates had a lower porosity whereas those made with larger aggregates had a larger porosity.

Three experimental runs, with 102 mm of rainfall simulated in each run, were carried out on each of these specimens. The specimens were flushed with N-hexane to recover the oil trapped in the pores of EPC after completion of each run.

Fig. 5 shows the relationship between cumulative water added and oil recovery for specimens made using large and small aggregates.

Figure 3. Schematic of distillation apparatus used to recover oil using N-hexane as the solvent.

Figure 4. Comparison of porosity between different aggregate specimens for preliminary study.

Figure 5. Relationship between cumulative water added and oil recovery for specimens made using large and small aggregates.
made with smaller sized aggregates (smaller pore sizes) have a better oil retention capability. Hence all the further experiments to investigate the effects of porosity, rainfall intensity, and rate of oil and water addition are carried out on specimens made with small aggregates sizes.

3.2 Effect of varying porosities

As elucidated in the previous section, EPC specimens with porosities of 14%, 20% and 25% made using aggregates of 9.5 mm $a_{\text{max}}$ were used to investigate the influence of porosity on oil transport behavior of EPC. For this set of experiments, each run consisted of 25 mm of water addition and an oil loading of 3 mL. A steady state (i.e. when the amount of oil recovered equals the amount of oil added) was achieved after addition of 36 g of oil and 300 mm of water for specimens with 14% and 25% porosity, while the specimens with 20% porosity achieved steady state after 40 g of oil addition and approximately 400 mm of water addition. The higher amount of oil addition required to achieve steady state also points to the fact that a porosity of approximately 20% is optimal for better oil retention in the pores of EPC.

Fig. 6 shows the cumulative amount of oil retained plotted against the cumulative water added for specimens having the three porosities mentioned above. It can again be noticed that the specimen with porosity of approximately 20% has a better oil retention capacity, while the specimen of approximately 25% porosity had the least retention.

Another representation of the oil retention capacity of EPC specimens is shown in Fig. 7 in which the percentage of pores filled in each specimen is shown. Initially there is a linear increase in percentage of pores being occupied by the retained oil with cumulative water addition, but after 200 mm of water addition for 14% and 25% porosity EPC specimens, and 300 mm for 20% specimen, the amount of pores filled plateaus off.

3.3 Influence of different rainfall intensities

A set of experiments was performed on seven EPC specimens prepared from 9.5 mm $a_{\text{max}}$ aggregates with porosities ranging from 13% to 25%. They were divided into three sets to simulate three different water additions of 25 mm, 51 mm and 102 mm each. Three experimental runs of oil and water additions were performed on each of the seven EPC specimens.

Figs. 8a, 8b, 8c shows oil recovery as function of porosity for varying amounts of rainfall simulations.
The oil recovery is very high for specimen with 14% porosity for 25 mm rainfall, (Fig. 8a); similarly the specimen with 25% porosity had more oil recovery than the 18% porosity specimen for 51 mm rainfall (Fig. 8b). The cumulative oil recovery for the three runs for the specimens with 13% porosity is greater than the cumulative oil recovery for specimens with 20% porosity (Fig. 8c) but not statistically significant. The specimen with an approximate porosity of 20% has smaller oil recovery in the three cases of varying amounts of rainfall and thus is presumed to have better oil retention capabilities.

3.4 Effect of rate of oil addition

The influence of the rate of oil addition was studied on EPC using specimens of porosities of approximately 13% and 20%. In this set of experiments the oil and water loading intensities were higher than those of the previous experiments (each run consisted of 6 mL (5 g) of oil addition and 50 mm of water addition). From Fig. 9 it can again be observed that the EPC specimens under consideration show a linear increase in amount of oil retained for initial 250 mm of water addition, and achieved a steady state at around 325 mm of water addition.

4 CONCLUSIONS

This paper describes the influence of material and experimental parameters on the oil retention capacity of EPC specimens under laboratory simulated oil loading and rainfall conditions. A later paper will explore the modeling aspects of oil transport and retention in EPC.

The following conclusions are derived from this study

- Oil retention for EPC made with larger sized aggregates (i.e. specimens with larger porosity) is less than that of specimens made with smaller aggregate (specimens with lower porosity). This can possibly be attributed to the less tortuous pore structure of specimens with larger aggregates.
- The specimen with a porosity of 14% has higher oil retention compared to the specimen with 25% porosity, while a specimen with 20% porosity showed the maximum retention for the same amount of oil addition. This indicates that a higher porosity is not necessarily a pre-requisite for better oil retention characteristics of EPC. An optimal value of porosity (that provides the physical space for oil retention) combined with a tortuous pore structure (that extends the time needed for the oil to flow out of the pore structure, and in turn aiding retention) is necessary for better retention characteristics. A material microstructural model that combines the
- The oil retention behavior of EPC varies with changes in amount of oil and water additions. In this case also it was observed that the specimen with 20% porosity had the maximum oil retention.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the financial support from the Clarkson Center for Environment and the Coulter School of Engineering at Clarkson University towards this research project.

REFERENCES

1. ACI 522 R. 2006. Pervious Concrete, American Concrete Institute, Farmington Hills, Michigan.


CO₂ uptake by concrete hardened in a simulated flue gas

Y. Shao & X. Zhou
Department of Civil Engineering, McGill University, Montreal, Canada

ABSTRACT: The possibility of using concrete building products to absorb flue gas carbon dioxide through their curing process is studied. Portland cement based products were exposed to a simulated flue gas with a CO₂ concentration of 25%, under 5 bar pressure and in two-hour duration. Their carbon uptake and immediate strength gain were measured. It was found that the flue gas CO₂ uptake was in a range of 7–9% based on dry cement as reference. Its corresponding two-hour strength exceeded 8 MPa in a multiple injection – release procedure. To investigate the maximum possible CO₂ uptake by cement, loose powder samples of same mass were carbonated under the same condition. The uptake could reach approximately 17%. Microstructure of the carbonated cement and concrete indicated that the material was mainly composed of poorly crystalline calcium carbonate incorporated in the calcium silicate hydration products.

1 INTRODUCTION

CO₂ sequestration has become an important component in global greenhouse gas mitigation exercise. The current strategy is focused on the removal, recovery and disposal of CO₂ at the point sources. The approaches include the geologic storage of CO₂ in aquifers and depleted oil wells, and mineral sequestration through carbonation.

Geological sequestration is the storage of CO₂ in a range of underground geologic formations. The flue gas is captured, recovered to a high CO₂ concentration and injected into the storage site. The capacity ranges from 14 million tons net of CO₂ in deleting oil reservoir over 15–20 years to 2200 tons CO₂ over 42 days into a deleted oil field (NETL 2004).

Mineral carbonation has been developed to prepare stable carbonate products by reacting CO₂ with magnesium silicate minerals such as olivine and serpentine. It was estimated that the necessary raw materials are available in vast quantities, far exceeding the known fossil fuel reserves (Lackner et al. 1995). The reaction took place in an aqueous bicarbonate/chloride medium at a temperature of 180°C, under a pressure of 150 atm, in one hour duration and using a finely ground mineral with 80% particles smaller than 37 µm. Carbonation reaction efficiency reached about 60–80%.

Carbon dioxide can also be converted to carbonates using calcium silicate materials. However, natural calcium silicate minerals containing high reactive calcium content are rare. One of the popular man-made calcium silicate materials is Portland cement. Directly using Portland cement powder to bind CO₂ in a mineral carbonation process is not practically feasible. Nevertheless, if building products made of Portland cement can be used to mineralize carbon dioxide to a calcium carbonate through its production, the method of CO₂ disposal can gain both environmental and economical benefits.

The production of Portland cement is a CO₂ releasing process. If the released CO₂ can be captured, recovered and mineralized into the final concrete products, the emission from cement and concrete industry can be reduced considerably. Portland cement concrete is the most used man-made material. The annual consumption in North America is more than one ton per capita and the demand for cement is growing at an average annual rate of 1.5% (Worrell et al. 2001). Cement and concrete industry is promoting the use of supplementary cementitious materials, such as slag and fly ash, to reduce cement consumption and thus the emissions (Malhotra 1999). CO₂ mineralization by concrete products through their curing provides a different and new approach for direct sequestration.

The reactivity and strength development of hydraulic and non-hydraulic calcium silicates activated by carbon dioxide were systematically studied by Berger (1972), Young (1974), and Goodbrake (1979). It was found that the carbonation products were primarily calcium carbonates and silica gel. The chemical reactions of carbonation are described in Eqs. (1) and (2):

\[
3\text{CaO} \cdot \text{SiO}_2 + 3\text{CO}_2 + \text{H}_2\text{O} \rightarrow \text{SiO}_2 \cdot \text{Ca}_2\text{O} + 3\text{CaCO}_3 \quad (1)
\]

\[
2\text{CaO} \cdot \text{SiO}_2 + 2\text{CO}_2 + \text{H}_2\text{O} \rightarrow \text{SiO}_2 \cdot \text{Ca}_2\text{O} + 2\text{CaCO}_3 \quad (2)
\]
The early age carbonation curing of concrete provides a direct means to consume carbon by converting the carbon dioxide to calcium carbonates, which are geologically stable, environmentally durable, and economically beneficial. For applications without reinforcing steel, the carbonated concrete products can perform better in achieving strength, durability and dimensional stability due to the near-complete depletion of calcium hydroxide. The rapid carbonation reaction with cementitious binders accelerates the strength development and shortens the time required for the production. It is best suited for manufactured concrete products, such as bricks and blocks, cement boards and precast concretes with non-metallic reinforcement.

Concrete technology has shown one more feature. It can use flue gas directly in the process with no need of carbon separation. This paper presents a study on the carbonation technology for CO2 uptake and fast strength gain through early age accelerated curing. A gas of 25% CO2 concentration was used to simulate the as-captured flue gas without processing. The carbonation time was fixed at two hours to make the process feasible for industry production and the properties of carbonated products were evaluated by their mass gain, strength development and microstructure characteristics.

2 EXPERIMENTAL

Two types of CSA Portland cements, Type 10 (T10) and Type 30 (T30), were used as CO2 absorbents and their chemical compositions are shown in Table 1. Type 10 is ordinary cement with a blaine number of 373 m²/kg and Type 30 is early strength cement with a blaine of 481 m²/kg. Their initial CO2 content in as-received cement was 0.54% for Type 10 and 1.78% for Type 30.

Quartz sand with particle size ranging from 0.2 mm to 2 mm was employed as aggregates of 25% by weight. Water to cement ratio was kept constant at 0.15. The mixture proportions of four typical batches are shown in Table 2. Rectangular plate specimens of 14 mm thick, 76 mm wide and 127 mm long were prepared by press forming under a constant pressure of 8 MPa.

The carbonation curing set-up is shown in Figure 1. A carbon dioxide gas of 99.5% purity was used to blend with air to make a gas of 25% carbon dioxide concentration to represent the as-received flue gas. The pressure and temperature curves were recorded using Measurement Group data acquisition system (System 5000) with pressure transducer and Type T thermocouple. The process was carried out at a constant pressure of 5 bar and in a period of 2 hours. For 25% CO2, multiple injection method was developed to offset the low concentration. Three plate specimens were placed inside the chamber immediately after press-formed and exposed to carbon dioxide gas with different concentrations.

CO2 uptake is characterized as percent mass gain by Eq. (3). It is based on the difference between the mass measured before carbonation and the mass after carbonation, with water loss correction and dry cement binder as reference. Water loss due to evaporation is caused by carbonation reaction and collected by absorbent paper immediately after the curing process.

\[
\text{Mass gain} = \frac{(\text{Mass})_{\text{after}} - (\text{Mass})_{\text{before}}} {\text{Mass}_{\text{dry binder}}} \times 100
\]

The mass gain was also quantified by infrared based CO2 analyzer. The CO2 content determined by the carbon analyzer was based on total mass after carbonation, including dry binder, water, quartz aggregates and calcium carbonates. Therefore the definition of mass gain would be different between direct weight increase by samples (Eq. 3) and infrared test results. Eq. (3) represents an average measurement while the infrared technique is a point analysis. Since infrared technology requires only 0.5 g powder for analysis, the results can still serve as an independent check for comparison.

Powders were collected both on surface 1-mm layer and at the core to examine the uniformity of carbonation. Three-point bending tests at a span of 101 mm and compressive strength tests were performed right after two-hour carbonation to determine the modulus of
rupture (MOR) and compressive strength (fc) of carbonated concrete specimens.

3 RESULTS AND DISCUSSIONS

3.1 CO₂ uptake in flue gas

Figure 2 shows typical temperature and pressure curves of cement paste carbonated using a simulated flue gas of 25% CO₂ concentration. The maximum gas pressure was 0.5 MPa. Four cycles were employed, each taking 15, 15, 30, 60 minutes respectively. Gas was injected to 0.5 MPa at which pressure the gas valve was closed. Carbonation reaction consumed CO₂ in flue gas and thus reduced the gas pressure. At designated time, the residual gas was released and fresh flue gas was injected for next cycle. Since the CO₂ concentration was low, the reaction between the cement and carbon dioxide was slow, leading to a temperature peak of about only 40°C.

The CO₂ uptakes by different measurements are given in Figure 3. The total mass gain using 25% concentration gas ranged from 7.2% to 9.3%. Type 30 cement didn’t show any benefit in CO₂ uptake. The addition of aggregates worked favorably to Type 10 cement, but not to Type 30. The results from infrared CO₂ analysis implied that surface and core were carbonated to different levels. Consistently, the surface layers demonstrated a CO₂ content in a range of 11–14%. Nevertheless the cores were poorly carbonated with very low carbon detected. It was interesting to notice that the average of CO₂ contents determined on surface and at the core was surprisingly close to the direct mass gain by Eq. (3). The partial carbonation phenomenon is depicted in Figure 4 using phenolphthalein indicator in which the dark (red) core was not carbonated.

3.2 Strength and microstructure

The strength gain of carbonated concrete processed using 25% CO₂ concentration is shown in Figure 5. While the moduli of rupture (MOR) were of close values for different batches, the compressive strengths were dependent on the cement content. The high the cement content, the more the absolute carbon dioxide was taken and the more the solids as calcium carbonates were formed, leading to a higher load carrying capacity. The carbonated Type 30 cement paste gained higher two-hour strength than Type 10 cement. With quartz aggregates, Type 10 cement concrete was stronger. The 2-hour green strength gained from flue gas carbonation could be high enough for batch rotation and thus allowed for accelerated production.

Microstructure of carbonated cement using simulated flue gas of 25% concentration is shown in the following image.
it is necessary to examine the maximum possible uptake in flue gas as well as the limiting conditions.

Carbonation of cement powder without compaction was carried out to examine the maximum possible CO$_2$ uptake by cement, since the powder contained the highest porosity, which could promote more carbonation reactions. Five batches of same mixture proportion were prepared with water to cement ratio of 0.15 and treated with different carbonation conditions as shown in Table 3.

For Process-1, five batches, MA1-MA5, were prepared with the same water/cement ratio at 0.15. The powder samples were mixed in a ceramic jar for 5 minutes when water was added, and then the mixed powder sample was immediately placed into the pressure vessel for carbonation.

For Process-2, the carbonated samples were ground with effort to avoid any loss of materials. The fineness of ground powder was the same as that of the original cement powder. By grinding, the carbonated surface of cement particles was broken and the non-carbonated core was exposed that would promote further carbonation in Process-2. A solution of 1% phenolphthalein in 70% ethyl alcohol was used as the indicator of carbonation. The non-carbonated core turned red and the carbonated surface was colorless. For MA1, the water added at the Process-2 was 0.15 of its carbonated powder by weight; for MA2 and MA5, it was 0.05. No water was added in other batches to examine water effect on reaction of carbonation. For MA1, MA2 and MA3, the carbonation procedures were the same as in Process-1, while for MA5, one time continuous injection was used over a period of 120 minutes.

For Process-3, the sample MA1 was dried in an oven for 12 hours to determine the amount of free water inside the sample. Then, the powder was ground, water was added using a ratio of 0.1 of its dried weight and carbonated by one time injection procedure for
2 hours. The sample of MA3 was ground, water was added using 0.03 of its weight and the specimen was carbonated for a continuous period of 15 hours. After the whole process completed, the CO₂ content of each powder sample was determined by infrared based carbon analyzer.

The cumulative mass gain using dry cement as reference is displayed in Figure 9 against the square root of time. The linear relationship of the carbon dioxide uptake against the square root of time in the first two processes (4.5 hours in total) for the cement powder shows the reaction is diffusion controlled, obeying Fick’s first law. It means that the initial rate of carbonation for powder material is controlled by the availability of carbon dioxide to the surface sites rather than by the rate of chemical reaction at the surface [Reardon et al 1989]. The cumulative mass gain is also summarized in Table 4.

The powder in Batches MA1, MA2 and MA3 was carbonated under the same conditions as in Process-2. The carbonated powders were then crashed using a grinder and mixed with additional water in Process-2 to receive more moisture and carbonation at water to powder ratios of 0.15, 0.05 and 0.0. The mass gain values showed that optimal water content existed to promote the reaction. In these three batches, the additional water to powder ratio of 0.05 was the best, with mass gain increasing from 1.1 to 1.5% more than the other two batches.

The reference Batch MA4 did not have water added or powder ground during the same carbonated duration, with 4-time gas injections. The mass gain of Batch MA4 was 14.7%, 1.4% lower than that of MA2, indicating that the complex process of grinding, moisturizing and increasing the frequency of injection did not increase the mass gain significantly during the second process.

To compare the moisturizing effect, Batch MA5 used a water-powder ratio of 0.05 in Process-2 for an additional 2-hour carbonation. By comparing Batch MA5 to Batch MA4, the mass gain was noted to increase by only 0.5%, although one more gas injection was added and the total carbonation duration was about the same. This result shows that the initial water content in Process-1 with water to cement ratio of 0.15 was adequate.

To study time dependence of mass gain, one-time continuous carbonation of the Process-3 was run separately on Batches MA1 and MA3 with a reaction time of 120 minutes and 900 minutes. Because of the slight high content of cumulative water in Batch MA1 from the first two processes, the powder was dried in

---

**Table 3. Carbonation of cement powder.**

<table>
<thead>
<tr>
<th>Batch</th>
<th>W/C</th>
<th>Time</th>
<th>W/P</th>
<th>Time</th>
<th>W/P</th>
<th>Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>MA1</td>
<td>0.15</td>
<td>15/30/30/60</td>
<td>0.15</td>
<td>G,15/30/30/60</td>
<td>D, 0.1</td>
<td>G, C-120</td>
</tr>
<tr>
<td>MA2</td>
<td>0.15</td>
<td>15/30/30/60</td>
<td>0.05</td>
<td>G,15/30/30/60</td>
<td></td>
<td>0</td>
</tr>
<tr>
<td>MA3</td>
<td>0.15</td>
<td>15/30/30/60</td>
<td>0</td>
<td>G,15/30/30/60</td>
<td>0.03</td>
<td>G, C-900</td>
</tr>
<tr>
<td>MA4</td>
<td>0.15</td>
<td>30/60/60/120</td>
<td>0</td>
<td></td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>MA5</td>
<td>0.15</td>
<td>15/30/30/60</td>
<td>0.05</td>
<td>G, C-120</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: G = ground by mortar and pestle; C = carbonation; W/C = water to cement ratio; W/P = water to powder ratio; 15/30/30/60 = carbonation cycles at 15, 30, 30, 60 minutes for each.

---

**Figure 9. Cumulative mass gain in cement powder exposed to flue gas.**

**Table 4. Mass gain of carbonated cement powder.**

<table>
<thead>
<tr>
<th>Batch</th>
<th>Process-1</th>
<th>Process-2</th>
<th>Process-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>MA1</td>
<td>10.1%</td>
<td>14.6%</td>
<td>15.9%</td>
</tr>
<tr>
<td>MA2</td>
<td>11.4%</td>
<td>16.1%</td>
<td></td>
</tr>
<tr>
<td>MA3</td>
<td>12.2%</td>
<td>15.0%</td>
<td>17.2%</td>
</tr>
<tr>
<td>MA4</td>
<td>14.2%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MA5</td>
<td>11.4%</td>
<td>15.2%</td>
<td></td>
</tr>
</tbody>
</table>

---

385
an oven at a temperature of 110°C for 12 hours and then ground and mixed with water at water to powder ratio of 0.1. In Batch MA3, a small amount of water with a water to powder ratio of 0.03 was added to compensate for the water loss during the powder grinding. The mass gain values were increased with processes-3 by 1.3% and 2.2% for Batch MA1 and Batch MA3, showing the slow reaction rate. The total accumulated mass gain values for Batch MA1 and MA3 in these three processes were 15.9% and 17.2%.

Figure 10 compares the results of the CO₂ uptake using the percent mass gain and carbon analysis method. The two test results are reasonably close except for the Batch MA3. The overall average value of CO₂ uptake falls in the range of 15–17%. This was the maximum possible value for the cement powder carbonated at the given conditions.

### 3.4 Comparison with carbonation using pure carbon dioxide gas

As reference, CO₂ uptake by Portland cement in pure gas is summarized in Figure 11. The percent mass gain was the direct weight increase due to the absorption of CO₂ by cement in two hours. Type 30 cement batches took more CO₂ than Type 10 cement, suggesting that Type 30 cement was more reactive than Type 10 cement owing to its finer particle size and larger surface area. The use of aggregates could slightly improve the uptake because of the increase in porosity of concrete. The CO₂ content at surface and in the core determined by infrared based CO₂ analyzer is also given in Figure 11. Although results were different from each other, they were in the same order of magnitude. In most cases, the surface layer contained high CO₂ content than the core. Nevertheless the C1 batch demonstrated it was possible to have higher CO₂ absorption in the core. The infrared analysis indicated that the CO₂ uptake was fairly uniform from the surface to the core in light of the use of 100% carbon dioxide and the average of surface and core was always lower than the direct mass gain, attributing to the different denominator used as reference in calculation. Overall, it was estimated that a 10 to 16% CO₂ uptake by cement in a concrete was readily achievable.

The strength gains of carbonated samples in pure gas are shown in Figure 12 together with mass gains. Compared to the two-hour strengths obtained with pure CO₂, concrete carbonated by flue gas had shown less strength. Its flexural strength (MOR) was only about 50%, compressive strength 20–25% with a CO₂ uptake half as that by pure gas. Obviously the crystalized solids created through carbonation curing were influenced significantly by the carbon dioxide concentration. Higher concentration promotes more carbonation reaction and creates more CaCO₃ solids which are attributed to the strength development at early age.

### 3.5 CO₂ Sequestration through concrete production

A 7–9% carbon dioxide uptake by cement via concrete production is readily achievable using cement...
and concrete with varied aggregates through its carbonation curing. This percentage of uptake is calculated based on the cement binder used in the product. There are many concrete products which are excellent candidates for the process. One example is the concrete masonry units (CMU) which are mass produced at a rate of 3000 units/hour by a typical block manufacturing machine and steam-cured currently in a chamber at a temperature of 70°C for about 24 hours. The CMU can be treated in the same chamber using flue gas CO₂ at ambient temperature to accelerate the strength gain and shorten the curing time. A standard concrete masonry unit (200 mm equivalent) contains about 1.36 kg (3 lb) cement. If 10% CO₂ uptake is assumed, one masonry unit can absorb about 0.136 kg carbon dioxide. One block machine operating for 16 hours per day produces 48,000 units, consuming 6,528 tons of CO₂. For a production, using 10 block machines over 42 days, 2,742 tons of carbon dioxide can be sequestered, which is close to the scale of small geologic storage projects. However, the mass production of CMU will continue and its annual CO₂ uptake can reach about 23,830 tons. Moreover, there is an incentive for the industry to get involved because of the accelerated curing which eliminates presetting time for steam-curing. If CO₂ absorption can be enhanced to 20% or more, the capacity of the sequestration will be doubled. A 20% uptake of CO₂ by concrete represents only 40% of its theoretical limit. Therefore a large window of opportunity exists to improve the technology for better sequestration. It is certain that an increased conversion of carbon dioxide to carbonates would result in stronger and more durable concrete products, as long as there is no steel reinforcement used in the concrete products.

4 CONCLUSIONS

Study has demonstrated that concrete has the potential to absorb carbon dioxide through its early age curing. The technology can help alleviate a serious environmental problem associated with the greenhouse effect and global warming.

It was found that concrete carbonated in a flue gas could consume up to 9% CO₂ in two-hour carbonation curing at 0.5 MPa pressure. Non-uniform carbonation across the section was divided into a harder condensed shell and a softer non-carbonated core. The early age strength of carbonated specimen was attributed mainly to the condensed surface layer. As the carbonated surface layer became thicker, both the modulus of rupture (MOR) and compressive strength (fc) increased, especially the compressive strength, which reflected the average strength of the shell and the core. The carbonation depth showed that the complete carbonation was not achieved within the 2-hour carbonation period.

Of all parameters that affect CO₂ uptake, the most critical one is the CO₂ concentration in the gas. The higher the carbon dioxide content, the more the calcium carbonates could be generated and the less the time and pressure are required to achieve the mass gain. Longer curing time promoted more CO₂ absorption. However the carbonation duration had a threshold value over which the rate of mass gain would be substantially decreased because of the densified surface layers and the loss of moisture. A gas of 20–25% CO₂ concentration represented a typical flue gas from cement kilns or thermal power plants. Although the low concentration could still partially harden the concrete in a reasonably short time and produce comparable performance to conventional curing in long run, the corresponding CO₂ uptake was not substantial because of the lack of carbon dioxide in flue gas.

The maximum possible CO₂ uptake by cement powder was studied through the changes of parameters such as the reaction time, the particle size and the additional water. The CO₂ uptake by the cement powder through grinding it to nearly its original particles size, adding more water and extending the exposure time fell in the range of 15 to 17%. These values gave the maximum possible uptake under the test conditions of less than 24 hours, being 5 bars gas pressure and 25% CO₂ concentration at the ambient temperature.

REFERENCES


Early hydration of calcium sulfoaluminate-based cements for structural applications

M. Marroccoli, M. Nobili, A. Telesca, G.L. Valenti
Department of Environmental Engineering and Physics, University of Basilicata, Potenza, Italy

ABSTRACT: The properties of a hydrated high-performance calcium sulfoaluminate (CSA) cement were investigated by means of XRD, DTA-TGA, SEM analyses and mercury porosimetry. The early hydration behaviour was strongly influenced by the high (0.78) stoichiometric water-solid mass ratio required by the reaction of \( C_4A_3S \) with calcium sulfate and water towards ettringite and aluminium hydroxide. The high rate of this reaction resulted into a rapid depletion of water within a relatively short curing period and both full consumption of reactants and hydration of other CSA cement components were hindered. However, \( C_4A_3S \) conversion degrees of 0.60 and 0.87 were obtained at 3 hours and 28 days respectively, for CSA cement pastes cured with a water-cement ratio equal to 0.45. Both the fast formation of ettringite and the rapid establishment of prevailing low-porosity regions can play a very important role in regulating the technical behaviour of high-performance CSA cements.

1 INTRODUCTION

The technical behaviour of calcium sulfoaluminate based cements is mainly related to the generation of ettringite, \( C_6A_3S_3H_{12} \), upon hydration of their key-component, \( C_4A_3S \). This compound can usefully react with water according to the following paths: i) combined with lime and calcium sulfate to give only ettringite, reaction (1); together with calcium sulfate alone to give ettringite and aluminium hydroxide, reaction (2):

\[
\begin{align*}
\text{(1)} & \quad C_4A_3S + 6C + 8S + 96H \rightarrow 3C_6A_3S_3H_{12} \\
\text{(2)} & \quad C_4A_3S + 2CS + 38H \rightarrow C_6A_3S_3H_{12} + 2AH_3
\end{align*}
\]

Besides \( C_4A_3S \), dicalcium silicate \( (C_2S) \), calcium sulfoaluminate \( (C_4S_2A) \) and a variety of calcium aluminates \( (C_3A, CA, C_{12}A_2, C_4AF, C_2AS, CAS_2) \) as well as lime and/or calcium sulfate (depending on the type of application) can occur in calcium sulfoaluminate (CSA) cements. Among these phases, \( C_2S \) is particularly interesting, due to its hydraulic activity and ability to regulate strength and durability of hydrated cements at medium and long ages.

Ettringite formed in the reaction (1) has expansive characteristics which can be exploited by special binders like shrinkage-resistant and self-stressing cements [Mehta 1973; Kurdowski et al. 1986; Muzhen et al. 1992]. There are two types of expansive \( C_4A_3S \)-based binders: a) cements having a high \( C_4A_3S \) content, used as additives for normal portland cement; b) modified portland cements containing \( C_4A_3S \) instead of \( C_3A \). In both cases \( Ca(OH)_2 \), derived from hydration of \( C_3S \) and \( C_2S \) contained in portland cement, produces the desired expansive effect due to its combination with \( C_4A_3S \) and \( CaSO_4 \).

Ettringite generated in the absence of lime (reaction 2) is not expansive and has the important property of giving high mechanical strength at early ages [Mehta 1980; Mudbhaktal et al. 1986]. Other interesting features, useful for structural applications, concerning these rapid-hardening CSA cements are high impermeability and chemical resistance as well as low dry-shrinkage and solution alkalinity [Wang Lan & Glasser 1996; Muzhen et al. 1997; Zhang & Glasser 1999; Glasser & Zhang 1999; Glasser 2002].

Compared to normal portland cement, all CSA cement formulations are characterized by better environmentally friendly aspects of their manufacturing process. To this regard [Beretka et al. 1992; Beretka et al. 1993; Majling et al. 1993; Belz et al. 1995; Beretka et al. 1996; Ikeda et al. 1997; Arjunan et al. 1999; Bernardo et al. 2003], important features are:

1) synthesis temperatures lower than those requested \( (1450^\circ\text{C}–1500^\circ\text{C}) \) by portland cement clinkers;
2) clinkers easier to grind;
3) reduced amount of limestone in the kiln raw mix and, consequently, reduced thermal input and \( CO_2 \) generation;
4) greater usability of industrial wastes and by-products whose reuse and disposal is quite complicated.

Moreover, high-performance CSA cements are very interesting not only for their technical behaviour, but also from the environmental point of view because they require a significant addition of calcium sulfate and enable both a decrease of clinker concentration and an enhancement of chemical gypsum utilization.

The aim of this work is to investigate the microstructural features of high-performance CSA cement pastes cured from 3 hours to 28 days by means of XRD, DTA-TGA, SEM analyses and mercury porosimetry.

2 EXPERIMENTAL

2.1 Materials and properties

CSA cement was obtained from a sulfoaluminate clinker produced in a rotary kiln at about 1350°C, using limestone, bauxite and natural gypsum as raw materials.

Chemical and mineralogical composition, respectively evaluated by XRF and XRD analysis, are shown in Table 1. Calcium sulfates were present under both anhydrous and hydrated form, respectively belonging or added to the clinker.

Table 2 indicates the mechanical strength of CSA and portland (class 52.5 R, as a reference term) cements, determined according to the European Standard EN 197-1.

Table 1. Chemical (major oxides) and mineralogical (major phases) composition of CSA cement, mass percent.

<table>
<thead>
<tr>
<th>Chemical</th>
<th>Mineralogy</th>
</tr>
</thead>
<tbody>
<tr>
<td>CaO</td>
<td>C₄A₃S</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td></td>
</tr>
<tr>
<td>SO₃</td>
<td>C₄A₃S</td>
</tr>
<tr>
<td>SiO₂</td>
<td>C₅S₂S</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>C₃A</td>
</tr>
<tr>
<td>MgO</td>
<td>C₆AF</td>
</tr>
<tr>
<td>TiO₂</td>
<td>CT</td>
</tr>
<tr>
<td>Na₂O</td>
<td></td>
</tr>
<tr>
<td>K₂O</td>
<td>0.1</td>
</tr>
</tbody>
</table>

Table 2. Compressive strength on mortar of CSA and portland cements at various curing times, MPa.

<table>
<thead>
<tr>
<th>Curing time</th>
<th>CSA</th>
<th>Portland cement</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 hours</td>
<td>6.1</td>
<td>–</td>
</tr>
<tr>
<td>8 hours</td>
<td>23.9</td>
<td>4.6</td>
</tr>
<tr>
<td>1 day</td>
<td>33.1</td>
<td>23.3</td>
</tr>
<tr>
<td>7 days</td>
<td>57.7</td>
<td>41.5</td>
</tr>
<tr>
<td>28 days</td>
<td>63.4</td>
<td>54.2</td>
</tr>
</tbody>
</table>

2.2 Samples preparation and characterization techniques

CSA cement samples were paste hydrated (water-cement mass ratio equal to 0.45) and investigated by XRD, DTA-TGA, SEM analyses and mercury porosimetry. The pastes, shaped as cylindrical discs (15 mm high, 30 mm in diameter), were cured in a FALC WBMD24 thermostatic bath at 20°C for times ranging from 3 hours to 28 days. At the end of each aging period, the discs were in part submitted to mercury porosimetry, in part broken for SEM observations or pulverized for XRD and DTA-TGA analyses after grinding under acetone (to stop hydration), followed by treating with diethyl–ether (to remove water) and storing in a desiccator over silica gel-soda lime (to ensure protection against H₂O and CO₂).

XRD analysis was performed for both qualitative and quantitative purposes using a BRUKER-D4 XRD apparatus. XRD patterns were obtained in the range 5°–55° 2θ (Cu Kα radiation), at the scanning rate of 0.02 2θ/sec.

DTA-TGA was carried out through a NETZSCH TASC 414/3 apparatus, operating between 20° and 1000°C with a heating rate of 10°C/min.

For SEM observations a PHILIPS XL-30 ESEM instrument was used. Specimens were metallized with gold by means of an EMITECH K 950 apparatus.

Porosity measurements were performed with a THERMO FINNIGAN PASCAL 240 Series porosimeter (maximum pressure, 200 MPa) equipped with a low-pressure unit (140 Series) able to generate a high vacuum level (10 Pa) and operate between 100 and 400 kPa.

3 RESULTS AND DISCUSSION

3.1 XRD results

As curing time increases, the peak-intensities of C₄A₃S and calcium sulfates rapidly decrease while those of the other cement compounds are unchanged. Ettringite and aluminium hydroxide are the only observed hydration products.

The high consumption rate of C₄A₃S and calcium sulfates (expressed as SO₃) as well as the quick formation of ettringite and Al(OH)₃ can be clearly seen from Figures 1 and 2, where the phase concentrations of reactants and products are respectively reported as a function of the curing time.

Within the investigated aging period, the reactants and ettringite concentration decreases and increases, respectively, up almost stationary values.

As far as Al(OH)₃ is concerned, the XRD peak intensity first increases, then is stable and at longer ages seems to increase again.
The last phenomenon is most likely due to an acquisition of crystallinity by some alumina gel. So the final increase of Al(OH)$_3$ concentration is only apparent. $C_4A_3S$ conversion degrees equal to 0.60 and 0.87 are obtained at 3 hours and 28 days of curing, respectively. The water-cement ratio of 0.45 corresponds to a $H_2O/C_4A_3S + CS$ ratio equal to 0.68; this value is lower than the stoichiometric water-solid ratio (0.78) for the reaction (2). This lack of water explains why:

a) phases less reactive than $C_4A_3S$ are not involved in the hydration process;

b) unreacted $C_4A_3S$ and calcium sulfates are still present in the hydrated systems.

The early stopping of the hydration and the reduced capillary porosity (due to the free water consumption) as well as the persistence of anhydrous phases (stable like their hydration products) and the environment without moisture (fast established owing to the self-drying process) can enhance the durability of the hydrated systems.

### 3.2 DTA-TGA results

Thermal analysis was used in this investigation to detect ettringite and aluminium hydroxide through the DTA endothermal peaks (at about 200° and 300°C, respectively) as well as the related TGA weight losses. Furthermore, TGA was utilized for the evaluation of the water content bound in the paste, useful for the calculation of the above mentioned conversion degrees.

The thermal data are generally consistent with the XRD results: typical thermograms for CSA cement pastes, cured at various aging times, are shown in Figures 3 and 4.

It can be noted that the signals concerning Al(OH)$_3$ are almost constant at longer ages, indicating that the related concentrations are stable.

### 3.3 SEM observations

Figure 4 shows micrographs of CSA cement pastes cured at 3 and 8 hours, 3 and 28 days. The fast rate of ettringite formation can be easily observed. Its particles already form at 3 hours, when the surfaces of the anhydrous grains appear corrugated, develop as prismatic crystals (having an hexagonal cross section) at 8 hours, become dominant at 3 days and finally assemble at 28 days in the form of large and close-packed clusters.

### 3.4 Porosimetric results

For each sample, two plots can be obtained from the porosimetric analysis: a) cumulative and b) derivative Hg intruded volume vs. pore radius. With increasing pressure, mercury gradually penetrates the bulk sample volume. If the pore system is composed by an interconnected network of capillary pores in communication...
with the outside of the sample, mercury enters at a pressure value corresponding to the smallest pore neck. If the pore system is discontinuous, mercury may penetrate the sample volume if its pressure is sufficient to break through pore walls. In any case, the pore width related to the highest rate of mercury intrusion per change in pressure is known as the “critical” or “threshold” pore width [Winslow & Diamond 1970]. Unimodal, bimodal or multimodal distribution of pore sizes can be obtained, depending on the occurrence of one, two or more peaks, respectively, in the derivative volume plot.

The porosimetric characteristics of CSA cement pastes cured at 6 and 24 hours, 3, 7 and 28 days are shown in Figure 5.

The pore size distribution is bimodal at 6 and 24 hours (the first and the second threshold pore radius being included in the range 200–50 and 30–12 nanometers, respectively) and unimodal at 3, 7 and 28 days (with a threshold pore radius ranging from 7 to about 4 nanometers).

The first peak, at higher porosity, is related to the lowest size of pore necks connecting a continuous system consisting of a network of capillary pores; the second peak, at lower porosity, corresponds to the pressure required to break through the blockages formed by the hydration products [Cook & Hover 1999; Bernardo et al. 2006].

The total Hg intruded volume and the preferred pore width (first threshold pore radius at 6 hours, second
Figure 4. SEM (SE) micrographs of CSA cement pastes cured at: (a) 3 hours; (b) 8 hours; (c) 3 days and (d) 28 days (lower magnitude on the upper side).
threshold pore radius later) as a function of the curing time are reported in Figure 6. It is clearly observed a decreasing trend, particularly remarkable within the first 24 hours of hydration.

The porosimetric data are consistent with the characteristics of CSA cement hydration pointed out by XRD, DTA-TGA and SEM analyses. The hydration products, generated in a large amount at early curing times, are able to reduce and isolate the interior space. At longer aging periods, the evolution of porosity proceeds very slowly because hydration is almost stopped.

Taking into account the strict relationship between the permeability degree and the first threshold pore width [Nyame & Illston 1980], the porosimetric behaviour of CSA cements shows that they are able to develop rapidly a high impermeability and to exploit, due to the considerable isolation from the external environment, their properties useful for durability.

4 CONCLUSIONS

From the investigation on a hydrated high-performance calcium sulfoaluminate cement cured at early ages it can be argued that:

a) only one reaction occurs in the system, namely the hydration of $C_4A_3S$ and calcium sulfate to give ettringite and aluminium hydroxide; its initial rate is elevated and high $C_4A_3S$ conversion degrees are reached within a relatively short curing period;
b) the reaction requires a high (0.78) stoichiometric mass ratio between water and reactant solids; so water is quickly used up for the products formation and other calcium aluminates as well as calcium silicates contained in CSA cement are not involved in the hydration, being less reactive than $C_4A_3S$; at 28 days of curing the reaction is almost stopped and both $C_4A_3S$ and calcium sulfate are still present in the hydrated systems;
c) according to the high initial rate of reaction, the development of the pore structure is very fast and prevailing low-porosity regions are rapidly established, owing to the blockages of the hydration products.

REFERENCES


Reactive magnesium oxide cements: properties and applications

L.J. Vandeperre, M. Liska & A. Al-Tabbaa
Department of Engineering, University of Cambridge, Cambridge, UK

ABSTRACT: Production of Portland cement (PC) causes significant CO₂ emissions. A recently proposed family of alternatives are reactive magnesium oxide (MgO) cements. These are mixtures of normal hydraulic cements, such as PC, magnesium oxide and a pozzolan, such as pulverised fuel ash (pfa). In this paper, an overview will be given of ongoing research on these cements. The physical characteristics of the pastes and the hydration reactions that occur initially are described and supplemented with a description of the reactions that occur later on. The effect of carbonation on these materials is discussed. The chemical changes that occur in these materials are then correlated with measurements of strength or stiffness as a function of time. To illustrate possible applications the results of 2 case studies are summarised: (1) a light-weight masonry unit made almost entirely out of pfa, and (2) the use of these cements to produce standard aggregate-cement masonry units.

1 INTRODUCTION

Reactive magnesium oxide cements are mixtures of magnesium oxide (MgO) and Portland cement (PC) to which a pozzolan may be added. Such blends were recently proposed [Harrison 2001] as a way to improve the environmental performance of cementitious mixtures relative to Portland cement. The formulation of the blends depends on the requirements of the product: small additions of MgO are claimed to be beneficial for improving the durability of Portland cement products, while blends where MgO is the main component are claimed to be a low energy and reduced emission alternative to Portland cement for products where only moderate strengths are required. It has been stressed that for the latter type of mixes, carbonation of the material is required if useful strengths are to be obtained in an economic way [Harrison 2003].

In this paper, an overview is given of quantitative results obtained for these materials as well as some case studies of possible applications. The aim of the research was to investigate the use of reactive MgO cements as a way to enhance the use of by-products or wastes as raw materials for construction products. Hence, the focus has been on moderate strength products such as masonry units. In line with the proposed design philosophy for the blends, work has therefore been limited to mixtures containing at least as much MgO as PC and the blends contained typically at least 50 wt% of an industrial by-product: pulverised fuel ash (pfa). All results are compared to the performance of blends with PC and pfa, that is without MgO.

2 MATERIALS

Most results, which will be presented here, have been obtained by blending one reactive MgO powder (XLM, Causmag Ltd., Australia), with one type of PC (Blue Circle CEM-I, Lafarge, UK) and one type of pfa (run-of-station pfa, Powergen, Ratcliffe-on-Soar, UK). The characteristics of these materials have been summarised in Table 1. Results published elsewhere show that the source of pfa and MgO is relatively unimportant, provided the latter is a reactive or caustic MgO [Vandeperre et al. 2006a, b]. Where appropriate, the

<table>
<thead>
<tr>
<th></th>
<th>r-MgO</th>
<th>d-MgO</th>
<th>PC</th>
<th>pfa</th>
</tr>
</thead>
<tbody>
<tr>
<td>CaO</td>
<td>1.2</td>
<td>5.6</td>
<td>64.3</td>
<td>5.8</td>
</tr>
<tr>
<td>SiO₂</td>
<td>1.2</td>
<td>7.5</td>
<td>14.1</td>
<td>42.8</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>0.2</td>
<td>0.9</td>
<td>2.8</td>
<td>10.4</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>0.2</td>
<td>3.4</td>
<td>10.3</td>
<td>29.2</td>
</tr>
<tr>
<td>MgO</td>
<td>97.2</td>
<td>83.0</td>
<td>0.6</td>
<td>1.3</td>
</tr>
<tr>
<td>K₂O</td>
<td>0.9</td>
<td></td>
<td>2.6</td>
<td></td>
</tr>
<tr>
<td>Na₂O</td>
<td></td>
<td></td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td>TiO₂</td>
<td>0.1</td>
<td></td>
<td>2.1</td>
<td></td>
</tr>
<tr>
<td>SO₃</td>
<td>6.9</td>
<td></td>
<td>4.0</td>
<td></td>
</tr>
<tr>
<td>L.O.I.</td>
<td></td>
<td></td>
<td>11.0</td>
<td></td>
</tr>
<tr>
<td>C.A.</td>
<td>0.27</td>
<td>33</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
results will be contrasted with results obtained using a dead burned MgO powder (DBM90, Richard Baker Harrison, UK). The difference between a reactive MgO and a dead burned MgO is that the latter is heated to a more elevated temperature during production, giving powders with a lower specific surface area and more perfect crystals. As a result the reactivity or the tendency to dissolve in contact with water is reduced. A typical test for the reactivity [Stuart 2005] is to add 2 g MgO powder to 100 ml of a 26.7 g/L solution of citric acid in water to which bromothymol blue is added as a pH indicator, and to measure the time until the indicator changes colour from yellow to blue, which for this indicator occurs at a pH between 6 and 7.6. The reactive powder neutralised the solution in 0.27 minutes, whereas the dead burned MgO needed 33 minutes to neutralise the acid.

3 PASTE CHARACTERISTICS

Initial work was carried out to determine the characteristics of pastes as a function of the relative quantities of MgO, PC and pfa used in the blend. Hereeto, the standard consistence and setting time were measured following the procedures in BS EN 196 Part 3 [BSI 1994].

As shown in Figure 1, it was found that the standard consistence, $SC$, of blends of the three powders could be described with a rule of mixtures and the standard consistence of pastes made with each of the three components alone:

$$SC = 0.255 \cdot W_{PC} + 0.77 \cdot W_{MgO} + 0.39 \cdot W_{pfa} \quad (1)$$

where $W_{PC}$, $W_{MgO}$ and $W_{pfa}$ are the weight fractions of respectively PC, MgO and pfa in the blend.

Hence, no synergistic packing effects occur when these powders are blended. Furthermore, PC and pfa require about the same amount of water to make a paste of standard consistence. The pfa used here requires slightly more water for a given consistence than PC, which is probably due to a combined effect of the fairly high carbon content as is clear from the high loss on ignition, see Table 1, and the fact that large irregular particles have not been removed as the pfa had not been classified.

Adding reactive MgO to the mix strongly enhances the water demand for workable pastes. As will be discussed below, this property of reactive MgO powders has a strong influence on the strength and stiffness of the set material.

The measurements of the setting time revealed two trends. A first trend was that as the pfa content of the blends was increased, the setting time also increased, see Figure 2. This is not surprising as the pfa used here has only a very limited hydraulic activity due to its low CaO content and therefore does not contribute to the initial setting of the blend.

A second trend, also shown in Figure 2, was that at all pfa contents, blends with either PC and pfa alone or MgO and pfa alone, set more rapidly than when MgO and PC were both present in the blend. More data on the setting of these materials is given in [Liska et al. 2006].

The data suggests that the dormant period during hydration is lengthened when both MgO and PC are
present. A possible explanation might be that MgO and PC clinker mutually reduce each other’s dissolution rate as both reactions raise the pH of the liquid in which they dissolve, but more work is needed to clarify the exact nature of the mechanism.

From a practical point of view, however, it is clear that the setting time of reactive MgO cements containing pfa is quite similar to the setting time of blends of Portland cement and pfa at same pfa content. Hence, if Portland cement is replaced by reactive MgO cements in a given product, only minor changes to the production cycle will be required.

4 HYDRATION

X-ray diffraction measurements made on crushed fragments of the materials using a powder diffractometer equipped with Cu Kα radiation (PW1050, Philips, Eindhoven, The Netherlands) are shown in Figure 3. All diffraction patterns show peaks consistent with the mullite and quartz present in the pfa. In blends containing PC, typical crystalline reaction products due to PC hydration such as portlandite and ettringite are also formed. Further, it is clear that MgO reacts with water to form magnesium hydroxide or brucite:

$$MgO + H_2O \rightarrow Mg(OH)_2$$ (2)

When no Portland cement is added, the peaks for the brucite align better with the expected positions for these peaks, whereas when PC is present the {101} reflection ($2\theta = 37.98^\circ$) is shifted toward lower $2\theta$ values. This could indicate that some material is being incorporated into the brucite. The only other feature in the patterns that could indicate the presence of another crystalline hydration product is the low intensity and broad peak around $11^\circ 2\theta$ which is absent in blends without MgO. At the time, it was not possible to attribute this peak to a known phase with a sufficient level of confidence, but one possible candidate was considered to be a hydrotalcite:

$$Mg_6Al_2(CO_3)_3(OH)_6(H_2O)_4$$ (3)

which is known to form during hydration of mixtures of MgO and Al₂O₃[Ahari et al. 2002]. Data collected later on, which will be presented below, suggests that this attribution is correct.
On the whole, however, and consistent with the fairly independent hydration of the different clinker phases in PC [Mindess et al. 2003], hydration of MgO and PC occurs in parallel, that is little or no products are formed due to chemical interaction between the PC clinker and MgO.

Importantly, based on the relative intensity of the area of the peaks for MgO after hydration and of the powder before exposure to water, it was estimated that even as early as 28 days 94% of the MgO had hydrated. The same experiment performed with a dead burned MgO, yielded a conversion of only 36% of the MgO. This suggests that the risk for unsoundness due to late hydration of MgO, which is the main reason for limiting the MgO content of Portland cement, is substantially reduced provided the MgO powder used has a sufficient reactivity.

5 EARLY AGE STIFFNESS AND STRENGTH

To optimise the water to solids ratio, blends with a range of water to solids ratios were prepared and cured in a closed container with moist air (RH > 98%). The dynamic modulus was determined by measuring the resonance frequency of cylindrical samples (RFDA Analyser, IMCE, Diepenbeek, Belgium) using a method outlined in [ASTM 2001], and the compressive strength was measured using the methods in BS EN 12390 Part 3 [BSI 2002], with the exception that the samples were tested dry.

Figure 4 shows the variation of dynamic elastic modulus, $E_{\text{dyn}}$, with the water to solids ratio. As indicated by the dashed lines, for any water to solids ratio, the stiffness tends to be higher for blends with 50 wt% pfa than for blends with 90 wt% pfa. Given the slow nature of the pozzolanic reaction [Wesche 1991], this is not surprising.

Another result, which is similar to what is observed for PC [Mindess et al. 2003], is that as the water to solids ratio is reduced, the stiffness increases until at some point a further reduction in water to solids ratio yields a reduction in stiffness. This was invariably observed at water to solids ratios were the mixture became so dry that crumbling occurred, which meant that the blends could no longer be sufficiently compacted.

Further, the stiffness for PC/pfa blends with 50 wt% pfa is somewhat higher than all other blends with 50 wt% pfa at the same water to cement ratio. To some extent this is an artefact because more bleeding occurs in mixtures without MgO, so that the effective water to solids ratio in PC/pfa blends of high water to solids ratio is slightly lower than the ratio used to make the blend.

The results of strength measurements are shown in Figure 5. Comparing Figure 4 and Figure 5 reveals that to a large extent stiffness and strength correlate. However, for the same weight fraction and the same water to solids ratio PC yields higher strengths than MgO. In terms of highest strength that can be achieved with a given composition of solids in the blend, PC outperforms MgO further because the high water demand of the reactive MgO, see section 3, means that the minimum water to solids ratio that can be used is higher for mixtures with more MgO and this leads to lower strength and stiffness.
The microstructures shown in Figure 6 help to understand the difference in strength between the different types of blends: at the water to solids ratio, which yields maximum strength, the microstructure of blends containing 50 wt% pfa and 50 wt% MgO still consists of particles, which have merely been bonded to one another in their contact regions, while in the case of 50 wt% pfa and 50 wt% PC, a continuous matrix of hydration products, which completely engulfs the pfa particles, has formed, and such a structure can be expected to be much stronger than linked particles.

6 LONG TERM BEHAVIOUR

Three types of blends containing 50 wt% pfa were prepared, see Table 2, and cured at room temperature (235 ± 5°C) under high humidity (>95% RH). The dynamic stiffness and unconfined compressive strength were measured using the methods described above at 28, 56, 112 and 224 days. In addition to this, the dynamic stiffness was also measured weekly on smaller samples. As the test is non-destructive, this allowed the same samples to be followed in time.

In addition to the mechanical properties, X-ray diffraction and pH measurements were carried out. For the latter, crushed fragments of the material were added to deionised water in a ratio of 1 g of solids for 2 g of liquid, and the pH was measured using a standard glass pH-electrode after 24 h equilibration on a shaking table.

The dynamic modulus results shown in Figure 7 illustrate that hydration of reactive MgO is a relatively rapid process, with the stiffness of samples of type M reaching 80% of the 28 day value in 1 day. Consistent with the delay in setting noted during measurements of the setting time, see Figure 2, the stiffness development of blends containing both PC as well as MgO is slower initially. The dashed line in Figure 7 shows the evolution of the stiffness of samples with the same composition but made with the dead burned MgO and a lower water content (w/s 0.3). The lower water demand of the dead burned MgO allowed to achieve a higher stiffness initially. However, after about 5 months, the stiffness

Table 2. Composition and water to solids ratio, w/s, of the blends studied for long term evolution of properties. All blends contained 50 wt% pfa.

<table>
<thead>
<tr>
<th></th>
<th>MgO</th>
<th>PC</th>
<th>w/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>M</td>
<td>50%</td>
<td>0%</td>
<td>0.5</td>
</tr>
<tr>
<td>MP</td>
<td>25%</td>
<td>25%</td>
<td>0.375</td>
</tr>
<tr>
<td>P</td>
<td>0%</td>
<td>50%</td>
<td>0.3</td>
</tr>
</tbody>
</table>

The water demand of reactive MgO is not an important factor for blends with 90 wt% pfa, where the minimum water to solids ratio is determined more by the pfa than by either PC or MgO. It would appear therefore that the design rule, which suggests that reactive MgO cements with a high MgO content should only be used in applications where only a limited amount of cement is needed, effectively expresses the problems due to the water demand of reactive MgO. It follows that if the water demand could be reduced while retaining the reactivity, more applications could become within the reach of reactive MgO cements. Therefore, more work is needed on tailoring MgO powders specifically for use in cements.
starts to decrease suggesting that cracking is occurring. Such cracking has so far not been observed for the samples made with the reactive MgO, suggesting that these indeed hydrate sufficiently early on to avoid cracking due to further expansive hydration of MgO. This is consistent with the estimates based on the x-ray results at 28 days reported above. The stiffness measurements are being continued to clarify this issue further.

The trends in stiffness and strength with MgO to PC ratio seen at 28 days persist for longer times as well: addition of PC to a blend yields stiffer and stronger materials, see Figures 7 and 8, than addition of MgO. In fact, as ageing is continued, the difference becomes even greater: the stiffness and strength of blends containing PC continues to increase significantly with curing time, whereas in absence of PC, the increase in strength and/or stiffness is very limited.

The reason for the continued increase in strength and stiffness of blends containing PC is probably the pozzolanic reaction of portlandite with the pfa through which more C – S – H gel is formed [Wesche 1991]. Consistent with this interpretation, the X-ray diffraction measurements, see Figure 9a, show that the portlandite content of the blends first increases as expected due to increased cement hydration.

However, after 2 months, when any further PC hydration becomes very slow, the portlandite content decreases as more is being consumed than created.

There is no a priori reason why brucite could not react with the fly ash to form an equivalent M S H gel. Indeed recent chemical synthesis experiments have shown that it is possible to produce such gels starting from solutions of magnesium nitrate and sodium metasilicate [Brew&Glasser 2005].

The amorphous nature of gels makes it difficult to reach conclusions on its formation using X-ray diffraction as the glassy signal due to pfa may well mask the signal due to M – S – H gel. However, as shown in Figure 9b measurements of the area of a brucite peak show that the brucite content does not change much with time, which indicates that most magnesium remains locked in brucite.

It should be pointed out that there is a further complicating factor: as shown in Figure 10, the pH is easily about 1 unit higher in blends containing PC than in the blends without PC. Therefore, the glass in the pfa will dissolve more rapidly in blends with PC than in those without it. Hence, it can not be concluded that brucite and pfa do not react. However, it is clear from the measurements that if it is occurring, it is a very slow process.

7 CARBONATION

From the above it is clear that blends without PC do not result in strong materials, although the difference in strength is more limited for blends with high pfa contents, i.e. low cement contents. Carbonation of blends with a high MgO content is said to improve the properties [Harrison 2003], and it is even claimed that construction products rich in MgO could absorb carbon dioxide throughout their life-time making the materials carbon neutral. Therefore two carbonation
experiments were conducted. The first experiment aimed to determine whether carbonation occurs, and how quickly, when the materials are simply exposed to an ambient CO2 level, while the second experiment aimed to determine whether forcing the carbonation could enable to achieve reasonable properties without adding Portland cement.

7.1 Natural carbonation

Samples containing 50 wt% pfa and 50 wt% MgO were produced and after 14 days of curing submitted to a cycle consisting of 1 day submersion in water followed by 2.5 days of drying in ambient conditions. The treatment was performed to promote natural carbonation as it is believed that repeated dissolution and re-precipitation of material aids carbonation of brucite [Deelman 2003].

X-ray diffraction patterns were taken after 45, 90, 180 and 365 days. Some of these are shown in Figure 11, and it is clear that the x-ray diffraction pattern hardly changes. The only distinct change is that the broad peak around 11° 2θ becomes stronger as the treatment is continued. As discussed above, this peak is probably due to a hydrotalcite. The fact that other peaks of this phase develop as the quantity present increases suggests the originally tentative identification is correct. Hence, the carbonation process involves incorporation of trivalent aluminium ions, probably from the pfa, on the sites of divalent magnesium ions, which causes a charge imbalance which is resolved by incorporating CO$_3^{2-}$ ions in between the hydroxide layers. More information on these phases is given in [Allada et al. 2005]. Hence, while normally the result of carbonation of brucite in environments low in CO$_2$ is hydromagnesite [Lippmann 1973], the presence of the pfa appears to have changed the carbonation product. The conclusion from this experiment is that natural carbonation does occur, but it is clear from the very limited decrease in brucite peaks after one year of exposure that the process is slow.

7.2 Forced carbonation

Small samples (50 mm × 20 mm × 5 mm) containing 50 wt% pfa and 50 wt% MgO, with a water to solids ratio of 0.4, were exposed to 5% CO$_2$ at 60°C in an autoclave for 14 days. The results are shown in Figure 12, and it is clear that the x-ray diffraction pattern changes significantly. The broad peak around 11° 2θ becomes much stronger as the treatment is continued. As discussed above, this peak is probably due to a hydrotalcite. The fact that other peaks of this phase develop as the quantity present increases suggests the originally tentative identification is correct. Hence, the carbonation process involves incorporation of trivalent aluminium ions, probably from the pfa, on the sites of divalent magnesium ions, which causes a charge imbalance which is resolved by incorporating CO$_3^{2-}$ ions in between the hydroxide layers. More information on these phases is given in [Allada et al. 2005]. Hence, while normally the result of carbonation of brucite in environments low in CO$_2$ is hydromagnesite [Lippmann 1973], the presence of the pfa appears to have changed the carbonation product. The conclusion from this experiment is that forced carbonation does occur, but it is clear from the very limited decrease in brucite peaks after one year of exposure that the process is slow.
ratio of 0.5, were cured for 14 days and then exposed to 20% CO₂ for 21 days. Both the curing as well as the carbonation was carried out in an environment with a high relative humidity (RH > 90%).

The choice for small samples was made to allow determining the change in properties due to carbonation when carbonation occurs throughout the sample. As compressive strength could not be measured on such samples, the toughness was measured instead by introducing a sharp notch in the centre of the sample and loading it in three point bending [Brown&Pomeroy 1973; Rooke&Cartwright 1976]. The size of the notch was determined on the fracture surface after failure. The dynamic stiffness was also determined as before by measuring the resonance frequency.

The carbonation changed the microstructure of the material dramatically: from rounded particles bound only in the contact regions before carbonation, see Figure 6a, the microstructure after forced carbonation consists of closely packed fibrous crystals, see Figure 12.

The newly formed fibrous crystals were identified as nesquehonite (MgCO₃·(H₂O)₃) using X-ray diffraction, see Figure 11, and the strong reduction in brucite peaks confirms that most of the brucite has carbonated under these conditions. The formation of nesquehonite at room temperature and high CO₂ levels is consistent with observations elsewhere as

![Figure 12. Scanning electron micrograph of a material containing 50 wt% pfa and 50 wt% MgO after 21 days in 20% CO₂.](image)

![Figure 11. X-ray diffraction patterns for samples with composition MgO:pfa 1:1 for various conditions of exposure as indicated in the figure (n: nesquehonite, h: hydrotalcite, b: brucite).](image)
summarised in a tentative practical phase diagram by Langmuir [Lippmann 1973].

As shown in Figure 13, both the dynamic elastic modulus as well as the toughness increase rapidly with carbonation. After approximately one week, the dynamic modulus is 15 GPa, which is much closer to the range of values observed for mixtures with 50 wt% pfa and 50 wt% PC (20–25 GPa) than the original 4 GPa.

The toughness increases simultaneously from 0.1 to 0.4 MPa m$^{-1/2}$, which is also much closer to the toughness measured for 50 wt% pfa and 50 wt% PC (0.3–0.6 MPa m$^{-1/2}$) as well as typical values for cement pastes and mortars (0.2–1 MPa m$^{-1/2}$) [Brown & Pomeroy 1973].

It is clear from these results that carbonation of reactive MgO cements indeed enhances the stiffness. Moreover, the increase in toughness suggests that the strength will increase as well. Experiments are being conducted to establish whether such improvements can be achieved in large samples. This remains to be seen as it is clear from much denser microstructure that is formed, see Figure 12, that further carbonation might be hampered by sealing of the microstructure by a carbonated rim.

8 APPLICATIONS

The initial characterisation of the strength that can be achieved with reactive MgO cements with a high MgO content suggests that such cement formulations are indeed best suited for applications were only low amounts of cements are required, and that for compositions without PC ideally a carbonation step should be included in the production process. With this in mind and also taking into consideration the aim to preferably use waste or by-products as raw materials, two cases studies were selected: light weight masonry units consisting almost entirely out of pfa and standard aggregate-cement masonry units and the results of these are reported below.

8.1 Lightweight masonry units based on pfa

Typical strength values for commercial light-weight masonry units in the United Kingdom are 4.2, 7 or 10 MPa. Therefore a choice was made to produce materials with a strength of around 7 MPa at 28 days. In addition, it was decided to use the same water to solids ratio for all compositions.

Therefore the pfa content was set at 80 wt% pfa and the water to solids ratio to 0.35, see Table 3. The samples were cured at room temperature (23 ± 5 °C). The dry density was determined in accordance with BS EN 772 Part 13 [BSI 2000b], the coefficient of capillary water absorption was determined in accordance with BS EN 772 Part 11 [BSI 2000a], and the unconfined compressive strength was measured as before on three samples of height 100 mm and diameter 50 mm after 28, 56, 112 and 224 days.

Table 3. Composition, water to solids ratio, w/s, and density at 28 days after drying at 70°C for lightweight masonry units based on pfa. All blends contained 80 wt% pfa.

<table>
<thead>
<tr>
<th></th>
<th>MgO</th>
<th>PC</th>
<th>w/s</th>
<th>Density kg m$^{-3}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>M</td>
<td>0 wt%</td>
<td>20 wt%</td>
<td>0.5</td>
<td>1318 ± 2</td>
</tr>
<tr>
<td>MP</td>
<td>10 wt%</td>
<td>10 wt%</td>
<td>0.375</td>
<td>1294 ± 1</td>
</tr>
<tr>
<td>P</td>
<td>20 wt%</td>
<td>0 wt%</td>
<td>0.3</td>
<td>1280 ± 3</td>
</tr>
</tbody>
</table>

Figure 13. (a) Change in stiffness and (b) change in toughness with carbonation time.
To test the change in mechanical properties upon carbonation of blends without PC, an additional set of samples of type M was cured for 2 days and then exposed to an environment containing 20 %CO₂ and a relative humidity of 80–90 %RH.

The density is reported in table 3 and all materials have a density below 1500 kg m⁻³, which is a typical value below which units are considered to be lightweight.

Figure 14 shows that the unconfined compressive strength at 28 days is indeed close to 7 MPa for blends of PC and pfa and blends of MgO, PC and pfa, but much lower for blends with only MgO and pfa. Furthermore, while the strength of the former blends continues to rise with ageing time, the strength of blends without PC remains fairly constant with further curing. This is similar to what was reported above for blends with 50 wt% pfa, confirming that any reactions that might be occurring between brucite and pfa are too slow to contribute to the properties in a realistic time-scale.

The coefficient of capillary adsorption of the materials is plotted together with literature values for a range of materials in Figure 15. Clearly, the measured values are quite similar to typical values for bricks and mortars.

Figure 16 shows that carbonating a sample made from a blend, which contains only MgO and pfa, increases the compressive strength from 1 MPa to 6 MPa. Hence, light-weight units with a strength of 4.2 MPa based on MgO and pfa alone are clearly achievable, and with optimisation perhaps even units with a strength of 7 MPa.

A cross-section through one half of a carbonated sample is shown in Figure 17. Upon grinding and polishing the cross-section, it was observed that a hard-wearing rim had formed, indicated by the dashed line as well as spherical spots in the interior, which are also hard-wearing relative to the surrounding matrix. Before carbonation the material is not hard-wearing and therefore these regions are regions where carbonation has proceeded extensively. This suggests that carbonation has not only occurred at the edge but also to a more limited extent in the interior of the material. Why the carbonation in the interior of the material tends to concentrate in specific regions is not clear at present.
Nevertheless, the significant increase in compressive strength shows that it is possible to achieve sufficient mechanical properties for light-weight masonry units using mostly an industrial by-product (pfa). The trade-off is that the cement content needed is fairly high (20 wt%), which makes it an expensive solution. To allow a reduction in the cement content, the use of aggregates in conjunction with reactive MgO cements was studied in a second case study.

8.2 Aggregate-cement masonry units

As before, the study was conducted for reactive MgO cement formulations with MgO, MgO and PC in equal quantities and PC. Two types of aggregates were considered: mixtures of sharp sand and gravel, and mixtures of different grades of Lytag (CEMEX Lytag, Retford, UK), which is a commercial product obtained by sintering pfa. Sieving results for the different aggregate sources are shown in Figure 18. The sand and gravel mixtures were included in the study to allow comparison with more standard results, while the Lytag-based aggregates were selected to replace natural resources by aggregates made from industrial by-products.

To obtain a reasonable grading with a fineness modulus between 3.2 and 4.2 [CCI 2001], gravel, sharp sand and pfa were mixed in a ratio of 10:7:1, where the pfa was used to obtain a sufficient amount of fines. For the grading of Lytag aggregates, two types of Lytag were mixed: Lytag (0–4 mm) and Lytag (4–8 mm) in a ratio of 1:1.31. As the former fraction already contains a sufficient amount of fines, no further pfa additions were made.

After initial work to optimise the production of cylindrical samples (Ø 50 mm × 70 mm) by uni-axially pressing, further work was carried out using compositions reported in Table 4 and 5 and using a consolidation pressure of 5 MPa. The water to cement ratio in...
the tables is expressed relative to the dry mass of PC and MgO, but the actual amount of water added is often lower as it is corrected for the water content of the aggregates, which is measured before mixing. Consistent with the results presented above, it was found again that more water is needed when the reactive MgO content is higher.

The evolution of compressive strength for both groups of samples is reported in Figure 19. As observed for samples without aggregates the strength increases with the amount of PC in the mix.

Most compositions with Lytag aggregates are weaker at 28 days than compositions with natural aggregates, but with time the influence of the type of aggregate on the strength decreases, so that by 112 days the strengths are fairly similar. Comparing Figure 19 with Figure 14 shows that at 112 days the strengths for mixtures with aggregates are quite similar to those of mixtures with 80 wt% pfa without aggregates.

However, for mixtures with aggregates these strengths are achieved for only 10 wt% cement, whereas in the mixtures with 80 wt% pfa, 20 wt% cement was used, which illustrates quite well the generally accepted idea that using well graded aggregates allows to reduce the cement content needed to reach a certain strength.

The coefficient of capillary absorption of the materials made with aggregates is also comparable with what is obtained for other building materials, see Figure 15. In comparison with the materials made with 80 wt% pfa, the values are higher, which is sensible as the pores between the coarser aggregate particles can be expected to be larger and hence water can permeate easier into the materials with aggregates.

Given the significant improvements that can be achieved by carbonation of mixtures without PC, samples of type Mna and Mlyt were also forced to carbonate by exposing them to 20% CO2 in high relative humidity (RH >90%). As shown in Figure 20, the improvements in strength were even more pronounced for aggregate based systems than for systems without aggregates. Where before the strength increased to close to but below what could be achieved by 28 days hydration of Portland cement, the strengths here are higher than what
is obtained at 28 days of Portland cement hydration. This is probably due to easier transport of CO₂ towards the interior of the aggregate based materials than towards the centre of the materials made with 80 wt% pfa as indicated by the difference in capillary absorption.

The fact that the strength is significantly higher than the strength range for typical masonry units (4.2–10 MPa), suggest that it might be possible to reduce the cement content further while achieve useful properties.

8.3 Other applications

Other applications of reactive magnesium oxide cements with a high MgO content currently being investigated are the ability of these reactive magnesium oxide cements to immobilise a range of contaminants such as metal ions, salts and hydrocarbons, and the production of artificial aggregates by pelletisation, but these results will be presented elsewhere.

9 CONCLUSIONS

Pastes of reactive MgO cements require more water for the same consistence proportional to the amount of MgO in the composition. Pastes based on MgO and pfa set faster than pastes based on PC and pfa, but when PC and MgO are both mixed with pfa, setting is slower than in mixes of PC and pfa.

Reactive MgO was confirmed to react extensively with water in a fairly short time period, with approximately 94% of the MgO converted into brucite at 28 days. In contrast, only about one third of a dead burned MgO hydrates in the same time span, leaving much unhydrated MgO in the body, which was confirmed to lead to unsoundness. To date, cracking has not been observed in materials made with reactive MgO. In mixtures containing both PC and MgO, the normal hydration products for PC were found in addition to brucite, suggesting that the hydration reactions occur in parallel.

At low pfa contents, the strength and stiffness that can be achieved increases strongly when the PC to MgO ratio is increased. Two factors contribute to the difference: (i) at the same water to solids ratio, PC tends to yield stronger material, and (ii) the high water demand of reactive MgO powders means that the minimum water to solids ratio that can be used increases with the MgO content of the mix, which reduces the strength and stiffness further. Therefore, it can be expected that further improvements in the properties that can be achieved could be obtained by tailoring MgO powders to retain high reactivity while reducing the water demand. At higher pfa contents more similar water to solids ratios can be used and this leads to smaller strength differences, so that with the sources of reactive MgO available commercially today, reactive MgO cements with high MgO contents are indeed better suited for applications where the cement content is limited.

With further ageing, blends with pfa and MgO show almost no further increases in strength of stiffness, while if PC is present further strengthening does occur. X-ray diffraction results showed that the portlandite content decreases with time consistent with a pozzolanic reaction with the pfa. The amount of brucite however remained stable and hence no analogous reactions between brucite and the pfa are occurring. It is not clear at present whether this is due to the lower pH or the stability of the brucite.

Natural carbonation by exposure to atmospheric CO₂ levels leads to the formation of a limited amount of hydrotalcite, but the reaction is slow. Forced carbonation leads to significant increases in stiffness, toughness and strength for samples without PC.

This information was then applied in two case studies for masonry units: one with 80 wt% pfa and no aggregates and one study where the properties using two types of aggregates were determined. It was shown that in the presence of 80 wt% pfa quite similar 28 day strengths could be obtained with blends of PC and blends of PC:MgO in a ratio of 1 to 1, but not with blends containing no PC. Moreover, mixes with PC strengthen more with further ageing due to the reaction of portlandite with pfa while blends without PC do not strengthen further due to the absence (or very slow nature) of such a reaction between brucite and pfa. Upon carbonation of samples made out of 80 wt% pfa and 20 wt% MgO the strength increased significantly from 1 to 6 MPa.

The same strengths achieved in mixes with 80 wt% pfa and 20 wt% cement can be achieved with only 10 wt% cement by adding well graded aggregates to the mix. Although strengthening is somewhat slower for mixes with Lytag aggregates than for mixes with gravel, sharp sand and pfa, by 112 days the strengths are very similar. Again materials made without PC in the cement were much weaker, but carbonation was found to increase the strength to levels over and above what was found to be possible with PC.

Hence, it is possible to obtain properties comparable to what can be achieved with Portland cement using reactive MgO cements. PC can be omitted from the composition entirely if it is feasible to include a carbonation step in the production process.

REFERENCES


CCI 2001. The manufacture of concrete paving blocks, Cement and Concrete Institute of South Africa.


Harrison, J. 2003. New Cements Based on the Addition of Reactive Magnesia to Portland Cement with or without added Pozzolan. CIA Conference : Concrete in the Third Millennium. Brisbane, Australia, CIA.


1 INTRODUCTION

Nowadays a broad array of products is available on the market for the protection of concrete surfaces [Basheer & Cleland 2006, Basheer et al. 1997, Ibrahim et al. 1997]. Several of these products are organic coatings consisting of volatile compounds. The air-polluting effect of these compounds during manufacturing and coating works has lead to the development of new formulations such as inorganic coating materials. Traditional inorganic coatings consist of calcium-silicate compounds, which exhibit a composition similar to cement [Moon et al.]. Promising results of an innovative technique based on microbial mineral precipitation have lead to research concerning the use of bacteria in concrete. This paper reports the effects of bacterial CaCO₃ precipitation on parameters affecting the transport processes and durability of concrete and mortar. Pure cultures of ureolytic bacteria were compared for their effectiveness in relation to conventional surface treatments, such as water repellents and coatings. Microbial calcite precipitation was visualized by SEM. The results indicated the presence of a newly formed layer on the surface of the mortar specimens, consisting mainly of calcite. Bacterial deposition of a layer of calcite on the surface of the specimens resulted in a decrease of capillary water uptake and permeability towards gas. The results obtained with cultures of the species Bacillus sphaericus were comparable to conventional surface treatments.

Microbial mineral precipitation involves various microorganisms, pathways and environments. Considerable research on carbonate precipitation by bacteria has been done by using ureolytic bacteria. These bacteria are able to influence the precipitation of calcium carbonate by the production of a urease enzyme. This enzyme catalyzes the hydrolysis of urea to CO₂ and ammonia, resulting in an increase of the pH and carbonate concentration in the bacterial environment [Stocks-Fischer et al. 1999]. Precipitation of calcium carbonate crystals occurs by heterogeneous nucleation on bacterial cell walls once supersaturation is achieved. The fact that hydrolysis of urea is a straightforward common microbial process and that a wide variety of microorganisms produce the urease enzyme makes it ideally suited for biotechnological applications [Hammes & Verstraete 2002].

Research has indicated that a concrete which is low in permeation properties lasts longer without exhibiting signs of distress and deterioration. Therefore, the permeation properties have been used principally for the comparison of the effectiveness of different surface treatments enhancing the durability of concrete [Nolan et al. 1995].

The goal of this research was to investigate the effects of bacterial carbonate precipitation on the durability of cementitious materials.
mortar and concrete in general. The durability was assessed from the permeation properties. Experiments were performed on mortar cubes of varying porosity (w/c). In order to gain a better insight into the efficiency of the bacterial treatments, results were compared to those obtained with conventional surface treatments.

2 MATERIALS AND METHODS

2.1 Mortar mixture proportions

The mortar samples were made using an normal portland cement (CEM I 52.5 N) content of 350 kg/m³, sand 0/4 content of 1050 kg/m³ and water-cement ratios of 0.5, 0.6 and 0.7. Average compressive strength of each mix was obtained from the measurements (according the Belgian standard NBN15-220) on three cubes (100 mm) after 28 days of water curing and amounted to 50.3 ± 3.96 MPa, 48.5 ± 3.40 MPa and 41.6 ± 3.91 MPa respectively. Cubes (40 mm) for capillary water suction experiments were cut from mortar slabs (900 x 600 x 40 mm). Cylindrical specimens (H = 50 mm and D = 150 mm) were drilled out of mortar slabs for the determination of gas permeability. For each mixture, a number of slabs were cast and cured for one day at 90% R.H. and 20°C. The specimens were demolded after 24 h and then stored in water for 27 days prior to the drilling of the specimens. Afterwards the specimens were stored under humid atmosphere (60% R.H., 20°C) until application of the surface treatment (at the age of 1–2 months).

2.2 Micro-organisms and growth conditions

Bacillus sphaericus LMG 225 57 (BCCM, Gent) was used for this study. Selection of this strain was based upon earlier work by our research group. This strain showed a high urease activity, a continuous formation of dense calcium carbonate crystals and a very negative ζ-potential [Dick et al. 2006]. Liquid culture media consisted of 3 g/L nutrient broth powder (Oxoid N.V., Drongen, Belgium), 2.12 g/L NaHCO₃, 10 g/L ammonium chloride and 10 g/L urea (VWR International, Leuven, Belgium). Liquid media were sterilized by autoclaving for 20 min. at 120°C. Cultures were incubated at 28°C on a shaker at 100 rpm for 24 h.

2.3 Treatment procedure

All treatments were applied on the side opposite to the troweled face. The biodeposition treatment was applied as follows: Mortar specimens were immersed for 24 hours (15 ± 5 mm depth) in a one day old stock culture of Bacillus sphaericus (ca. 10⁷ cfu/mL). After this inoculation, specimens were wiped with a paper towel in order to remove some bacteria from the surface. In this way ureolytic activity primarily resulted from bacteria inside the specimens. Following this wiping, specimens were immersed in solutions of varying composition in order to investigate the effects of the external calcium source (Table 1). In the absence of a calcium source, plugging of the pores results from the presence of bacterial biomass and/or precipitation from calcium ions present in the pore solution.

A lot of research on biodeposition was conducted with CaCl₂ as the calcium source [Adolphe et al. 1990, Bang et al. 2001, Stocks-Fischer, Galinat & Bang 1999]. The specimens were removed from the solution after 3 days.

Conventional surface treatments were applied with gentle brushing (following technical data instructions). An overview of the different treatments is given in Table 1. Choice of treatments was based upon commercial availability and care was taken to cover a wide range of treatments acting according different mechanisms (coatings and sealants).

2.4 Water absorption

To determine the increase in resistance towards water penetration a sorptivity test, based on the RILEM 25 PEM (II-6), was carried out. The mortar specimens (triplicate) were coated at the four edges adjacent to the treated side, to ensure unidirectional absorption through the treated side. The coating existed of two layers of polysiloxane and one layer of silicon paint.

---

Table 1. Overview of the different treatments ranked according to mechanism and composition.

<table>
<thead>
<tr>
<th>Group</th>
<th>Subgroup</th>
<th>Composition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Biodeposition</td>
<td>Bacillus</td>
<td>No calcium source</td>
</tr>
<tr>
<td></td>
<td>sphaericus</td>
<td>Calcium chloride</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Water based dispersion, one component</td>
</tr>
<tr>
<td>Surface coatings</td>
<td>Acrylates</td>
<td>Water based, gel-like</td>
</tr>
<tr>
<td>Penetrating sealants</td>
<td>Silanes</td>
<td>Water based, Alkoxysilane/siloxane</td>
</tr>
<tr>
<td></td>
<td>Silane/siloxane</td>
<td>Sodium silicates in water</td>
</tr>
<tr>
<td></td>
<td>mixture</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Silicates</td>
<td></td>
</tr>
</tbody>
</table>

*a Nutrient components in media of biomineralisation experiments had the following concentration: (basic medium) urea, 10 g/L; sodium bicarbonate, 2.12 g/L; ammonium chloride, 10 g/L; nutrient broth, 3 g/L; (additions) calcium chloride, 25 g/L.
After coating, the test cubes were dried at 45°C in a ventilated oven, establishing a mass equilibrium of less than 0.1% between two measurements at 24 hour intervals, to ensure low uniform moisture content in the matrix. The specimens were then exposed, to 10 ± 1 mm of water, with the treated side facing downwards (water level about 2 mm above the base of the specimen). This was done in an atmosphere of 20°C and relative humidity of 60%. The water level was kept constant through addition of tap water. At regular time intervals (15, 30 min; 1, 1.5, 3, 5, 8, 24, 72, 96, 120, 144 and 168 h) the specimens were removed from the water and weighed, after drying the surface with a wet towel. Immediately after the measurement the test specimens were submerged again. The sorptivity coefficient was obtained by using the following expression:

\[ \frac{Q}{A} = k \sqrt{t} \]  

where \( Q \) is the amount of water absorbed (cm\(^3\)); \( A \) is the cross section of the specimen that was in contact with water (cm\(^2\)); \( t \) is the time (s); \( k \) is the sorptivity coefficient. \( Q/A \) was plotted against the square root of time, then \( k \) was calculated from the slope of the linear relation between the former.

### 2.5 Gas permeability

Measurement of permeability towards gas was performed according to the RILEM-CEMBUREAU method [RILEM & 116-PCD. 1999]. The underlying principle is the Hagen-Poiseuille relationship for laminar flow of a compressible fluid through a porous body with small capillaries under steady state conditions. From this relationship, the specific permeability coefficient \( K \) can be calculated as follows:

\[ K = \frac{4.04 \cdot p_2 \cdot Q \cdot L \cdot 10^{-16}}{A \left( p_1^2 - p_2^2 \right)} \]  

where \( K \) is defined as the gas permeability coefficient, \( p_2 \) is the atmospheric pressure [bar], \( Q \) is the volumetric gas flow rate [mL/s], \( L \) is the thickness of the specimen [m], \( A \) is the cross section of the specimen [m\(^2\)] and \( p_1 \) the applied oxygen pressure [bar].

Measurement of the oxygen flow rate was performed using a Martin Sommer oxygen permeability apparatus (Schmidtheim, Germany). \( K_{\text{oxygen}} \) (m\(^2\)) calculated is the mean of three \( K \) values obtained at three pressure stages (2, 3 and 4 bar). Measurements were performed on 3 cylinders (with the same treatment) at the same time. Before the test, cylinders were dried at 60°C establishing a mass equilibrium of
less than 0.1% between two measurements at a 24 hour interval. Afterwards they were sealed with 2 layers of aluminium foil and put in an atmosphere of 20°C and relative humidity of 60% for one day.

3 RESULTS

3.1 SEM examination

Figure 3 shows scanning electron micrographs from the surface of an untreated mortar sample (Figure 3a) and a sample treated with bacteria and calcium chloride (Figure 3b–d) (w/c 0.5). The treatment of mortar cubes with bacteria and a calcium source resulted in the presence of crystalline deposits on the surface (Figure 3d). On closer observation, rhombohedral deposits of around 10 µm are visible within a dense matrix of irregular crystalline deposits (Figure 3c). Figure 3d shows an enlarged portion of the calcium carbonate crystals found in Figure 3c. Rhombohedral crystals, characteristic for calcite, are present in the middle of the micrograph. Most of the carbonate deposits were present as calcite crystals as was confirmed by XRD analysis in our previous research [De Muynck et al., De Muynck et al. 2005]. The rod shaped holes inside the crystals show the space that was occupied with the bacteria.

Figure 3. Scanning electron micrographs showing the surface of (a) an untreated mortar cube and (b) a cube treated with bacteria and calcium chloride. (c) Magnification of the area in (b) Rhombohedral crystals can be observed. (d) Enlarged image of a section in (c). Rod shaped holes (see arrow) indicate bacterial mediation of crystallization. Rhombohedral crystals are characteristic for calcite.
3.2 Water absorption

Figure 1 summarizes the effects of the water-cement ratio and the surface treatment on the water absorption of the mortar specimens. The influence of the water-cement ratio on the capillary water uptake was most pronounced for untreated mortar specimens and specimens treated with bacteria. The water absorption increased with increasing water-cement ratio for these cubes. The presence of bacteria resulted in a significant decrease of the water uptake compared to untreated specimens. When a calcium source was added to the medium an additional significant decrease of the water absorption coefficient was noticed. Acrylates, silanes or silanes/siloxanes decreased the water absorption of mortar specimens by more then 90% for all the water-cement ratios. With exception of the most porous specimens (w/c 0.7) similar results were obtained for the biodeposition treatment in the presence of calcium chloride. Over a period of 168 hours the cubes treated with bacteria and calcium chloride absorbed nearly five times less water than the control cubes (results not shown). The water absorption rate in this period was similar to that of the silanes and acrylates.

3.3 Gas permeability

Specimens with a higher water-cement ratio showed higher gas permeability (Figure 2). The presence of a surface treatment resulted in a decreased permeability, the effect being more pronounced with the more porous specimens (w/c 0.6 and 0.7). The biodeposition treatment in the presence of calcium chloride resulted in a decrease of the permeability of about 31%, 47% and 63% respectively with increasing w/c. The conventional surface treatments resulted in larger reductions of gas permeability compared to the biodeposition treatment.

4 DISCUSSION

The deposition of a layer of calcium carbonate on the surface resulted in a decrease of the permeation properties of mortar specimens. Plugging of the pores as a result of the presence of calcium carbonate crystals and biomass, accounts for the decreased water absorption and gas permeability.

As a result of the biodeposition treatment, specimens with a w/c 0.6 showed significantly less water absorption than untreated specimens with a w/c 0.5. Specimens with w/c 0.7 on which the biodeposition treatment was applied however showed a slightly higher water absorption coefficient than the latter. Conventional surface coatings showed similar water absorption regardless the water-cement ratio. The higher water absorption coefficient of the most porous specimens applied with the biodeposition treatment could be due to an incomplete coating. Repeated application of bacteria and a calcium source could result in an additional decrease of the water absorption. Nemati and Voordouw noticed an additional decrease of the permeability of sandstone cores after injecting CaCO$_3$ forming reactants for a second time [Nemati & Voordouw 2003]. The influence of repeated applications of bacteria and a calcium source on the permeation properties of concrete will be investigated in future research.

One of the factors to be considered in coating selection is the durability of the coating. As calcium carbonate is solubilised in acidic environments, there is a need to investigate the effect of acidic rain on the durability of the biodeposition treatment. Bacterially induced calcite crystals however, are assumed to be more resistant to dissolution since it has been experimentally demonstrated that biologically deposited calcite is less soluble than inorganically precipitated calcite [Morse 1983]. In future research the durability of the calcite layer under varying conditions will be investigated.

5 CONCLUSIONS

Biodeposition has the potential to improve the durability of concrete structures. The transport mechanisms in concrete are influenced by the surface treatment with the ureolytic bacteria and a calcium salt. Best results were obtained with the combination of Bacillus sphaericus and calcium chloride, with which a similar protection as some of the conventional surface treatments was obtained.

Further research however is needed to investigate the impregnation depth and the durability of the treatment under environmental conditions.

ACKNOWLEDGEMENTS

This research was funded by a Ph.D grant of the Institute for the Promotion of Innovation through Science and Technology in Flanders (IWT-Flanders IWT/SB nr. 41260). The financial support of the Fund for Scientific Research Flanders (project nr. G.0054.02) is also gratefully acknowledged.

REFERENCES


De Muynck, W., Cox, K., Verstraete, W. & De Belie, N., Bacterial carbonate precipitation as an alternative surface treatment for concrete, Construction and Building Materials submitted.


1 INTRODUCTION

The need to implement sustainable development practice in all sectors of construction has been widely recognised and, in particular, improvements in the approach to the design, construction, operation and rehabilitation of major transport infrastructure projects have already been proposed. Some have also been implemented [Britpave 2000]. Whole life costing for highway maintenance work in the UK was pioneered in the 1980’s. For example, environmental, social and financial needs have been carefully balanced for bridges and access structures and a minimum design life of 100 years or more is now typical. However, despite these advances, UK, in comparison to other economically advanced nations, has not in general embraced sustainability and whole life costing considerations for the highway pavements due to under investment and short term non-sustainable solutions [The Highways Agency & Britpave 2003].

The later may also be due to the reluctance of pavement engineers to incorporate new and innovative solutions into new pavements. Hence, existing analytical and design methods for new pavements are based on simplified models that do not always represent the actual conditions satisfactorily. Also, re-design of pavements to be repaired, is usually based on a practical (mechanistic) approach, relying on past experience and previous, long-term observations and can be costly as it does not take advantage of the remaining life of the worn pavement [Ullidtz 1987]. In fact, the increasing use of heavier and more complex types of loads generated by vehicles and aircraft, emphasises the urgent need for improved design methods for new pavements and for better, more environmentally friendly ways of rehabilitating existing roads and airport runways with the emphasis on sustainability. It has now been widely accepted that complete removal and replacement of a worn pavement is a waste of an existing asset, environmentally unfriendly and unsustainable, and should only be considered as a last resort.

2 DEFECTS & WEAKNESSES IN CONCRETE PAVEMENT OVERLAYS

The benefits of structurally improved BCO have now been recognised but not many applications have been
successful, due to several problems still waiting to be resolved [Hughes 2003]. These problems and their associated modes of failure are briefly mentioned below:

2.1 **Sub-base and sub-grade severe decaying**

Subbase and/or subgrade deterioration is a common mode of failure usually associated with drainage problems. If shear transfer across a crack (or movement joint) in the worn pavement has been lost or seriously impaired, the subbase and subgrade reaction on either side will also be inadequate. It is essential therefore that serious deficiencies in the subgrade reaction are eliminated by employing either crack-and-seat or concrete injection techniques, before any overlays are applied.

2.2 **Shear failure across old cracks and transfer to BCO**

The shear resistance across a long established crack in a worn pavement can sometimes start to deteriorate rapidly even if little loss in shear transfer is evident. The minimum depth for the overlay should therefore be established, by considering only the shear resistance of the uncracked BCO on a diagonal plane. Hence, if the maximum design cracks in the BCO can be restricted to acceptable limits, the shear resistance of the cracked BCO above an existing old pavement crack should be greater than, or equal to, the shear resistance on a diagonal plane [Hughes 2006].

\[ \tau_{crack} \geq \tau_{dia \, plane}, \quad \text{if} \quad \text{crackwidth} \rightarrow 0 \]  

2.3 **Bond failure between overlay (new) and old pavement**

Bond breaking between the old, worn out pavement and the new overlay is another common mode of failure. It has been demonstrated that this is not the result of the horizontal stresses generated by vehicle braking forces alone. Rather, it is the combination of the latter, plus other factors such as flexural loads, environmental effects (thermal gradients) and stress concentrations at defects [Lau et al 1994].

2.4 **Reflection cracking in the BCO**

These are existing cracks in the old pavement, finding their way up the surface of new overlay due to continuous positive and negative rotations of the two portions of pavement under traffic and other loads. Reflection cracking cannot be avoided, hence must be suitably resisted by restricting the crack width in the BCO. This, in turn, can be achieved by restricting the angle of rotation of the crack, pivoting about the reinforcement of the overlaid pavement [Hughes 2003].

2.5 **Flexural failure**

BCO, although of high quality concrete with a higher bond strength than normal, are still a brittle material and therefore liable to fracture without yielding. Hence, another possible mode of failure is that of flexure. The only way to avoid this kind of failure is by allowing the overlay to crack, but control cracking by introducing the right amount of continuous reinforcement. Hence provide the cracked pavement similar resistance to that of the uncracked pavement.

2.6 **Delamination at discontinuities**

Delamination is a mode of failure occurring mainly at discontinuities such as edges and movement joints where stress concentrations can be high. It is due to the combined effect of thermal and traffic loads. It can also occur at the ends or sides of the pavement, if no end restraint and/or continuous transverse and longitudinal reinforcement is present.

3 **NOTES FOR SUCCESSFUL STRUCTURAL PERFORMANCE OF BCO**

The concrete in the overlay should be of high-quality, preferably polymer modified and the bond strength at the interface should be higher than the tensile strength of the existing concrete (as interface stresses are of shearing nature a higher bond should be achievable) [Karadelis et al 2003]. Good bond strength and good preparation of the existing pavement surface is absolutely essential. Delatte et al [1998] noted from their very successful full-scale test sections in El Paso, Texas that rougher surface preparation substantially improved performance. A mean roughness of 1.30 mm was observed for the shot-blasted section and an even better 1.71 mm for hydrocleaning. They found that cold milling can cause cracking below the prepared surface and should be avoided.

Delamination tends to occur due to thermal cracks first at stress concentrations at the interface and to other discontinuities in the BCO (eg. edges and movement joints). The continuous reinforcement in the BCO greatly restrains any possible curling and warping in the pavement at cracks if the transverse as well as the longitudinal reinforcement is adequate. Vertical restraint at all transverse expansion joints or other discontinuities in BCO pavements by either a wide flange beam joint or ground beam anchorage has been proposed [Gregory 1984], [Garnham 1989]. However, this may sound as an old fashion solution and therefore other, more economical and sustainable solutions should be sought.

Reflection cracking in the BCO above all live cracks in the worn pavement subjected to flexure is inevitable. It is therefore vital that the rotation angle
at the ‘hinge’ due to the crack in the semi-rigid pavement is strictly limited by restricting the crack width in the BCO. When the BCO cracks the flexural strength, stiffness and fatigue life, as well as the shear strength must remain at least adequate. The finer the crack, the higher the flexural stiffness and the higher the long-term fatigue life. However, decreasing the design crack width increases the cost of the reinforcement (smaller diameters and/or increased amounts) but maintains sufficient overall pavement stiffness to ensure adequate preservation of the subgrade. Obtaining data for the optimum design crack widths to use for an ideal balance between an adequately extended life and economy is a most crucial objective of the proposed research.

4 EXPERIMENTAL, PILOT STUDY. AIM AND OBJECTIVES

At Coventry, we have been experimenting for some time with a polymer modified concrete and with the help of our Industrial Partners we have been able to develop a material with superior properties such as high compressive and tensile strengths of the order of 90 MPa and 12 MPa respectively and also good bonding characteristics and promising resistance to delamination. However, if this new concrete is intended for use as a pavement overlay it has to have additional special properties in order to minimise costs. For instance, it has to be easily applicable, preferably by an asphalt paver and pressed by a vibrating roller. That is, it has to have the properties of high strength concrete and the versatility, ease and speed of application of asphalt [Karadellis et al 2003]. A major drawback to the later and a parameter closely associated with costs is the conventional steel reinforcement as it includes design, purchasing, transportation and placement costs. This has to be replaced with alternatives if a more economical and faster solution, part of the overall sustainable design to pavement rehabilitation is sought.

Therefore, based on the problems listed in Section 2, above, a pilot study was planned aiming to investigate the following:

(a) the possibility to replace the main steel reinforcement with a more suitable material, examine the mode of failure of a series of specimens and propose solutions to enhance the flexural resistance of the BCO pavement.

(b) the tendency of new concrete (overlays) to break away (de-bond) from the old pavement (i.e.: study the mechanism of delamination) and to find ways to quantify and control the delamination process.

(c) the problem of delamination at the edges and corners due to curling and warping generated by thermal stresses.

### 4.1 Preliminary work

All preparatory work such as the design and manufacturing of the necessary formwork was carried out at the University’s workshops. Marine plywood was used to construct a series of prismatic moulds in order to produce the appropriate small beam and slab specimens for testing. These moulds were transported to Aggregate Industries laboratories and their recently acquired facilities were used to design, mix and cast the polymer modified concrete. Preliminary, basic results such as cube strengths were extracted and sent back to University together with a number of specimens for further testing. An indicative table is shown below (Table 1).

As it was mentioned earlier, a major drawback of the concrete roads compared against their asphalt counterparts is the very slow speed of construction of the former. This is owned to the time taken for the reinforcement to be placed and for the concrete to develop its full strength (Figure 1).

Hence, alternatives have to be found. First, the advantage of using polymeric concrete for road paving is that it develops adequate strength and can be passed to traffic within the first three days of its life. Second, a new type of reinforcement has been introduced, that of a synthetic (modified polyolefin) reinforcing fibre, which has a contoured surface similar to steel to maximise bond, a high tensile strength of up to 550 MPa and the trade name of ‘BarChip’ [Bernard S 1999].

A wholesale replacement of steel reinforcement is therefore sought, by providing a ‘cocktail’ of synthetic fibres in the mix and a mesh as main and secondary

<table>
<thead>
<tr>
<th>Trial Mix &amp; Comments</th>
<th>Strength N/mm²</th>
<th>Density Kg/m³</th>
<th>Strength N/mm²</th>
<th>Density Kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>7 Days</td>
<td>28 Days</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TM06/113</td>
<td>36.54</td>
<td>2338</td>
<td>46.88</td>
<td>2330</td>
</tr>
<tr>
<td>Plain Concrete</td>
<td></td>
<td></td>
<td>43.33</td>
<td>2327</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>46.87</td>
<td>2334</td>
</tr>
<tr>
<td>MEAN</td>
<td></td>
<td></td>
<td>45.69</td>
<td>2330</td>
</tr>
<tr>
<td>TM06/114 F1</td>
<td>66.3</td>
<td>2297</td>
<td>74.59</td>
<td>2293</td>
</tr>
<tr>
<td>Polymer. Mod. Concrete</td>
<td></td>
<td></td>
<td>72.73</td>
<td>2299</td>
</tr>
<tr>
<td>MEAN</td>
<td></td>
<td></td>
<td>73.44</td>
<td>2294</td>
</tr>
<tr>
<td>TM06/114 F2</td>
<td>64.52</td>
<td>2298</td>
<td>76.02</td>
<td>2309</td>
</tr>
<tr>
<td>Poly. Concrete + Fibres</td>
<td></td>
<td></td>
<td>67.26</td>
<td>2303</td>
</tr>
<tr>
<td>MEAN</td>
<td></td>
<td></td>
<td>76.09</td>
<td>2302</td>
</tr>
<tr>
<td>TM06/117</td>
<td>46.15</td>
<td>2292</td>
<td>62.45</td>
<td>2274</td>
</tr>
<tr>
<td>Poly. Concrete + Fibres + H₂O</td>
<td></td>
<td></td>
<td>66.59</td>
<td>2291</td>
</tr>
<tr>
<td>MEAN</td>
<td></td>
<td></td>
<td>62.34</td>
<td>2282</td>
</tr>
</tbody>
</table>
reinforcement, in an effort to keep construction time and costs as low as possible.

Figure 2(a) below shows a better performance of BarChip fibre reinforced concrete compared to its steel rivals. Specifications of the materials and details are shown in Table 2. Figure 2(b) shows a representative Load v Extension curve taken from tensile tests on Barchip ‘wire’. The maximum tensile load recorded before failure was 259 N. The mean diameter of Barchip was recorded as 1.003 mm. Hence, its tensile strength was estimated to be 327.43 Nmm\(^{-2}\) (MPa). Note that engineering stress (strength) does not give a true indication of the deformation characteristics of a ductile material because it is based entirely on the original dimensions of the specimen, whereas it is known that these dimensions change continuously during the test. However, for the present, pilot study, it was considered to be acceptable.

4.2 Tests with BarChip as main reinforcement

A series of laboratory tests were carried out on small, prismatic, polymer modified, concrete beams (360 \( \times \) 60 \( \times \) 60 mm) reinforced with BarChips to assess their performance. Similar plain concrete prisms as well as prisms with 1.00 mm diameter steel wire reinforcement are also plotted alongside, for comparison. The ‘BarChip ratio’ obtained was much lower than the corresponding minimum steel ratio allowed by BS8110 and Eurocode 2. However, the point was made. A sample of the results is shown in Figures 3 & 4.

It is recognised that the results are preliminary and that more tests are needed for confidence to increase. However, indications are that replacing conventional with continuous synthetic reinforcement may be possible, with substantial reductions in deflection and strain achieved by increasing the number of BarChip ‘wires’. That is, BarChip reinforcement can replace steel successfully and make a contribution to flexural resistance and bond strength. The later should also be more effective in controlling minor cracks as BarChip can be strong enough to crack the immature concrete. That is, if the BarChip ratio exceeds some Critical ratio still to be defined.

<table>
<thead>
<tr>
<th>Specimen Description</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Barchip synthetic fibre.</td>
<td>Dosage rate: 10 kgm(^{-3})</td>
</tr>
<tr>
<td>Length = 48 mm</td>
<td></td>
</tr>
<tr>
<td>Steel fibre. High performance, hooked end</td>
<td>Dosage rate: 40 kgm(^{-3})</td>
</tr>
<tr>
<td>Steel mesh. Type F41</td>
<td>4 mm wire @ 100 mm c/s</td>
</tr>
</tbody>
</table>

Table 2. Materials and specifications.

Figure 3. Flexural tests. Load v Deflection. Comparison between beams with different amount of BarChip reinforcement.
4.3 Assessing the bond between old pavement and new overlay

A series of PC concrete prisms were cast in special moulds blocked at 45°. The 45° surfaces were treated to different degrees of roughness, according to the number of times a wire brush was passed over, as follows: 0, 10, 20, 30, 40, 50 and 60-passes. After fully setting, polymeric concrete was added and a bond was created between the two inclined surfaces. The prisms were subjected to compression and the load to failure was recorded. Table 3 summarises the tests and shows two indicative types of failure. It is obvious from this table that a good bond can be achieved relatively easily. However, bond is limited to excess brushing, linked to high exposure of aggregate and damaging of the bonding surface.

Additionally, a series of flexural tests were carried out on beam specimens made of old PC concrete and covered with a layer of polymer modified concrete, in order to estimate the horizontal shear at the interface. Dimensions and specifications were as per Figure 6, above.

As before, in order to create a rough surface, the top face of the old concrete was treated with a wire brush prior to full setting, by applying 50, 100, 150, 200 and 250 brush passes. The treated surface was vacuum cleaned and a 25 mm layer of polymeric concrete was applied. Three pairs of demec points were attached at the vertical face of the ‘composite’ beam, at distances 10 mm, 60 mm, and 90 mm from the bottom and four point loading tests were carried out. Table 4 summarises the tests and shows the two modes of failure.

Unfortunately, due to the relatively low number of tests carried out it was not possible to draw tangible and credible conclusions by studying Table 4, above. However, it can be stated from the same table that bond strength increases with number of brush passes (rougther surface), as expected. There is however a limit to the bond strength achieved, after which the strength decreases. In this case this could be due to coarse aggregate dislocation and loosening from the parent material; hence creating a plane of weakness between the two concretes.
The strain distribution per load increment across the depth of the beams, at midspan, was also plotted. Some indicative results are shown in Figure 8, above. Whole load increments are shown only, for clarity. The position of Neutral Axis was estimated from the graph to be at approximately 47.5 mm above the bottom of the beam; slightly lower than halfway up or down, attributed to the influence of the polymeric concrete layer.

Figure 9 shows the variation of strain with load measured at midspan. Marker labelled ‘@ 100 mm’, shows readings from the electrical resistance strain gauge placed on top, whereas the rest show readings taken by the mechanical strain gauge. A typical concrete-in-flexure behaviour has been demonstrated with promises of good bond strength.

Figure 10 shows the variation of maximum deflection (at midspan) with load for Beams 2F (50 brush-passes) to 6F (250 brush-passes). The familiar trend continuous as beams with rougher interface between ‘old’ and polymeric concrete tend to behave as ‘composites’, withstand more load and undergo larger deflections.

4.4 Curling and warping at the edges and corners

These tests incorporated a series of small concrete slabs (300 × 300 × 30 mm), approximating their behaviour under thermal and traffic loads and establishing their mode and mechanism of failure. These slabs were reinforced with both BarChip fibres and wires as shown in Figure 12(b).

The tensile warping stresses developing in the slab are given by [Bradbury 1938]:

$$\sigma_w = \frac{(c)(E)(e)(\Delta T)}{2}$$

where: $\sigma_w =$ slab edge warping stress; $C =$ Coef’t, a function of slab length and radius of relative stiffness; $E =$ Modulus of Elasticity of PCC; $e =$ Thermal coefficient of PCC (0.000005/F, or 0.000009/°C); $\Delta T =$ Temperature differential between top and bottom of slab

It is obvious from equation (2) above that one way to minimise warping stresses caused by temperature difference between the top and bottom of the slab is to keep $\Delta T$ minimum. Therefore, the ultra-thin white-top slab with non conventional reinforcement was adopted. The slabs were supported on quarry sand. An infra-red (IR) lamp was placed above to simulate ambient (sun) temperatures. The temperature at top and bottom surfaces, the deflection and strain at the corners and edges were measured. Specimen preparation and instrumentation is shown in Table 5 and Figure 12 respectively.
Due to time restraints the following 240-minutes long ‘day’ was proposed:

(a) The IR lamp was placed at 400 mm above the surface of the slab and was switched on for 45 min. to simulate sunrise.
(b) The IR lamp was lowered to 300 mm and left on for 30 min (noon).
(c) The lamp was lifted up to 400 mm and left on for 45 min (sunset)
(d) The lamp was turned off for 120 min. (night time).

A series of readings were taken during the above time period; a sample of the results is presented below. Figure 13 shows the variation of temperature difference with time between the top-side of the slabs exposed to IR radiation and their bottom-side, for slabs with different amounts of synthetic fibres. It is obvious that the performance of all these slabs is very similar, as the temperature difference increases in the ‘morning’ when the IR lamp is ‘ON’ and decreases in the ‘evening’ when it is ‘OFF’. The temperature difference changes sign when the temperature at the
top becomes less than that at the bottom. Finally equilibrium is reached after a long period (not clear in Figure 13).

Figures 14 & 15 show the variation of corner and edge displacements during the same time period, demonstrating a trend for the displacement to decrease with increasing amount of reinforcement. However, there seems to be a limit to the rule, rather obvious in Figure 14, where the $5 \text{kgm}^{-3}$ amount of synthetic fibres graph is plotted above the corresponding $4 \text{kgm}^{-3}$. This is probably due to the fact that the particular slab has been 'saturated' with fibres and therefore lost some of its stiffness toughness and strength.

5 CONCLUSIONS

This paper demonstrates the need for fundamental improvements in the maintenance costs, traffic management, long-term performance and rehabilitation of concrete pavements based on a more sustainable approach. This can be possible by applying environmentally friendly, long-term solutions such as optimising the design, and redrafting the construction and rehabilitation procedures of their repairs. Even greater benefits can be claimed by combining existing design trends, philosophy and know-how with synthetic fibre and bar reinforcement. Specifically:

1. It is possible to go for a wholesale replacement of the traditional steel reinforcement with new, innovative, corrosion free, easier and faster to apply synthetic alternatives such as BarChip which can make a positive contribution to flexural resistance, crack control and bond strength of the rehabilitated pavements.

2. It is possible to achieve a good bond between the old pavement and the overlay by treating the surface of the old pavement. However, there seems to be an optimum degree of roughness before bond performance is reduced again.

3. Bonded concrete overlays perform and behave like reinforced concrete with strain distributing linearly across the depth of the beam under tests.

4. Warping at the corners and curling at the edges caused by thermal loads decreases with amount of synthetic fibres and reinforcement. Once again, there seems to be a limit of fibres each concrete specimen can handle after which the later loses stiffness and toughness and fails prematurely.

Finally, it is clear that further research is needed for continuously reinforced bonded concrete overlays to be successfully applicable. This research work should be consisting mainly of site data collection but also accurate numerical representation.

ACKNOWLEDGEMENTS

The Authors would like to express their gratitude to Aggregate Industries for their continuous technical, financial and in-kind support. Also, thanks are directed to Elasto Plastic Concrete Europe, for providing our Group with the reinforcing material and their technical support.

REFERENCES


Effect of loading level, cooling regime, polypropylene fibers, and coating type on the behavior of high strength concrete columns in fire

H.I. El Sayad
Faculty of Engineering, Banha University, Egypt

M.M. Abdel-Razek & H.S. Haddad
Housing and Building Research Center, Cairo Egypt

ABSTRACT: High-strength concrete (HSC) columns were found to be more prone to explosive spalling due to their low permeability and high brittleness. Adding polypropylene fibers to the mix could be a solution since the fibers melt during the early stages of heating and create channels in the concrete. Hence, the water vapor finds an escape without causing spalling. Coating of concrete members with special mortars, e.g. perlite and vermiculite rendering, could be another solution since the coating works as an insulation, which prevents heat from penetrating to the heart of concrete. Previous researchers pointed out that the data on the effect of fire on the behavior of HSC loaded columns is scarce. In addition, the authors did not cite any literature on the effect of cooling regime on the fire behavior of HSC loaded columns. This paper presents an experimental program on the behavior of HSC columns with or without polypropylene fibers and/or those coated with perlite or vermiculite mortar under fire. The research includes testing seventeen reinforced HSC columns heated to 750°C for one hour under three loading levels (30%, 40% and 50% of the control specimen’s ultimate load). To study the effect of cooling regime, the columns were left to cool in air or quenched by water. The results indicated that a preload level, not more than 40% of the columns’ ultimate load, leaving the column to gradually cool in air, inclusion of polypropylene fibers and coating with perlite mortar greatly improved the residual capacity after fire. The column with the above specification had a capacity almost identical to the control column, which was not heated.

1 INTRODUCTION

Conflagration has become one of the main calamities in cities around the world. As a result, design for fire resistance and fire safety evaluation of concrete structures has become an urgent issue. In order to analyze and evaluate the fire safety of concrete structures in a more reasonable way, it is important to study the effect of fire on material properties in detail. Concrete spalling under high temperatures is a major factor that reduces its fire resistance. Explosive spalling is the most dangerous type of concrete spalling. It happens with high explosive energy causing concrete shrapnel to fly with high speed causing more damage to the surrounding environment including smashing adjacent windows; letting more oxygen get into the fire area and hence leading to an increase in fire severity. This raises serious concerns about studying this phenomenon and identifying methods of preventing concrete spalling [Ali et al., 2001]. Explosive spalling happens due to excessive pore pressure inside the concrete. During concrete exposure to fire, free and combined water inside the concrete starts to evaporate. If the concrete has a low permeability, like HSC, the pore pressure starts to build up and induces stresses on the internal structure of the concrete. Vapor stresses can build up to high levels with temperature increase and if these stresses overcome the tensile strength of the concrete, local failure happens releasing high energy and leading to a chain of concrete micro-structural failure in the adjacent parts and the violent explosion of concrete takes place [Anderberg 1997]. It has been reported that even a small volumetric contents of polypropylene fibers can improve the behavior of HSC in fire [Meda et al. 2002].

An extensive review by Ali et al. [1997] cited in Ali et al. [2001], showed that high-strength concrete is more prone to explosive spalling than normal strength concrete due to its high density and low permeability. Spalling is governed by many complicated factors such as the rate of temperature elevation, mineral constituents of aggregates, thermally induced stresses, density of concrete, moisture content and so on [Luo et al. 2000]. Coatings of concrete structures have a great effect on the behavior of structures under fire conditions. If a
coating is non-combustible, such as perlite or vermiculite rendering, it can improve the fire safety of structures. Perlite is a glass volcanic rock of rhyolitic composition usually containing 2–6% of water. It shows thermal and chemical inertness and is friendly to the environment as stated elsewhere [Abdel-Razek et al. 2004a]. Perlite plasters offer up to 4–6 times more resistance to heat transmission compared to the ordinary sand plaster. Vermiculite is a member of the flake-silicate group of minerals, resembling mica in appearance. It is commonly used as flame insulation since it has a remarkable ability to expand many times more than its original volume when heated. Vermiculite plaster is 4 times more resistant to heat transmission compared to sand plaster. The use of perlite and vermiculite is recommended in the Egyptian Code, the Arab Unified Code, the Jordanian Code and the British Code for improving the fire resistance [Abdel-Razek et al. 2004b].

Despite that some investigations [Ali et al. 2001, Aldea et al. 1997 and Copier 1983] considered the effect of loads on explosive spalling of high-strength concrete columns, the data from tests carried out on stressed columns is scarce [Phan & Carino 1998]. This data is of particular interest as it simulates the conditions in structural elements when exposed to fire [Phan & Carino 1998]. In addition, only a few investigations were cited on the effect of cooling method on the residual capacity of samples (e.g. [Nassif 1999]) or test columns (e.g. [Issa & Anwar 2004]). The authors did not cite any studies on the effect of cooling method on the residual capacity of loaded columns.

This paper is an experimental study which aims to investigate the effect of some factors related to the fire safety of concrete structures, in particular, level of loading, cooling regime, presence of polypropylene fibers and coating type (perlite and vermiculite) on the residual load capacity, deformations and temperature distribution across the sections for high-strength concrete columns subject to fire. The tests were conducted at the Housing and Building Research Center, where the test facilities permit fire exposure of loaded columns and the application of different cooling regimes. Seventeen columns were tested to cover the range of variables in the test program.

### 2 EXPERIMENTAL PROGRAM

#### 2.1 Test samples, variables and materials

The experimental program was designed to cover a rational range of loading levels, cooling regimes, coating types and presence of polypropylene fibers in the mix. A total of 17 columns were prepared for testing. Details of samples and variables are shown in Table 1. The first column specimen was tested directly in ambient temperature to determine the maximum load capacity of the column models (ultimate load) and was considered as a control column. The other 16 columns were divided into four groups A, B, C and D each comprising 4 columns. Samples in groups (A), (B) and (C) were preloaded to 30, 40 and 50% of the ultimate load of the column models, obtained from testing sample No. 1, respectively. They were then subjected to elevated temperature in the loaded state and then cooled either gradually in air or rapidly by water quenching. The columns were prepared with or without polypropylene

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Group</th>
<th>Loading level (% of ultimate load)</th>
<th>Code in figure 4</th>
<th>Fibers in mix</th>
<th>Type of cooling</th>
<th>Type of coating</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Control</td>
<td>No</td>
<td>C-I</td>
<td>No</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>2</td>
<td>A</td>
<td>30</td>
<td>C-I-40 A</td>
<td>No</td>
<td>Air</td>
<td>None</td>
</tr>
<tr>
<td>3</td>
<td>A</td>
<td>30</td>
<td>C-I-40 W</td>
<td>No</td>
<td>Water</td>
<td>None</td>
</tr>
<tr>
<td>4</td>
<td>A</td>
<td>30</td>
<td>C-II-40 A</td>
<td>Yes</td>
<td>Air</td>
<td>None</td>
</tr>
<tr>
<td>5</td>
<td>A</td>
<td>30</td>
<td>C-II-40 W</td>
<td>Yes</td>
<td>Water</td>
<td>None</td>
</tr>
<tr>
<td>6</td>
<td>B</td>
<td>40</td>
<td>C-I-55 A</td>
<td>No</td>
<td>Air</td>
<td>None</td>
</tr>
<tr>
<td>7</td>
<td>B</td>
<td>40</td>
<td>C-I-55 W</td>
<td>No</td>
<td>Water</td>
<td>None</td>
</tr>
<tr>
<td>8</td>
<td>B</td>
<td>40</td>
<td>C-II-55 A</td>
<td>Yes</td>
<td>Air</td>
<td>None</td>
</tr>
<tr>
<td>9</td>
<td>B</td>
<td>40</td>
<td>C-II-55 W</td>
<td>Yes</td>
<td>Water</td>
<td>None</td>
</tr>
<tr>
<td>10</td>
<td>C</td>
<td>50</td>
<td>C-I-70 A</td>
<td>No</td>
<td>Air</td>
<td>None</td>
</tr>
<tr>
<td>11</td>
<td>C</td>
<td>50</td>
<td>C-I-70 W</td>
<td>No</td>
<td>Water</td>
<td>None</td>
</tr>
<tr>
<td>12</td>
<td>C</td>
<td>50</td>
<td>C-II-70 A</td>
<td>Yes</td>
<td>Air</td>
<td>None</td>
</tr>
<tr>
<td>13</td>
<td>C</td>
<td>50</td>
<td>C-II-70 W</td>
<td>Yes</td>
<td>Water</td>
<td>None</td>
</tr>
<tr>
<td>14</td>
<td>D</td>
<td>40</td>
<td>C-I-PRE</td>
<td>No</td>
<td>Air</td>
<td>perlite</td>
</tr>
<tr>
<td>15</td>
<td>D</td>
<td>40</td>
<td>C-I-PRE</td>
<td>Yes</td>
<td>Air</td>
<td>perlite</td>
</tr>
<tr>
<td>16</td>
<td>D</td>
<td>40</td>
<td>C-I-VER</td>
<td>No</td>
<td>Air</td>
<td>vermiculite</td>
</tr>
<tr>
<td>17</td>
<td>D</td>
<td>40</td>
<td>C-II-VER</td>
<td>Yes</td>
<td>Air</td>
<td>vermiculite</td>
</tr>
</tbody>
</table>
fibers. Samples in the fourth group (D) were coated with perlite or vermiculite mortar and were with or without fibers. The layer of coating had a thickness of 2.0 cm. The samples in group D were loaded to 40% of the ultimate load and were left to cool in air after exposure to temperature.

The tested specimens were produced from ordinary Portland cement, natural sand and crushed basalt with a maximum size of 15 mm. Silica fume and superplasticizer were added to increase strength. Mixing was performed in a rotating mixer in the Concrete Research Laboratory at Housing and Building Research Center. Mix proportions for concrete used in this study are shown in Table 2. The liquid/cementitious materials ratio of the mix was 0.29 in order to produce HSC. Low melting point polypropylene fibers (12 mm long, 0.1 mm diameter) were used in mixes with fibers at a dosage of 900 gm/m³. Table 3 shows the mix proportions of the two coating plasters used in this investigation. The selected coating mix proportions represent the mixes with adequate flow, bond and compressive strengths to resist the service loads. A square galvanized wire mesh 2.0 cm x 2.0 cm, with density 1.61 kg/m², was fixed around the columns using 1 mm diameter wire, prior to coating with perlite or vermiculite mortar. The columns were demolded after 24 hrs from casting, covered with wet burlap, and stored under laboratory conditions for 28 days before heating in the furnace.

2.2 Column design details and positions of thermocouples

The concrete columns had a square cross section of 150 mm and were 1100 mm high. Each column was reinforced with four 12 mm diameter steel bars (f_y = 360 MPa) as shown in Figure 1 and connected with 10 stirrups (6 mm diameter of f_y = 240 MPa) at 150 mm intervals in the middle and 75 mm at the ends. Stirrups were located more often near the ends to prevent any possible local column failure near the loading points. The columns were simply supported at both ends. The column design details are given in Figure 1.

Thermocouples used were type J-Iron (+) versus constant (−), which can be used for temperature range from −210 to 1200°C. In this investigation, three thermocouples for each column were connected to the data logger. The thermocouples were positioned at mid height in each column. The first thermocouple was

Table 2. Mix proportions for studied columns.

<table>
<thead>
<tr>
<th>Type of mix</th>
<th>Cement (kg)</th>
<th>Sand (kg)</th>
<th>Crushed basalt (kg)</th>
<th>Silica fume (liter)</th>
<th>Water (liter)</th>
<th>Fiber (gm)</th>
<th>Superplasticizer liter/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>No fibers</td>
<td>550</td>
<td>600</td>
<td>1250</td>
<td>55</td>
<td>150</td>
<td>–</td>
<td>23</td>
</tr>
<tr>
<td>With fibers</td>
<td>550</td>
<td>600</td>
<td>1250</td>
<td>55</td>
<td>145</td>
<td>0.9</td>
<td>28</td>
</tr>
</tbody>
</table>

Table 3. Mix proportions for coating plasters.

<table>
<thead>
<tr>
<th>Coating type</th>
<th>Cement (kg)</th>
<th>Perlite (m³)</th>
<th>Vermiculite (m³)</th>
<th>Water (liter)</th>
<th>Glass fibers (gm)</th>
<th>Air entraining agent (liter)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perlite mortar</td>
<td>500</td>
<td>1</td>
<td>–</td>
<td>275</td>
<td>800</td>
<td>4</td>
</tr>
<tr>
<td>Vermiculite mortar</td>
<td>500</td>
<td>–</td>
<td>1</td>
<td>400</td>
<td>800</td>
<td>4</td>
</tr>
</tbody>
</table>
positioned to get milli-volt reading of the temperature at the concrete surface, the second was positioned to get milli-volts readings at the steel surface (i.e. just beneath the cover). The third and last thermocouple was placed at the center of the column to obtain the temperature at the heart of the concrete. The positions of the thermocouples are shown in Figure 1.

2.3 Testing setup and instrumentation

The well-known temperature-time curve ISO 834 [Meda et al. 2002], representing cellulose fire in a residential building, is compared to the time temperature curve for the furnace used in this investigation in Figure 2. It can be seen from the figure that the temperature-time curve used in this investigation follows the same trend of ISO 834 curve but with slightly lower values of temperature.

An electrical symmetrical furnace was used in this study. The furnace is a part of the facilities of the test laboratory at Housing and Building Research Center. Figure 3 shows the test set-up including a typical column during testing.

The control specimen was tested till failure under static load only without heating or preloading. The other 16 specimens, comprising the four groups, were put inside the furnace and loaded by the hydraulic jack to the preload level depending on their group (see Table 1). After the temperature in the furnace reached 750°C, the load and temperature were kept constant for 60 min. Upon the completion of the exposure duration, the furnace was turned off and the specimens were left to cool to ambient temperature for the samples which were cooled by air. Alternatively, water quenching was applied to the columns for rapid cooling by water. For coated specimens, the specimens were brought out, the coating materials, caps and wire mesh were removed and new caps were made. After the columns were completely cooled, they were statically tested to failure in a single load cycle to determine the residual load capacity.

3 EXPERIMENTAL RESULTS AND DISCUSSION

3.1 Overall behavior, cracking patterns and failure modes

The development of the cracking patterns and modes of failure in the studied columns are shown in Figure 4. It can be seen from the figure that, generally, no explosive spalling was observed for columns containing polypropylene fibers (Groups A, B & C) and those coated with perlite or vermiculite (Group D) after exposure to fire in the furnace for one hour. However, the overall performance of the columns in the fire tests was
not affected by the presence of polypropylene fibers and a more important factor was found to be the axial load as will be explained in the next paragraph. In addition, Figure 4 shows that columns left to cool by air showed better stiffness properties (less chipping off in concrete parts) than those cooled rapidly by water quenching. This is in agreement with Nassif et al. [1999] who reported that rapid cooling by water quenching has a significant effect on the stiffness properties of the structural members compared with those cooled slowly by air. Luo, et. al. [2000] reported also that water cooling resulted in a significant thermal shock and caused a bit more severe deterioration in strength compared to furnace cooling.

For columns containing no fibers and without coating, the behavior of the column under fire conditions was dependent on the level of loading and the cooling regime. For example, no explosive spalling was observed for columns in Groups A and B. However, an explosive spalling was observed for the column in Group C after 45 minutes in the furnace and concrete pieces 30 to 60 cm long and 7–12 cm wide fell after explosion. The inclusion of polypropylene fibers was found to reduce or eliminate spalling, inspite of the fact that the columns in the current investigation had a higher moisture content that is higher than that normally expected in practice. This is in agreement with the findings of Purkiss [2000] and is shown clearly in Figure 4. A close up view of the column after the test (Figure 4) shows that, except for coated specimens (Group D), the concrete cover for most parts of the columns spalled off and the main longitudinal reinforcement and the stirrups were fully exposed. For the column in Group C cooled by water quenching, buckling of the main reinforcement at failure was observed. For coated specimens in Group D, it was observed that major parts of the plaster completely fell off after the column was exposed to air and left to cool. Similar observations were found by Ali et. al. [2001], Meda et al. [2002] and Abdel-Razek et. al. [2004].

3.2 General observation on the load-axial deformation relationships for the studied columns

Figure 5 shows the effect of different studied variables, namely, loading level, cooling regime, presence of polypropylene, and coating type on the residual load-axial deformation relationships. It can be seen from the figure that generally, the capacity of the studied columns were decreased after exposure to 750°C at the furnace, to different degrees. However, the reduction of capacity or residual load was dependant on the studied variables. It is also clear that the axial deformation at failure was affected by the studied parameters.

The general appearance of most of the load-axial deformation curves indicates that the curves become less steep after the samples were exposed to heating. This is in agreement with Wu et al. [2002] who found that increasing the temperature of a an applied single thermal cycle makes the stress-strain curves for HSC flatter. In addition, it is clear that in almost all the cases
Figure 5. Effect of studied variables on the residual load-deformation relationships.
the axial deformation at failure is greater for water cooled samples compared to the air cooled ones in spite of the fact that the reverse was true for the observed residual loads. The data of Issa and Anwar [2004] showed a similar trend for all temperature exposure times with regards to the residual load, but for the axial deformation, their data showed an opposite trend for a fire duration of 60 minutes only. With higher fire durations, the trend in axial deformation was similar to that observed in the current investigation. In the next few sections, the effects of the studied parameters will be discussed in detail.

3.3 Effect of pre-load level for the columns

Table 4 shows the residual capacity of the studied columns without fibers, as a percentage of the original capacity (as measured from testing column No 1). It can be seen that the residual capacity was highest when the columns were preloaded with 40% of its ultimate capacity during heating. This is in agreement with the comments of Malhotra [1956], who suggested that the imposition of a stress during heating (load application) retards the development of cracks in a specimen, which would be free to extend in an unstressed specimen. However, it would appear that this statement would not be accurate for higher pre-loading levels. Indeed, Ali et al. [2001] found that there was a slight, but consistent reduction in failure time with the increase in pre-loading level when the samples were heated.

Specimens in Group C had lower residual loads compared to those in Group (B) as shown in Figure 5 and Table 4. It was noticed that the deterioration was more pronounced for specimens without fibers especially for water quenched ones. For example, the residual loads for the air-cooled and water-cooled specimens without fibers were lower than their twins in Group B by 25.9% and 48.0%, respectively. For specimens containing polypropylene fibers, the residual loads for the air-cooled and water-cooled specimens were lower than their twins in Group B by 9.3% and 10.3% only, respectively. Although Hoff et al. [2000] argued that the addition of polypropylene fibers has virtually no beneficial or adverse effect on the residual strength after exposure to elevated temperature, Chen and Liu [2004] found the presence of fibers in HSC, to some degree, mitigated the deterioration due to heating. The results of the current investigation support the findings of Chen and Liu [2004].

3.4 Effect of cooling regime after temperature exposure

Nassif et. al. [1999] found that the damage index for air-cooled concrete after firing to 320°C is at the same level as that for concrete heated to 220°C and quenched by water. Therefore, it is well established that water-cooling is more damaging to concrete than gradual air-cooling. The results of the current investigation show that the damage by water-cooling was increased with the increase in preload level. The residual loads of the studied columns are summarized in Figure 6. It can be seen that the air-cooled column in Group A without fibers had a residual load higher than that for the water quenched one by 5% only. The difference was increased to 6.7% for Group B samples and further to 34.6% for Group C samples. For specimens containing fibers, the difference was 13.6%, 21.2 and 22% for Groups A, B and C respectively. This indicates that the higher the loading level, the higher the incurred damage after water quenching for samples without fibers.

3.5 Effect of the use of polypropylene fibers in the column mix

It was observed also that the columns contain polypropylene fibers have ultimate loads slightly less than those without fibers. This may be due to the fact that the presence of polypropylene fibers may reduce the compressive strength of concrete, hence the columns with fibers may have had a lower ultimate capacity than the ones without fibers before heating. A similar observation was made by Purkiss [2000] who reported that the fibers significantly lower the cube compressive strength and he argued that the reduction might be attributed to the increase in pore volume caused by the fibers during concrete production.

It is also clear from Figure 6 that presence of fibers can reduce the damage caused by water-cooling when the samples are highly stressed. This is observed in spite of the fact that fibers disappear by melting at an early stage of heating. For example, the Group C column without fibers exhibited 34.6% more damage, when water quenched, compared to the air cooled one. In the same group the sample with fibers exhibited only 22% more damage under the similar conditions. Therefore, the presence of fibers seems to protect highly stressed columns from damage by
3.6 Effect of perlite and vermiculite coatings

It can be seen from Figure 5 and Table 4 that coating the specimens in Group D led to a significant improvement in the behavior of such specimens after exposure to fire in the furnace and cooling in air compared to the specimens in Group B which were loaded to the same loading level (40% of the ultimate load) and left to cool in air also. It is also clear that coating the columns with Perlite mortar provided better insulation to the concrete heart than that provided by vermiculite mortar. For example, the specimen without fibers in Group D, coated with Perlite had a 26.6% higher residual load compared to its twin in Group B, whilst coating the specimen with vermiculite resulted in a residual load higher than its twin in Group B by 3.3% only. In addition, specimens containing fibers showed better behavior than those without fibers. For specimens containing fibers, the specimen coated with perlite had a residual load higher than that for its twin in Group B by 47.8%, whilst coating the specimen with vermiculite gave a residual ultimate load higher than that of its twin in Group B by 28.2%. It is interesting to notice that adding polypropylene fibers to the specimen mix and coating the specimen with perlite mortar resulted in a behavior of the specimen almost similar to that of the control specimen as can be seen from the curves for Group D containing fibers in Figure 5. It would appear that the perlite mortar acted as a shield that prevented extensive cracking and/or fiber melting during heating. Therefore, the columns coated with perlite were only slightly damaged by heating. This is in agreement with the findings of Abdel-Razek et. al. [2004a, b] who concluded perlite coating with thickness of only 2.5 cm maintained 94.44% of the initial load capacity while vermiculite coating maintained only 80% of the initial load capacity.

3.7 Temperature-time relationships of the studied columns

Figure 7 shows temperatures development with time as recorded from the thermocouples located at the column surface, steel reinforcement and the column center (see Figure 1) for different studied columns exposed to fire from all the four sides in the furnace at 750°C for one hour. It can be seen from the figure that, generally, the temperature develops sharply during the first 10 minutes and then the increase of temperature with time becomes slow. For example, the temperature of the concrete surface reached 528°C after 10 minutes and 630°C after one hour in the furnace for specimens in Group A without fibers.

The inclusion of polypropylene fibers in the mix did not reduce the rate of heat transfer to the inner parts of the columns. In fact, with the high pre load (Group C), the temperature in the center of the column with fibers became very close to the temperature on its surface. After one hour in the furnace the temperature in the heart of the concrete was 32% lower than that at the surface for the sample without fibers. For the column with fibers the corresponding value was only 20%. Perhaps, the cracking due to loading and channels opened by fiber melting may have helped in transferring the heat towards the column center.

The effect of plastering using perlite or vermiculite mortars is very clear in insulating the steel and concrete heart from the high temperature on the surface. For example, the temperature values at steel reinforcement under the perlite plaster were less than those at the surface by 80% and 67% after burning in the furnace for 10 minutes and one hour, respectively. The heart temperature after 10 minutes and one hour in the furnace was less than that of the surface by 85% and 76%, respectively. For specimens coated with vermiculite, the temperature at steel reinforcement was 72% and 47% compared to that at the plaster surface after 10 minutes and one hour in the furnace. At the heart of the concrete, the temperature values were lower than those at the plaster surface by 80% and 62% after 10 minutes and one hour in the furnace, respectively.
4 CONCLUSIONS

Based on the experimental program carried out in this investigation, the following conclusions can be drawn:

1. Explosive spalling of HSC columns occurred when the column was subjected to a preload of 0.5 of the ultimate load and no fibers were used in the mix. Other columns did not exhibit explosive spalling.
but still lost parts from the cover during heating in a non-violent manner.
2. Heating of the HSC studied columns resulted in making the load axial deformation curve less steep, leading to the reduction in the ultimate load and an increase in the axial deformation.
3. A certain degree of preloading, up to 40% of the ultimate load, increased the HSC column residual capacity after fire. However, loading above this level caused severe cracking, hence the capacity was sharply reduced. The addition of polypropylene fibers to the columns mix helped in protecting the highly loaded columns from deterioration during heating.
4. Water cooling caused more reduction in the residual load of the column with higher preload levels during heating.
5. Not only did the presence of polypropylene fibers protect the columns from explosive spalling, but also the deterioration by water quenching in highly loaded columns containing fibers was less than those without fibers. This was probably caused by the co-operation of cracks and channels from fiber melting in transferring the cooling water more quickly to the inner parts of the concrete, hence the thermal gradients were relieved.
6. Perlite and vermiculite coatings offer good protection to the concrete columns from the effects of heat. Perlite mortar was more efficient than vermiculite mortar in that respect. However, the inclusion of fibers in the column mix and coating it with perlite greatly improved the residual capacity of the column.

REFERENCES


Malhotra, H. L. 1956. The Effect of Temperature on the Compressive Strength of Concrete, Magazine of Concrete Research, August 1956, pp. 85–94.


Effect of simulated desert climate and sustained moderate temperature on some properties of concrete with and without polypropylene fibers

H.I. El Sayad
Faculty of Engineering, Banha University, Egypt

ABSTRACT: In this investigation, the effect of simulated desert climate on moisture loss (as it is related to drying shrinkage), ultrasonic pulse velocity (internal crack detector), absorption (durability index) and compressive strength (mechanical properties pointer) were studied. Concrete mixes, having w/c ratios of 0.35, 0.45 and 0.55 and polypropylene fiber volumes of 0, 0.1, 0.2 and 0.3%, were prepared and subjected to one of three conditioning regimes after 14 days of moist curing. The samples were either kept in laboratory air, placed in an oven operating two daily temperature cycles between 75°C and room temperature or placed in a continuous heating oven at the same temperature. It was found that heating adversely affected all studied properties of concrete. The moisture loss and absorption were increased by a factor that ranged between 4.23–29.96 or 2.8–18.67, respectively. The compressive strength was reduced by 3.16 to 33.82% compared to the samples kept in laboratory air. The inclusion of fibers improved the performance of concrete mixes compared to the no fiber mixes subject to temperature, to a certain degree. Fiber volumes 0.1–0.2% are recommended in these conditions depending on the type of mix. The presence of polypropylene fibers interferes with the elastic waves traveling through concrete and hence reduces the pulse velocity. Heating of fiber concrete seems to magnify this interference and therefore ultrasonic pulse velocity readings could not be used as internal crack indicators in the current study.
on samples stored in natural environments (Vandewalle [2000] in Belgium and Barr et al., [2003] in Northern Iran). However, these investigations were conducted in moderate climate countries. Okba et al. [2004] examined the surface cracks in fresh concrete slabs, with and without polypropylene fibers, cast at the Touska district. However, in their study no shrinkage or moisture loss measurements were made. Whereas in the out door shrinkage studies of Alsayed [1998] and Fattuhi & Al-Khaiat [1999], which were carried out in the Gulf region, no polypropylene fibers were used in the mixes. Balaguru [1994] reported that at temperatures above 32°C the Young’s modulus of the fibers may be reduced, and therefore fiber deformation especially for thin fibers, may increase. It can be argued that there is a need to investigate the effect of high ambient temperatures on the drying shrinkage cracking tendency of polypropylene fibers concrete. In the current investigation, the moisture loss of samples subject to simulated desert climate was investigated, as an indicator to shrinkage, since Barr & El-Baden [2003] found that the relationship between shrinkage and moisture loss during the first 100 days of drying was linear for all grades of concrete.

One other major problem with concrete in hot climates is its decreased durability [El-Dieb 1994]. Dhir et al. [1994] considered that the absorption of concrete could be used to specify its durability irrespective of curing, grade or mix constituents. Limited data on the absorption and permeability of fiber concrete has been cited in the literature. Bayasi & Zeng [1993], studied w/c = 0.4 mixes having polypropylene fiber volumes 0, 0.1, 0.3 and 0.5%. They found that for $F1/2$ in long fibers, slightly longer than those used in the current investigation, there was a small increase in rapid chloride permeability with the use of fibers. With longer fibers ($F3/4$ in) the increase was significant. Sanjuan et al. [1990] found that the use of 0.1 to 0.2% fibers in mortars reduced the water absorption if the w/c was less than 0.5, but the effect was reversed with higher w/c ratios. They also reported that CO2 diffusion was increased with the increase in fiber volume. Ibrahim [2000] tested the permeability of concrete with various fiber volumes, but all less than 0.1%, using Autoclom. She reported that the air cured samples at 45°C exhibited very high permeability regardless of the fiber volume. However, the permeability was reduced with the increase of fiber volume for the water cured samples. No data has been cited in the literature on the effect of heat cycles on absorption of fiber concrete.

In this investigation, the effect of simulated desert climate on some properties of concrete is studied. The tested properties include moisture loss (as it is related to drying shrinkage), ultrasonic pulse velocity (internal cracking or discontinuity detector), absorption (durability index) and compressive strength (mechanical properties pointer). Concrete mixes having w/c ratios of 0.35, 0.45 and 0.55 and fiber volumes of 0, 0.1, 0.2 and 0.3% were prepared. Samples were water cured for 14 days. After that, three cubes from each mix were kept in laboratory air and then tested for the studied properties. Another three samples were placed in an oven operating two daily temperature cycles between 75°C and room temperature. Whereas the last three samples were placed in a continuous heating oven at the same temperature to examine if the heat cycles are more or less damaging than sustained heating (which concrete sometimes encounters in industrial applications). The samples were conditioned for 16 days before testing for the studied properties.

2 MATERIALS, MIX PROPORTIONS, SAMPLE PREPARATION AND TEST PROGRAM

2.2 Materials

Ordinary Portland cement conforming to ESS 373/1991 was used in preparing the test samples. The fine aggregate was natural sand, whereas the coarse aggregates was crushed dolomite which was supplied in two sizes 5–10 mm and 10–20 mm. Both conformed to ESS 1109/2002. Tap water was used in mixing and curing the test specimens. The fibers used were polypropylene monofilament crimped fibers having a length of 12 mm and a diameter of $18\mu m$ ($L/D = 667$).

2.3 Preparation of the test samples and test program

The dry cement and aggregates were first mixed in a laboratory mixer. Half the water was added and mixing resumed. The mix was left undisturbed for few seconds after that the remaining water and superplasticiser were added. When uniformity of the mix was observed the fibers were evenly sprinkled by hand on to the fresh concrete whilst keeping the mixer running. Finally the slump was measured and nine standard 15 cm cubes were prepared from each mix giving 108 samples in total.

The test program is shown in Figure 1. Cast samples were kept in their moulds for 24 hours under wet burlap. After demoulding, they were water cured for 14 days. At that point the cubes were weighed and
ultrasonic pulse velocity (UPV) measurements in accordance with the recommendations of the Egyptian Code [2003] were taken for each cube (stage 1). Samples from each mix were divided into three groups, each having three cubes to be subjected to the different conditioning regimes. The first group was kept in laboratory air undisturbed, the second group was placed in an oven operating a heat cycle, 6 hours of heating at 75°C followed by 6 hours in which the oven was automatically switched off by a timer.

Figure 1. Test program.

Table 1. Mix proportions for concrete mixes (kg/m$^3$).

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>w/c</td>
<td>0.35</td>
<td>0.45</td>
<td>0.55</td>
<td>0.35</td>
<td>0.45</td>
<td>0.55</td>
<td>0.35</td>
<td>0.45</td>
<td>0.55</td>
<td>0.35</td>
<td>0.45</td>
<td>0.55</td>
</tr>
<tr>
<td>Water</td>
<td>117.5</td>
<td>153.5</td>
<td>189.5</td>
<td>112.5</td>
<td>149.5</td>
<td>186.5</td>
<td>107.5</td>
<td>145.5</td>
<td>183.5</td>
<td>102.5</td>
<td>141.5</td>
<td>180.5</td>
</tr>
<tr>
<td>Fine Aggregates</td>
<td>670</td>
<td>640</td>
<td>609.3</td>
<td>670</td>
<td>640</td>
<td>609.3</td>
<td>670</td>
<td>640</td>
<td>609.3</td>
<td>670</td>
<td>640</td>
<td>609.3</td>
</tr>
<tr>
<td>Coarse Aggregates</td>
<td>1245</td>
<td>1188.5</td>
<td>1130.5</td>
<td>1245</td>
<td>1188.5</td>
<td>1130.5</td>
<td>1245</td>
<td>1188.5</td>
<td>1130.5</td>
<td>1245</td>
<td>1188.5</td>
<td>1130.5</td>
</tr>
<tr>
<td>Admixture</td>
<td>5</td>
<td>4</td>
<td>3</td>
<td>10</td>
<td>8</td>
<td>6</td>
<td>15</td>
<td>12</td>
<td>9</td>
<td>20</td>
<td>16</td>
<td>12</td>
</tr>
<tr>
<td>Fiber</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>1.8</td>
<td>1.8</td>
<td>1.8</td>
<td>2.7</td>
<td>2.7</td>
<td>2.7</td>
</tr>
<tr>
<td>Slump (cm)</td>
<td>10.5</td>
<td>11.5</td>
<td>12</td>
<td>13.5</td>
<td>12.5</td>
<td>13.5</td>
<td>12.3</td>
<td>13</td>
<td>14.3</td>
<td>10.5</td>
<td>11.3</td>
<td>13</td>
</tr>
</tbody>
</table>
and thereafter this cycle was repeated. The third group, however, was placed in an oven which was continuously heated to the same temperature. The different groups remained in these conditions for a further 14 days. At that time the samples from the first group were weighed to evaluate their moisture loss then they joined their counterparts in the constant heating oven for drying for 48 hours. After these conditioning treatments, all cubes were taken out of the ovens and weighed to evaluate their moisture loss, and again the UPV was measured (stage 2). All samples were immersed in water for 48 hours and then weighed to evaluate their absorption and finally the compressive strength test [26] was carried on all samples.

3 RESULTS AND DISCUSSION

3.1 Moisture loss
The moisture loss results are shown in Figures 2 and 3. It can be seen that the moisture loss of samples placed in both ovens was significantly higher than those stored in laboratory air. The moisture loss values of the heated samples were 4.23 - 29.96 greater than those for the samples kept in air. This reflects the drastic dehydrating effect, and hence drying shrinkage tendency, which desert climate may have on concrete subject to this environment. It is clear from the figures that the moisture loss values for samples placed in the cycled heating oven were slightly lower than those from the constant heating oven. As expected, constant heating caused more moisture to be lost from the concrete samples.

In general, the moisture loss increased with the increase in w/c ratio. This agrees with the fact that concrete with low water content shrinks less than that with high moisture content [Babaei 2005]. The moisture loss was also reduced with the increase in fiber volume. However, with the 0.3% fiber volume and w/c 0.45 and 0.55 mixes, the moisture loss was slightly higher than the corresponding mixes with 0.2% fiber volume. This is probably because of the ease of moisture evaporation from the 0.3% mixes as it was reported that in some cases with higher fiber volumes large amounts of air can be incorporated in the matrix [Ramakrishnan 1989, Bayasi & Zeng 1993]. Beddar & Belagraa [2004] argued that the presence of fibers increases the size and volume of pores, therefore the evaporation is increased.

The effectiveness of the fibers in moisture loss reduction depended on the conditioning regime to which the samples were subjected. Moisture loss reduction, for samples with fibers compared to those without fibers and stored in air, was 62.2–85.1%. The corresponding values for heat cycled and constantly heated samples were 16.2–30.7% and 14.2–27.1%, respectively. In addition, it should be noted that for samples

![Figure 2. Moisture loss of different mixes after the conditioning regimes.](image-url)
stored in the laboratory, the effectiveness of the fibers in reducing the moisture loss depended on the w/c of the mix, the fibers were less effective in w/c \( \frac{0.55}{0.55} \) mixes (see Figure 3). For samples subject to heating, better control on moisture loss was exhibited by the inclusion of 0.3% fibers in the w/c \( \frac{0.35}{0.35} \) mix, whereas with w/c \( \frac{0.45}{0.45} \) or 0.55 mixes, 0.2% fibers mixes showed the highest moisture loss reduction. Therefore, it may be argued that from the moisture loss point of view in hot climates, 0.2–0.3% fiber contents are beneficial depending on the w/c ratio of the mix.

3.2 Ultrasonic Pulse Velocity

The percentage loss of UPV, e.g. difference between readings at stages 2 and 1 as a percentage of readings at stage 1 (see Figure 1), is plotted in Figure 4 for the different mixes in the current investigation. It can be seen that the UPV loss for heated mixes was higher than those kept in laboratory air, partially because the UPV of concrete is reduced by drying [Bungey & Millard 1996] and mainly because of the cracking induced by the temperature. There was no general trend in the data to indicate which heat treatment caused more damage to the samples (cyclic or continuous heating). In spite of the fact that the inclusion of fibers reduced the moisture loss (see Figure 3), the UPV loss seemed to increase with the increase in fiber content (see Figure 4).

Pulse velocity measurements were carried out on concrete samples with polypropylene fiber volumes of up to 1.5% by Katwan et al. [2002]. Their results showed that the pulse velocity is reduced as the fiber volume is increased. They argued that ultrasonic waves tend to travel in straight paths, however, the presence of fibers, which can be considered as a low density inhomogeneity in the matrix, results in scattering of the wave and this prolongs the time taken by the wave to cross the medium. Similar observations were made by Leung et al. [2003] when studying the resonant frequency of fiber concrete. The effect of heat on this phenomenon has not been cited in the literature, but the current data, which shows higher UPV loss for heated mixes with high fiber volumes, may point out that the wave scattering may be increased when the fiber concrete had been subject to heating. Therefore, it can be argued that the UPV loss of heated samples shown in Figure 4 was not purely the result of dehydration and internal cracking.

3.3 Absorption

The absorption values of concrete with different fiber volumes, subject to the conditioning regimes, are shown in Figure 5. It can be seen that the conditioning
Figure 4. Effect of type of mix and conditioning regime on ultrasonic pulse velocity.

Figure 5. Effect of fiber inclusion and conditioning regime on absorption.
regime had a profound effect on the absorption results. Heating increased the absorption values by a factor ranging between 2.84–18.76 compared to the samples kept in laboratory air. It is also clear that, the w/c = 0.55 samples exhibited a lower increase in absorption due to heating compared to the w/c = 0.35 and 0.45 mixes. For example, the absorption of the mix with w/c = 0.55 and 0.1% fibers, subject to either cyclic or constant heating, was 7.79 times greater than that for the same mix kept in laboratory air. The corresponding increases for the w/c = 0.35 and 0.45 mixes were 16.61 and 18.76 for cyclic heating or 16.54 and 18.67 for constant heating, respectively. However, higher fiber contents (0.2 and 0.3%) resulted in a lower increase in absorption leading to a maximum increase by a factor of 4.36 due to heating.

It can be seen from Figure 6 that the inclusion of 0.1% fibers in samples kept in laboratory air resulted in a reduction in absorption by a factor of 0.65, 0.41 and 0.84 for the w/c = 0.35, 0.45 and 0.55 mixes. Higher fiber volumes lead to an increase in absorption, i.e. the absorption of the w/c = 0.35 mix with 0.3% fibers was triple that without fibers. This trend was not observed in the heated samples, which almost all exhibited reductions in absorption (factor = 0.71–0.98) relative to the no fibers samples. The single exception for that observation was in the w/c = 0.35 mix with 0.3% fibers which exhibited a small increase in absorption in the order of 1.1 relative to the no fibers sample in the case of constant heating. It is also clear that, for samples subject to cyclic heating, 0.2% fibers seemed to give the lowest absorption values, whereas with the constant heating 0.1–0.2% fiber contents gave low absorption results. It seems that the inclusion of 0.2% fibers in concrete subject to desert environment would offer some improvement to its absorption, relative to the samples without fibers.

3.4 Compressive strength

The compressive strength results are shown in Figure 6. It can be seen that the trend lines for the different conditioning regimes followed the same pattern. Soroushian et al. [1992] reported that the inclusion of 0.1% polypropylene fibers in a w/c = 0.39 mix that was moist cured for 7 days then air dried till the age of 28 days resulted in a reduction of 23% in the compressive strength values compared to the control mix without fibers. In the current investigation, the w/c = 0.35 mix, which was kept in the laboratory, exhibited a small increase in strength (2.15%) due to the inclusion of 0.1% fibers. Mixes with w/c = 0.45 and 0.55 benefited more from the use of 0.1% fibers (11.54 and 23.81% increase in strength, respectively, compared to samples with no fibers).

Samples with no fibers and subject to cyclic heating lost 5.4, 15.4 and 33.8% of their compressive strength values compared to those kept in air, for w/c = 0.35, 0.45 and 0.55, respectively. This confirms the findings of Rasheeduzzafar & Al-Kurdi [1993] who reported that low w/c samples are less damaged by cyclic heating. The inclusion of 0.1% fibers seemed to reduce this effect, particularly for the w/c = 0.35 mix.
fibers increased the compressive strength, but with higher fiber volumes, the strength started a downward trend regardless whether the samples were heated or not. The combined effect of fiber inclusion at a volume of 0.1% and heating resulted in a higher increase in the percentage of compressive strength relative to that for the samples kept in laboratory air. For example, w/c = 0.35 and 0.55 samples with 0.1% fibers exhibited an increase of 24.3 and 40.0% in strength with the constant heating regime, but the increase was only 2.2 and 14.7% when the samples were kept in air, respectively. However, in all cases, the heated samples exhibited a lower strength compared to the samples left in air. It is also clear that the compressive strength of w/c = 0.35 samples suffered the most from the combined effect of the inclusion of high fiber volumes (0.3%) and heating. For example, cyclic heating of these samples reduced their strength by 37.5% compared to the no fiber samples. However, the w/c = 0.55 samples, with the same fiber volume and subject to the same heating regime, exhibited a slight increase in strength of 2.2% compared to the sample without fibers. The samples kept in laboratory air did not show such a difference in compressive strength change, as the strength of w/c = 0.35, 0.45 and 0.55 with 0.3% fibers was reduced by 22.6, 20.5 and 23.5%, respectively. Therefore, a small fiber volume of 0.1% is beneficial from the strength point of view in all cases, but higher fiber volumes can be used with high w/c mixes only, if required for the enhancement of other properties. High fiber volumes are not recommended for low w/c mixes in desert climates as the strength can be impaired.

4 CONCLUSIONS

Based on the tests carried out in this investigation, the following conclusions can be drawn:

1) Heating (either cyclic or constant to 75°C) caused severe damage to the investigated properties of concrete, but no definite conclusion can be drawn regarding whether cyclic or constant heating is more damaging to concrete.

2) The moisture loss values due to heating were increased by a factor that ranged between 4.23 and 29.96 relative to the values for samples kept in air.

3) The inclusion of fibers reduced the moisture loss. However, the effectiveness of the fibers in moisture loss reduction depended on the conditioning regime to which the samples were subjected. Moisture loss reduction, for samples with fibers compared to those without fibers and stored in air, was 62.2–85.1%. The corresponding values for heat cycled and constantly heated samples were 16.2–30.7% and 14.2–27.1%, respectively.

4) The presence of polypropylene fibers interferes with the elastic waves traveling through concrete and hence reduces the pulse velocity. Heating of fiber concrete seems to magnify this interference and therefore ultrasonic pulse velocity readings could not be used as internal crack indicators in the current study.

5) Heating increased the absorption values by a factor that ranged between 2.84 – 18.76 compared to the samples kept in laboratory air. It seems that the inclusion of 0.2% fibers in concrete subject to desert environment (cyclic heating) would offer some improvement to its absorption (factor = 0.71–0.98), relative to the samples without fibers.

6) Samples with no fibers and subject to cyclic heating lost 5.4, 15.4 and 33.8% of their compressive strength values compared to those kept in air, for w/c = 0.35, 0.45 and 0.55, respectively. The inclusion of 0.1% fibers increased the compressive strength, but with higher fiber volumes, the strength started a downward trend regardless whether the samples were heated or not. In all cases, the heated samples exhibited a lower strength than the unheated ones.

7) A small fiber volume of 0.1% is beneficial from the strength point of view in all cases, but higher fiber volumes can be used in high w/c mixes only, if required for the enhancement of other properties. High fiber volumes are not recommended for low w/c mixes in desert climates as the strength can be impaired.

REFERENCES


ESS 1109/2002, Specification for Concrete Aggregates from Natural Sources.


Rheological and engineering properties of SCLC

A.A. Maghsoudi
Assistant prof., Civil Eng. Dept., Shaheed Bahonar University of Kerman, Kerman, Iran

S. Mohamad pour
M.Sc. Student, Shaheed Bahonar University of Kerman, Kerman, Iran

ABSTRACT: Lightweight concretes have been successfully applied in the building constructions for decades because of their favourable material properties, especially their low specific weight in connection with a high strength, a high capacity of thermal insulation and a high durability. The development leading to a self compacting light weight concrete (SCLC) represents an important innovative step in the recent years. This concrete combines the favourable properties of a lightweight concrete with those of a self compacting concrete. Research work is aimed on development of (SCLC) with the use of light aggregates “Light expand clay aggregate (Leca)”. In this investigation, first by trial and error, different mix design of SCLC were casted and tested to find out the values of slump flow, L-box, V-funnel and 28 day compressive strength. Based on the results obtained, the best so-called standard mix design was selected for further investigation. For two selected mix, engineering properties of SCLC, such as compressive and flexural strength, E-modulus, shrinkage and swelling (expansion) values for three different curing conditions were measured at short and long ages (upto-90 day). The results are shown that use of SCLC can improve the engineering properties as well as the durability of structural concrete made of light weight materials such as Leca.

1 INTRODUCTION

Research work is aimed on development of Self Compacting Light Concrete (SCLC) with the use of light weight aggregates “Leca”. The SCLC is new building material which due to properties, the use of SCLC is very convenient for many cases such as, at rehabilitation of old building constructions, where the use of ordinary SCC would lead to overloading and to necessity of additional strengthening of existing structures. Other very favorable use is for production of precast components with very complicated shapes.

Decisive problems to solve at preparation mix design of this type of concrete are water absorption of Leca aggregates. Furthermore it is evaluation of applicability of conventional mix designing of SCLC. Leca aggregates if well produced are suitable for use in SCLC by reason of spherical shape improving rheological properties of fresh concrete mix. Water absorption of aggregates which has strong influence on rheology, in this research has been compensated by aggregate is ensured by thorough mixing of aggregate in water. Disadvantage of Leca aggregates is its low compressive strength, which resulted in reduced compressive strength of concrete. SCLC combines the already know advantages of lightweight dense concrete and self compacting concrete [Hubertova 2005]. In structural applications, the self weight of the concrete structure is important since it represents a large portion of the total load. Hence by use of the lightweight aggregates it is possible to reduced member size of the structures and foundation force [Caijun & Xiaohong 2005]. Therefore, by reducing the self-weight of the structures, considerable savings could be attained, not only in materials but also in construction costs.

Experimental research is required to understand the mechanical properties of SCC including the light weight aggregate, Leca. The objective of this research study is to i) design and construct so called standard mix design of SCLC and ii) provide information on mechanical properties of SCLC for short and long term ages.

2 MATERIALS USED

To prepare the mix design, Type II Portland cement was used and its physical properties and chemical composition are given in Table 1. The aggregate with nominal maximum particle size of 9.5 mm and well graded sand for SCC were employed. The particle size
distributions and physical properties of both Leca aggregate and sand were well within ASTM C-33 and ASTM C-127 limits respectively, as shown in Table 2. A poly-carboxylic-ether (PCE) super plasticizer was incorporated in all mixture. It was liquid with a specific gravity of 1.13 and solid content of 40.2%. Also filler (lime stone powder) with a nominal mean particle size of 0.3 mm was used.

3 REOLOGY OF FRESH SCLC

The term self-compacting lightweight concrete describes a highly flowable lightweight concrete which de-airs without the supply of compacting energy and which simultaneously features a high resistance to sedimentation and to the segregation regarding the buoyancy of the lightweight aggregate, respectively. To ensure these properties, the classic methods of concrete technology only partly achieve their aim. It is however possible to ensure the desired flowability of the concrete by adding super plasticizers or by increasing the paste content, but this entails also a growing tendency of the concretes to segregate. The key to a successful development and manufacturing of SCLC lies above all in a careful regulation of the rheological properties of the mortar matrix and the powder paste matrix of the concrete.

The rheological behavior of fresh building material suspensions, as there is fines paste or mortar, is a result of the interaction between the properties of an elastic solid and a viscous fluid. The elastic and viscous properties can be separately recorded by means of rheological measuring methods.

The interactions of the elastic and viscous properties of a material are, among others, represented in the so-called flow curve (Figure 1). It describes the relation between the applied shear stress ($\tau$) and the resulting shear rate ($\gamma$). Whereas Newtonian fluids as water or silicon oil show a purely viscous flowing – the flow curve runs through the origin and its gradient is constant with building material suspensions at first an elastic deformation of the sample can be ascertained. Only when a critical shear stress, the so-called yield stress ($\tau_0$) is exceeded, the deforming behavior is dominated by the viscous properties of the material. This is reflected in a proportionality between the applied shear stress ($\tau$) and the shear rate ($\gamma$).

Materials which have a yield stress and which show a linear flow curve are called Bingham solids. The tangent gradient of the regression line describes the plastic viscosity ($\mu$) of a mixture. Figure 1 demonstrates

![Figure 1. Flow curve [Muller & Haist 2004].](image-url)
as an example the flow curves of a Newtonian fluid, an ideal Bingham solid as well as a real cement paste. Different from the idealization of the Bingham model, at very low shear gradients, real fines pastes show a strong increase of the shear stress with a rising shear gradient. This behavior is especially important for segregation stability of a mixture.

A further special feature of the building material suspensions is that their rheological properties distinctly depend on the shear history and the age. This means that their rheological properties change in the course of time as well as in consequence of the flowing process. Especially the ability of the building material suspensions to rebuild a stabilizing structure during the state of rest which follows an intensive shearing, has a positive influence on the processing abilities as well as on the stability and on the homogeneity of the respectively prepared concrete mixtures during and after the casting. Decisive for all mentioned rheological properties is, among others, the water content of the mixture.

The results of the rheological investigations of fines pastes and mortars with lightweight fine sand and lightweight sand show that the yield stress as well as the plastic viscosity of the examined suspensions decrease considerably when the water contents rises. Furthermore, both characteristics are influenced by the material composition of the mixtures and by the properties of the single solid raw materials (particle size distribution, shape of the particles, etc.). In order to ensure a high flowing ability as well as a good de-airing of the concrete, a low yield stress and viscosity are necessary. At the same time, both characteristics have to be chosen high enough to prevent the lightweight aggregate from buoying upwards or blocking, respectively. These requirements, contradictory in principle, have to be finely adjusted within the framework of an optimizing process [Muller & Haist 2004]. A part of this investigation, was the study of properties of fresh concrete for SCLC (see item 5).

4 SCC MIXTURE PROPORTIONS

At present three different concepts for the production of SCC are distinguished. In contrast to normal concrete (NC) for the production of SCC the powder content is increased (Powder Type), a viscosity agent (Viscosity Agent Type) or both possibilities are combined (Combination Type) [Dehn 2000].

Whereas, here Powder Type was chosen to produce SCLC too. In this investigation, first by trial and error procedure, different mix design were caste and tested to find out the fresh concrete properties of SCLC such as value of the slump flow, J-ring, V-funnel, L-box and hardened concrete properties of SCLC such as the average value of tree cube specimens at 28 day compressive strength (the full report of trial tests are given in [Mohamad pour 2006]. Based on these results, the following two mixes called as SL1, SL2, (see Table 3) was selected for further investigation of properties of fresh and short and long term age of hardened SCLC. In this Table, S and L are defined as self compacting concrete and light weight aggregate, Leca, respectively and 1 and 2 are defined as mix number 1 and 2 respectively.

The concrete mixtures had water-cementitious material ratios (w/cm) of 0.38 and 0.35. The 10% silica fumes by mass of cementitious materials as cement replacement was used. The volume content of the coarse aggregates (Leca) and powder materials (cement, silica fume and lime stone powder) for mixes SL1 and SL2 was kept constant at 175 and 550 (kg/m³) respectively. Leca, sand, lime stone powder, cement, and silica fume were mixed first for 1 min, then PCE that was mixed in water was added last for 1 to 2 min. Then all the materials were mixed for 2 to 4 min.

5 PROPERTIES OF FRESH SCLC

There is as yet no universally accepted standard for characterizing of SCLC. Nevertheless, a few testing methods seem to reappear several times in literature and tend to become internationally recognized as suitable methods to characterize the self normal compacting concrete [Poppe 2001]. Hence, almost same procedure was employed to produce SCLC too. Immediately after the mixing, the value of slump flow, J-ring, L-box and V-funnel test were determine by the following methods.

5.1 Slump flow test

The slump flow test was used to evaluate the free deformability and flowability of SCLC in the absence of obstruction. A standard slump flow cone was used for the test and the concrete was poured in the cone without compaction and leveled. Slump flow value represented the mean of two perpendicular diameters of concrete after lifting the cone [Sakata et al. 1996].

<table>
<thead>
<tr>
<th>Mix no.</th>
<th>SL1</th>
<th>SL2</th>
</tr>
</thead>
<tbody>
<tr>
<td>w/cm</td>
<td>0.38</td>
<td>0.35</td>
</tr>
<tr>
<td>Water kg/m³</td>
<td>256.40</td>
<td>240.33</td>
</tr>
<tr>
<td>Cement kg/m³</td>
<td>360</td>
<td>450</td>
</tr>
<tr>
<td>Silica fume kg/m³</td>
<td>40</td>
<td>50</td>
</tr>
<tr>
<td>Lime stone powder kg/m³</td>
<td>150</td>
<td>50</td>
</tr>
<tr>
<td>PCE L/m³</td>
<td>4.950</td>
<td>4.675</td>
</tr>
<tr>
<td>Leca kg/m³</td>
<td>175</td>
<td>175</td>
</tr>
<tr>
<td>Sand kg/m³</td>
<td>1133.80</td>
<td>1153.40</td>
</tr>
</tbody>
</table>
A slump value ranging from 500 to 700 mm for a concrete to be self compacted in normal SCC [Nagataki & Fujiwara 1995]. By this test in addition to assessing the deformability of the concrete, it is possible to observe segregation of aggregates near the edges of the spread out concrete. The slump flow test for SCLC is shown in Figure 2.

### 5.2 J-ring test

The J-ring test is used to determine the passing ability of the SCLC (Fig. 3). The equipment consists of a rectangular section (30 mm × 25 mm) open steel ring, drilled vertically with holes to accept threaded sections of reinforcement bar. These sections of bar can be of different diameter spaced at different intervals; in accordance with normal reinforcement considerations, 3 (the maximum aggregate size) might be appropriate. The diameter of the ring of vertical bars is 300 mm, and the height 100 mm.

After the test, the difference in height between the concrete inside and that just outside the J-ring is measured. This is an indication of passing ability, or the degree to which the passage of concrete through the bars is restricted [EFNARC 2002].

### 5.3 V-funnel flow time test

The V-funnel test consists of V-shaped container as shown in Figure 4. The deformability through restricted area can be evaluated using V-funnel test [Ozawa et al. 1994]. In this test, the funnel shown in Figure 4, was filled completely with SCLC and the bottom outlet was opened, allowing the concrete to flow out. The time of flow from the opening of outlet to the seizure of flow was recorded. Acceptable value for SCC range between 4 to 10 s [Chai 1998].

### 5.4 L-box test

The test assesses the effect of reinforcement on free flow of concrete constrained by formwork. The L-box test for SCLC shown in Figure 5. With the L-box apparatus, it is possible to measure different properties such as flowability, blocking and segregation of the concrete [Sonebi & Bartos 1999]. Concrete is allowed to flow from the vertical column section into the horizontal trough. The basic test result is the “blocking ratio” h₂/h₁, it is the ratio between the height of the concrete surface in the vertical column part of the apparatus (h₁) and the height of the concrete surface in the trough at its far end (h₂), after the passage through vertical reinforcing bars. There are two additional marks on the horizontal trough at 200 mm and 40 mm from the sliding door. In addition to the basic result, times T₂₀ and T₄₀ (in seconds), which it takes for the concrete to reach the marks, are sometimes...
measured [Bartos 2005]. The ratio between these two heights (h2/h1), which is usually 0.7–0.9, was used to evaluate the ability of the SCC mixture to flow around obstruction [Nehdi & Ladanchuk 2003]. This limit, however, has been proposed to be within 0.8 and 1.0 by EFNARC guidelines [EFNARC 2002].

Many versions of L-box equipment have been used. The version selected for this study has inside dimensions of; column: 200 mm x 100 mm, 600 mm tall; trough: 200 mm x 150 mm, 700 mm long and uses a set of three vertical reinforcing bars. For this study, a gap of 55 mm between the 12 mm diameter bars was selected where the top aggregate size was 20 mm. This test requires 12.7 liters of concrete. Blocking caused both by oversize coarse aggregate or its excessive content can be detected, as well as blocking generated by moderate severe segregation. The mix can be regarded as possessing a segregation resistance, if the particles of coarse aggregate are seen to be distributed on the concrete surface all the way to the end of the horizontal part. Blocking usually manifests itself as coarse aggregates wedged between the reinforcement bars [Bartos 2005].

The results of properties of fresh self compacting light concrete used in this investigation are well between the mentioned values and presented in Table 4.

### Table 4. Results of properties of fresh SCLC

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Slump flow diameter cm</th>
<th>J-ring cm</th>
<th>V-funnels h2/h1</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL1</td>
<td>72</td>
<td>1.2</td>
<td>5.0</td>
</tr>
<tr>
<td>SL2</td>
<td>67</td>
<td>1.5</td>
<td>5.5</td>
</tr>
</tbody>
</table>

6.1 Casting and curing of test specimens

After casting, the molded specimens were covered with two layers of plastic and left on the casting room at 20°C for 48 h. They were then demolded and cured in three different storage conditions. For the first 7 days, all specimens are cured in water which is saturated with lime. Then some specimens are cured in air, (D) condition, with a surrounding temperature of 20°C ± 3 and relative humidity of 30 ± 5%, some samples even after 7 days are also kept at same condition up to 28 days age and then (after 28 days age) they are cured in air, (W) condition and the remaining after 7 days are cured in 5% sulphate sodium solution, (S) condition. The specimens (10 x 10 x 10) cm density of light weight concrete after they were demolded was 1800–1900 kg/m³. Which is about 600 kg/m³ less than the normal SCC (it is noted that, different attempts was made to produce the SCLC with a density lower than 1900 kg/m³, but their 28 day compressive strength were low. Also, for the same mix but only receiving Leca at different time (date) from the supplier, it was found that the compressive strength was varied considerably. It seems such a founding is due to not well product of Leca in the factory which need to be reconsidered by the manufacturer).

6.2 Compressive strength tests and results

For two cases of studied at three storage condition, the total number of 48 concrete cube specimens of (10 x 10 x 10)cm were casted and tested at 3, 7, 28 and 90 days age. The results for average value of three specimens at each storage condition, and each age are shown in Figures 6 to 8. Meanwhile, the slope of the
lines (m), which are presenting the growing up rate of compressive strength between ages are founded and given in Table 5. As indicated in figures approximately the amount of compressive strength at 28 and 90 days for specimens SL1 and SL2 in (W) is more than (D) and (S) storage conditions. Also, the compressive strength for SL1 and SL2 samples is approximately similar for both (D) and (S) storage conditions.

6.3 Stress-strain curve of SCLC

To observe the stress-strain behavior of SCLC specimens, for some cub samples, the electrical strain gages were fixed and during the test (Fig. 9), the data from the load cell and electrical strain gage were recorded by the data logger for any load increment and the typical stress-strain diagrams for SL1 and SL2 samples are plotted and shown in Figures 10 and 11.

The obtained ultimate strain and slope of the curves at $f_{cu}/3$ (i.e., $E_c$) are 0.00241, 0.00222 (which is less than the minimum value suggested in building codes for traditional concrete, i.e., $\varepsilon_{cu} = 0.003$) and 13500, 16400 MPa for SL1 and SL2 respectively. The amount of $E_c$ obtained by this method is closed to those obtained by the Universal Testing Machine.

Table 5. Growing up rate of compressive strength.

<table>
<thead>
<tr>
<th>Mix no.</th>
<th>SL1</th>
<th>SL2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$m_{1-7}$</td>
<td>W,D,S</td>
<td>0.400</td>
</tr>
<tr>
<td>$m_{7-28}$</td>
<td>W</td>
<td>0.433</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>0.376</td>
</tr>
<tr>
<td></td>
<td>S</td>
<td>0.404</td>
</tr>
<tr>
<td>$m_{28-90}$</td>
<td>W</td>
<td>0.029</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>0.030</td>
</tr>
<tr>
<td></td>
<td>S</td>
<td>0.009</td>
</tr>
</tbody>
</table>
The SL1 and SL2 were in 90 days age and (D) storage conditions.

6.4 Flexural specimens tests and results

For two cases of studied at two (W) and (D) storage conditions the total number of 16 prism specimens of (10 × 10 × 45 cm) were casted and tested at 28 and 90 days age and the average values of the test results are shown in Figures 12 to 15.

For two mixes, By comparison of the strength results, it can be concluded that for both conditions, a better behavior for (W) storage condition at 28 and 90 days are obtained. The flexural strength of SL2 specimen is higher than SL1 which indicates that similar to normal concrete, by increasing the compressive strength, the flexural strength of SCLC will be increased.

6.5 Modulus of elasticity tests and results

This type of tests were carried with a Universal Testing Machine which was able to draw the load deflection curves and the full report of the author's proposed method of finding the modulus of elasticity appeared in [Mohamad pour 2006]. However, here only the results are shown in Table 6 and Figures 16 to 17. Also the growing up rate of modulus of elasticity is shown in Table 7. As shown in figures, for all storage conditions at different ages, the modulus of elasticity of SL2 specimens is higher than SL1. Also modulus of elasticity SL1 and SL2 at (W) storage condition is more than those for (D) condition at 90 days age.

6.6 Shrinkage tests and results

When exposed to dry environment after an initial moist curing, the shrinkage of concrete my be divided into two components: drying shrinkage and autogeneous shrinkage.
According to ACI 116R, the drying shrinkage is defined as “shrinkage resulting from loss of moisture,” whereas the autogenous shrinkage is defined as “change in volume produced by the continued hydration of cement, exclusive of the effects of applied load and change in either thermal condition or moisture content. The autogenous shrinkage is a consequence of the withdrawal of water from the capillary pores by the hydration of cement a process known as self desiccation.

Typical values of autogenous shrinkage of ordinary concrete are approximately \(40 \times 10^{-6}\) at the age of 1 month and \(100 \times 10^{-6}\) after 5 years, which are relatively low compared with those of drying shrinkage. Because of this, autogenous shrinkage has been ignored for practical purpose for ordinary concrete. For concrete with a low w/c, however, particularly when it contains silica fume, autogenous shrinkage may be important. Because of this, the shrinkage of concrete exposed to a dry environment is a combination of the drying shrinkage and autogeneous shrinkage.

According to ACI material journal [Zang et al. 2005], the incorporation of 5% silica fume reduced the shrinkage of concrete significantly. Because the pozzolanic reaction of silica fume refines the pore structure and densifies the cement paste, the loss of water from the cement paste and that absorbed inside the light weight aggregate would probably reduced. This water absorbed inside the aggregate provided a means for continued cement hydration and pozzolanic reaction, thus further densifying the cement paste. This might have contributed to the lower shrinkage of the lightweight concrete with silica fume.

It is well known that, in normal concrete (NC), different factors are effecting drying shrinkage such as relative humidity, type of aggregate used, w/c, modulus of elasticity of aggregate used, the amount of aggregate, ... However, experimental research is required while considering SCLC.
For each mix of Table 3, 4 prism specimens and the total number of 8 prism specimens of (10 × 10 × 45 cm) were casted and at the age of one day, the Demec points were fixed on two opposite surface of the specimen and then they are kept either at (D) or (W) storage condition. At different age the amount of shrinkage were measured by the mechanical strain gauge (with the gauge length of 10 cm) and the results of average of two opposite surface reading of three specimens were plotted on Figures 18 to 21. As expected the amount of shrinkage for the samples kept at (D) condition is more than the value for (W) condition. It is because; the curing time for (W) condition was 3 weeks more than (D) condition. The amount of shrinkage for the (SL1) samples which contains w/cm = 0.38, is more than the (SL2) samples having w/cm = 0.35. The results are indicating that, similar to normal concrete, the amount of shrinkage has a direct relationship with the w/cm for SCLC too. Growing up rate of shrinkage at early ages is more than the last age in SL1 and SL2 at both storage conditions.

6.7 Swelling and results
The specimens (10 × 10 × 45) were casted and at the age of one day, the Demec points were fixed as with shrinkage specimens and then they were kept in water
which is saturated with lime. The amounts of swelling at different ages were measured by mechanical strain gauge and the results are shown in Figure 22. The results are indicating that, amount of swelling for SL1 is more than SL2 specimens. It seems that similar to shrinkage, the swelling value for SCLC will be increased as the ratio of w/cm is increased and decrease as compressive strength is decreased.

7 CONCLUSIONS

Based on experimental research on SCLC, the following conclusions are obtained:

1. It was possible to produce an internationally suitable self compacting light concrete (SCLC) mixes in Iran.
2. Different storage condition will affect the compressive strength of (SCLC) and the highest strength of specimens for long time ages is reached under the (W) storage condition.
3. By use of well grained aggregates, Leca and 400 and 500 kg/m³ of cement containment, it was possible to produce a self compacting light concrete mix with compressive strength of 20.8 and 28.5 MPa at 28 days respectively. This strength is allowed in structural reinforced concrete codes.
4. The increase in flexural strength of (SCLC) specimens is depending upon their 28 days compressive strength value.
5. For longer initial time of storage, the amount of shrinkage for samples kept at (W) condition is lower than (D) condition.
6. By increasing amount of compressive strength, the swelling value will be decreased.
7. The growing up rate of the shrinkage results are much regular than the swelling results. However, for better judgment it is suggested that for different type of filler the long time tests to be carried out.

REFERENCES

1 INTRODUCTION

Concrete is a major building material and its durability needs to be maintained to ensure a proper service life. One aspect of durability under consideration is the material’s volume change, or shrinkage. Concrete experiences volume changes throughout its service life. Shrinkage strain develops in the concrete as moisture is lost to the environment or by self-desiccation. Excess shrinkage can lead to cracking followed by greater infiltration by harmful materials and a shorter lifetime.

The recent development of high performance concretes (HPC) like self compacting concrete (SCC) and silica fume concrete (SFC) has led to increased studies aimed at mitigating the effects of shrinkage. SCC recipes are often associated with high contents of binder, fillers and plasticizing admixtures, which all may contribute to increased drying shrinkage.

SCC, a new kind of HPC, also known as super workable concrete is characterized by high resistance to segregation, and is considered as a concrete which can be placed and compacted under its own weight without any vibration and fills all parts of the form work even when access is hindered by dense reinforcement (Collepardi 2003). The development of SCC was first reported in 1989 by Ozawa et al. (1989).

In order to simulate numerically the effect of drying on the deformation and stresses developed in concrete, free shrinkage strain needs to be quantified. The unrestrained shrinkage strain is expressed as the product of the change in moisture content (moisture loss) and coefficient of moisture contraction, $\alpha_{hygro}$.

2 EXPERIMENTAL PROGRAM

2.1 Mix proportions of concrete

In this research normal concrete (NC) with varying water cement ratios, self compacting concrete (SCC), enhanced self consolidating concrete (ESCC) and silica fume concrete were used. Mix proportions of these concretes are shown in Table 1. Concrete mix designs for normal concrete was carried out following the ACI 211.1–91 specifications. Since SCC and ESCC are new concretes, several trial mixes were carried out by varying the quantities of cement, admixture, superplasticizers and aggregates to obtain a workable mix design. The mix design for silica fume concrete is taken as that of Royal Commission, Saudi Arabia located in the industrial city of Jubail, Saudi Arabia.

2.2 Geometry and preparation of specimens

A standard ASTM prism specimen of size $75 \times 75 \times 300$ mm was used to determine both moisture loss and shrinkage for all types of concretes. All the specimens were cured in molds for 24 hours sealed in plastic wraps. They were then demoulded and cured under wet burlap for 7 days. The samples
were stored in a controlled environment with a RH of 40 ± 2%. For moisture loss measurement, specimen weights were taken at regular intervals. In the starting phase readings were taken number of times a day, which was later on reduced to few times a week. The interval of taking readings was governed by the change in weights of the specimens or the amount of moisture being lost, in order to have sufficient data to generate the moisture loss curves properly. Weights were taken using electronic digital weighing balance with an accuracy of 0.01 grams and the same balance was used for all specimens. Afterwards they were dried in an oven at 105°C to determine the evaporable moisture content in each case (Sakata et al. 1980) and hence generate the curves for moisture loss percentage. To determine the free shrinkage strain, embedded TML strain gauges of type PMFL-50-2LT were placed in the specimens at the time of casting. The specimens after curing were immediately connected to digital data logger to record the shrinkage strains. All the tests were done for a period of two months at room temperature (25°C ± 5°C). The shrinkage and moisture loss for prism specimens of size 25 × 25 × 300 mm was also determined to study the effect of specimen size on shrinkage and moisture loss relationship.

3 RELATIONSHIP BETWEEN SHRINKAGE AND MOISTURE LOSS

The moisture loss values are shown plotted against the free shrinkage strain for all concretes in Figure 1. The relationship between moisture loss and shrinkage was found to be almost linear. Sakata (1983) has also reported similar behavior for moisture loss and shrinkage tests conducted on normal concrete using specimens of size 100 × 100 × 400 mm and presented almost a linear relationship between moisture loss and shrinkage, commenting that it is possible for shrinkage strain to be predicted by moisture loss so far as strain at the surface is concerned. Penev and Kawamura (1991) conducted moisture loss and shrinkage tests on soil-cement mixtures and lean concretes and reported that the relationship between moisture loss and shrinkage is linear. Rahman (1999) also conducted moisture loss modified cementitious repair mortar and fluid micro concrete) by using specimens of size 25 × 25 × 285 mm and 40 × 40 × 300 mm. He also reported that the relationship between moisture loss and shrinkage is linear. Thus shrinkage can be expressed as a function of moisture loss in the form

\[ \varepsilon_{sh} = \alpha_{hym} \ast \Delta M \]  

(1)

where

\[ \varepsilon_{sh} = \text{free shrinkage}, \]
\[ \alpha_{hym} = \text{coefficient of moisture contraction}, \]
\[ \Delta M = \text{moisture loss percentage}. \]

The concept of \( \alpha_{hym} \) introduced in this research leads to the definition of a new material constant similar to coefficient of thermal expansion. Use of Equation (1) can also be valuable in estimating limiting values of shrinkage by simply conducting a moisture...
loss experiment on a standard ASTM specimen 75 × 75 × 300 mm at the exposure condition of interest.

The values of $\alpha_{\text{hygro}}$ obtained from regression for the concretes under consideration are as follows:

- For NC1 $\alpha_{\text{hygro}} = 10.5 \times 10^{-6}$ mm/mm
- For NC2 $\alpha_{\text{hygro}} = 11.1 \times 10^{-6}$ mm/mm
- For NC3 $\alpha_{\text{hygro}} = 12.0 \times 10^{-6}$ mm/mm
- For SCC $\alpha_{\text{hygro}} = 11.4 \times 10^{-6}$ mm/mm
- For SFC $\alpha_{\text{hygro}} = 9.2 \times 10^{-6}$ mm/mm
- For ESCC $\alpha_{\text{hygro}} = 15.0 \times 10^{-6}$ mm/mm

The cement content for the normal concretes (NC1, NC2 and NC3) was taken as 420 kg/m³. It is known that richer mixes would yield higher shrinkage (Sakata 1983). Thus $\alpha_{\text{hygro}}$ would have to be determined for such concretes and for other mixes using different coarse to fine aggregate ratios.

The next section shows that the coefficient of moisture contraction $\alpha_{\text{hygro}}$ is independent of specimen size provided that both moisture loss and shrinkage measurements are made on the same specimen.

### 4 EFFECT OF SPECIMEN SIZE ON COEFFICIENT OF MOISTURE CONTRACTION $\alpha_{\text{hygro}}$

To study the effect of specimen size on $\alpha_{\text{hygro}}$, moisture loss and shrinkage plot are shown superposed for two specimen sizes 25 × 25 × 300 mm and 75 × 75 × 300 mm in Figures 2–4 for the three new generation concretes. Figures 2–4 show that the relationship between moisture loss and shrinkage is almost similar with small discrepancy. This confirms the postulated invariant form as given by Equation (1) and its independence of specimen size.

### 5 CONCLUSIONS

A new concept of $\alpha_{\text{hygro}}$, referred to as coefficient of moisture contraction, is introduced. It is similar to
that of coefficient of thermal expansion $\alpha_{\text{therm}}$. The introduction of $\alpha_{\text{hygro}}$ makes stress computations due to moisture movement rather convenient, much in the same manner as thermal stress computations. Loss of moisture $\Delta M$ can be obtained from the associated boundary value problem governed by Fick’s law (analogous to change of temperature $\Delta T$ from the associated heat transfer problem), followed by computations of free shrinkage strain using the suggested form as given by Equation (1) and the associated stress buildup due to internal or external restraint can then be found by using a standard finite element package. Typical problems would include drying shrinkage stresses in patch repair and column jacketing.

Values of $\alpha_{\text{hygro}}$ have been presented for new generation concretes and normal concretes in an effort to build up a database similar to the base that exists for $\alpha_{\text{therm}}$ for various materials. This approach over a period of time would obviate the need to measure shrinkage strain in the laboratory as is standard practice at present.

**ACKNOWLEDGMENTS**

The support of Civil Engineering Department at King Fahd University of Petroleum & Minerals (KFUPM) is deeply acknowledged. This work was completed under a grant from Research Committee of KFUPM under Project CE/Mix Design/267.

**REFERENCES**


1 BACKGROUND AND OBJECTIVE

1.1 General durability aspects of SCC

The mechanism of Thaumasite Sulphate Attack (TSA) was examined based on international research. Other durability issues than TSA are mentioned briefly based on an international overview so that TSA will not be dealt with separately. The primary durability properties are chloride ingress, fire resistance, internal freezing and thawing resistance, salt freezing and thawing resistance and sulphate resistance. All the mentioned properties were recently studied [Persson 2001]. Salt freezing and thawing scaling and internal freezing and thawing resistance did not alter much from the corresponding properties of normal concrete (NC) [Persson 2003a]. In the Great Belt tunnel and also in the Channel tunnel large scale spalling of the concrete occurred during real fire owing the low w/c, in High-Performance Concrete (HPC), that was used in these projects. After the real fire at the Great Belt tunnel only about 10% of the tunnel wall remained due to the extensive and long-term explosive spalling. At the real fire at the Channel tunnel explosive spalling to a depth of 260 mm took place at a 500-m long part of the 500-mm tunnel wall. The HPC was too dense for moisture to evaporate fast enough during the fire. Filler in SCC also increased the risk of fire spalling as compared to NC due to the low water-powder ration of SCC that were corresponding to the low w/c of HPC. Introduction of polypropylene fibres was a way of securing the fire resistance and thus required in SCC in order to obtain resistance to fire spalling [Persson 2004a]. The chloride ingress in SCC with limestone filler was larger than that in NC [Persson 2004b]. The larger chloride ingress in SCC than in NC was related to lower cement content in SCC than in NC (the cement was partly replaced by limestone filler) [Fagerlund 2005, Boubitsas 2005].

1.2 Effect of mineral additives

SCC contains large amounts of fine particles such as limestone filler, fly ash, silica fume, and glass or quartzite filler in order to avoid gravity segregation of larger particles in the fresh mix. Normally the sum of cement and all filler varies between 450 and 650 kg/m$^3$ for SCC, i.e. about 200 kg/m$^3$ more fines than for NC. The increase of fines in SCC as compared to the amount of fines in NC prevents the larger aggregate from segregation in the fresh mix due to differences in gravity to the water and the cement in the fresh mix. Large amount of limestone filler in the concrete affects the durability substantially especially chloride ingress and fire resistance even though economical and ecological reasons talks for replacing Portland cement, as much as possible by filler [Persson 1999]. As much as 7% of the industrial fabrication of carbon dioxide, CO$_2$, is related to the production of cement.
One way to reduce the CO₂-relaease is to use HPC where cement is more efficiently applied, or to replace the cement by filler. However, large amount of limestone filler increases the carbonation [Andenaert, de Schutter 2002] and reduce other durability properties of SCC compared with NC [Stark 1999]. A protective layer of NC for the steel reinforcement bars maybe needed outside the SCC under severe conditions as at the seaside of Japan [Sakai 1998]. When sulphates are present at a concrete construction with large amounts of limestone filler there is a substantial risk of TSA [Sakai 1998]. TSA for concrete with limestone filler differs clearly from normal sulphate attack since Ca₆[Si(OH)₆].12H₂O.2(SO₄)₂(Al₂O₃).12H₂O (Thaumasite) is formed instead of Ettringite Ca₆[Al(OH)₆].12H₂O.2(SO₄)₂(SO₄)(H₂O)₂ [Sharp 2003]. TSA is only formed under low temperature conditions and is most abnormal since silicon atoms are surrounded by an octahedron of OH-ions Sharp 2003]. The firm solution is therefore quite complex and may contain sulphate-rich and carbonate-rich forms Thaumasite [Sharp 2003]. The reason for this risk is the high specific surface of the limestone filler that causes a solution of CO₂ in the pore water especially at low temperature. Comparisons between mortars with OPC, OPC CEM I with 5% limestone filler and CEM II with 15% limestone filler shows TSA formation during sulphate attack even 5% limestone filler [Torres 2003]. The TSA is somewhat prohibited when the sulphate solution also contained sodium chloride (synthetic seawater) but still there [Torres 2003]. Mortar with OPC CEM I with 5% limestone filler was much more damaged by TSA than mortar with OPC. Only after 3-year TSA on concrete with limestone filler may observed in 1.4–3 g/l sodium sulphate solutions [Crammond 2003]. This international research referred to clearly shows that even low content of limestone filler in the concrete affects the TSA resistance even though low-alkali cement is used. TSA is also much more frequent at in-situ concrete that at precast probably due to quality differences, i.e. lower-dense concrete (higher w/c) is used in-situ than in precast applications [Crammond 2003]. The role of pH seems to be negligible as compared to the TSA [Zhou 2003]. In England large scale TSA was observed in 1998 at 30-year old high-way foundations structures made of concrete with limestone filler due to Pyritic clay [Cripps 2003]. TSA may thus take place in buried concrete foundations containing limestone aggregate and subjected to an external source of sulphate ions from the surrounding groundwater with about 1 g/l sodium sulphate solutions [Trägårdh 2003]. Only after 14 days there was little change in water soluble sulphur acid in contact with the Pyritic clay (filled around the foundation afterwards). After TSA on a concrete the CSH-binding in the cement paste converts into a more or less porous mass causing completely deterioration of the concrete [Cripps 2003]. In 1998 the issue of TSA even caused a question in the British House of Commons [Sharp 2003]. TSA can not be prevented by sulphate resistant low-alkali cement since the limestone filler in the concrete and sulphate acids interact. Low-alkali cement has a low C₁₂A content to avoid external sulphate attack. Otherwise formation of Ettringite or gypsum would destruct the concrete due to the increase of volume of the compound. For NC without limestone filler sulphate resistance is not an issue when low-alkali cement is used. For SCC with limestone filler TSA may be a problem even though low-alkali cement is used since the limestone filler provides the source internally of the concrete. Gypsum is due to sulphate attack on concrete without limestone filler with cement with normal alkali content).

1.3 Objective
The objective of this research was to compare over a 5-year period of time the performance of SCC with low-alkali cement and limestone filler under influence of a solution of sodium sulphate with the corresponding properties of NC with low-alkali cement but without limestone filler. The tests were performed in accelerating conditions since the main reason was to compare NC with SCC under sulphate attack. Measurements of length, weight and FRF were performed. A comparison with the influence of curing in seawater and distilled water on the corresponding properties was performed over 3 years. The effect of mix proportions of the SCC, i.e. filler content, filler type, mixing order and pouring pressure (at top or at bottom of 5.5 -m high mould), was also studied but not in the full-scale matrix. The accelerated test was to be performed under reasonably severe conditions as compared to the worst conditions expected in real environment for concrete. Only a few years existed to see the TSA effect. The test conditions were also to follow international standards and recommendations for test of sulphate attack. The ongoing research eventually will continue until any damage is observed on NC without limestone filler.

2 MATERIALS AND METHODS

2.1 Materials
Sulphate resistant, low-alkali Portland cement, Table 1, and limestone filler were used in the experiments together with crushed and natural aggregate, super-plasticiser (polycarboxylic ether) and an air-entrainment agent (fatty oils). The motivation for using low-alkali cement was for the concrete to resist normal sulphate attack. This type of low-alkali cement is in common use when sulphates may exist or accumulate close to the concrete. Mineralogical granulometric
properties of the material and the grading of the fresh concrete mix proportions are given in Figures 1–2. The following parameters were studied partly (not in the full-scale) [Persson 2001]:

1. Compaction type, self-compacting SCC (O) and vibrated, NC (R)
2. Mixing order, new (N) and ordinary (O)
3. Amount of limestone filler, increased (B) and ordinary (O)
4. Type of limestone filler, crystalline (K) and sedimentary (S)
5. Pouring pressure, 0.3 m (O), and 5.5 m (T)

A total of 7 different concrete were studied, six SCC and one NC, all with w/c = 0.39, which is a w/c normal for concrete in severe conditions, Table 2. NC of type RO and SCC of type KN were performed in full-scale in large quantities at all the Southern Link site, Stockholm [Persson 2001]. NC of type KN thus is a typical bridge deck, foundation or retaining wall concrete [Persson 2001]. From the concrete, larger specimens, 0.23 m in diameter and 0.30 m in length, were prepared, two of each combination. The concrete was sealed in a steel container directly after casting and covered by steel. From the large specimen 3 cylinders 50 mm in diameter and 150 mm in length were core drilled. The grounds for the new reversed mixing order was a better dispersion and frost resistance in a separate investigation of four types of limestone filler for SCC [Pettersson 2000]. The motivation to use an increased amount of filler was to simulate mixing mistakes and its effect on TSA of SCC. Finally, the pressure increase was performed in order to simulate the effect on TSA of pouring SCC into a 5.5 m high wall. SCC was not vibrated after casting. NC was vibrated 10 s each time after filling one-third of the steel container. Samples were taken at 28 and 90 days' age.

In parallel 100-mm cubes were cast in order to obtain the strength of the concrete, three at each age and combination of time and concrete mix proportions. The cubes were wrapped in adhesive aluminium foil in order to be sealed cured.

Two types of mixing order were used [Persson 2001, Pettersson 2000]:

1. Ordinary mixing order with all dry material mixed with the water with air-entrainment for ½ min. and

### Table 1. Chemical compositions and properties of the cement CEM I 42.5 BV/SR/LA.

<table>
<thead>
<tr>
<th>Component</th>
<th>(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CaO</td>
<td>65</td>
</tr>
<tr>
<td>SiO₂</td>
<td>21.6</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>3.5</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>4.4</td>
</tr>
<tr>
<td>K₂O</td>
<td>0.58</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.05</td>
</tr>
<tr>
<td>MgO</td>
<td>0.78</td>
</tr>
<tr>
<td>SO₃</td>
<td>2.07</td>
</tr>
<tr>
<td>Ignition losses</td>
<td>0.47</td>
</tr>
<tr>
<td>CO₂</td>
<td>0.14</td>
</tr>
<tr>
<td>Clinker minerals: C₃S</td>
<td>21</td>
</tr>
<tr>
<td>C₃S</td>
<td>57</td>
</tr>
<tr>
<td>C₃A</td>
<td>1.7</td>
</tr>
<tr>
<td>C₄AF</td>
<td>13</td>
</tr>
<tr>
<td>Water demand</td>
<td>25</td>
</tr>
<tr>
<td>Initial setting time</td>
<td>145 min.</td>
</tr>
<tr>
<td>Density</td>
<td>3214 kg/m³</td>
</tr>
<tr>
<td>Specific surface</td>
<td>305 m²/kg</td>
</tr>
</tbody>
</table>

### Figure 1. Material grading curves.

### Figure 2. Particle grading in fresh mix.
then mixed with the superplasticiser for another 2\(\frac{1}{2}\) min.

2. New reversed mixing order with all material (also water, air-entrainment superplasticiser) except for limestone filler mixed for 1\(\frac{1}{2}\) min. with additional mixing with filler for 2\(\frac{1}{2}\) min.

During the slump flow tests SCC performed stable without any separation which stability also was observed in the full-scale production in Stockholm.

### 2.2 Methods

The experiments consists of accelerated tests based on CEN recommendations in order to compare over a long period of time the performance to TSA of NC without limestone filler with the performance of SCC with limestone filler, both the concrete based on low-alkali cement. All curing and testing took place at 5°C, which is close to the average temperature of the ground in Nordic countries. One-third of the specimens were placed in a solution of 18 g/l of sodium sulphate in distilled water being a three time higher concentrations than what may be expected in sulphuric grounds [Persson 2003b, Johansson 2003, 20Moir 2000], one-third in 1% sodium chloride seawater from Barsebäck, Sweden, and the rest (one-third) of the specimens placed in distilled water. The solutions were renewed every third month. A propeller was placed in the basin in order to secure a good circulation of the solution around the specimens. The method for investigation of the damage has been measurement of length, mass by weighing and of FRF. At the start of testing, either at 28 or 90 days’ age, and at 100, 300, 900, 1400 and 1700 days.

### Table 2. Mix proportions and properties of tested concrete (kg/m\(^3\)).

<table>
<thead>
<tr>
<th>Material/mix composition</th>
<th>KN</th>
<th>KN8</th>
<th>KO</th>
<th>KOB</th>
<th>KOT</th>
<th>RO</th>
<th>SO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushed aggregate 8–16 mm</td>
<td>363</td>
<td>355</td>
<td>367</td>
<td>371</td>
<td>363</td>
<td>862</td>
<td>402</td>
</tr>
<tr>
<td>Natural sand 0–8 mm</td>
<td>853</td>
<td>836</td>
<td>864</td>
<td>872</td>
<td>855</td>
<td>715</td>
<td>786</td>
</tr>
<tr>
<td>Natural sand 0–2 mm</td>
<td>316</td>
<td>309</td>
<td>320</td>
<td>135</td>
<td>316</td>
<td>146</td>
<td>422</td>
</tr>
<tr>
<td>Limestone filler</td>
<td>183</td>
<td>180</td>
<td>186</td>
<td>375</td>
<td>184</td>
<td>0</td>
<td>94</td>
</tr>
<tr>
<td>Cement</td>
<td>418</td>
<td>409</td>
<td>423</td>
<td>427</td>
<td>419</td>
<td>431</td>
<td>416</td>
</tr>
<tr>
<td>Air-entrainment (g/m(^3))</td>
<td>585</td>
<td>1203</td>
<td>106</td>
<td>213</td>
<td>117</td>
<td>474</td>
<td>125</td>
</tr>
<tr>
<td>Superplasticiser</td>
<td>2.97</td>
<td>3.2</td>
<td>3.39</td>
<td>4.13</td>
<td>3.69</td>
<td>7.32</td>
<td>2.99</td>
</tr>
<tr>
<td>Water</td>
<td>163</td>
<td>160</td>
<td>165</td>
<td>167</td>
<td>163</td>
<td>168</td>
<td>162</td>
</tr>
<tr>
<td>w/c</td>
<td>0.39</td>
<td>0.39</td>
<td>0.39</td>
<td>0.39</td>
<td>0.39</td>
<td>0.39</td>
<td>0.39</td>
</tr>
<tr>
<td>Air content (%)</td>
<td>5.6</td>
<td>8</td>
<td>5.5</td>
<td>4.9</td>
<td>6.3</td>
<td>5.8</td>
<td>5.6</td>
</tr>
<tr>
<td>Slump (mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>110</td>
</tr>
<tr>
<td>Slump flow (mm)</td>
<td>720</td>
<td>735</td>
<td>620</td>
<td>780</td>
<td>640</td>
<td>710</td>
<td></td>
</tr>
<tr>
<td>Flow time until 500 mm (s)</td>
<td>5</td>
<td>8</td>
<td>10</td>
<td>7</td>
<td>8</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Aggregate content (% vol.)</td>
<td>0.64</td>
<td>0.63</td>
<td>0.65</td>
<td>0.65</td>
<td>0.65</td>
<td>0.65</td>
<td></td>
</tr>
<tr>
<td>Compressive strength, 1-day (MPa)</td>
<td>20.8</td>
<td>26.0</td>
<td>14.0</td>
<td>23.8</td>
<td>23.8</td>
<td>16.0</td>
<td></td>
</tr>
<tr>
<td>7 days (MPa)</td>
<td>50.0</td>
<td>63.0</td>
<td>38.0</td>
<td>56.0</td>
<td>56.0</td>
<td>45.0</td>
<td></td>
</tr>
<tr>
<td>28 days (MPa)</td>
<td>63.3</td>
<td>83.5</td>
<td>49.8</td>
<td>74.5</td>
<td>74.5</td>
<td>60.5</td>
<td></td>
</tr>
<tr>
<td>90 days (MPa)</td>
<td>76.0</td>
<td>92.0</td>
<td>58.0</td>
<td>81.0</td>
<td>81.0</td>
<td>70.0</td>
<td></td>
</tr>
<tr>
<td>Density at mixing</td>
<td>2297</td>
<td>2250</td>
<td>2323</td>
<td>2348</td>
<td>2300</td>
<td>2325</td>
<td></td>
</tr>
<tr>
<td>Density at 28 days' age</td>
<td>2345</td>
<td>2299</td>
<td>2351</td>
<td>2436</td>
<td>2436</td>
<td>2440</td>
<td></td>
</tr>
<tr>
<td>Density at 90 days' age</td>
<td>2385</td>
<td>2309</td>
<td>2380</td>
<td>2416</td>
<td>2428</td>
<td>2415</td>
<td></td>
</tr>
<tr>
<td>28-day increase of density (%)</td>
<td>2.1</td>
<td>2.2</td>
<td>1.2</td>
<td>3.7</td>
<td>5.9</td>
<td>4.9</td>
<td></td>
</tr>
<tr>
<td>90-day increase of density (%)</td>
<td>3.8</td>
<td>2.6</td>
<td>2.5</td>
<td>2.9</td>
<td>5.6</td>
<td>3.9</td>
<td></td>
</tr>
<tr>
<td>1400-day scaling 28 d (kg/m(^2))</td>
<td>0.53</td>
<td>0.90</td>
<td>3.8</td>
<td>0.14</td>
<td>0.35</td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>1400-day scaling 90 d (kg/m(^2))</td>
<td>0.96</td>
<td>0.92</td>
<td>3.1</td>
<td>1.09</td>
<td>0.28</td>
<td>1.82</td>
<td></td>
</tr>
<tr>
<td>1700-day scaling 28 d (kg/m(^2))</td>
<td>3.54</td>
<td>5.02</td>
<td>12.5</td>
<td>0.83</td>
<td>1.79</td>
<td>6.02</td>
<td></td>
</tr>
<tr>
<td>1700-day scaling 90 d (kg/m(^2))</td>
<td>5.56</td>
<td>4.92</td>
<td>6.99</td>
<td>4.66</td>
<td>3.54</td>
<td>5.49</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3. Concrete strength development.
days of exposure, length, weights of the specimens and FRF were established (1400 and 1700 days TSA only) [Persson 2001, Persson 2003b]. The test specimen was dried off with a wet cloth before weighing removing all peeled off material with a brush. The ongoing comparative test did not as yet include the measurement of water suction but this test may be included in the test as soon the time of testing is ended. For the detection of macro voids, pressure technique under water then also may be applied. FRF is closely correlated to the elastic modulus of the specimen. A decrease of FRF is a sign of internal deterioration of the concrete for example caused by TSA. The precise measurement location for the length and FRF measurement was marked into the rim of the ends of the cylinders. The cubes were tested for strength at 1, 7, 28 and 90 days’ age. Increase of density between the fresh state and the hardened is a uncomplicated way of quantifying the degree of compaction of concrete. In the fresh state the density was obtained before casting of the concrete – after hardened also at 28 and 90 days’ age. The diameter, length and weight of all specimens were measured. The fresh density was measured on a 8-l sample of concrete. The same batch of concrete was then cast in larger specimens, 0.23 m in diameter and 0.30 m in length. Directly after core drilling of the small sample, cylinders 50 mm in diameter and 150 mm long, the density was taken of these, from the same source as the fresh density was established, both at 28 days’ age and at 90 days’ age. At 1700 days of exposure surface of all specimens except for NC were totally destroyed (measurement of FRF/length was not feasible at this time). The quality of the microstructure is very important like documentation by thin section but has not yet been performed since it is an ongoing research.

3 RESULTS

3.1 Nine hundred days’ age

The list of symbols makes it easier to read the Figures. There is no acceptance criterion given in this comparative test series since the objective was to show if the risk of TSA enhances over a long time of period or not when using limestone filler in SCC as compared to the risk of TSA in NC without limestone filler. The strength development of the concrete is shown in Table 2 and Figure 3. The strength development was clearly influenced by the filler mineral additives in the concrete (substantially increased strength) and by the air content (lower strength). A continuous increase of the length, weight and FRF over 900 days was observed for all concrete types dependent on the way of mixing. The increase of the length, weight and FRF was independent of type of compaction, SCC or NC, independent of the content of limestone filler, independent of the content of air and independent of the pouring pressure, Figures 4–6 [Persson 2001].

Only KO28 a decrease of weight was observed when curing in sodium sulphate. This decrease was owing to a crack-through the specimen at the rim of the end of it. The increase of length, weight and FRF over 900 days was due to continuous hydration and water absorption, compensating for chemical shrinkage due to hydration. The water absorption did not differ much between SCC and NC at this point. The weight increase may also be explained by deposits inside the concrete. The tests were prolonged until significant change was observed of length, weight and FRF between the types of concrete studied. The grounds for the new reversed mixing order was a better dispersion and frost resistance in a separate investigation of four types of limestone filler for SCC [Pettersson 2000]. A significant decrease of weight was observed for SCC with new mixing order exposed to a solution of 18 g/l of sodium sulphate in distilled water. No corresponding decrease as related to FRF of these SCC was observed. The decrease of weight was due to a surface scaling attack of the specimen, clearly visible. The specimen performed as it did after salt freezing and thawing scaling tests [Persson 2003a]. Obviously limestone filler should be added together with cement in order to avoid this kind of deterioration of the surface. Otherwise, when filler was added afterwards, the water was primary attached

Figure 4. FRF change after 900 days at 5°C. FRF = fundamental resonance frequency.
to the cement and a proper mixing of cement and limestone filler was prohibited. The new mixing order is referred to the improved dispersion of limestone filler [Pettersson 2000].

3.2 One-thousands-four-hundred-day TSA

After 1400 days only TSA was studied, since only large length change and mass changes were observed after accelerated treatment with sodium sulphate, cp. Figures 4–6 with Figures 7–9. For almost all concrete an increase of FRF, and then also of the elastic modulus, was observed which indicated that no internal deterioration took place of the concrete due to TSA except for SCC, SO 28 (decrease of FRF), and SCC, KO 28 (divided specimen). For vibrated NC continuous 5% increase of FRF was observed – more or less as for SCC. After about 8% increase of FRF the NC, KO 28, was divided – SCC specimen SO28 exhibited about 7% increase of FRF before a sudden decrease of

![Figure 5. Change of length after 900 days of exposure at 5°C.](image)

![Figure 6. Change of mass after 900 days of exposure at 5°C.](image)

![Figure 7. FRF change after 1400 days of exposure in 18 g/l of sodium sulphate in distilled water at 5°C.](image)

466
FRF to about −3% FRF was measured. Continuous water absorption probably led to an increase of FRF. Accelerated treatment with sodium sulphate caused much larger expansion than with fresh or sea water probably due formation of Thaumasite, cp. Figure 5 with Figure 8. Since low-alkali cement was used little or no gypsum was formed. The largest expansion after 900 days of TSA was observed for NC, RO 90, about 7‰, and for SCC, KO 28, about 6‰. Between 900 and 1400 days’ age continuous length increase was observed, up to 8‰ for NC, RO 90, which is much. For SCC, SO 28, 10‰ increase was observed and decrease of FRF in parallel which indicated that the specimen was undergoing internal destruction. For some of the SCC specimens the length may not be measured at 1400 days’ age (KO 28 and KO 90) since the measurement point disappeared due to surface destruction of the specimens. The most significant development after 1400 days of TSA was the weight decrease of SCC after 900 days’ age (normal mixing order) and after 450 days’ age (new mixing order), cp. Figure 6 with Figure 9. For NC continuous mass increase took place. Calcium hydroxide, Ca(OH)$_2$, probably leached from the SCC at a distance of 1–3 mm from the surface under the influence of external sulphate attack and internal carbon acid attack originating from the fine dispersed limestone filler. The surface of SCC, still with a flat and smooth surface and without any sign of destruction, exhibited white residuals, probably from escaped Ca(OH)$_2$, brook down when touched because an empty space was left just behind the surface. The surface deposits are not yet investigated with respect to composition since the tests are ongoing. However, all loose deposits were removed with brush before weighing. After brushing up to 12% mass loss was observed for SCC, KO 28. For SCC except for KOB 28, KOT 28 and KOT 90, the surface scaling, still after taking into account the water absorption, exceeded 0.5 kg/m$^2$ (up to 3.8 kg/m$^2$ for specimen KO 28), which was much, Table 2. With 0.5 kg/m$^2$ surface spalling the aggregate of the surface is clearly visible indicating severe TSA on the gel. However, even though large mass losses were observed for most of the SCC, almost no specimen exhibited internal percolation of Ca(OH)$_2$ leading to loss of FRF and elastic modulus decrease. After 1400 days of TSA all surface of SCC was affected. However, the sulphate concentration and the time of exposure are not linear parameters from the experimental point of view [Johansson 2003].

3.3 One-thousands-seven-hundred-day TSA

After 4 years until 5 years a sudden huge scaling of the concrete surface took place caused by TSA, more or less for all concrete with low-alkali cement and with
limestone filler, Table 2, Figure 10. For NC with low-alkali cement but without limestone filler no scaling at all took place due to TSA since no carbon acid was available in the concrete in this case, Figure 10. The mechanism of TSA is a interaction by carbon and sulphate acids which attack the gel forming Thaumasite. If no limestone filler was presented as in NC with low-alkali cement in this investigation, no weight decrease was observed. No normal sulphate attack with Ettringite formation was observed in the gel on NC since low-alkali sulphate resistant cement was used. Low-alkali cement NC without limestone filler withstood a high concentrated solution of sulphate at 5°C for 5 years.

4 DISCUSSION

4.1 Effect of amount of limestone filler in mix proportions

The limestone particles were much more soluble in the pore water forming carbon acid which caused deterioration of the cement gel together with the sulphate attack, forming Thaumasite, than when much larger limestone aggregate particles were mixed with cement and covered. At 90 and 450 days of contact the effect of a solution of sodium sulphate on SCC was small but accelerated from 450 until 900 days of experiment, Figures 4–9. No accelerating effect of a solution of sodium sulphate was observed in SCC with increased amount of limestone filler, Figure 9.

Larger amount of limestone filler, i.e. lower water-powder, w/p = 0.21, when w/c was held constant, did not lead to any increase of interior deterioration, which was expected. Results of SCC with w/p = 0.21 (about the same amount of limestone filler as cement in the mix proportions) showed about the same FRF as for NC probably due to lower permeability of SCC with limestone filler than that of NC. Still SCC with w/p = 0.21 showed larger surface scaling than NC with w/p = 0.39 at w/c = 0.39 for both the concrete types, indication the observed deterioration of the surface (shell formation maybe owing to percolation of Ca(OH)\textsubscript{2}, 1–3 mm from the face of SCC).

4.2 Effect of compaction technique

Three different compaction techniques were used in the experiments:

1. Pressure compaction, 5.5 m pouring height (SCC, KOT, Table 2, Figure 11)
2. Self-compaction, 0.3-m pressure from a pouring height (other SCC than KOT, Table 2)
3. Vibration (NC).

The largest increase of density, i.e. increase of density between fresh state and hardened one, was observed for SCC, KOT, with pressure compaction, about 6% as followed by NC, RO, about 5% increase of density from fresh state till the hardened one, Table 2 and Figure 11. The lowest increase of density, i.e. the lowest degree of compaction was observed for SCC, KO, which is the same type of concrete as SCC, KOT, but with only 0.3-m pressure compaction, less than 2% of increase of density between fresh and hardened state, Table 2 and Figure 11. SCC with large amount of filler, KOB, or with small size limestone filler, SO, exhibited degrees of increase of density in between concrete as mentioned above, about 3%. Figure 11. SCC with new mixing order also showed about 3% degree of increase of density. A tendency relationship between the mass losses, $\delta m/m$ (%), and the increase of density between the fresh and the hardened state, $\delta \rho/\rho$ (%), was obtained for all SCC (%), Figure 12 ($R^2 = 0.68$):

$$\langle \delta m/m \rangle_{\text{SCC}} = 17.3 \ln(\delta \rho/\rho) - 35.55 \quad \{1 < \delta \rho/\rho < 6\%; \ R^2 = 0.68\}$$

(1)
δ\(m/m\) denotes the mass losses between start of testing and 5 years’ age (%)

\(\ln(\delta \rho/\rho)\) denotes the natural logarithm of density increase between fresh and hardened state.

The accuracy of Figure 12 is acceptable \((R^2 = 0.68)\) taking into account that the starting age of the test differs, 28 or 90 days, and also the pouring pressures and mixing order differs. Figure 12 indicates a tendency that the TSA attack of SCC is dependent on the compaction of the SCC, i.e. differences between density at start of testing and at fresh state influenced the sulphate resistance.

5 SUMMARY AND CONCLUSIONS

Accelerated testing of Thaumasite sulphate attack, TSA, has been performed on concrete with low-alkali cement, both normal compacted, NC, without limestone filler and self-compacting, SCC, with limestone filler. The solutions of the accelerated test was according to the one proposed by CEN, 18 g/l, at 5°C. The experimental results and international studies clearly shows a potential risk of using limestone filler in self-compacting concrete even with low-alkali cement since Thaumasite formation may destroy the concrete. NC with low-alkali cement but without limestone filler thus withstood a solution of high sodium sulphate content at 5°C for at least five years. The following detailed conclusions may be drawn at 900 days of exposure:

1. For NC continuous increase of length, weight and fundamental resonance frequency, FRF.
2. For SCC cast under 5.5 m pressure also a continuous increase of length, weight and FRF.
3. For SCC with normal mixing cast without pressure up to 4 kg/m² TSA surface scaling.
4. For SCC with new mixing order cast without pressure up to 1 kg/m² TSA surface scaling.
5. For SCC (except with sedimentary limestone filler) a continuous increase of FRF.
6. The amount or type of limestone filler did not affect the amount of TSA surface scaling.

At 1700 days the following conclusions were drawn:

1. A rapid increase of surface scaling of SCC with limestone filler, between 2 and 12 kg/m².
2. No scaling of NC without limestone filler.
3. A link between the TSA mass losses and the increase of density (degree of compaction) between fresh and hardened concrete state.
4. Less TSA of SCC with 5.5 m gravity pressure compaction than of SCC without pressure.

ACKNOWLEDGMENT

I am most grateful to Professor Fagerlund, G., for his support and to Johansson, S.-E., Cementa, Heidelberg Cement Ltd, for his advises.

REFERENCES


LIST OF SYMBOLS

B increased amount of filler
FRF fundamental resonance frequency (Hz)
K crystalline limestone filler (40 µm)
N new mixing order
NC (R) normal (compacted) concrete (vibrated)
O ordinary mixing order
S limestone filler (15 µm)
T 5.5 m hydrostatic pouring pressure instead of 0.3 m
8 8% air content.
Effect of polypropylene fiber addition on restrained plastic shrinkage cracking of SCC

A.A. Maghsoudi  
*Civil Eng. Dept., Shaheed Bahonar University of Kerman, Kerman, Iran*

R. Rahgozar  
*Civil Eng. Dept., Shaheed Bahonar University of Kerman, Kerman, Iran*

M. Motavaselian  
*Shaheed Bahonar, University of Kerman, Kerman, Iran*

**ABSTRACT:** This paper reports a laboratory investigation on the contribution of polypropylene fibers (15 mm long) to the reduction of crack widths and area caused by restrained plastic shrinkage of different self-consolidating concrete (SCC) and self-consolidating light concrete (SCLC) mixes during the first 24 h after casting. The polypropylene fiber (PF) fresh concrete to be tested was laid on top of a fully hardened and grooved based concrete that provided the bottom restraint and this resulted in cracking in freshly (PF) concrete placed overlay. Prismatic (PF) concrete specimens (1060 \( \times \) 76.2 \( \times \) 38.1 mm) freshly cast on top of a grooved and hardened concrete substrate were investigated under adverse environmental conditions, high temperature, namely, 25°C, 35°C, 45°C, low relative humidity, and high volume and velocity of airflow. The parameters of this study were three polypropylene volume fractions of 0.1%, 0.2% and 0.3% in three different matrix types of (SCC), and three drying environments. The test showed that polypropylene fibers were effective in controlling the plastic shrinkage cracking of (SCC). The use of 0.3% volume fraction of polypropylene fibers was found to be successful in combating plastic shrinkage cracking in all concrete mixes tested even under a particularly severe environment.

1 INTRODUCTION

Self-consolidating concrete (SCC) is a relatively new type of concrete which can be defined as concrete that is able to flow in interior of the formwork, passing through the reinforcement and filling it in a natural manner, being consolidated under the action of its own weight. Adding the benefit of (SCC) to those resulting from the addition of discrete fibers to cement based materials, a high performance material, designated by polypropylene fiber reinforced self-consolidating concrete (PFRSCC), is obtained.

The present work is part of research program for influences the polypropylene on self-consolidating concrete, (SCC) properties. Compared to the effects of polypropylene fibers on properties of fresh and hardened concrete, the fature properties of concretes were significantly influenced by the polypropylene fibers. In this research the effect of this fiber on the reduction of crack by restrained plastic shrinkage of different self-consolidating concrete mixes is investigated.

Plastic shrinkage cracking of concrete occurs within the first few hours after the concrete is placed and before it attains any significant strength; it results in an unsightly and non uniform appearance on the concrete surface. Plastic shrinkage cracks become critical weak points for aggressive substances to penetrate into the internal portion of concrete leading to the acceleration of other detrimental forms of concrete deterioration. Consequently, the performance, serviceability, durability, and aesthetic qualities of concrete structure are reduced. Controlling plastic shrinkage cracking in concrete is essential for developing more durable and longer-lasting structures at minimum life-cycle cost [Antonio 2005].

When concrete is in the plastic state, moisture loss may occur both by evaporation from the surface of concrete or absorption by dry surface or formwork. Although some of the water lost this way is replenished by bleeding, if the surface moisture loss exceeds 0.5 kg/m²/h, negative capillary pressures develop in the concrete causing internal compressive strains.
If concrete is restrained, these compressive strains may result in tensile stresses far in excess of those needed to cause cracking in young concrete with poorly developed strength [Midness 2003].

One of primary causes of plastic shrinkage cracking is the loss of water from evaporation that leads to a built-up tensile shrinkage stress when concrete is subjected to sufficient restraint [Antonio 2005].

The contraction induces tensile stress in the surface layers because they are restrained by the non-shrinkage inner concrete, and, since the concrete is very weak in the plastic state, plastic cracking at the surface can readily occur if the tensile stresses are greater than the tensile strength of the concrete. Plastic shrinkage is greater than the greater the rate of evaporation of water, which in turn depends on the air temperature, the concrete temperature, the relative humidity of the air and wind speed.

One highly effective technique of controlling plastic shrinkage cracking is by reinforcing concrete with fibers. Randomly distributed fibers of steel, polypropylene, etc., provide bridging force across cracks and thus prevent them from growing [Nemkumar 2006]. Of all fibers used today for this purpose, polypropylene is considered to be the most effective for normal concrete (NC). Polypropylene is inexpensive, inert in high PH cementitious environment and easy to disperse. However, the exact influence of polypropylene fiber geometry, diameter, length, fibrillations, etc., is not well understood in NC [Nemkumar 2006]. However, considering the new type of concrete i.e., (SCC), an agent research is needed in this field.

To prevent plastic shrinkage cracking, the most widely accepted method is the use of randomly distributed fibers, particularly fine synthetic fibers in volume fraction below 0.5% [Antonio 2005]. In spite of well known role of fibers in this regard, there is no generally accepted method of assessing the effectiveness of fibers in controlling shrinkage induced cracking. In the past, several techniques have been proposed for studying shrinkage induced cracking in cement-based materials. These include, a ring type specimen [Bantia et. al 1993], a linear specimen whit anchored ends [Swamy & Stavrides 1979], a linear specimen held between a movable and a fixed grip [Bloom & Bentur 1995], and a plate type specimen [Kraai 1985]. These tests are well idealized in nature but do not represent the actual condition of restraints in practice [Bantia & Cheng 2000]. A technique producing restraints that is close to the one in reality was recently developed [Bantia et. al 1996]. In this technique, a layer of fresh concrete is placed directly on a fully hardened substrate base.

Different investigators used prismatic concrete specimens with dimension [Bantia et. al 1996; Naaman & Gurrero 2000; Naaman et al. 1999; Balaguru & Shah 1992] 1016 × 76.2 × 38.1 mm in dimension were cast on top of a transversely grooved already hardened concrete substrate base that was designed to produce a high restraint to enhance the potential of cracking. This method is believed to simulate well the actual situation of restraint in practice for slab placed directly on grade. These specimens were exposed to adverse environmental conditions immediately, and high velocity and constant large-volume airflow over the surface of the concrete. No report was observed for (SCC).

As previously mentioned, water evaporation and unrestrained shrinkage of concrete during 24 h after mixing are two of the most important factors to generate tensile stress in concrete. The influence of fibers on these two mechanisms, however, is not clearly understood. It is believed that the reduction of water evaporation and unrestrained shrinkage lead to a decrease in built-up tensile stress. Thus, cracking is potentially lessened in fiber-reinforced cementitious systems [Antonio 2005].

The main objectives of this study are: (1) to evaluate the effect of number of fibers on the plastic shrinkage cracking characteristics of (SCC); (2) to make a recommendation on the optimal amount of fibers needed to effectively control plastic shrinkage cracking for (SCC); and (3) to describe the mechanisms involved. These objectives will be supported by tests on plastic shrinkage cracking resistance, evaporation, free shrinkage, and bond strength development. Only the results to plastic shrinkage cracking resistance of fiber-reinforced cementitious composites for (SCC), however, are presented in this paper.

2 RESEARCH SIGNIFICANCE

Although plastic shrinkage remains a major issue in many ordinary concrete (OC) placements, however, considering the novel (SCC) a lack of information regarding the plastic shrinkage and the structural performance of (SCC) is one of the main barriers to its acceptance in the construction industry. The lack of published studies dealing with the structural performance of (SCC) especially in the very early age (24 h) of casting demands initiation of new research.

Polypropylene fibers are increasingly being used in concrete slabs for the control of plastic shrinkage cracking of (OC). The concept of (SCC) is relatively new, but over the last few years has gained worldwide acceptance as a high-performance construction material. Plastic shrinkage cracks can lead to the development of larger cracks as drying shrinkage occurs. Corrosive agent can enter the concrete through these cracks and can eventually cause severe deterioration of the concrete and its reinforcement. Controlling plastic shrinkage cracking in (SCC) is essential and, a structural element with superior performance made
with an affordable and durable (SCC) will contribute significantly to gain confidence of engineers, builders and clients. A test program was conducted in order to understand the influence of polypropylene fiber on plastic shrinkage cracking of (SCC).

3 RESEARCH PROGRAMME

3.1 Substrate base preparation

The mix proportions used for preparing substrate bases are listed in Table 1. Prismatic concrete specimens were similar to [Naaman & Gurrero 2000; Naaman, et al. 1999; Balaguru & Shah 1992], 1016 × 76.2 × 38.1 mm in dimension were cast that was designed to produce a high restraint to enhance the potential of cracking.

Substrate preparation; the concrete was cast into substrate with the dimension of 38.1 × 76.2 × 1016 mm of steel mold. A piece of polystyrene plastic 25.4 mm thick and 38.1 mm wide was placed with 38.1 mm spacing to create the mixture proportion for the concrete substrate. Because the restraint of the substrate is a key in the formation of plastic shrinkage cracks, the substrate must exhibit high stiffness [Antonio 2005]. The mix shown in Table 1 was chosen for its high strength and low shrinkage properties. The prepared bases were cured for a minimum period of 28 days in order to allow it to achieve sufficient strength. This method is believed to simulate well the actual situation of restraint in practice for slab placed directly on grade [Antonio 2005].

3.2 Materials

The materials used in this research investigation were: Portland cement (type (II) and (V)), coarse aggregate of crushed limestone whit maximum size of 10 mm and the fine aggregate of clean river sand. The polypropylene fibers of 15 mm length, 0.023 mm diameter and having a density of 910 kg/m³ was used (Table 2).

A series of plastic shrinkage tests for the overlay concrete was carried out. A number of polypropylene fiber was evaluated at different volume fractions that varied from 0.1 to 0.3%.

3.3 Environmental testing conditions

The plastic shrinkage test set up used for this investigation was similar to the one used by Antoine, et al [Antonio 2005]. A radiator heater provided an ambient temperature of 25°C to 45°C during the tests. Two high-velocity fans provided a constant high flow of air over the specimens to ensure rapid evaporation. A chamber was constructed to hold six plastic shrinkage specimens and to supply additional heat and airflow. The relative humidity varied between the duration of the whole testing program 23.5 ± 3.5%. Figure 1 illustrates the chamber and its components of the test set up.

3.4 Material and mixing procedure for (SCC)

A series of plastic shrinkage tests from specimen (AA1)–(AA8) for the overlay concrete was carried out and are listed in table 3. A number of polypropylene fiber was evaluated at different volume Fractions that varied from 0.1 to 0.3%.

Matrix specimen preparation—The matrix proportions by weight of cement for the overlay concrete are listed in Table 3. The maximum size of coarse aggregate used in this study was 10 mm.
The mixing procedure of each batch of concrete was consistently timed. The dry ingredients were mixed for approximately 1 min. Part of the total water (30–50%) was then added slowly and allowed to mix for 2 min. Fiber were then slowly added to the mixture to ensure the dispersion of fibers. The mixing then continued for 1 min. After the bowl was scraped to ensure no segregation of sand and cement at the bottom, the rest of the water was added and mixed for 1 min. Finally, in each series, two prismatic concrete specimens (1016 x 76.2 x 38.1 mm) cast simultaneously on top of a grooved and stiff concrete substrate Figure 2 were investigated under adverse environmental conditions, namely, high temperature, high volume, and velocity of airflow.

### 3.5 Test procedure

Plastic shrinkage crack occurred during the first few hours after casting the overlay concrete while the concrete was still in a semi-fluid or plastic state. In SCC cracks could appear very quickly and in a random order.

It was observed that the time elapsed between the first appearance of the crack and its growth to maximum length and width was approximately 15 min. Similarly, the time elapse between the appearances of the first crack to crack stabilization was approximately 50 min to 70 min.

Plastic shrinkage crack data were monitored in terms of crack widths and lengths measured by a crack width measurement with a very good accuracy, a handheld microscope, and a magnifying glass. The crack characteristics, such as crack width and length of shrinkage cracks were measured after 24 h. A longer duration was chosen to make sure that all cracks had developed and stabilized. The width of the crack varies along length and with (OC); it is real challenge to measure the width of crack at various locations along the length [Antonio 2005]. Minimum of four points were measured to the nearest 0.05 mm. Length of each crack was measured with the help of a piece of string and a measuring tape. The lengths were measured to the closest 1 mm.

From the cracking data, several values were obtained namely, a summation of length of all cracks or total crack length, maximum crack length, average crack length, maximum crack width, and total crack area defined as the summation of the products of crack length by crack width. The experimental results presented in this research are the average from two specimens.

### Table 3. Mix proportions for overlay concrete (by weight).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Type of SCC</th>
<th>Type of cement</th>
<th>Fiber content</th>
<th>PCE, Liter</th>
<th>Lime stone powder</th>
<th>Micro silica</th>
<th>Water</th>
<th>Lika aggregate</th>
<th>Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>AA0</td>
<td>SCLC II</td>
<td>1</td>
<td>0.1</td>
<td>4.66</td>
<td>0.41</td>
<td>0.11</td>
<td>0.51</td>
<td>0.486</td>
<td>–</td>
</tr>
<tr>
<td>AA1</td>
<td>SCLC II</td>
<td>1</td>
<td>0.2</td>
<td>5</td>
<td>0.41</td>
<td>0.11</td>
<td>0.58</td>
<td>0.486</td>
<td>–</td>
</tr>
<tr>
<td>AA2</td>
<td>SCLC II</td>
<td>1</td>
<td>0.3</td>
<td>5.5</td>
<td>0.41</td>
<td>0.11</td>
<td>0.61</td>
<td>0.486</td>
<td>–</td>
</tr>
<tr>
<td>AA3</td>
<td>SCC II</td>
<td>1</td>
<td>0.1</td>
<td>4.66</td>
<td>0.27</td>
<td>0.11</td>
<td>0.61</td>
<td>–</td>
<td>2.16</td>
</tr>
<tr>
<td>AA4</td>
<td>SCC II</td>
<td>1</td>
<td>0.2</td>
<td>5</td>
<td>0.27</td>
<td>0.11</td>
<td>0.58</td>
<td>–</td>
<td>2.16</td>
</tr>
<tr>
<td>AA5</td>
<td>SCC II</td>
<td>1</td>
<td>0.3</td>
<td>5.33</td>
<td>0.27</td>
<td>0.11</td>
<td>0.61</td>
<td>–</td>
<td>2.16</td>
</tr>
<tr>
<td>AA6</td>
<td>SCLC V</td>
<td>1</td>
<td>0.1</td>
<td>4.66</td>
<td>0.41</td>
<td>0.11</td>
<td>0.76</td>
<td>0.486</td>
<td>–</td>
</tr>
<tr>
<td>AA7</td>
<td>SCLC V</td>
<td>1</td>
<td>0.2</td>
<td>5</td>
<td>0.41</td>
<td>0.11</td>
<td>0.81</td>
<td>0.486</td>
<td>–</td>
</tr>
<tr>
<td>AA8</td>
<td>SCLC V</td>
<td>1</td>
<td>0.3</td>
<td>5.5</td>
<td>0.41</td>
<td>0.11</td>
<td>0.9</td>
<td>0.486</td>
<td>–</td>
</tr>
</tbody>
</table>

4 RESULT AND DISCUSSIONS

First by trial and error procedure for different (SCC) mix design, it was tried to find out any possible problem which could be appeared during the mixing of (SCC) with different amount of polypropylene, in this regard more information is given by [Motavaselian 2007]. Finally a successful mixing procedure was found.
out and selected as the suitable one, and, for the range of fiber content used in (SCC), no problems were encountered during mixing or placement. The fibers mixed well with the matrix; with no balling effect. Therefore uniformity of polypropylene fiber distribution was not separate variable (it is remained that such an achievement are reported by [Midness 2003; Nemkumar & Rishi 2006] for (OC) but not for (SCC)). Control specimens were cast for each (SCC) mix in each drying environment so that the crack area and maximum width of crack of fiber reinforced specimens could be compared with that of specimens without fibers. All specimens reinforced with fibers showed a reduction in crack area and reduced maximum width of crack compared to the control specimen, emphasizing crack arrest properties of the fibers.

The obtained results for all the three (SCC) mixes tested under different environmental conditions are given in Table 4–6. Table 4 shows the effects of fiber on (SCC) and (SCLC) mixes at 25°C. As it is depicted,
fiber, 100 percent effectually eliminated the cracks on the surface of specimens in each three (SCC) type. However it is remained that, the 25°C as a surface evaporation is little, consequently this result is acceptable.

The plastic shrinkage cracking test results for 35°C are shown in table 5. Reduction in length, width and number of cracks are occurred for majority of the (SCC) specimen. It is obvious that, as before, here even for 35°C, no crack is occurred for all the specimen except (AA3) which contained 0.1% fiber. This could be due to small amount of fiber added (i. e, 0.1% fiber).

Table 6 shows the effect of fiber on both (SCC) and (SCLC) at 45°C. The best results are obtained (no crack) for (SCC) containing cement type II (AA3–AA5). However (SCLC) with cement type II (AA0–AA2) and (SCLC) with cement type V (AA6–AA8) are in the next performance position. This seems to be due to (i) Type of cement and (ii) the use of Lika as aggregate and its ability to water absorption, its more likely to produce more cracks in the concrete. Result for all the three (SCC) mixes tested under different environmental conditions are given in table 4–6 for 0.1%, 0.2% and 0.3% fiber volume fractions. Areas of cracks were calculated by multiplying the length of crack by the average width of each crack. To clearly understand the effectiveness of polypropylene fiber in controlling the plastic shrinkage cracking of (SCC), Figures 3–10 were also plotted. Figures 3–10 show that there is general decrease in total crack area and in the width observed and crack length in all the (SCC) mixes are directly proportional to the rate of free surface water evaporation.

All (SCC) mixes tested showed an average crack reduction of 85% to 90% even at very low fiber volume fraction of 0.1% and nearly 95% to 100% crack reduction was obtained for all specimens at 0.2 and 0.3%.
0.3% fiber volume fraction. Test results of control specimens showed that crack area and maximum crack width increased for all (SCC) mixes from 0.1% to 0.3% tested under all drying environmental condition and increase in cement – aggregate ratio of concrete mixes from mix (AA0) to mix (AA8).

5 CONCLUSIONS

Polypropylene fibers are very effective in reducing the extent of shrinkage cracking and the crack widths in (SCC) and (SCLC) mixes.

Even at very low fiber volume fraction of 0.10%, an average crack reduction of 85 to 95% was achieved.

Plastic shrinkage cracking can be successfully overcome in (SCC) by adding 0.3% fiber volume fractions.

Total crack area and the maximum crack width are directly proportional to the rate of free surface water evaporation.

REFERENCES


High performance grout connection for pre-cast concrete structures

M.M. Alizadeh Kharazi
Sharif University of Technology, Tehran, Iran

ABSTRACT: Connections have very important role in pre-cast concrete structures, especially in seismic zones, and grout connections increase strength and resistance of pre-cast concrete structures for lateral forces. High performance grout connections (HPGC) provide new opportunities to design cost-effective pre-cast concrete structures. HPGC have been proposed to decrease cost of insulation, erection and crane cost of pre-cast concrete structures and to overcome weakness of traditional pre-cast concrete structure connections. This research focuses on properties of grout materials properties which produced in Iran and the effect of propylene fiber on the performance of grout connection. In the first stage, more than 65 MPa (28 days) compressive strength was achieved for Iranian traditional grout which the results (failure in the rebar and performance of the sleeves) demonstrated sufficient strength of the grout. New type connections (sleeve) were examined and the results are reported in this paper. The experimental results demonstrated better performance of these types of connections. These connections are also considered to be more suitable for pre-cast concrete construction in regarding to failure of the rebar (ultimate strength) in stead of the sleeves connection.

1 INTRODUCTION

Many connection elements for pre-cast concrete were designed and tested. The splices play important role in pre-cast concrete construction connections (especially in seismic). Commonly, it is a difficulty of pre-cast concrete constructions performance in seismic. This experimental work propose a new type of splice for pre-cast concrete construction in seismic. Some types of tensile grouted sleeve connections were investigated and tested to determine their performance. The investigation consisted of the determination of the ultimate load capacity and the complete force-deformation behavior of the connection until failure occurred.

Transmission of external forces acting on a splice connection element is based on friction, keying (bearing) and bond agents. To activate friction resistance (between internal surface of sleeve wall and grout, and between surface of rebar and grout), an expansion force has to be produced which expansion of grout plays this role. For keying activation, supporting forces must act locally by means of sleeve wall and rib deformation of rebar, and for bonding action an adhesive mortar(grout) produces the bond between rebar (or strand) surface with grout and sleeve wall surface with grout.

Therefore, grout was injected into the sleeve. Each component of a connection system (the grout, the sleeve and the rebar) itself and in interaction with others influences the strength and behavior of a connection, and makes the mechanical behavior of connection system. Therefore, improvement of mechanical characteristics of grout, shape of rib deformation and sleeve strength would lead to an increase in the ultimate load of connection and change failure process and force-deformation curves.

2 EXPERIMENTAL PROGRAMME

2.1 Grout selecting

In regarding to commonly, lower strength of grout mortar which produced by adding expander additive to ordinary mortar which mixed in site, and more cost of epoxy grout, powder ready mix grout (which supply by manufacture) was selected in this study. In regarding to high strength and suitable performance of S220 grout of Sarapoush Company, for final specimens was prepared. General properties of this product are shown in table 1. For improving of mechanical characteristics of grout especially volume expansion of grout and increasing of toughness, synthetic fibers were used. Grout was used, it was made by adding chopped fibers of Glass FRP, Carbon FRP and Polypropylene (PP) fibers (general properties of used PP fibers are shown in table 2), for which the compressive strength is presented in table 3.

The results show that 19 mm PP fibers (table 4) with mixture ratio 1.6 kg at cubic metre of grout (table 5) have optimal efficiency.
Thirteen grout sleeve specimens in three types (according to rebar or strand size and getting minimum grout cover) were made with tapered steel cone shell elements by E60 welding electrode and full penetration weld. Sleeves increased from a smaller to bigger diameter at mid length and then decreased to the same smaller diameter symmetrically.

### 2.2 Specimen design

After preparing the steel sleeves, rebar or strands were placed and fixed inside the sleeves. Firstly rebar was passed from middle of sleeve, secondly anchorage length of rebars was symmetry to sleeve and thirdly rebars were in the same alignment Figure 1. A kipping cap was used at the bottom end to prevent the grout exiting and to centralize the rebar and the strand. Grout after mixing was injected from upper sleeve opening in type I specimens immediately and in type II and III specimens with 15 minutes stop time (delay time for decreasing of grout air entrain). 5 cm cubic control specimens were prepared at beginning and end of grouting. General description of sleeves is illustrated in table 6.

The possible failure modes of corrugate elements are:

- Pull out rebar or strand
- Rebar rupture
- Breakage of grout
- Breakage of sleeve or combination of manner.

Each of them has a specific failure process.
The aim of these tests was to study the influence of various rebar diameters on the behavior of the existent connection. The tests should reveal probable changes in force-deformation characteristics, the ultimate load and the corresponding failure modes. For this reason, connection elements were tested under static tensile loading with three type of sleeve connection and a range of 16 mm to 32 mm of rebar diameter.

2.3 Test setup

All sleeves were pulled by a 1000 kN capacity computerized universal machine with defined stroke rate. The machine drew load-stroke graph for each test. S12.7 mm strand (S2S13) chopped and slipped between wedge grips and the machine could not continue to ultimate strength ($F_u$). In addition, the 5 cm cubic specimens of grout were compressed with defined stroke by that machine. All test results are shown in table 7, Figure2, Figure 3 and Figure 4.

3 TEST RESULT AND DISCUSSION

3.1 Sleeves

3.1.1 Type I

Setting: Three specimens with perpendicular ribs rebar $\Phi 14$ were made and three of them had 250 mm over the rebar.
The specified compressive strength of the grout was 71 MPa.

Force-deformation curve: For all specimens, in relation to bond between grout and rebar, and outstanding performance of the connection, failure of connection was rebar rupture. Thus the force-deformation of connection was the same as steel rebar behavior.
3.1.2 *Type II*

Setting: Two specimens were made with particular Post-Tensioning strands. The specified compressive strength of grout for one was 67 MPa and for other specimen was 72.5 MPa.

Force-deformation curve: The force-deformation behavior of number 1 specimen (figure 2) consists of two main parts. In the first part, load-stroke behavior of the specimen was elastic until the load reached approximately 130 kN. As the surface of the strand was plain, the bond between strand and grout was weak. Therefore, strand pull out from connection and curve continued with cyclic process in the second part. When the load level reached 140 kN, adhesive bond between strand and grout was lost and when the test was continued, the maximum load of each cycle decreased smoothly. For the helix shaped strand, in each cycle the pull out was 3 mm approximately and the specimen got twisted. At a Constant minimum load value (70 kN approximately) strand pulled out. Grout did not fail but the adhesive bond between the strand and grout failed. With the decreasing of anchorage length of the strand, the maximum capacity of each cycle decreased.

3.1.3 *Type III*

Setting: Results of type I tests illustrated grout sleeve connection have a great bearing ability. For Φ32 rebar and getting minimum grout cover (10 mm), increase of sleeve dimensions was necessary. Specimens were made with Φ20, Φ25, Φ28 and Φ32 without overlap. The specified compressive strength of grout was either 67 MPa or 72.5 MPa.

Force-deformation curve: Transmission of external forces on a connection element is due to friction, keying (bearing) and adhesive bonding. For better activation of friction, an expansion force should be produced. For keying bonding acting need an adhesive mortar produces the bond between rebar (or strand) and connection element. Therefore, grout was appointed to injection into sleeve. Each component of a connection system _the grout, the sleeve and the rebar_ itself and in interaction with others-influences the mechanical behavior of connection system. Therefore, an improvement of mechanical characteristics of the grout would lead to an increase in ultimate load capacity of the connecting element and would change the failure process and the force-deformation curve.

Φ20:(Figure 3) – The force-deformation curve of this specimen consists of three parts. In the first part, rebar bear load and force-deformation of connection was the same as rebar actions. In the next part, grout adhesive bond and bearing (keying) were lost and rebar pull out and the process continued in part 3. With the decrease of anchorage length, force decreased. Rebar’s rib caused a rough surface between rebar and grout. As a result, an unsmooth form was produced in part 3 of the curve.

Φ25, Φ28 and Φ32: For all specimens, in relation to better bond between grout and rebar, and excellent performance of connection, failure of connection was...
rebar rupture. Therefore, force-deformation of connection was the same as steel rebar behavior.

3.2 Grout

Grout is an expansive mortar suitable for any place that needs expansion with high strength. Grouts are of various types but three types of them are usual: ready mix grout (cement base), epoxy grout and grout admixture. The epoxy grout needs water for mixing. The advantages of the epoxy grout consist of very high strength and suitable expansion, and the disadvantages consist of high cost and difficulty to work with. The grout admixture which mixed in site produces an expansion (by grout additive) in ordinary cement mortar that is controllable by mixture ratio but it has low strength. Therefore, in view of the availability, cheapness and ease to work, ready mix grout was selected.

For improving the mechanical performance of the grout, especially to increase the of toughness and expansion control, synthetic fibers were used (19 mm chopped polypropylene fiber) and compressive and tensile specimens were tested. Results have shown that, Glass and Carbon fibers did not distribute homogeneously. Also 19 mm PP fibers had better tensile performance than 6 mm and 12 mm fibers (table 4). For the determination of an optimal mix ratio, a range of volume ratio 1 kg to 2.2 kg fiber in cubic meter of grout was tested and it was found that the volume ratio 1.6 kg fibers in cubic meter of grout have the optimal efficiency in grout mortar (table 5). In addition, using fibers through mix water produced homogenous mixture compared to adding fibers in dry mixture or after adding water.

For control specimen, after molding, a flat glass plate was placed at the top of mould that produced closer condition to test specimen. The control specimens were tested with a loading rate 5 kN per second. Finally, three 5 cm cubic specimens, with two different slumps were tested for the determination of the grout force-deformation curve. The force-deformation curves were the same for all specimens and they consisted of three parts. In the first part, the matrix and fibers had an elastic behavior. By increasing the load and crack propagation, force transmitted to fibers in the second part. Increasing of load in the third part was due to an increase in contact surface under loading (Figure 4).

3.3 Anchorage length calculations

\[
\frac{c+K_p}{d_b} \geq 2.5 \quad \text{ACI 408.3R-01 COMMENTARY}
\]

R4.2 (higher value of the parameter correspond to pullout failures).

\[
\frac{c+K_p}{d_b} \geq 2.5 \quad \text{ACI 12.2 COMMENTARY} \text{ (a pull out failure is expected and an increase in cover or transverse reinforcement is unlikely to increase the anchorage capacity)}.
\]

4 SUMMARY

– Test results (table 7) prove acceptable performance of traditional ready grout mixture.
– Test results (table 7) prove a satisfactory performance of the sleeve.
– Only \( \Phi 20 \) ordinary rib deformation rebar and \( \text{S12.7} \) strand were pulled out during the test.
– All Φ14 ordinary rib deformation rebar, and Φ25, Φ28, Φ32 high related rib area rebar failed as rebar.
– The maximum bond stress between a rebar and the surrounding grout in a sleeve is more than possible in reinforced concrete for comparable compressive strength.
– Performance index which is defined as predicted anchorage lengths to anchorage length of tests ratio, showed excellent performance (more than 233%) for Φ32 rebar (table 7).
– High relative rib area rebar depicts more efficiency than ordinary rib deformation.
– Ordinary rib deformation has low performance index (137%) for Φ20 (for the same length and diameter sleeve).
– 75 cm length sleeve can produce 1511 MPa for S12.7 mm strand.
– S12.7 mm strand (S2S12) presented good cyclic ductile behavior after maximum load (Figure 2).
– Figure 1 illustrates that load rise sharply 196.7 kN (adhesive bond and bearing stress between grout and rebar) and drop to 62.1 kN (friction stress between grout and rebar); then load decreased gradually due to a decreasing of the embedment length of rebar in sleeve. The second part of the curve had demonstrated no brittle behavior of sleeve for S1p8 specimen.
– Figure 2 demonstrates load grow dramatically 128.7 kN (adhesive bond and bearing stress between grout and strand) and drop to 69.7 kN (bearing and friction stress between grout and strand); then continue satisfactory cyclic load-stroke (ductile) behavior with slowly decreasing load capacity due to decreasing of embedment length of strand in sleeve for S2S12 specimen.
– Other specimen load-stroke graphs are the same as rebar load-stroke graph with no significant slip of rebar in sleeves.

5 CONCLUSIONS

The test results and the performance of splice demonstrate that this type connection satisfy type 2 mechanical splice of ACI 12.14.3.2, and it can be used in special moment frames constructed using pre-cast concrete according to ACI 21.6.b

REFERENCES

ACI 2006. Building Codes requirements for Structural Concrete (ACI 318-02) and Commentary (318R-02). Farmington Hills, MI, USA: American Concrete Institute.
ACI 2002. Splice and development length of high relative rib Area reinforcing bars in tension (408.3-01) and commentary (408.3R-01). Farmington Hills, MI, USA: American Concrete Institute.
ACI 2002, State-of-the-Art Report: Bond under Cyclic Loads (408.3-01) and Commentary (408.3R-01). Farmington Hills, MI, USA: American Concrete Institute.
Sustainable concrete arch bridges

J. Radic, Z. Savor & J. Bleiziffer
Faculty of Civil Engineering, University of Zagreb, Zagreb, Croatia

ABSTRACT: One of the most important aspects of sustainable development in the field of civil engineering is strategic planning and management of structures. Understanding various damaging effects of aggressive environment on structures and vice-versa plays an essential role in assuring sustainable construction. The paper discusses issues regarding sustainable design and construction of bridges, using in-service performance of large concrete arch bridges located at Croatian Adriatic Coast (Šibenik, Pag, Krk, Maslenica and Skradin Bridges) as benchmark. The most famous among the bridges addressed in the paper is the Krk Bridge as the largest conventional reinforced concrete arch structure in the world. Many problems and deficiencies were encountered during the lifetime of Šibenik, Pag and Krk Bridges, resulting in huge, complex and expensive repair works. Lessons learned from their performance in service were adopted in the design of more recent Maslenica and Skradin Bridges.

1 INTRODUCTION

1.1 Sustainability issues

The importance of sustainable development for the future of society has been recognized worldwide. According to United Nations Report of the World Commission on Environment and Development from 1987, the term sustainable development is defined as meeting the needs of the present without compromising the ability of future generations to meet their own needs [UN 1987].

Like in most sectors of human activity, sustainability issues are becoming a major concern in the 21st century structural engineering.

Philosophy of sustainable structural design is based on assessing long-term ecology, economy and socio-cultural effects of our built environment [Maydl 2004]. Thus, the entire life-cycle of a structure needs to be considered – from project planning, through design, construction and in-service performance, up to the end of its life and subsequent removal.

Increased interest in performance and long-term aspects of structures led to many changes in design and construction. It became evident that the strategic planning and management of structures is essential to assure sustainability of our built environment.

1.2 Sustainable bridges

Structures are designed considering and combining several design requirements: (1) load-carrying capacity
and stability, (2) functionality, (3) economy, (4) ecology and (5) aesthetics. Generally, design is dominated by balancing strength and functionality requirements, and the cost of construction. In some cases, for instance in urban environments, aesthetics may be a key factor in selecting among various design solutions. Environmental issues are generally addressed as compliance with minimum requirements. Like aesthetic requirements, environmental factors are bringing about an increase in the initial cost, and are consequently often disregarded as unnecessary. Good sustainable bridge design should put greater importance on environmental issues.

Another issue of great importance is that to design sustainable bridges, long-term effects have to be considered, thus eliminating the criteria of minimising the construction costs and changing towards considering life-cycle aspects in planning and design. The service life of bridge structures is generally higher than for buildings. While design specifications mandate bridges to be designed for 100 years service life, service life for buildings is estimated to be around 50 years [CEN 2002]. In practice, residential buildings maintain or even surpass this design assumption, but office buildings and especially industrial buildings are often of a substantially shorter service life. Consequently, the impact of bridges on our environment should be considered exceptionally long-term.

The practice has shown that considering sustainability in building design results in measurable benefits: reduced operation costs and increased workers productivity [Yates 2001].

A parallel can be drawn between building operation costs and bridge maintenance costs. Life-cycle cost indicators in bridge planning and design would include costs associated with maintenance and/or replacements throughout the service life of a bridge. Thus, a higher initial cost of sustainable bridge design might provide savings in the long run.

Ancient Roman stone arch bridges are often pointed out as extreme examples of sustainable bridge design [Oehsendorf 2004]. High initial cost is justified by extremely long service life (thousands of years). Figure 2 shows one such bridge – the Aqueduct of the Roman Emperor Diocletian’s Palace (today the very center of the second largest Croatian city of Split). The aqueduct was constructed during the years 284 to 305 to carry freshwater from the source of Jadro River over a distance of 9 km, and serves its purpose to this day. In addition to endurance, Roman stone arch bridges are environmentally friendly structures – they do not require substantial maintenance work, and when the end of life is reached, stones can still be used for new constructions.

Issues regarding sustainable design and construction of bridges shall be analysed studying the relation between design and construction solutions and performance in service of large reinforced concrete arch bridges in Croatia.

2 CONCRETE ARCH BRIDGES

2.1 **State-of-the-art**

During the whole of construction history, the arch has been the resistant structure par excellence, and still is today. Its strength is due to its shape or in its geometry converted into a resistant mechanism where compression predominates [Troyano 2003].

Concrete arches have been favoured for large spans up to the middle of the 20th century. Then, other structural systems were developed for large spans, which made arches non-competitive, primarily due to large construction costs. In the past, the heavy falsework was needed for the arch construction, which was often more demanding than the arch itself, creating problems in the supply of materials, construction, foundations and stability [Radic et al. 2004]. Even today, when...
Arches are usually erected using the cantilever procedure thus avoiding the need for scaffolding, expensive auxiliary equipment is required during the construction process, because of the significant differences in internal forces during the construction stages and at the final stage, once the arch is completed.

In today's technology, reinforced concrete arches are generally considered well suited for spans ranging from 200 to 400 m, which coincides with span range of steel girder or truss bridges and cable-stayed bridges.

Arch bridges have considerable advantage because of their inherently interesting form, strong visual appeal and attractiveness, reducing the adverse environmental and social impact imposed by huge man-made structures. But, for an arch span to be competitive economically over other bridge types, the right site with good soil to provide adequate foundation condition is needed.

Such surroundings are an important structural requirement to resist the thrust at the arch springing, but are also providing for aesthetically exceptionally successful structures. The most preferable sites for an arch structure are river canyons or valleys with high bluffs. When the arch is placed below the deck, such topographic features provide visual and structural resistance to both horizontal and vertical reactions at the springing points.

Such favourable topographic conditions can be easily found along the Croatian Adriatic coast. Consequently, six large span reinforced concrete arches were constructed over the years to provide important road crossings.

2.2 Adriatic arch bridges

Four Adriatic arch bridges – the Sibenik Bridge, the Pag Bridge and the Krk Bridges (two arches) were built during the sixties and the seventies and are usually referred to as the first generation of Croatian Adriatic arches (Figs 1, 3 & 4). Maslenica and Skradin Bridges (Figs 5 & 6) belong to the second generation of Adriatic arches, not just because they were constructed much more recently, but due to introduction of significant improvements in terms of sustainable bridge design.

These are world-renowned bridges that stand out as major developments in bridge engineering and technology. The Sibenik Bridges is the world’s first reinforced concrete arch constructed entirely by the cantilever method. The Krk Bridge was constructed using another innovative method in which concrete spandrel columns, temporary steel tension tie top chords and diagonals were combined to form a truss. With a span of 390 m the Krk Bridge is still the largest conventional reinforced concrete arch span in the world, 26 years upon its completion.

The bridges are set in very harsh environment in terms of frequent strong winds (Bora) that carry sea-salt and spray exposed structural elements. Additionally, high average annual temperature, high moisture content and occasional winter temperatures below 0°C accelerate chloride penetration and reinforcement corrosion [Radic et al. 2006].

Over the years many deficiencies and advanced stage of deterioration processes were identified on older bridges, as a consequence of extremely aggressive exposure conditions, but also of conceptual design and errors and negligence on site.

There are many similarities in the design of Sibenik, Pag and Krk Bridges. They were designed using very thin structural elements: extremely slender columns and the superstructure consisting of precast prestressed concrete girders (14 cm thick web) connected by a very thin deck plate (12–13 cm). The deck plate was additionally prestressed in transverse direction. Design values of concrete cover for columns (25–32 mm reinforcement bars) and the superstructure were only 1.5 to 2 cm.
Sibenik Bridge is somewhat less exposed to aggressive maritime environment than other Adriatic arch bridges and only minor rehabilitation work was performed so far. The bridge was thoroughly inspected last year. This was just the third major inspection performed on this bridge over its 40 years history. The most affected areas are at superstructure supports, designed as half-joints, where cracks opened in roadway slab above, allowing ingress of water (Fig. 7). Additionally, cracks and delaminating were detected at most structural elements. The chloride ions content testing showed that critical values are exceeded at the arch springing.

Half-joints in the superstructure at middle supports were used also on the Pag Bridge, with the same consequences. Repair works on this bridge started already after a decade of its service, but did not prove efficient in terms of stopping the corrosion process. Major reconstruction started in 1991 with the repair of the arch and was finally finished in 1999 when the original concrete superstructure was dismantled and replaced by a completely new superstructure made of steel (Fig. 8).

Lighter steel superstructure had to be used to allow for increase in traffic loads over the past decades, but has caused major concerns in how it will affect the arch behaviour. Since the arch axis is designed as a thrust line for certain permanent load, the distribution of lighter permanent load can be unfavourable and adversely affect the arch behaviour. The calculations revealed that the arch is capable of withstanding new loading within the designated threshold level only if the arch reinforcement contributes in the compressive zone and if the actual measured compressive concrete strength (increased over the years) is accounted for.

The repair works on the Krk bridge started several years after its completion with installation of laminated bearings and strengthening adjacent structural...
elements. In the 1990s the works were broadened to include the repair of columns of the smaller arch.

Different repair techniques had to be devised for spandrel and approach columns. The smaller arch has been recently repaired by removal of the contaminated concrete, its subsequent replacement with shotcrete and adding protective coating (Fig. 9). The repair of the larger arch presents major challenge to bridge engineers. The arch is supported on submerged inclined struts (Fig. 10). After many years of research and testing various corrosion protection systems, cathodic protection method was selected for this part of the structure. All of these repair works are not only expensive, but technically demanding tasks and very difficult to perform.

The experience of performance in service of previously described large reinforced concrete arch bridges and severity of the aggressive maritime environment was carefully considered while designing Maslenica and Skradin Bridges.

Maslenica Bridge introduced significant improvements with respect to considering long-term performance of the structure already at planning stage. Design took into account highly aggressive environmental effects. High quality concrete was used with water-cement ratio less than 0.4. Portland cement with 20% slag addition was utilized. Structural members and concrete cover were significantly increased in size compared to the first generation of Adriatic arch bridges to achieve a robust and durable structure. This led to the increased initial cost, but was justified with lower probability of costly and complex repairs in the future, which were experienced on the older bridges.

Skradin Bridge is a rarely used combination of reinforced concrete arch and superstructure composed of steel and concrete.

Instead of a conventional prestressed concrete superstructure made of precast girders and a cast in place slab, it consists of steel girders and reinforced concrete deck-plate. This enabled omitting the cap beams on columns and reducing the size of the arch cross-section, resulting in 35% less mass of the Skradin Bridge when compared to the Maslenica Bridge. Smart combination of two materials reduced the resources consumption.

The short overview of older Adriatic arch bridges performance in service clearly indicates the importance of efficient and effective management as a primary response to sustainability challenge. Outstanding management efforts are essential to ensure preservation of such large and significant bridges and their environment. Croatia is currently placing large efforts to develop a reliable and efficient motorway asset management system, an important part of which is bridge management system. In the course of this work it was established that major structures should be hand-picked from the entire bridge stock and treated separately because these are very complex structural systems that have huge environmental, social and cultural impact. The intention is to develop bridge-specific maintenance and management strategy and programme, as well as employ dedicated bridge engineers for each of the bridges. The programme would be tailored to individual structural aspects and present structural condition.

Additionally, effective management of bridges for sustainability should be adaptive and learn from success or failures of past actions. Here, monitoring plays a crucial role as it provides information on condition of a system to assess whether the structures are managed in a sustainable fashion.

Both Maslenica and Skradin Bridges are equipped with a range of sensors for long-term monitoring of both structural performance and durability related performance in order to facilitate the future maintenance activities by triggering timely adjustments and interventions. The monitoring system also records indicators of environmental conditions. Unfortunately, Maslenica Bridge monitoring project was stopped due to lack of
funds, but the monitoring system installed on Skradin Bridge, consisting of a lesser number of gauges, is set up and running. The structural health monitoring data would be used for structural condition evaluation and deterioration prediction, thus triggering timely and appropriate maintenance and rehabilitation work.

3 CONCLUSIONS

There are six large reinforced concrete arch bridges on Croatian Adriatic coast, built from 1966 until 2005. The bridges are major achievements of modern bridge engineering. Many problems and deficiencies were encountered on older Adriatic arch bridges resulting in complex and expensive repair works. The design of Maslenica and Skradin arch bridges, which were constructed recently in Croatia, took into account the experience from the in-service performance of older arch bridges. The study of large Adriatic arch bridges was used to analyse how principles of sustainable development may be introduced in the design and management of concrete arch bridges, both to protect our man-made and natural environment. Sustainable bridge design shall generally incur higher initial cost, but life-cycle aspects should be analysed to select the optimal solution.

REFERENCES


Determination of the amount of hydrated cement in hardened concrete and mortar by gravimetric thermal analysis on macro and semi-micro scales

F. Gomà
Polytechnical University of Catalonia. Barcelona, Spain

ABSTRACT: This paper describes the scientific-technical bases for the application of the new proposed procedure by Thermal Gravimetric Macro Analysis (TGMA) monitored by DTA for determining the quantity of the hydrated cement in hardened concretes and mortars. The loss on ignition as determined by Thermal Analysis (TG) within the temperature range of 102°C to 410°C under the given conditions, includes the water of constitution of the phases existing in the concrete: calcium silicate phases and aluminate, ferrite, sulphate phases with exception of the portlandite. This Loss On Ignition (L.O.I.) is proportional to the amount of cement in the concrete for a determined employed cement. When these values are compared with those obtained from the pure pastes of the cement used in the concrete under the given conditions, the cement content in the concrete can be determined. The proposed method allows the direct application to whole samples on macro and semi-micro scales. This method is simple, quick with multi-application that enables representative results to be obtained from varied multiple application zones in structural elements. In combination with ASTM C – 642 method for determine physical parameters, it allows the water-cement ratio to be determined operationally. Certain possible interferences that could be produced due to the presence of the any types of the aggregates are determined and enable the procedure application limit to be established. In combination with the “Soluble Fractions Analysis” method by the same author, it becomes possible apply the method to the concrete containing active additions, base materials for the necessary sustainability. The results of the same homogenised samples are compared with the classic method based on the determination of the modified soluble silica by the same author. The modification consisted of the correction of significant errors that continue to exist in current methods with which the standard deviation is reduced by a factor of three and providing an operationally valid procedure. The results show that the TGMA procedure hydrated cement content to be obtained with equivalent precision and operational time savings and also permits multiple results to be obtained at the same time. Finally, its application is suggested for the tracking of the quality of onsite-poured concrete in order to reveal the distribution of water-cement ratio (W/C) variations in determined structural elements and for the tracking of cement hydration in onsite-poured concrete in function of curing and its effectiveness.

1 INTRODUCTION

The accumulation of research papers on hydrated phase knowledge, for both pure species as well as hydrated Portland cement pastes (hcp), has led to a significant amount of information relating to their structures and water of hydration levels. The applications of DTA and DTG techniques applied to pure hydrated Portland cement pastes and their isolated synthesised phases in the literature from the second half of the twentieth century have been thoroughly examined. An extensive summary of those papers was produced [Ramachandran, 1969] and then later by [Taylor,1997], in which it can be appreciated just how it is possible to predict the characteristic thermal analysis curve for a determined cement from its phase composition.

Their studies allowed general conclusions to be made here that permit a first interpretation of the obtained results, together with the method possibilities.

It can be seen from these studies that the complex sequences of the layout of the layers and/or cavities formed in hcp structures and its water or ion content are not always the same. These lead to units with more or less symmetry in the water links fixed in the structure of the hydrated phase molecules, having different energy levels according to their locations. Consequently, they are released at different temperatures when subjected to thermal analysis. In the chosen
temperature range of 102°C to 410°C for sample calcination, the main hcp phases are the CSH, the CAH phases and their variety of AFm phases and the various mono-sulfo-aluminates and tri-sulfo-aluminates. The carbo-aluminates and Portlandite are released at significantly higher temperatures. For this reason, the carbonation of the concrete does not directly affect the results.

Because of all these, we can see that the losses on ignition as determined by TG Thermal Analysis offer possibilities for determining the hydrated cement contents in mortars and concretes. We have not discovered any background in the literature for this specific application, therefore, a proposal of a new method for determining the amount of cement in hardened concretes by this technical is given for first time.

The properties of this new procedure make it possible to, not only establish the average quality of a mass of concrete, or the quality of a particular part of the mass of the concrete that may be in question [3], but also to obtain better knowledge of the from the tracking of the reaction with the active additions.

This point is significant in obtaining monitoring information on concrete with type II cements, with active additions and/or active additions introduced in the dosing of general use concrete in the role of sustainable construction.

2 BRIEF SUMMARY OF THE BACKGROUND FOR THE DETERMINATION OF THE AMOUNT OF CEMENT IN HARDENED CONCRETE

The determination of the amount of cement in hardened concrete is carried out using the classic gravimetric method after digesting the sample with hydrochloric acid, filtration of the insoluble residue and the determination of the soluble silica in the filtrate. The procedures are described in BS [BS,1988] and ASTM [ASTM, 2001], and specially in a document from the Concrete Society on the Analysis of Hardened Concrete, [CONCRETE SOCIETY,1990]. These procedures are the best available [Gomà, 2005], but the standard deviation in the gravimetric determination of the silica is significantly large and the results provide too high a variation to offer efficient operation so that it is not currently used regularly unless in order to provide complementary data.

This state of the circumstances was not emphasized until the papers of Professor Neville in the last few years [Neville, 1999, Neville, 2000].

The errors discovered in the classic gravimetric method for determining silica in silicates in the author’s papers [Gomà,1975, & Gomà,1976] on the determination of silica in silicates, and the application on soluble silica [Gomà,2005, & Gomà 2006], still unknown in the cited current methods and in the general silica determination methods, [CEN 196–2,1996 & ASTM C-114,2001] and still unknown in the soluble silica determination [BS 1881 Part 124, 1998 & ASTM 1084,2001], describe the state of this situation. Its correction in the author’s cited procedure has led to the proposal of a new general method for silica determination and for the determination of soluble silica in hardened concrete in order to obtain the amount of cement [Gomà, 2005 & Gomà, 2006].

The application of this new method to the determination of the soluble silica in hardened concrete in order to establish its cement content has reduced the value of its standard deviation by a factor of three. With this, a truly operational method has been achieved with technical significance. The application of this new method for determining the cement content in concrete with active additions and for discriminating the silica in these additions can be found in the author’s cited papers [Gomà, 1986, & Gomà et al., 1999& Gomà, 2007].

The determination of soluble silica with these procedures requires a time of some four hours from the time of preparing representative samples. Having a quicker analytic procedure that is equally operational and based on a different principle permits more flexible information for its application to the control of hardened concrete in relation to its water/cement ratio and to obtaining more information on concrete composition, curing and hydration in onsite-poured concrete.

The thermal analysis applied to the determination of combined water makes it possible to determine the amount of cement in hardened concrete and mortar, if carried out under specific conditions ascertained in our research work with this Thermal, Gravimetric, Macro-Analysis technique (TGMA), which is given in the general scope of the procedure.

3 GENERAL SCOPE OF THE PROCEDURE

This essentially consists of the determination through TG analysis, on a macroscopic scale, of the losses on ignition of a whole, without any prior grinding, hardened concrete sample within a temperature interval of 102°C to 410°C from a sample state dried at 102°C in an oven in an open atmosphere until constant weight. Its weight is determined and then it is calcined to a constant weight at a temperature 410°C, achieved in an asymptotic manner and maintained to constant weight. The difference in weight in this interval corresponds to the loss of water of constitution due to the hydrated phases of the CSH silicates and the hydrated phases of the aluminates ACH and AFm and of its respective mono sulfo-aluminates and Ettringite.

The measurements of these losses from 102°C and 410°C are characteristics of each type of cement,
hydrated in pure paste subjected to curing by immersing in water up to the necessary time for consideration of practically total hydrolysis (approx. six months). For the usual cements in a given geographic area, they have to be previously measured, with the preparation of \(20 \times 20 \times 20\) mm. micro-cubes of pure paste of each cement type, as described in the details of the procedure.

Any eventual interferences of the aggregates have to be taken into account as indicated in the procedure description depending on whether they are available or not.

4 BASES FOR THE PROPOSED NEW PROCEDURE

In our research work employing the TG, DTG and DTA techniques, we have found that it is possible to obtain a constant and reproducible material state in hydrated cement samples when a time is maintained for its dehydration in an oven until constant weight in an open atmosphere at a constant temperature of 102°C, when the temperature variation is less than \(\pm 0.2^\circ\text{C}\). This is possible inside an ordinary oven with normal, non-forced ventilation. The 102°C temperature allows guarantee that, at any pressure value, the hygroscopic water vapour pressure inside the sample is higher than atmospheric pressure, so that complete dehydration balance is reached for the hygroscopic water at constant weight. This temperature is as close as possible to that of the combined water dehydration that we are going to determine and therefore ensures a negligible initial minimum error.

The determination of the loss on ignition by TGMA curve is performed using a muffle oven with heaters at the top, bottom and the sides. This is fitted with a FUZZI temperature control system [FUZZY, 2000] that allows the temperature to be gradually increased up to the predetermined value without exceeding its value, with a deviation of less than 0.5°C. In order to avoid any effects of direct radiation from the heater elements on the samples, these are placed inside a stainless steel container covered with a refractory ceramic plate, with a bulk density of 1.0 g/cm³, and 1 cm thick that is simply supported so the atmosphere is free, but not enclosed. Inside the container, the bottom is covered with 2 cm of sand and the samples are placed on this sand bed.

This point is essential to achieving constant weight. The calcination time at various temperatures has to be sufficient to reach dehydration at constant weight. Stainless steel is valid up to a temperature of 550°C. For higher temperatures up to 950°C, the stainless steel sample container has to be replaced by previously-weighed, individual porcelain crucibles for each sample, placed directly on the bottom of the muffle.

The chosen temperature interval for the determination of the water of constitution is from 102°C to 410°C. The temperature interval above 410°C up to 550°C will allow the determination of amount of portlandite and finally the loss from 550°C to 950°C is determined in order to obtain the CO₂ content, from of aggregates and carbonation effect.

Within the temperature interval from 102°C to 410°C, the corresponding losses on ignition at suitable intermediate temperatures can also be determined in order to have more information when desired. The accumulative sum of these losses constitutes the usual TG curve. If the percentages, referred to dry sample weight at 102°C for the losses on ignition corresponding to each temperature are determined individually and represented on a graph in function of the temperature, a diagram is obtained that permits the establishment of the dehydration peak intensities in function of a given temperature interval, together with the existing dominant phase in this interval, in the hydrated concrete or mortar. This manual method of obtaining the DTG curve enables multiple simultaneous determinations to be carried out, unlike an automatic single-sample unit, which also requires cooling before continuing. This circumstance makes it possible to work with many samples simultaneously and to obtain information distributed over extensive areas of the concrete, together with a more exact mean value of the studied structural element, providing the procedure with special operational interest.

The determination of the complete physical parameters in accordance with ASTM designation C-642, [ASTM-C-642, 2001], both for micro-cubes and the hardened concrete samples has also been performed to establish the hygroscopic water content and to express the composition in kg/m³. The weighing accuracy is 0.02 g for a mass from 100 to 400 g on a macro scale, and 0.001 g for 15 g masses on a micro scale. It has to be taken into account that the total amount of concrete mass in the oven as well as the muffle oven are related to their calorific powers in order to obtain results that are correct in pre-established times and sufficient for obtaining constant weight.

5 DESCRIPTION OF THE PROCEDURE

5.1 Sampling

The conditions for the explicit sampling in sections 3, (3.1 to 3.5) in [BS 1881, Part 124, 1988] are the same as those we applied to our experimental determinations on concrete samples because they are the ones providing the best guarantees. These conditions are usually applied only when there is some doubt about the quality of the concrete, when the average quality of a mass of concrete, or the quality of a particular part of this mass may be in question.
The application concept of this procedure to the concrete is different to the usually-cited current methods, which are used to analyse previously ground and homogenised samples. This procedure determines the amount of cement in a whole mass sample of between 100 and 400 g, measured with an accuracy of 0.02 g. The reason for this lies in the ability to obtain various results of cement contents and water/cement ratios of a specific concrete high-strength structural element and/or slender elements, in order to establish the achieved degree of homogeneity and the water/cement ratio variation in the actual element, especially when constructing in aggressive soils. The sample is extracted from the structural element to be studied by drilling and sawing the indicated mass into parts.

For this reason, the recommended method of obtaining of the samples would be extraction by drilling 75-mm diameter cylinders and cutting slices from two parallel cuts, preferably with a diamond saw and avoiding, as far as possible, any loss of material from the cut faces. The slice should be approximately 20-mm thick and may be cut by sawing into two or three parts. Each one of these parts should have a mass of between 200 g and 400 g. On a macroscopic scale, the extracted concrete cylinders are directly dried in a ventilated oven (not forced draught), which is controlled to maintain the temperature at 102°C ± 0.2°C, to constant weight, which is determined to 0.02 g. The required time is from 12 to 24 hours and depends on the equipment mass/power ratio.

5.2 Calcination

The samples, dried at 102°C, are subsequently placed inside a stainless steel container, with an approximately 10-mm thick piece of refractory porcelain, having a bulk density of 1.0 g/cm³. The container is closed with a sheet of the same material that is simply placed on top so that the atmosphere is open, not closed. The container, with the samples, is placed inside a muffle oven with heaters at the top, bottom and sides. This is fitted with a FUZZI temperature control system that allows the temperature to be gradually increased up to the predetermined value without exceeding this value, with a deviation of less than ±0.5°C. In order to avoid any effects of direct radiation from the heater elements on the samples, these are placed inside a stainless steel container on a 1-cm thick refractory ceramic plate, with a bulk density of 1.0 g/cm³, located at the bottom. This container is covered with a sheet of the same material and conditions and simply supported so the atmosphere is free, but never enclosed.

The weight is determined after achieving constant weight at a temperature of 410°C. The time required depends on the mass/heat power ratio of the system.

5.3 Determination of the L.O.I. of the pure pastes of different cements

On a semi-micro scale, the losses on ignition are determined for the different types of fully-hydrated cement. They are mixed with water in a cylinder, employing carefully adjusted stirring to prevent any loss of material, which would alter the proportion.

The water/cement ratio must be accurate to 0.5; they are placed in 20 × 20 × 20 mm³ moulds, then they are compacted and hydrated in a wet chamber for 24 hours at 20°C; the obtained micro-cubes are then immersed in water at the same temperature until a practically fully hydrated state is attained, at least over six months. Each micro-cube has an approximate weight of some 15 to 16 g. A series of these micro-cubes is dried in an oven, under the described conditions for 24 hours or more (to be checked) to constant weight. The individual weights are determined, with an accuracy of 0.001 g. Then the loss is determined by calcination from 102°C to 410°C to constant weight under the same conditions described for concrete, inside the box or a stainless steel container. The values of these losses on ignition from 102°C to 410°C for each cement type are the calculation bases for each concrete.

In order to determine the physical parameters of the micro-cubes, a minimum set of three of each cement type is immersed in de-ionised water at room temperature for at least 24 hours.

Finally, in order to eliminate any occluded air, they are boiled at this temperature in a vacuum for fifteen minutes. They are jointly weighed to 0.02 g accuracy, after wetting them with a damp cloth to obtain their saturated weight and then they are weighed immersed on a hydrostatic balance. The micro-cubes are first dried in air and then in an oven, under the described conditions, at 102°C for 24 hours to constant weight and then the complete physical parameter values are calculated in accordance with the same calculation specification in [ASTM,C-642, 2001]. The losses on ignition at 550°C and then at 950°C are determined under the same conditions and accuracy in porcelain crucibles in the usual fashion, without the stainless steel container protection.

6 INTERFERENCES

The following interferences from among those that could affect the results are taken into account:

a) The most important could be the small magnesium oxide content, generally in the form of carbonate from the calcareous aggregate, with inter-atomic replacement of Ca by Mg. In most cases, the mean value oscillates between 1.5% and 2.5%, calculates as MgO %. The decarbonation of this magnesium carbonate is produced between 380°C and 550°C. Therefore, it affects the loss on ignition value up to
410°C. When the interference value is more than 20% of the loss value in this interval, the procedure is not applicable and, when the concrete contains dolomite aggregates, this method is not useful.

b) The presence of clay in the aggregates for the concrete and/or mortar produces interference. Clays usually have some endothermic peak around the 200°C mark. This interference is normally very low and can be disregarded.

c) The presence of organic additions similarly produces interference by pyrolysis in this temperature region, up to 410°C. This interference may be negligible in ordinary concretes, but can be significant in special performance concretes. Great care must be taken when the limestone contains geological carbon.

The way of establishing these possible influences is to determine the loss on ignition within the same temperature range of 102°C to 410°C for a representative sample of the aggregates contained in the concrete, if these are accessible. If they are not, there are two cases:

a) Non calcareous aggregates. They can be extracted from the representative sample of concrete or mortar that has been previously reduced to a size of less than 63 microns, with prior digestion with 10% hydrochloric acid until the hydrated cement disappears, together with any limestone in the aggregate. Generally by a double digestion with intermediate filtration and washing are needed. The loss on ignition is determined for this sample, between 102°C and 410°C, referring its result to the weight of the aggregate sample dried at 102°C.

b) Lime or calcareous aggregates. In the soluble acid extraction of the sample must be to determine MgO content by EDTA method with elimination of the silica previously and to deduce the interference values due to MgO content.

7 CALCULATION

The calculation of the cement content in percentage from a sample of concrete was based on the criterion that the L.O.I. in the interval from 102°C to 410°C is proportional to the cement content in the concrete and when the sample is a pure cement paste the L.O.I. in this case corresponds to 100% of the cement.

The general equation is as follows:

\[
\frac{(a - b)}{a} \cdot 100 - \frac{e}{100} \left( \frac{a - c \cdot a}{100} \right) = \frac{d}{100}
\]

where:

a = the dry weight of concrete at 102°C
b = the weight of concrete calcined at 410°C
c = % of cement in the concrete
d = % loss on ignition of the pure hydrated cement paste
e = % loss on ignition of the concrete aggregates

The method of finding the unknown cement percentage c in concrete is as follows:

\[
c = \frac{100 \cdot \left( \frac{a - b}{a} \cdot 10^4 - e \cdot a \right)}{a \cdot (d - e)}
\]

8 EXPERIMENTAL DETERMINATIONS

8.1 Cement analysis, reference standard and micro-cube on a semi-micro scale

Three clinkers from different sources were taken and analysed in accordance with CEN methodology [CEN196–2, 1996]. The clinker compositions were selected so that there were various typical phase compositions calculated according to Bogue and the results are provided in Table 1.

Three typical cements were prepared with these three clinkers, with the addition of 99% pure dehydrated gypsum in order to form a series of cements with a 3.3% SO₃ content and jointly ground down in a laboratory ball mill to grain-size values that correspond to the usual current qualities (reject using a 63-micron sieve, from 0% to 5%). Details on Micro-cube preparation are given in the method description.

Table 1. Chemical analysis of the portland cement clinkers.

<table>
<thead>
<tr>
<th>Clinker</th>
<th>A</th>
<th>B</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>TiO₂</td>
<td>0.13</td>
<td>0.19</td>
<td>0.16</td>
</tr>
<tr>
<td>SiO₂</td>
<td>23.5</td>
<td>21.4</td>
<td>23.4</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>4.1</td>
<td>5.9</td>
<td>5.2</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>5.1</td>
<td>6.0</td>
<td>0.35</td>
</tr>
<tr>
<td>CaO</td>
<td>63.0</td>
<td>63.2</td>
<td>67.3</td>
</tr>
<tr>
<td>MgO</td>
<td>0.72</td>
<td>2.1</td>
<td>1.76</td>
</tr>
<tr>
<td>SO₃</td>
<td>0.80</td>
<td>0.28</td>
<td>0.47</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.11</td>
<td>0.40</td>
<td>0.15</td>
</tr>
<tr>
<td>K₂O</td>
<td>0.85</td>
<td>0.10</td>
<td>0.68</td>
</tr>
<tr>
<td>L.O.I.</td>
<td>1.60</td>
<td>0.38</td>
<td>0.50</td>
</tr>
</tbody>
</table>

Insoluble Residue

|   | 0.80 | 0.74 | 0.90 |

Free Lime

|   | 0.45 | 0.52 | 0.85 |

Bogue Compounds

| C₃S   | 43.0 | 46.3 | 62.1 |
| C₂S   | 34.9 | 26.4 | 20.2 |
| C₃A   | 2.2  | 5.5  | 12.9 |
| C₄AF  | 15.5 | 18.3 | 1.1  |

Values in percentages.
The typical cements were: two type I cements: CEM I 52.5 R; one cement type I, white: CEM BL I 52.5 N.

Pure pastes were prepared using these cements with water, at a 0.1 g accuracy in proportions and with water/cement ratios of 0.5 for all of them. They were put into 20 mm moulds and then vibrated, and the obtained micro-cubes were cured by immersion at 20°C for 180 days until a hydration state that was considered almost complete.

Different series of Micro-cubes were prepared under the conditions given in the description of the procedure. Once the micro-cubes were cured, they were dried in air and then in an oven at 102°C to constant weight, under the described conditions and individually weighed with 0.001 g accuracy. Then, the loss on ignition was determined by TGMA (micro-scale) at the temperatures given in the procedure.

The results are given in Table 2.

8.2 Concrete analysis on a macro scale

Two concretes were prepared with cement using clinker C with 3.3 % SO₃. The concretes A and B had the following specifications:

Concrete A. Cement CEM I-52.5 R. 300 kg/m³, Cement: Fines: Coarse: 1.0:2.0:3.4, Aggregates: Fine and Coarse (maximum 18 mm.) Mycritic Limestone, w/c 0.6, Bulk Density 2,220 kg/m³, Soluble SiO₂ Content of Cement 19.5%, Insoluble Residue 0.95%, Soluble SiO₂ Content of Aggregates 0.22%

Concrete B. Cement CEM I-52.5 R. 230 kg/m³, Cement: Fines: Coarse: 1.0:2.7:3.5, Aggregates: Fine, sand and Coarse (maximum 18 mm.), Granitic, w/c 0.6, Bulk Density 2190 kg/m³, Soluble SiO₂ Content of Cement 19.5%, Insoluble Residue 87.0% Soluble SiO₂ Content of Aggregates 0.25% The obtained concretes were put in 100 mm cubic metal moulds. They were vibrated and cured after conservation in wet chambers at 20°C immersed at the same temperature for six months. The aggregates of concretes A and B were tested with others current aggregates. The values are shown in Table 3.

At the end of this period, they were air-dried and sawed into six slices, measuring 100 × 100 × 16 mm. The six slices were divided into two groups of three slices, in an alternate fashion, so that each group was a representative sample of the concrete that was as homogeneous as possible.

Each slice from the other series of three slices of A and B concretes was sawed into three parts so that each slice provided four pieces equivalent to a mass of 150 to 400 g. Six of the nine pieces obtained from

| Table 2. Thermal analysis of the pure pastes of clinker and cements. |
|-----------------|--------|--------|-----------------|---------|---------|
|                | Δ102°–410°C | Xi    | σ     | C.V.     | Δ410°–550°C | Δ550°–950°C |
| Clinker A      | 7.2    | 5.6   | 2.2   | 7.2      | 0.12      | 1.7%       |
|                | 7.4    | 5.2   | 2.8   | 7.4      | 0.12      | 1.7%       |
|                | 7.1    | 5.1   | 2.7   | 7.1      | 0.12      | 1.7%       |
| Cement 3.3% SO₃| 6.3    | 3.9   | 2.4   | 6.2      | 3.8       | 2.3        |
|                | 6.4    | 3.8   | 2.3   | 6.2      | 3.8       | 2.3        |
|                | 6.2    | 3.9   | 2.4   | 6.2      | 3.9       | 2.4        |
| Clinker B      | 7.9    | 5.2   | 2.8   | 8.0      | 5.2       | 2.8        |
|                | 8.1    | 5.3   | 2.7   | 8.1      | 5.3       | 2.7        |
| Cement 3.3% SO₃| 5.6    | 4.1   | 2.4   | 5.6      | 4.0       | 2.5        |
|                | 5.4    | 4.1   | 2.5   | 5.6      | 4.0       | 2.5        |
|                | 5.6    | 4.0   | 2.5   | 5.6      | 4.0       | 2.5        |
| Clinker C (White) | 8.3    | 5.7   | 2.7   | 8.2      | 5.8       | 2.7        |
|                | 8.2    | 5.8   | 2.6   | 8.2      | 5.8       | 2.6        |
|                | 8.2    | 5.8   | 2.6   | 8.2      | 5.8       | 2.6        |
| Cement 3.3% SO₃| 7.4    | 5.3   | 2.8   | 7.4      | 5.3       | 2.8        |
|                | 7.3    | 5.2   | 2.7   | 7.3      | 5.2       | 2.7        |
|                | 7.4    | 5.2   | 2.7   | 7.4      | 5.2       | 2.7        |

Weight of Each Micro-Cube 15 to 16 g. Precision Weighing 0.001 g. Scale Semi-Micro.
All results have been referred to dehydrated mass on 102°C.

The typical cements were: two type I cements: CEM I 52.5 R; one cement type I, white: CEM BL I 52.5 N.
each group A and B, were analysed by the proposed TGMA method. These were dried to constant weight in an oven at 102°C, under the conditions described in the procedure and the mass of each one was determined with a 0.02 g accuracy.

Then the losses on ignition from 102°C to 410°C were determined under the conditions described in the method.

Their respective physical parameters were determined for the remaining three parts of each, in accordance with ASTM, [ASTM C-642, 2001] in the described manner, as the densities of the micro-cubes modified by boiling in vacuum at a maximum temperature of 35°C, to avoid phases transformations.

The results are given in Table 4 and 5.

One series of group A, together with one series of group B were fully ground down to a fineness of just below 63 microns. The amount of cement in the homogenised sample was determined using the author’s method [Gomà, 2005 & Gomà, 2006] for the determination of quantity of cement from the soluble silica method.

The results are shown in table 6.

9 DISCUSSION OF THE RESULTS

The statistical results of the analysed samples under conditions of repeatability provide very precise results on a semi-micro scale for cements with 15 g of sample and an accuracy of 0.001 g. This accuracy was established with pure cement samples that were completely homogeneous in nature. The actual repeatability accuracy is the equivalent of concrete samples ground down to a homogeneous fineness that passes completely through a 75-micron sieve. The comparison of these results with those obtained by the Gomà soluble silica method [Gomà, 2005 & Gomà, 2006] on the same
homogeneous samples confirms its validity. If the accuracy of the procedure is good, then when applied on a macro scale to whole concrete pieces, the obtained dispersions will be attributable to variations in the composition of the concrete poured and settled onsite. The ability of being able to work with whole samples, without prior treatment provides the possibility of quickly establishing the composition dispersion of the concrete poured onsite in a simple and operational fashion. It is, perhaps, the lack of flexible methods that is responsible for the scarce amount of data on concrete parameters both on poured-onsite concrete settled and cured, and in the field studies. The influence of interference is controllable and can be determined in each test.

The DTA diagrams prepared for pure pastes of the analysed cements provide information about the composition of their dominant phases. The evidence can be seen in the displacement of the endothermic peak towards higher temperatures when the ettringite proportion is increased. The determination of the SO$_3$/C$_3$ ratio in aged field concretes, which is recommended by many authors and specially by [Hime, 1996 & Hime, 2006], provides the possibility of comparison in the diagnosis of pathologies caused by sulphate attack.

10 CONCLUSIONS

The proposed procedure has good repeatability accuracy on both tests scales. It is an operational procedure because of its simplicity and speed. – TGMA procedure allows multiple simultaneous determinations that produce highly effective mean values.

– The application of the two methods, that of the soluble silica from the same author [Gomà, 2005] and the TGMA, to the same representative samples of concretes in a cured or hardened state in the short term, provides information on the cured state in function of time and its efficiency.

– The similar application of the “Soluble Fractions” method [Gomà, 1999 & Gomà, 2007] from the same cited author provides information on whether there are any active additions or not in the concrete.

To also identify the type of addition, its quantitative or semi-quantitative evaluation, as applicable to the discrimination the amount of silica in the active concrete additions from the silica in the cement.

– The combination of these methods allows the evaluation of the composition of concretes with active additions, its water-cementitious material ratio and its degree of hydration in current sustainable construction options.

ACKNOWLEDGEMENTS

To Professor Carmen Iduarte Despuig, my dear wife and colleague since university, in the 50s, for her expertise in preparing the calculations, her review, and for our shared work of an entire lifetime.

To ASLAND, S.A., Portland cements in Spain, for the technical equipment they have provided since the 70s in order to carry out this research in my position.
as Managing Director of their Central Research Laboratories in Barcelona, Catalonia (Spain).

REFERENCES

FUZZY control, Fuji electric Co.Ltd Model PXW.
Gomà, F., Sobre la determinacyó gravimètrica de la sílice, Tesi Doctoral, 1975, Universitat de Barcelona, Departament de Química Analítica. (Lectura, 04-12-1975), Seminari de Ciències Químiques.
Gomà, F., Discovery of the causes de error in the determina- tion of the soluble silica in hardened concrete with their correction to obtain an accurate water/cement ratio. Seventh CANMET/ACI International Conference on durability of Concrete Supplementary papers pp. 159–171 June, Canada 2.006
Assessment of the fracture of three-point bending concrete specimens

A. Ziliukas
Department of Mechanics and Solids, Kaunas University of Technology, Kaunas, Lithuania

A. Augonis
Department of Building Materials, Kaunas University of Technology, Kaunas, Lithuania

ABSTRACT: In structural concrete elements opening up of structural and non-structural cracks can lead to irreversible damage of the structure. A more precise prediction of the process of fracture would help us to avoid this occurring and the terrible consequences of an accident arising from it. Methods of assessing concrete fracture are not sufficient in order to determine the mechanical behaviour of concrete in case defects appear in it. The available analytical methods not always allow us to calculate all parameters of fracture while the experimental methods are expensive and sometimes their application is infeasible. In the work, along with analytical and experimental methods, digital methods are suggested. Here it was aimed at calculating the critical crack opening and fracture energy of concrete prisms being bent. In accordance with three-point bending scheme concrete specimens with cut cracks inside were bent. Concrete specimens have been formed from different mix. Bending force was added at deformation speed of 0.4 mm/min. At the same time the changing deflection of concrete specimen and lower opening of the crack were recorded. Stress intensity factor $K_I$ was calculated by the ANSYS computer program because the traditional analytical equation was found to be inappropriate due to the distance between the supports (250 mm) and measurements (100 × 100 × 400 mm). With the aid of the stress intensity factor $K_I$ derived by the computer program and crack mouth opening displacement (CMOD) the determined experimentally critical crack opening were calculated. Also calculation of fracture energy was made.

1 INTRODUCTION

Concrete is a brittle material and non-resistant to tensile stress. Therefore cracks opened during stress rapidly develop. Cracks can result from mechanical and temperature effects. In order concrete might be more plastic and that its fracture might be not so sudden it needs reinforcing. For this purpose fibres are used. Mechanical and strain properties of concrete are usually improved by fibres of 25–80 mm long and 0.8–2 mm in diameter. Their quantity in concrete mix depends upon the type of fibre and the effect aimed at [Augonis & Stuopys 2005].

In order to determine the effectiveness of steel fibres three or four point bending tests are frequently carried out on concrete beams reinforced by fibres. Effective are considered such fibres and such a quantity of them in case in load-deflection curves (beam affected by bending load) after crack opening the loading does not decrease or decrease slightly and up to the final beam fracture remains as large as it is possible [Augonis & Stuopys 2005].

In designing the composition of the concrete mix it is important to choose appropriate granulometry of its aggregates and pay greater attention on small aggregates fractions. This would enable not only segregation and bleeding to be avoided but also cement quantity to be decreased for obtaining the specified strength of concrete. For this one of the ways is the use of fly ash in concrete composition. The use of fly ash depends on its chemical and granulometrical content and on the way of its collection. Chemical activity of fly ash is one of the most important properties determining the field of its use. In comparison to other active mineral aggregates and SiO$_2$ silica fume, fly ash of heat power plants is more chemically active than rotten stone, geize or diatomite and by far cheaper than SiO$_2$ silica fume.

Some minerals containing fly ash in concrete (for example amorphous SiO$_2$, metakaolin, CaO and MgO) can make chemical reactions with cement hydration products and form stable hydro silicate compounds. Therefore fly ash is to be attributed to active microfillers group because its active part turns into hydro silicate compounds while the other part that does not react with the binding agent remains as on inert microfiller [Rudzionis et al. 2005].

The stress intensity factors ($K_{IC}$, $K_{IIC}$, $K_{IIIC}$) depend upon the mode type (when the crack is opened by
tearing, by splitting shear and by revolving it). At the top of the crack due to its growth crack opening is constant and it is called CTODc. The energy necessary for the start of crack growth is called Gc and the energy necessary for the crack to separate two planes is called Gf. In this work it was aimed at calculating the parameters (stress intensity factor KIC, crack tip opening displacement CTODC and fracture energies Gf and Gc) of concrete fracture. Also characteristic length, lch = EGf/ft², was calculated, which indicates concrete’s brittleness. For this purpose concrete beams with a notch below were formed and a three point bending (TPB) test was carried out. Application of fibres and fly ash in concrete mix influenced these parameters of fracture.

2 MATERIALS AND METHODS

For the reinforcing of concrete steel fibres Dramix RC-80/60-BN of round diameter with bent ends have been chosen. Concrete mix has been prepared from Portland cement CEM I 42.5 R, gravel of 4/16 mm fraction, sand of 0/4 mm fraction and water. Also superplasticizer Rebamix F2 has been used. A part of Portland cement was replaced with fly ash (10 and 20 %). Concrete mix content is presented in Table 1.

The size of the TPB specimens was 100 x 100 x 400 mm, with the effective span of 250 mm. The depth of the notch was 10 mm. Concrete beams were bent at the speed of 0.4 mm/min. Concrete deflection, CMOD and time till the major crack (first load peak) opening were measured experimentally. Fracture energy Gf was calculated according to the obtained load – deflection curve. In different experiments the compressive strength fc and the modulus of elasticity E were obtained (Table 3). All the experiments were carried out after 28 days of hardening.

KIC was calculated by a digital way according to the TPB specimen dimensions and maximal bending load determined experimentally. The models were constructed and calculations were made with the ANSYS programme.

3 THE RESULTS OF THE EXPERIMENTS

Experimental results obtained during the major crack opening are presented in Table 2.

While bending concrete at a constant speed of 0.4 mm/min the concrete (C) without fibres fractured most rapidly (in 27 seconds). The CMODc of this concrete was also largest (147 µm) while deflection the least (109 µm). The first load peak in the concrete (FRC) reinforced by fibres appeared in 42 seconds. It occurred when the deflection was 180 µm and the CMODc – 138 µm. Of interest is the fact that at the

moment of major crack opening in concrete without fibres the deflection was less than CMODc while in concrete with fibres vice versa (Table 2).

While bending concrete beams without fibres fracture is sudden (Figure 1). Used fibres increase strain of concrete beams till the major crack opening and after it. In the bending curve of concrete beams reinforced by fibres it is possible to distinguish several zones: increase in deflection and bending load till the major crack opening, more or less decrease in sudden bending load, the zone of an increase in load and deflection and the zone of an increase in stable deflection when the force remains constant or slowly diminishes (Figure 2).

In other composition of concrete with fly ash (F10 and F20) the deflection slightly diminished (over 14 µm) than in concrete without fly ash (F0). A change of cement over fly ash reduced the time interval till the major crack opening (Table 2). The curves of deflection of bending concrete specimens with fly ash are shown in Figure 3.

4 FRACTURE PARAMETERS

The major parameters of fracture are stress intensity factors (KIC, KIIC, KIIIC, where I, II, III – different
modes), CTODC, which is a constant value and fracture energies GF and GC. However, in the literature there are more parameters indicating fracture, such as fracture process zone (FPZ) and effective fracture process zone length cf [Gettu et al. 1998, Zhang & Wu 1999]. The parameters of fracture calculated in this work are presented in Table 4.

4.1 Stress intensity factor $K_{IC}$

Calculated values of $K_{IC}$ are strongly influenced by the content of aggregates and are size dependent [Carpinteri & Ingraffea 1984, Carpinteri 1986, Zhang & Li 2004]. For specimens of three point bending $K_{IC}$ can be calculated according to equation (1) given below [Karihaloo 1995].

$$K_{IC} = 6YM_{max}\sqrt{a}/(BW^2)$$  \hspace{1cm} (1)

where $Y$ – function of geometry, $M_{max}$ – bending moment due to the maximally applied load, $B$ – width of the beam, $W$ – depth of the beam, $a$ – notch depth.

$$Y = A_6 + A_1(a/W) + A_2(a/W)^2 + A_3(a/W)^3 + A_4(a/W)^4$$  \hspace{1cm} (2)

$A_i$ coefficients is known only for two span to depth ratios (S/W = 4 and 8). However, linear interpolation of $Y$ is permitted within, and outside of, these two ratios denoted $Y_4$ and $Y_8$ respectively [Karihaloo 1995].

$$Y = Y_4 + \frac{(S/W)-4}{4}[Y_8 - Y_4]$$  \hspace{1cm} (3)

### Table 3. Mechanical properties of determined by experiments.

<table>
<thead>
<tr>
<th>Denotation</th>
<th>Compressive strength, $f_c$ (MPa)</th>
<th>Tensile strength*, $f_t$ (MPa)</th>
<th>Modulus of elasticity, E (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>44.1</td>
<td>2.19</td>
<td>30.935</td>
</tr>
<tr>
<td>FRC</td>
<td>41.8</td>
<td>2.91</td>
<td>33.030</td>
</tr>
<tr>
<td>F0</td>
<td>53.6</td>
<td>2.57</td>
<td>32.282</td>
</tr>
<tr>
<td>F10</td>
<td>49.1</td>
<td>2.17</td>
<td>31.385</td>
</tr>
<tr>
<td>F20</td>
<td>51.2</td>
<td>2.20</td>
<td>35.079</td>
</tr>
</tbody>
</table>

* Where, tensile strength calculated according to equation (6).

### Table 4. Fracture properties of concrete.

<table>
<thead>
<tr>
<th>Denotation</th>
<th>Fracture toughness, $K_{IC}$ (MPa-m$^{1/2}$)</th>
<th>Fracture energies, $G_F$ and $G_C$ (N/m)</th>
<th>Characteristic length, $l_{ch}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>0.524 (0.533)*</td>
<td>8.9 (137)</td>
<td>884</td>
</tr>
<tr>
<td>FRC</td>
<td>0.697 (0.710)*</td>
<td>14.7 (6500)**</td>
<td>25353</td>
</tr>
<tr>
<td>F0</td>
<td>0.615 (0.625)*</td>
<td>11.7 (146)</td>
<td>714</td>
</tr>
<tr>
<td>F10</td>
<td>0.521</td>
<td>8.6</td>
<td>906</td>
</tr>
<tr>
<td>F20</td>
<td>0.527 (0.530)*</td>
<td>7.9</td>
<td>899</td>
</tr>
</tbody>
</table>

* Where, $K_{IC}$ was calculated by a digital way.
** Where, fracture energy $G_F$ is calculated up to the deflection of 5 mm.
With the aid of these equations $K_{IC}$ was calculated and was found to be 0.524 N/m²m¹/² for the concrete (C) without fibres and 0.697 N/m²m¹/² for the concrete (FRC) with steel fibres. For composition (F₀) $K_{IC}$ of concrete 0.615 N/m²m¹/² was obtained while for that of concrete with fly ash (F₁₀ or F₂₀) 0.521 N/m²m¹/² and 0.527 N/m²m¹/², respectively. As seen from Table 4, the results are very similar to those obtained digitally. In case it is infeasible to calculate $K_{IC}$ according to the known equations its calculation can be made by digital modelling.

4.2 Critical crack tip opening displacement ($CTOD_{C}$)

In accordance with the British standard BSI-DD19 and BSI-COD for the metals typical of brittle fraction $CTOD_{C}$ can be calculated when $K_{IC}$ and $CMOD_{C}$ are known [Ziliukas 1988]. These values are determined by equations (4–8) doing three point bending test [Ziliukas 1988]. In accordance with these methods an attempt was made to calculate $CTOD_{C}$ of concrete.

$$K_{IC} = \frac{Y \cdot P}{B \cdot W^{1/2}}$$ (4)

$Y$ – refers to the constant without dimensions which depends upon $a/W$ ratio.

$$V' = γ \cdot σ_{max} \cdot W \cdot (1 - v^2)/E$$ (5)

Where $σ_{max}$ is maximal tensile stress calculated valuating the plasticity (in the equation for metals $σ_{yd}$ is used) of concrete being bent.

$$σ_{max} = 3.5 \cdot M_{max} / B \cdot W^2 [N/m²]$$ (6)

$$CMOD_{cr} ≥ 2 \cdot V'$$ (7)

$$CTOD_{cr} = \frac{0.45 \cdot (W - a)}{0.45W + 0.55a + H_0} \cdot [CMOD_{cr} - V']$$ (8)

where $H_0$ – thickness of the clip gauge holder.

According to this equation $CTOD_{C} = 0.0790$ mm is calculated for the concrete (C) without fibres and $CTOD_{C} = 0.0715$ mm for the concrete (FRC) with fibres. These values are larger twice or more in comparison to the values of $CTOD_{C}$ presented in the literature [Bazant & Becq-Giraudon 2002, Kim et al. 2004, Reis & Ferreira 2003] for concrete. Nevertheless, $CTOD_{C}$ can be calculated according to the equations presented below. In case distinct $CTOD_{C}$ correlation between the methods is not obtained the method assigned for calculating $CTOD_{C}$ of very brittle metals should be rejected. For concrete $CTOD_{C}$ is calculated according to equation (9) [Karihaloo 1995; Lam et al. 1998].

$$\frac{6P_{max} \cdot Stc}{EBW^2} \cdot \frac{1}{\gamma t} (1 - β^2) -(1.149α + 1.081β)(β - β^2)^{1/2}$$ (9)

where $ae$ – the effective crack length.

$$α = (a + H_0)/(W + H_0)$$ and $β = a/a_e$ (10)

$$V1(α) = 0.76 - 2.28α + 3.87α^2 - 2.04α^3 + 0.66/(1 - α)^2$$ (11)

4.3 Fracture energies $G_F$ and $G_C$

Other fracture models, such as Bazant’s size effect model or the two parameter fracture model measure fracture energy near the peak load [Landis & Baillon 2002]. The total fracture energy $G_F$ were determined from the areas under the load-deflection curves of the three-point bending test [Bazant et al. 2002, Guinea et al. 2002, Wong et al. 1999]. Physically, we interpret this energy as the two faces of a unit surface of craze apart by a distance [Guo et al. 1999]. Fracture energy $G_F$ can be calculated according to the load-deflection curve when the fracturing cross-section area of a specimen is known. $G_F$ calculated in this way is presented in Table 4. Fracture mechanic parameters like the fracture energy $G_F$ and the characteristic length, $l_{ch}$ can be calculated for steel fibre reinforced concrete [Kutzing & Konig 1999]. However, in the concrete reinforced by fibres its value is very large since fibres hinder complete fracture of a specimen, although the major crack is opened for a long time. The large the value of $G_F$, the more significant the effectiveness of fibres is. In the literature other ways of calculating fracture energy $G_F$ are presented [Karihaloo 1995].

For elastic brittle material $G_F = G_C$, however, for concrete, which is a quasi-brittle material, $G_F$ is higher than $G_C$ because of the stable crack growth before failure takes place in case of quasi-brittle materials. Moreover, $G_F$ is not constant but vary with the size – they increase with an increase in depth of the beam. The variability of $G_F$ with specimen size and for its significant departure from $G_C$ is mainly due to the violation of the two basic assumptions: firstly the work done by the external load goes solely into stable crack extension and secondly the energy required to create a crack of unit area is independent of geometry and loading configuration. The critical energy release rate $G_C$ is related to $K_{IC}$ as [Bharatkumar et al. 2005]:

$$G_C = (K_{IC})^2/E$$ (12)
4.4 Characteristic length, \( l_{ch} \)

The characteristic length was first defined by Hillerborg. To evaluate concrete brittleness calculation of characteristic length, \( l_{ch} \) was suggested [Guinea et al. 2002]

\[
l_{ch} = E \cdot G_F / f_t^2
\]

(13)

The less the value of \( l_{ch} \) is obtained the more brittle the material is. If tensile strength \( f_t \) is unknown in the literature [Guinea et al. 2002, Karhalalo 1995] ways of calculating it are presented when only compressive strength \( f_C \) is known. Since fracture energy \( G_F \) of concrete reinforced with fibres is large characteristic length, \( l_{ch} \) is also large. Its results are presented in Table 4.

5 CONCLUSIONS

A comparison was made between the concrete and the concrete reinforced with steel fibres after TPB testing had been done. At the moment of major crack opening the CMODC was obtained more than 1/3 larger (180μm) in the concrete reinforced by steel fibres than in the concrete without fibres. All calculated fracture parameters of the concrete reinforced by Dramix RC-80/60-BN steel fibres are large. \( K_{IC} \) necessary for forming critical crack length or critical load was found to be 25 % larger than concrete without fibres. Fracture energy \( G_F \) and characteristic length, \( l_{ch} \) were also large. Fibres increased fracture energy \( G_C \) up to 40%. The results show that concrete reinforced with steel fibres is more resistant to fracture.

The replacement of cement with fly ash very insignificantly reduced the compressive strength, however bending strength was relatively lessened up to 15%. Therefore stress intensity factor \( K_{IC} \) diminished too. Fly ash decreased fracture energies \( G_F \) and \( G_C \). However fly ash when used in large amount reduced characteristic length, \( l_{ch} \) up to 21 %.

REFERENCES


The economics of recycling in the US construction industry

Christian Meyer
Columbia University, New York, USA

ABSTRACT: In a free-market society, business decisions are driven by economic incentives, i.e. mostly profit, subject to government regulations and interventions. Decisions about the use of recycled materials in construction are no exception. In fact, based on purely economic factors, virgin materials are usually preferable for various reasons. But the playing field is often skewed, because the US economy does not value sustainability issues such as disposal costs and life cycle assessment as it should. This becomes obvious when comparing the situation in the United States with that of Japan and many European countries with greatly different degrees of governmental interventions.

This paper addresses some of the economic drivers that influence the extent to which recycled materials are utilized in the American construction industry. It examines the major differences between the United States and other developed countries, which have a number of root causes. Recommendations are made regarding actions necessary for the United States to “catch up” with countries such as Germany, The Netherlands or Japan.

1 INTRODUCTION

The United States is the wealthiest nation on earth, and it has not become this by chance. It did so as the result of several factors, namely a vast, relatively thinly populated continent with immense natural riches, opened up by adventurous and entrepreneurial pioneers. To some extent similar conditions existed at approximately the same time in South America and in the Asian part of Russia. Here is not the place to analyze the causes of why, given the similar initial conditions, the outcomes as measured by economic wealth were so vastly different. But it is safe to state that in each case the development was based largely on a comparatively wasteful exploitation of natural resources, basically without any consideration of conservation and sustainable development. It might also be mentioned that, except for a disastrous Civil War, the development of the North American continent could proceed in relative peace, especially compared with Europe, where, for example, the shortages caused by two world wars made the recycling and reuse of resources at times a matter of survival.

In the United States, the concepts of recycling and the reuse of natural and other materials have traditionally been of low priority if not nonexistent. This is no longer the case. A dramatic change in attitude can be felt today throughout the country. This change came relatively suddenly, gaining significant momentum some time in the early 1970s. A key event was the celebration of Earth Day in 1975, when a large part of the American public became aware of the limits of the nation’s resources and grew concerned about the deteriorating environment, whether soil, water, or air. If any single picture could capture the cause of this awakening, it was the famous photo taken by our astronauts of “Spaceship Earth”, which dramatically illustrated the fact that our planet is indeed finite in size and in resources, that we had better learn how to live within our means, and that these problems were global in nature, affecting all of humanity.

2 ECONOMICS 101

The economics of recycling cannot be discussed without first stating a few elementary facts. Above all, in a free-market economy the price of a service or commodity is determined by supply and demand. But even in such a free-market economy, government can and regularly does intervene with incentives (for example, in the form of tax write-offs) and disincentives, such as fees, penalties, or outright prohibitions, if it thinks this is in the best interest of the public. In general, government, with its legislative power, defines the boundary conditions for both supply and demand to develop. If these boundary conditions remain constant, changes in supply and demand for certain goods and services take place at their natural and usually slow pace, unless important inventions or innovations with wide ranging consequences accelerate that pace. In an open society
with representative government, also changes in public perception will eventually lead to changes of the legislative framework, under which the economy develops. For example, once the causal relationships between industrial air pollution and public health were established beyond a doubt, it was just a matter of time until public pressure led to legislation of the Federal Clean Air Act.

At present we are experiencing a fundamental change in public attitudes, spearheaded by a vocal and growing environmental community that has taken a strong foothold in the building and construction industry. This development is best illustrated by the exponential growth of membership of and the significant influence exerted by the U.S. Green Building Council, to name just one, maybe the largest pertinent organization. Thanks to the USGBC and other similar organizations we are now witnessing a fundamental overhaul of basic paradigms and value systems that already show signs of changing the entire building and construction industry. The current U.S. Administration, rather than actively promoting change, has primarily been only a reactive player at best, but there are indications that both the legislature and the executive branches of government are increasingly accommodating “the will of the people”.

One example to illustrate how government intervention can quickly change the balance between supply and demand is the requirement that manufacturers of certain products be required by law to take back their products at the end of their useful life. European car manufacturers have already learned years ago to redesign their automobiles such that they can be disassembled with ease and such that most of the parts and components are readily recycled or even reused. These car manufacturers did not do this out of concern for the environment or because of modest potential economic benefits, but simply because they were required by law to do so. In other situations it was possible for such manufacturers to reap economic benefits without legislative requirements. For example, the recovery of rare and precious metals can become economically attractive as soon as the market value of such materials exceeds a certain threshold, or traditional sources for such materials are suddenly becoming inaccessible.

Within the general national economy, the construction industry occupies a special place, because it moves by far the largest quantities of material, and these are generally bulk materials that cost relatively little, compared with most manufactured goods. Concrete, for example, is the most widely used material worldwide. In the United States it is estimated that well over 500 million cubic yards of it are produced annually. This translates to almost 2 cubic yards for every man, woman and child each year, and it may cost just $75 per cubic yard, or approximately 2 cent per pound (in New York City it retails for a little more, ranging from about $100 to $115 per cubic yard). There are not many examples of any commodities that inexpensive. (For comparison, at the time of this writing, regular gasoline costs about 30 cent per pound at the pump.) These parameters, namely extremely large quantities and very low costs (and profit margins), are characteristic of the construction materials industry and define both the difficulties and opportunities for any efforts of recycling.

3 CASE IN POINT: THE CONCRETE INDUSTRY

When analyzing the economics of recycling in the concrete industry, the various ingredients of concrete need to be analyzed separately with regard to their “value”, as expressed by their market price, as well as the profit margin. Discounting for now the various chemical admixtures, cement is by far the most expensive ingredient, at about $110 per ton in the New York area. The aggregate retails for about $20 per cubic yard, but that depends to a large extent on the cost of transportation. This means, cement may cost 10 times as much as aggregate. Therefore any efforts to use supplementary cementitious materials that are less costly than Portland cement are likely to realize economic benefits, regardless of any other advantages, such as improved performance or environmental benefits. This explains why no governmental intervention was required for the industry to recognize the advantages of using fly ash or ground granulated blast furnace slag as partial replacement of Portland cement.

The economic situation is quite different for aggregate, because this is so much less expensive than cement. Based on current economics considerations alone, it will be difficult to justify the substitution of a recycled aggregate for virgin material, whether natural gravel or crushed stone. Let us consider several materials that might serve as substitute for natural aggregate.

3.1 Recycled concrete aggregate

Construction waste constitutes a sizeable fraction of all solid waste in developed countries. In the United States it is estimated that about 150 million tons of construction waste is generated annually, and concrete debris, whether from demolished buildings or highway pavements, constitutes slightly over 50% of this amount. Almost 60% of this amount, or 45 million tons, are landfilled, with tipping fees as high as $50 per cubic yard or more. Note that such disposal costs alone may amount to more than half of the cost of new concrete. Still, recycling such concrete in the form of aggregate for new concrete is more the exception than the rule in the United States. In Japan, on the other hand, landfill capacity is rapidly diminishing, and so is the availability
of suitable virgin limestone aggregate. These are sufficient reasons for the Japanese concrete industry to rely increasingly on recycled concrete aggregate.

The economic feasibility of recycled concrete aggregate may be described in simplified form by defining the cost of virgin aggregate, \( C_{va} \), and of recycled concrete aggregate, \( C_{ra} \), as follows:

\[
C_{va} = C_{pv} + C_{tv} (+ C_d) \tag{1}
\]

\[
C_{ra} = C_{pr} + C_{tr} + C_{sd} \tag{2}
\]

where:

- \( C_{pv} \) = cost of processing virgin aggregate (e.g. crushing and grading),
- \( C_{tv} \) = cost of transportation of virgin aggregate,
- \( C_d \) = cost of disposal of demolished concrete (if there is such),
- \( C_{pr} \) = cost of processing recycled concrete aggregate (sorting, cleaning, crushing, grading),
- \( C_{tr} \) = cost of transportation of recycled concrete aggregate, and
- \( C_{sd} \) = a strength or quality deficit that needs to be quantified, since it is known that the performance of concrete made with recycled concrete aggregate is generally inferior to that of concrete made with virgin aggregate.

Purely economic factors, i.e., discounting any additional factors such as local, State, or Federal rules or regulations, thus will make the use of recycled concrete aggregate a feasible proposition if and only if \( C_{ra} < C_{va} \).

And even in this case, other factors such as old habits or business relationships with long-time suppliers may outweigh a slight economic advantage, thus preventing a concrete producer from switching supplies.

The cost of any bulk material in general and aggregate in particular depends to a large extent on the cost of transportation, which is basically proportional to the distance to the nearest source of suitable aggregate. Thus it is obvious that the distances to the sources of virgin and recycled concrete aggregate alone may be decisive in determining the economic feasibility of using recycled concrete aggregate.

### 3.2 Specialty aggregates

The economics can change considerably when considering specialty aggregates, which endow the concrete with properties that the regular aggregate does not. The most important example is lightweight aggregate, which is popular for either the reduction of dead weight or improved thermal properties or both. The improved performance has a certain economic value for which the customer is willing to pay a premium. For example, in the New York City area, lightweight aggregate may cost three times as much as regular aggregate.

Aggregates for special architectural concrete applications are governed by their own rules of economics. For example, terrazzo is a very popular material for floors in buildings, and producers can pay a premium for specialty aggregates such as marble ships imported from Italy, because they can pass on these cost premiums to the owners for the special architectural effects they produce. The cost of terrazzo concrete is a multiple of that of regular concrete largely a result of the increased labor cost involved), which allows the producer to pay a considerable premium for the marble chips.

This last example points the way in which the use of recycled materials may make economic sense. If it is possible to identify certain concrete properties made possible with the use of such recycled materials no further economic incentives will be necessary to promote their use. For example, preliminary studies at Columbia University have indicated that material dredged from the Port of New York and New Jersey (which is typically highly contaminated), after undergoing a special treatment process, can serve as a coating for wall panels with superior fire retarding and smoke suppressing properties. Other coatings with similar performance characteristics may cost an order of magnitude more than the dredged material, which otherwise needs to be deposited at great cost. This example illustrates that targeted research may lead to breakthroughs that do not require government intervention to level the playing field.

### 4 CASE IN POINT: WASTE GLASS AS AGGREGATE

#### 4.1 Technical aspects

The use of post-consumer glass has been contemplated for some time, but early attempts have failed because of the reaction between the alkali in the cement and the silica in the glass, the so-called alkali-silica reaction (ASR). The resulting ASR-product, a gel, swells in the presence of moisture, which can lead to severe damage of the concrete. This problem is a technical one and can be solved. As a matter of fact, extensive studies were undertaken at Columbia University, and many test results have been reproduced at other universities. Among the proven methods to avoid ASR or its damaging effects is the use of metakaolin or fly ash as partial substitute of Portland cement.

Before discussing specific concrete products made with recycled glass aggregate it is helpful to point to the various technical advantages of such aggregate:

1. Glass has no water absorption to speak of. This is a plus, because it improves the flow properties of fresh concrete and therefore allows the use of a lower water/cement ratio.
2. Glass has excellent hardness and abrasion resistance, which makes it an ideal aggregate for floor tiles and paving stones.

3. Glass is a very durable material and resistant to many chemicals.

4. Maybe most important is the esthetic potential of colored glass and the various effects produced by light reflection and refraction. If used in combination with white cement and color pigments, it gives architects and other design professionals an important new tool with basically unlimited combinations of color-coordinated cement matrix and glass particles.

5. It has also been shown that very finely ground glass powder has pozzolanic properties. This means that part of the Portland cement may be replaced by glass powder.

6. Glass powder has been shown to be a very good filler material.

4.2 Economic aspects

The use of post-consumer glass as aggregate for concrete products is associated with a number of cost items. The largest cost factor in large metropolitan areas such as New York City is the collection. New York City pays recyclers well over $100 per ton to collect and dispose of the glass, which residents place at the curbside, commingled with metals and plastic containers. The glass needs to be separated from the other materials before it can be cleaned and crushed. Sorting the glass by color increases its market value. For example, the glass industry will take back only clear glass (flint) for remelting. Automatic equipment is available to sort glass particles by color. Cleaning and crushing operations are also performed automatically. Compared with the cost of collection, these other costs of recycling glass for use as an aggregate for concrete are often minor, while the cost of transportation, as stated earlier, is very much a function of distance between the point of collection and the concrete producer.

In spite of the various technical advantages of glass aggregate listed above, practical applications are subject to important economic constraints. To evaluate these, we have to categorize such applications as either commodity products or value-added products. Commodity products, such as concrete masonry units or paving stones, are characterized by large volumes, low values and low profit margins, and the use of glass as aggregate is not very likely to affect the market value of the end product. For that reason, a concrete producer will not use recycled glass aggregate if he has to pay more for it than for the aggregate it replaces, that is, very inexpensive sand and gravel or crushed stone. Although such commodity products have the potential of absorbing large amounts of waste glass (one single paving stone manufacturer in the New York area could single-handedly use all waste glass collected in New York City), the economic outlook is not very promising. To avoid costly ASR-suppressing admixtures such as metakaolin, one might grind the glass fine enough to pass mesh size #50. But in this case it is be impossible to see the glass, so that the potential of added value due to the esthetic aspects of colored glass aggregate would be lost.

Value-added products, such as terrazzo tiles and table top counters, are characterized by low values but high profit margins. In this case, the glass would replace much more expensive specialty aggregates. To make full use of the esthetic benefits, larger size glass particles are needed, which call for ASR-suppressing additives. But the market value of the end product is considerably higher than products using regular aggregate, so that the added costs are immaterial. It is known that customers are willing to pay substantial premiums for high-quality products, such as natural stone. A large slab of Italian marble for a conference table may retail for hundreds if not thousands of dollars. A similar table top counter made of glass concrete may not fetch the same kind of price, but still a price high enough so that the material costs become all but irrelevant. An added advantage is the fact that such a slab may not only be engineered to given specifications, it can also be reinforced with short and randomly oriented fibers (for example, recycled carpet fibers) to improve the material’s toughness and energy dissipation potential. It also can be reinforced with textile fabrics – a new and very promising technology. Such fabrics of fiber meshes can even be prestressed to further enhance the mechanical performance of the products.

Other economic factors are even less quantifiable, such as the novelty aspect and the label of “environmental friendliness”. For example, the LEED™ rating system of the U.S. Green Building Council is gaining increasing popularity among builders and developers, who recognize the value, both tangible and intangible, of a LEED rating. It is said that apartments in the Solaire, the first “green” high-rise residential building in the United States, fetch higher rents than comparable units in conventional high-rise buildings, simply because of the cachet associated with living in a “green building”. The LEED rating is based on credit points for a variety of environmentally friendly attributes such as energy-saving measures and use of recycled materials.

5 CONCLUSIONS

Engineering research is most gratifying if it leads to tangible results, especially if these results are compatible with the demands of sustainable development. To the researcher, “problems” and “difficulties” pose challenges to be overcome. The true test of the viability of a solution is to what extent it satisfies the economic aspects without the need of governmental intervention,
so that the market can decide the economic viability strictly on the basis of supply and demand. The same holds true when considering the use of recycled materials in construction. The key to success lies in the ability to identify properties inherent in the recycled materials which are unique or superior to those of traditional materials. However, such superior qualities are often not sufficient to overcome economic barriers, which all too often are based on traditional value systems that disregard the demands of sustainable development. To create a sustainable economy, it is the duty of governing bodies to assure a level playing field by including environmental costs, for example, by performing scientific life cycle cost analyses of materials and structures.
1 INTRODUCTION

Construction of the Kern Center began in April of 2003 and was completed in September of 2004. The new 210,000 square-foot athletic facility located in downtown Milwaukee, Wisconsin includes a basketball court, ice hockey rink, weight-lifting area, wrestling room, field house and office and classroom space. Based on invoices from the construction manager, Hunzinger Construction, over the 21 months it took to finish the new building more than 785 tons or an estimated 7,500 cubic yards, of waste materials were sent to a landfill rather than recycled, because of space constraints at the site. The purpose of this National Science Foundation (NSF) funded Research Experience for Undergraduates (REU) project was to determine the potential for cost savings and landfill diversion if the construction management company and MSOE had been able to recycle rather than landfill these materials. Actual trash invoices were used for quantities of construction waste. Average Wisconsin construction waste composition data from WasteCap Wisconsin, Inc., an environmental organization which has managed construction waste on many local construction projects, was used to estimate the composition of Kern Center waste materials. The composition assumed for this project was wood 28%, concrete/masonry 14%, drywall 13%, cardboard 8%, metal 8%, trash 28% and 4% reused. Assuming that it would have been possible to recycle 75% of the construction waste, it is estimated that between $19,400 and $25,800 could have been saved and over 590 tons or 5,800 cubic yards of waste materials diverted from local Wisconsin landfills. Based on this analysis, if issues with site constraints could have been resolved, we concluded that it would have been cost-effective to recycle construction waste during this project.

2 BACKGROUND ON CONSTRUCTION WASTE

According to Jenna Kunde of WasteCap Wisconsin, it is estimated that 12% of materials in landfills nationwide are made up of construction waste materials. Over 850,000 tons of construction and demolition debris is thrown in Wisconsin landfills each year; making up approximately 28% of overall Wisconsin landfill composition [WDNR].

There are seven basic categories into which most Wisconsin construction waste fits. They are wood, drywall, cardboard, metal, concrete and masonry, reuse and trash. EPA construction waste composition estimates are shown in Figure 1 and WasteCap Wisconsin waste composition estimates are shown in Figure 2. WasteCap Wisconsin waste composition estimates are based on the results of actual projects in the Milwaukee and Madison areas.

2.1 Wood

Waste wood scraps are often chipped, dyed and reused as landscaping mulch. Wood over four feet long can sometimes be reused as lumber. Once the wood is
turned into mulch, it can be colored for the cost of $3 per cubic yard and then sold at around $28 per cubic yard [Goodrich]. Wood with an energy content of 6930 Btu/lb can also be used as a fuel [JUCA].

2.2 Drywall
Scrap gypsum board completely free of contaminants can be ground up and used as a soil amendment for local farmers and landscapers or sent back to a plant to be made into new drywall. Recycling drywall is sometimes hard however because it is somewhat dependent on local need. The average value of gypsum used for agriculture is about $17.40 per ton [Founie].

2.3 Cardboard
Cardboard collected onsite is most often sorted, bundled and sold in bulk to be made into new products such as paper, cardboard, paper office supplies and packaging materials. The average market value per ton of cardboard during the time of construction was $73.83 [WDNR].

2.4 Metal
Scrap metal (mainly steel) is usually gathered and sold to a metal recycler to be processed and recycled into new products such as steel construction materials, auto bodies and steel cans. Steel is 100% recyclable and recycled metal often yields a profit when recycled even after a hauling cost is deducted. Ferrous metal prices during the time of construction averaged $98.68 per ton [Eminton].

2.5 Concrete & masonry
Excess concrete and masonry generated onsite is usually hauled to a concrete crusher to be made into backfill or aggregate in new concrete or rock. This crushed concrete can be used as aggregate in asphalt or road based gravel; by volume concrete contains an average of 60% to 70% aggregate [Portland Cement Association].

2.6 Reuse
Materials that can be reused onsite include wood pieces over four feet long, wooden pallets, soil from site work, and cardboard. Oftentimes the project landscaper can use mulch from waste wood, ground up gypsum and crushed rock for onsite landscaping projects [Kunde].

2.7 Aluminum cans and plastic bottles
Many construction sites also recycle aluminum cans and plastic bottles. According to the EPA, commingled cans and bottles make up on average .02 percent of the waste stream. While this is a very small percentage compared to the other materials, recycling cans and bottles has many advantages. Since most commingled waste is generated during lunch, most commingle can be collected in recycling containers set out near trash cans in the lunch area.

As well as being easy to recycle, recycling cans saves energy; it takes about 95 percent less energy to recycle an aluminum can than it does to make a whole new can from bauxite ore [Denver Recycles]. Aluminum cans have a very high average market value; during the time of construction aluminum cans were valued at $1000 per ton [WDNR].

2.8 Weight to volume conversions
Each material has a different weight to volume ratio, Table 1 lists the cubic yards per ton of the materials from highest to lowest.

3 METHODOLOGY
The first step in this analysis was to calculate the total weight and volume of waste taken from the site and
the cost; actual trash invoices were used for these figures. Once the total weight and volume of waste was calculated, the waste composition was determined using EPA and WasteCap Wisconsin construction waste percentages shown in Figures 1 and 2.

To determine the cost of each scenario, the number of loads of each material was calculated. The most frequently used roll off container size for these materials is 30 cubic yards; to calculate the number of loads of each material, the total volume of each material was divided by 30. After the number of loads of each material was calculated, it was multiplied by the hauling and dumping costs of a 30 cubic yard container for that specific material.

Metal and trash include separate hauling and dumping costs. For other materials it is a flat rate per load regardless of tonnage.

The cost of each load is calculated using a local hauler cost of $80 per load and a landfill tipping fee of $28 per ton [Kunde]. The equation for cost calculation is below.

\[
h_c \cdot l + d_c \cdot t = C\]

where \(h_c\) = cost of hauling one load; \(l\) = number of loads; \(d_c\) = cost of dumping one ton; \(t\) = total tons of material; and \(C\) = total cost.

To determine the theoretical cost savings if MSOE had recycled, the calculated cost of recycling is subtracted from the cost of sending everything to the landfill.

4 ASSUMPTIONS

During the building of the Kern Center all construction waste generated was hauled as trash to local landfills. The analysis using EPA numbers is based on a 92% recycling rate, this is not an easily attainable recycling rate to achieve. WasteCap Wisconsin’s percentages are based on their experience with Milwaukee area construction. WasteCap numbers are based on a 75% recycling rate, a more realistic recycling goal [Kunde].

To calculate the total weight and volume of waste taken from the site, actual trash invoices were used. However, not all invoices gave actual tonnages, each dumpster had a maximum tonnage and only the weight of those loads which went over the maximum tonnage was documented. Because more than half of the loads were overweight, an estimate of half a ton less than the maximum roll off weight was used for those tonnages not listed on the invoices.

All trash and recycling costs used in this analysis are based on the rates of a local waste and recycling hauler; this was not the actual hauler used during construction.

### Table 2. Costs used in cost savings calculations.

<table>
<thead>
<tr>
<th>Material</th>
<th>Cost per 30 cubic yards</th>
<th>Cost per ton</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete/Masonry</td>
<td>75.00</td>
<td>0.0</td>
</tr>
<tr>
<td>Drywall</td>
<td>100.00</td>
<td>0.0</td>
</tr>
<tr>
<td>Wood</td>
<td>150.00</td>
<td>0.0</td>
</tr>
<tr>
<td>Ferrous Metal</td>
<td>100.00</td>
<td>-98.68</td>
</tr>
<tr>
<td>Trash</td>
<td>80.00</td>
<td>28.00</td>
</tr>
<tr>
<td>Cardboard</td>
<td>75.00</td>
<td>0.0</td>
</tr>
</tbody>
</table>

### Table 3. Materials breakdown based on EPA percentages [Kunde].

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight</th>
<th>Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trash</td>
<td>62.95</td>
<td>552.72</td>
</tr>
<tr>
<td>Cardboard</td>
<td>102.30</td>
<td>4871.38</td>
</tr>
<tr>
<td>Concrete/masonry</td>
<td>62.95</td>
<td>125.9</td>
</tr>
<tr>
<td>Drywall</td>
<td>157.38</td>
<td>642.11</td>
</tr>
<tr>
<td>Metal</td>
<td>102.30</td>
<td>846</td>
</tr>
<tr>
<td>Wood</td>
<td>299.02</td>
<td>1868.89</td>
</tr>
<tr>
<td>Reuse</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>TOTAL</td>
<td>786.9</td>
<td>8907</td>
</tr>
</tbody>
</table>

Scrap metal credit amounts were based on average scrap metal prices during the construction months when scrap metal was generated [Eminton]. Table 2 shows the recycling and trash prices used in the cost savings calculations.

5 RESULTS AND DISCUSSION

During the construction of the Kern Center, 786.9 tons or 5809.5 cubic yards of construction waste was dumped into local Wisconsin landfills.

Table 2 and Table 3 illustrate a weight and volume breakdown of the waste material generated from the Kern Center.

The estimated total cost of removing this waste as trash tallied $41,400. If MSOE had decided to recycle, between 590 and 724 tons of waste could have been diverted from local landfills. That is equivalent to between 5,810 and 8,350 cubic yards of material; enough material to cover an entire football field with almost three feet of trash.

Recycling on this project could have potentially saved between $19,400 and $25,800. Tables 4 and 5 show a breakdown of potential savings by material.
6 CONCLUSIONS AND RECOMMENDATIONS

Advantages of recycling on the MSOE Kern Center outweigh the disadvantages. On this project alone over 785 tons of trash was dumped in local landfills; if the project had recycled; only 196.7 tons would have been taken to landfills.

Aside from recycling being the environmentally friendly thing to do, it is also a relatively easy task to achieve. For site workers, the major difference between recycling and not recycling is the sorting of materials. Each material must be placed into its own container. However, during new construction the separation is not very difficult because materials are generally used one at a time, reducing time spent sorting.

Small construction sites, like the Kern Center, sometimes have difficulty finding space for the various dumpsters needed. This problem can be resolved however, by using the construction schedule to make a plan of what dumpsters will be needed when. For example, after the steel structure was completed on the Kern Center, hardly any metal waste was generated, eliminating the need for a large metal dumpster on site. On most small sites, flexibility and improvisation are necessary to have a successful recycling program.

The United States Green Building Council (USGBC) Leadership in Energy and Design (LEED) green building guidelines offers two points for recycling during construction in the Materials & Resources category. One point can be earned for Credit 2.1, Construction Waste Management, Divert 50%, and an additional point can be earned for Credit 2.2, Construction Waste Management, Divert 75% [LEED].

Not only is recycling on the job site beneficial in an environmental sense, but in a financial sense as well. Had the Kern Center recycled during construction, it is estimated that between $21,000 and $27,500 in hauling and dumping costs could have been avoided.

Though it may not yield a direct profit, the fact that recycling occurred onsite is a very marketable detail; the future of the construction industry is growing greener and publicizing a construction recycling program could be a great way for both owner and construction crew to advertise their environmental awareness.

REFERENCES


Impact of regulations on fly ash marketing in the United States

G.C. Plunk
Boral Material Technologies, San Antonio, TX, USA

D.C. Goss
American Coal Ash Association, Aurora, CO, USA

ABSTRACT: The marketability of a specific source of fly ash is dependant on several factors; quality (relative quality and quality perception), location relative to demand, competing materials, competing fly ash sources and market pricing dynamics. Quality and economics are arguably the most critical factor when determining the marketability of a specific fly ash source and are closely interrelated.

This paper will focus on the impact of current and proposed future federal environmental regulations on fly ash marketability related to quality in the United States. Specifically this paper will discuss the effect of plant combustion modifications (over fired air and low NOx burners), the installation of SCR’s and SNCR’s (ammonia injection), the installation of scrubbers (wet and dry), the addition of additives to adjust opacity, fuel changes to control SOx emissions and proposed mercury capture technologies on fly ash quality and marketability.

The paper will also discuss how the fly ash marketing industry has reacted to fly ash quality changes and worked with stakeholders to overcome the general decline in fly ash quality as market demand and quality expectations have steadily increased. Specifically, the paper will illustrate how the development of improved quality control methods and tests, the development of cost effective beneficiation processes, along with greater flexibility in fly ash handling, logistics and transportation have helped to offset the general decline of fly ash quality and how these changes were made possible by the economics of product value. The paper will also address what steps the U.S. CCP industries are taking to find beneficial uses for non-specification fly ash as well as discussions of current process technologies and treatments used to improve the quality and marketability of non-specification fly ash.

1 INTRODUCTION

The CCP most used in the United States is fly ash to produce concrete. In 2005, about 110 million tonnes of CCPs were produced, fly ash accounted for approximately 63 million tonnes; 12 million tonnes were used in such applications as concrete products, structural fills and cement raw feed for clinker. Fly ash marketing is an industry worth $1 billion (or $530 million lbs.) a year, according to the U.S. Environmental Protection Agency. Fly ash enhances concrete strength and durability, eliminates one ton of CO2 for each ton of portland cement it replaces, and is typically less expensive than portland cement. Research continues finding additional uses for fly ash as a supplementary cementitious material.

Though emission controls can increase carbon content in fly ash, limiting its use in concrete, new technologies have emerged able to mitigate or remove carbon. Electric utilities are realizing the benefit and are willing to invest and partner in these “beneficiating” technologies to ensure fly ash remains a source of revenue, in preference to disposal liabilities. The recent implementation of two new rules will further challenge industry to find ways of maintaining fly ash quality for concrete and may drive some companies to explore other applications.

2 NEW REGULATORY REQUIREMENTS

2.1 Clean air interstate rule & Clean air mercury rule

In 2005, the U.S. EPA implemented the Clean Air Interstate Rule (CAIR) and the Clean Air Mercury Rule (CAMR). CAIR predominantly affects utilities operating in 28 eastern states in which emissions of sulfur dioxide (SO2) and nitrogen oxide (NOx) will be capped. When fully implemented, CAIR will reduce SO2 emissions in these states by more than 70% and NOx emissions by more than 60%. Electric utilities in
the U.S. release between 45 and 48 tons of mercury into the environment each year. When fully implemented, CAMR will permanently cap and reduce mercury emissions from coal-fueled power plants by approximately 70% to 15 tons. Methods to implement CAMR will be determined by operators over the next several years, with full compliance required by 2018. Similar to regulations for the U.S. utility industry, the EPA proposed mercury regulations in February 2006 that will apply to cement kilns producing Portland cement. The EPA noted in comments they recently became aware of a concern about using fly ash as a raw feed for the kiln process. The comments suggested it might be appropriate to ban fly ash in cement production because mercury in the ash would be released to the air from the calcining process. If a power plant removes mercury from its flue gases, and that mercury attaches itself to the fly ash subsequently used in cement production, the re-release of the mercury from the kiln negates the collection of that same mercury from the power plant. Fly ash producers, cement manufacturers, the Department of Energy, and others are working with the EPA to better understand this process. It is not clear that fly ash alone contributes to mercury release from cement kilns. Natural materials (such as shale, limestone, clay, etc) may also be involved.

2.2 Energy Policy Act & the Safe Accountable Flexible Efficient Transportation Equity Act: a Legacy for Users

In 2005, the U.S. Congress also passed the Energy Policy Act (EPACT) and the Safe Accountable Flexible Efficient Transportation Equity Act: A Legacy for Users (SAFETEA LU). Part of the language of these bills is to conduct a study and prepare a Report to Congress within 30 months on several elements of the two bills. One is a renewed effort to encourage the use of concrete in projects where federal funding is used. The second is the support of the use of “recovered mineral components” such as coal combustion fly ash (in addition to ground granulated blast furnace slag, and other “waste materials or byproducts” diverted from the solid waste stream). These two bills have the potential to significantly influence procurement officials and specifiers to look more closely at the use of concrete and to call for supplementary cementsitious materials in the concrete. SAFETEA LU also opens the doors for the use of CCPs in ways other than concrete.

The EPA is the agency tasked to lead the compilation of the Report to Congress and the American Coal Ash Association (ACAA) and other industry organizations have been asked to participate in an industry/government work group to gather data in support of the study.

3 CAPTURE TECHNOLOGIES & FLY ASH QUALITY

The location of mercury capture technology affects fly ash quality. In Europe, many power plants place the collection system after the baghouse or precipitator to minimize impact on fly ash quality. When collection systems are placed before the precipitator or baghouse, fly ash may be impacted by additives or sorbents used to remove mercury. Technologies that introduce particles such as activated carbon, silicates or other powdered materials may be detrimental to fly ash quality as these particles ultimately end up in the fly ash. Preliminary testing indicates the amount of mercury adsorbed onto the fly ash will not require its classification as a hazardous or toxic material. The challenge is to meet the new environmental standards and still be viable from a business standpoint.

3.1 Activated carbon injection (ACI)

Activated Carbon Injection (ACI) technology injects activated carbon into the flue gas duct between the air preheater and electrostatic precipitator or baghouse. The gas temperatures are typically in the 250 to 350 degrees Fahrenheit range. Baghouses achieve higher mercury removal than electrostatic precipitators because of the buildup of the carbon layer on the bag filters. Because ESPs are less efficient at removing mercury, power plants may have to retrofitted with baghouses to increase mercury collection efficiencies. Various configurations of the basic technology are possible, with the simplest being the injection of activated carbon directly ahead of an existing ESP. According to the EPA, this may be the least costly approach in terms of capital expenditures, because no new particulate control device is required; however, when considering “life-cycle” costs, this approach could be more costly.

3.2 Coal switching & blending

Another strategy available for electric utilities to comply with new emission regulations is the changing of coal type or the blending of different coal types. When power plants make changes to fuel the quality of the CCPs produced will be affected. The degree of impact can range from very slight to severe. If a utility is able to purchase millions of tons of coal at a lower price than established contracts, that is a significant economic incentive. The cost savings for such a decision are substantial. In some regions, the change by utilities from bituminous to sub-bituminous fuels has significantly reduced the availability of Class F fly ash. This trend has created a new challenge, the introduction of Class C fly ash into areas where they have never been used before. It is not a simple matter to substitute one type of coal fly ash for
another, without extensive testing and field performance. As mentioned above, another impact of fuel blending has been the reduction of concrete-quality fly ash available to the marketplace. Previous and current state-approved sources of coal ash may also need to be re-evaluated based on the use of the new or blended fuels.

3.3 Flue gas desulfurization

A Flue Gas Desulfurization system typically uses limestone or lime to capture sulfur. Depending on the design of the system, FGD gypsum, i.e. synthetic gypsum, may be produced. More plants will add this system in the next ten years, causing the output of FGD gypsum to more than double. In addition to wallboard, FGD gypsum can be used in agricultural applications, and in the production of portland cement. Other types of FGD materials have much more limited applications and the markets for their end-uses are growing very slowly.

4 MARKETING CHALLENGES

With the emerging regulatory challenges that affect fly ash, utilities, their marketing partners and many others are seeking innovative alternatives to not only maintain, but increase utilization. Additionally, the CCP marketing industries along with the EPA and Department of Energy have set a goal of increasing overall beneficial use (utilization) from the present levels of 40% to 50% by 2011. To do so will require innovative thinking, new technologies, research, and the development of partnerships to increase the awareness of the value using CCPs can provide.

The fly ash marketing industry has responded well to quality changes, working with stakeholders to overcome the general decline in quality as market demand and quality expectations have steadily increased. Greater flexibility in fly ash handling, logistics and transportation has helped offset this decline. The U.S. CCP industries are taking steps to find beneficial uses for non-specification fly ash as well as working to develop process technologies that will improve the quality and marketability of non-specification fly ash. In addition, better quality control methods and tests are being developed, along with cost-effective beneficiation processes. There is no single scenario that addresses these issues in the same way. Location, operating company philosophies, historical use and many other elements affect decision making, from plant to plant and even from unit to unit.

Fly ash quality is determined by the coal rank or fuel type, power plant boiler design, and emission control devices installed, distance to marketplace, transportation options, competing materials, fly ash sources and a myriad of additional variables.

When fly ash produced is unsuitable as an admixture in concrete, or as an enhancement to portland cement, it may be used in other ways. For example, fly ash and bottom ash are used as raw feed stock in manufacturing portland cement. Minerals such as iron, aluminum and silica contained in these ashes replace natural materials that might have to be extracted from pits or quarries. When these opportunities are unavailable, ash may be used in structural fills or land reclamation projects. Blending CCPs with natural materials can provide technically equivalent mixes compared to conventional materials. The proper design of a structural fill or reclamation activity will have no negative impact on the environment, may help reduce project costs, and eliminate landfill disposal.

5 U.S. & GLOBAL PARTNERSHIPS

5.1 The U.S. EPA’s Coal Combustion Products Partnership

In 2003, the EPA initiated the “Coal Combustion Products Partnership,” or “C2P2”. This program is intended to foster awareness of the environmental, technical and economic benefits of using coal ash in a variety of ways. The agency recognized that in many situations, CCPs could be used to further national environmental goals such as the reduction of greenhouse gases, conservation of natural resources and lessening the need for coal ash landfill space. The program is aimed at identifying the actual or perceived barriers that limit the beneficial use of CCPs in highway and construction applications. Some of these barriers exist because there is a lack of understanding by end users and government agencies of the potential benefits that CCPs can have. With C2P2, the sum of these efforts is anticipated to result in an overall increase in the use of CCPs.

The EPA’s C2P2 website (http://www.epa.gov/c2p2/) provides the user with a number of valuable resources, including case studies. There are no costs to join nor are there any specific goals, other than a shared interest in increasing fly ash utilization.

Approximately 140 organizations have joined C2P2 with more joining each year. Workshops have been conducted in cities across the U.S. to encourage potential end-users, designers, engineers and state and local planners to better understand the uses of fly ash. These workshops have been held in capital cities to attract personnel from state departments of transportation. Additional workshops are planned for 2007, in conjunction with the Green Highways Partnership, described later. As the result of C2P2, the EPA issued a booklet (jointly endorsed by ACAA, the Utility Solid Waste Activities Group, DOE, and the Federal Highway Administration) titled “Using Coal Ash in Highway
5.2 Beneficial use summits

In 2002, the EPA hosted the first national summit on the beneficial use of industrial byproducts. Those in attendance included producers of non-hazardous wastes and state agencies responsible for the review and approval of beneficial use of these materials. This permitted an open and valuable dialog for sharing experiences in land applications. In some cases, partnerships were formed to further investigate potential for the beneficial use of many materials. Attendance has increased from 70 in 2002 to nearly 300 participants in 2005.

The purpose of these summits is to bring together state and local officials, as well as federal agency personnel and representatives of industry to exchange ideas about the use of organic and inorganic byproducts in positive ways. The summits typically focus on land applications, but also explore innovative uses for byproduct streams that unless otherwise used beneficially, would be discarded in some manner. Because many U.S. states, counties and cities have staff dedicated to recycling, the summits become forums for these individuals to learn more about the many underutilized materials and their positive impacts on the environment.

5.3 Industrial resources council

A relatively recent partnership is the Industrial Resources Council (IRC). The similarity of issues faced by other byproduct industries prompted ACAA to begin discussions with a number of associations, including Foundry Industry Recycling Starts Today (FIRST), Construction Materials Recycling Association (CMRA), the National Council for Air and Stream Improvement (NCASI), which represents the wood and paper products industry, the Rubber Manufacturers Association (RMA), and the National Slag Association (NSA). All these organizations are industry groups that see the beneficial use of their materials as sound from an environmental, economic and technical perspective. It was natural that their common interests gave rise to the idea of forming a partnership to address common opportunities and barriers. The IRC is envisioned as a clearinghouse for information pertaining to the beneficial use of coal ash, foundry sands, wood and paper industry byproducts, slag for the iron and steel industry, selected construction and demolition debris, and scrap tires.

The EPA favors the formation of partnerships like the IRC as it will enable the agency to better use its own resources. Instead of many similar programs for beneficial use of five or six or more byproducts, the agency can work with the IRC instead. There may be funding opportunities for the IRC from the EPA to support outreach and education related to a variety of industrial materials. Although the IRC is an entirely voluntary effort by the associations identified above, the concept of this partnership has significance. It is one of the first examples of diverse industries with competing products forming an alliance to address issues of common interest. Members of the IRC are still dedicated to their own products, but they also look for projects to combine their materials with those produced by other IRC members in order to convince specifiers and end users of the value of use and reuse of the materials. In many ways, these relationships fit very nicely with the goals of the Green Highways Partnership described below.

5.4 Green Highway Partnership

Another new effort, the Green Highway Partnership (GHP) is underway in the mid-Atlantic region of the U.S. The GHP consists of developing public-private partnerships with federal/state transportation and regulatory/resource agencies, contractors, industry, trade associations, academic institutions, and non-governmental organizations (NGOs). Its goal is to help planners, developers, contractors, end-users and others integrate environmental protection into transportation planning activities involving design, construction operations and maintenance activities.

GHP will act as a resource gathering, streamlining and disseminating information to identify opportunities for partnership initiatives, including helping to support and promote stewardship efforts in concert with the American Association of State Highway and Transportation Officials (AASHTO) Center for Environmental Excellence. The opportunities through GHP will include pilot projects to improve partnerships and research as well as demonstrate sustainable solutions and provide for market-based incentives. Three specific opportunities for partnerships and joint ventures in the mid-Atlantic region have been identified:

- Watershed-Driven Stormwater Management (Planning, Design, Products and Practices)
- Recycling and Reuse (Industrial By-Products and Recycled Material Uses/Implementation)
- Conservation and Ecosystem Management (Principles and Practices)

A recognition program is being established by GHP to identify projects, and activities in the mid-Atlantic region for excellence in achieving Green Highway goals of “better than before” defined for each focus area. It is anticipated that the Green Highways Partnership may be expanded to a national program in the future.
The U.S. EPA is the agency tasked to lead the compilation of the Report to the U.S. Congress and ACAA and other industry associations have been asked to participate in an industry-government work group to gather data in support of the study. This opportunity for partnering, which supports the goals of the beneficial use summits, C²P², the Green Highway Partnership and the Industrial Resources Council, will help build a greater awareness among government agencies of the possible uses for CCPs in transportation projects, from the national to the local level.

6 CONCLUSIONS

The use of CCPs is growing and new applications are being discovered for a variety of materials. At the same time, there are current and future challenges for producers and end-users. Environmental pressures make land filling a much less desirable alternative than beneficial reuse. Regulatory agencies must be assured that the end use will not create new environmental issues.

The fly ash industry has advanced to address many of these challenges by improving material management practices and through the implementation of new technologies. An increasingly significant proportion of fly ash in the future will be a processed, engineered product.

Though specific cases may differ in location, quantities and end uses, properly managed applications using fly ash will achieve desired physical, environmental, economic and social results. Working together we can address change and challenges in positive ways. The opportunity to conserve other materials, to recycle what some consider to be an industrial waste and to benefit from cost savings that can be realized in such uses is important to the United States, the United Kingdom and other countries worldwide.
Quantifying VOCs in products off-gassed during the construction of MSOE's Kern Center and proposed strategies for reducing employee exposure

S. Botic
Environmental Resources Management, Walnut Creek, CA, USA

C. Diggelman
Milwaukee School of Engineering, Milwaukee, WI, USA

ABSTRACT: During the construction of MSOE’s Kern Center, construction workers were exposed to 3.6 tons of Volatile Organic Compounds (VOCs) off-gassed by products used in the caulking, painting, terrazzo floor, and gymnasium floor phases of the project. VOCs pose significant health hazards to people working with or around the products as many of the VOCs are also classified hazardous air pollutants (HAPs). HAPs off-gassed during construction included: isophorone diisocyanate, toluene diisocyanate, m-xylene, and toluene. The most effective approach to reducing VOCs is through product substitution with low-VOC products. Through the development of the Environmental Risk Value (ERV), the products off-gassing VOCs were assigned a risk value that includes the volume and toxicity of each product. Alternatives were found for products with the six highest ERVs. A Product Selection Flowchart (PSF) was developed and used for alternatives to the 6 products evaluated based on ERV, performance measures, and cost. The products used during the construction of the Kern Center were found to have the highest ERV and lowest costs when compared to the alternatives. Also, the performance measures for the products used did not outperform the majority of proposed alternatives. Incorporating the PSF before the submitted stage of a project will allow the construction manager to select products that pose a lower health risk to construction workers. The ERV and PSF are tools that quantify health hazards and identify alternative products that are comparable in performance and cost while reducing employee health risks.

1 INTRODUCTION

From April 2003 until September 2004, the Milwaukee School of Engineering (MSOE) campus was the construction site for a new NCAA division III, 216,000 ft² athletic facility called the Kern Center. The Kern Center includes a basketball arena, hockey rink, recreational gymnasium, running track, numerous athletic amenities, classrooms, and offices [Diggelman 2003].

During construction, many products containing Volatile Organic Compounds (VOCs) were used in multiple phases including painting, caulking, and floor sealing. Construction workers, both those working directly with and those working in the building at the same time were exposed to 3.6 tons of VOCs off-gassed both while the materials were being applied and later during curing.

Exposure to VOCs may result in short- and long-term health effects to employees. Long term exposure to VOCs has been linked to respiratory diseases and cancer. Short-term exposure is usually treatable and includes the following health effects: Eye, nose and throat irritation, headaches, dizziness, physiological irritation, changes in skin temperature, and fatigue. Systemic and carcinogenic effects are a risk for people exposed to VOCs repeatedly at elevated levels [California Department of Health Services 1999]. Many workers are exposed to VOCs regularly on various construction sites throughout the year. The known health effects of VOC exposure are related to the individual compounds and rarely consider total risks to the workers for multiple, simultaneous exposures.

The best reduction technique for hazardous VOCs in a workplace or construction site is to select low-VOC products. In order to identify products with high VOC risks and propose safer alternatives, the total toxicity of these products needs to be quantified. Also, parameters such as cost and product performance need to be analyzed and compared in order to choose the best available products.

This report provides a means by which contractors can quantify the environmental risk of products that off-gas VOCs. Using this risk value with performance measures and a cost analysis will aid in identifying the
2 ENVIRONMENTAL ISSUES

2.1 Health effects of VOCs

Exposure to VOCs may result in short- and long-term health effects according to the United States Environmental Protection Agency (USEPA). Long-term health effects may show up years after long and repeated periods of exposure to VOCs and include respiratory diseases, asthma, reproductive effects, and cancer. Studies of individual VOCs in animals have further shown the potential for these long term effects at elevated levels of exposure. With non-cancer-causing chemicals, there are defined levels of exposure that, if avoided, will eliminate adverse health effects; however, for carcinogens, there is no threshold of exposure and even the slightest exposure may result in the development of cancer. Short-term health effects are treatable and may occur after a single, high dose exposure [USEPA 1998]. Most of the information on the health effects of VOCs are concentrated on individual compound exposure instead of the various combinations of the upwards of hundreds of compounds in one workplace, with many of the compounds not identified by the manufacturer.

The Toxic Substance Control Act (TSCA) requires manufacturers to list the chemical composition of a product for all constituents greater than 1%; however, many of these unidentified constituents can mix and accumulate to produce hazardous levels of VOCs. Additional animal studies have shown the combined exposure to VOC compounds over an extended time results in excess cancer risks [Olcerst 1999].

Building materials may be characterized in one of three categories depending on their rate of VOC emissions decay. Materials with slow decaying VOC emissions release low levels of VOCs at constant rates for long periods of time, typically greater than a year. Materials with intermediate decaying VOC emissions have low to high initial VOC emissions after installation that can continue for several weeks or months. Examples of materials that are classified as having intermediately decaying VOC emissions include floor and wall coverings, and caulks applied in continuous beads. Materials with rapidly decaying VOC emissions include wet-applied products that dry quickly such as paints and sealants. Ninety-five percent of the VOC emissions for these products occur within hours of their application, thus exposing employees to high concentrations for a short period of time.

VOCs emitted during construction may also sorb onto surfaces of many different materials including carpets, fabric upholstery, insulation, and other porous materials defined as sinks. The greater the surface area of the sorbing material, the higher the risk. The amount of VOCs sorbed onto sinks depends on the total volatility, polarity, and concentration of the substances emitted and temperature and volume of the indoor area where they are emitted [California Department of Health Services 1999].

2.2 Economic impacts of hazardous chemicals on a construction site

According to the electronic Library of Construction Occupational Safety and Health (eLCOSH), approximately 3.7% of cases with days away from work are a result of employee exposure to various hazardous and toxic substances on site. An annual survey of occupational injury and illnesses determined that these exposures caused construction employees to miss 7169 days of work [Centers for Disease Control and Prevention 2002].

The results of this study are conservative because these numbers do not include restricted work and the available data only covers the private sector and excludes the self-employed. Illnesses also comprise 1.7% of the total causes of days away from work. Although employees may contract illnesses from numerous sources, prolonged exposure to hazardous substances may result in weakened immune systems, thus contributing to the decrease in the overall health of employees. The USEPA identified increased incidents from occupants of buildings resulting from sick building syndrome (SBS) that results in eye, nose, and throat irritation, sneezing, stuffy nose, fatigue, headache, nausea, irritability, and forgetfulness as a result of VOCs [USEPA 1998]. In addition to losses due to missed time at work, employers also spent 5.17% of their total payroll for workers’ compensation claims.

The United States Occupational Safety and Health Administration (OSHA) estimates that employee productivity loss, as a result of poor air quality, to be approximately 3% [OSHA 1997]. Of the leading causes of death from injuries in construction, exposure constitutes 14.7% of these causes.

The 1999 Census of Fatal Occupational Injuries reported construction employee exposure to hazardous substances caused 181 deaths; however, many of the most serious work-related illnesses in construction, such as cancer, take years of repeated exposure to develop and were not reported as cases of injury or fatality [Centers for Disease Control and Prevention 2002]. The overall benefits for reducing VOCs in a workplace include increased employee productivity, decreased health care costs, and reduced liability exposure.
3 COATING TECHNOLOGY ALTERNATIVES

Currently, solvents are the most widely used carrier for coating a substrate. Adequate solvent substitution amounts to using the liquid form of substrates that are naturally present as gases in the atmosphere such as water and carbon dioxide. Liquid or supercritical carbon dioxide is limited to industrial uses due to the high pressure requirements for application. Therefore, water has become the most widely used coatings solvent. The term “water-borne” describes coatings in which the predominant solvent is water. The major drawback that comes from using water as a primary solvent is the variability in drying times due to changes in relative humidity. In order to maintain consistency of curing, organic co-solvents are used to reduce limitations of waterborne coatings with limited regard to the relative humidity of the environment. In many formulations, the ratio between the amount of water and organic solvent is 80:20 [Matheson Jr. 2002]. Waterborne coatings also have a greater sensitivity to the degree of surface cleanliness which requires increased preparation to obtain a strong bond to the substrate. For a liquid coating to adequately cover a solid surface, the critical surface tension of the solid must be greater than the surface tension of the substrate. The inability to sufficiently wet a surface can be due to the presence of contaminants such as oil and grease preventing a coating from wetting the surface as the surface tension of the coating may be slightly higher than that of the contaminants.

The four key mechanisms of adhesion include primary bonding, secondary bonding, chemisorption and mechanical adhesion. Although all four mechanisms may occur simultaneously, each exhibits a different degree of bonding effectiveness. In most cases, primary bonding is the most effective; however, when the substrate is smooth, mechanical adhesion is the critical mechanism obtained by roughening and cleaning the substrate. With these considerations, a solvent-based coating has greater application flexibility with less surface preparation, less appearance defects, and greater viscosity management than its water-based counterpart; however, it results in greater VOC emissions [USEPA 1996]. The major activity for modern coating reformulation is to balance the chemistry of the water-borne and organic solvents to reduce the VOC limits while maintaining the coating performance, consistent cure time, and low costs [Matheson Jr. 2002].

Another reformulation approach to reducing VOC emissions has been to increase the solids content of coatings to meet the regulated levels of VOCs while retaining the high performance characteristics of the low solids, high solvent coatings. This has been a difficult task as a reduction in the solvent concentrations without any other changes to the formulation has led to higher levels of viscosity, thus increasing the film thickness of the coating. Additives are then required to increase the cross-linking during curing resulting in greater costs to the overall coating. Advantages of the high solids coatings include easier storage due to the lower solvent concentrations, reduced storage and handling costs, fewer required coats due to thickness of the coating, and compatibility with low solids coating application equipment and techniques. The higher solids coatings have a tendency towards excessive flow and slower cure times; therefore, low molecular weight resins are required to help speed up the cure time [USEPA 1996]. These resins become volatile at elevated temperatures resulting in poor film formation and increased VOC emissions.

A newer alternative coating technology is ultraviolet (UV)-cured coatings. The advantages that make UV-cured coatings a popular alternative include fast cure times, high performance, and low solvent levels. UV curing involves the polymerization of the coatings directly onto the substrate to which it is applied by cross linking the simple compounds in the coating formula, thus converting the liquid material into a solid film [Field 2003]. The UV curing begins by introducing ultraviolet light onto the coating. The photoinitiators in the coatings absorb the UV energy forming free radicals that trigger the polymerization process. The curing time in this process is complete in a matter of seconds.

Additional additives such as pigments and adhesion-promoters may be included to achieve specific performance properties of the UV cured coating. The cost of the system is significantly more than its solvent-based counterpart, but the technology’s one hundred percent conversion versus the thirty-three percent conversion for solvent-based coatings. UV raw materials are more expensive, but these up-front costs are offset by lower curing costs, higher operating speeds, and lower VOCs.

4 PERFORMANCE MEASURES FOR COATINGS

Many tests are performed on coatings and paints to assess their performance. Testing includes resistance to scratching, impact and water, salt spray testing, hardness, curing time, and adhesion. These tests are carried out to determine why products have failed and, if it is a new product, whether it performs up to the requirements of many international standards. Specifications for many of the products used on a construction site do not include full results from all of the testing performed on these products and, as a result, comparisons between products are difficult to make. To help correlate product comparison, the American Society for Testing and Materials (ASTM)
develops standard test methods, specifications, practices, guides, classifications, and terminology in 130 areas covering subjects such as metals, paints, plastics, textiles, petroleum, construction, energy, the environment, consumer products, medical services and devices, computerized systems, and electronics [ASTM International Standards Worldwide 2004].

5 STRATEGY FOR EVALUATING VOC-EMITTING PRODUCTS AND ALTERNATIVES

Figure 1 illustrates a step-by-step strategy used in this project to evaluate VOC-emitting products and alternatives.

5.1 Task 1 – identify target materials and products

Target VOC off-gassing products for a construction site are identified by the quantities of the product used and toxicity information of high VOC products. The product volumes and MSDSs were obtained from the construction management firm and used to identify high VOC-emitting products used from building enclosure to substantial completion. Based on information on MSDSs and product specifications, products are targeted. Specific materials were identified as potential significant emitters of VOCs at a hazardous level. In general, these materials were used during the painting, caulking, and sports and terrazzo flooring phases of the Kern Center construction.

5.2 Task 2 – collect VOC-related product information from manufacturers

Individual VOC components in the products were identified by cross-referencing the Chemical Abstract Services (CAS) numbers with lists of VOCs provided by the Occupational Safety and Health Association (OSHA) and the USEPA. The mass balance approach was used to estimate VOC emissions. It was assumed that all of the volatile compounds were off-gassed during mixing, application, and curing of the products onsite. In order to apply the mass balance approach, the total weight of VOCs per liter of material and total liters of material were needed. The total liters of material used were ascertained from the construction manager and the total grams of VOCs per liter were calculated from information on the MSDSs.

The following is an example using the Kern Center's metal primer. From the MSDS, six VOCs were identified as shown in table 1. The total VOCs emitted from the acrylic resin metal primer is the sum of the individual compounds emitted.

5.3 Task 3 – evaluate VOC-emitting materials and products and alternatives

Although the above value provides a total amount of VOCs, individual compounds have different toxicity levels associated with them. Further analysis is required to develop a more complete hazard value. Using the Annual and Short-term Guideline Concentration (AGC/SGC) tables compiled by the New York State Department of Environmental Conservation.
values of individual annual toxicity limits were obtained for each of the VOCs. The guideline concentrations in the tables are based on limits published by the USEPA, 2003 American Conference of Government Industrial Hygienists (ACGIH) Threshold Limit Values (TLVs) and TLV ceiling limits, and NYSDEC.

5.3.1 Environmental risk value
Using the guideline concentration tables, AGC values are obtained for all of the VOCs identified in product MSDSs. Dividing the total VOC (mg) by the AGC value (mg/m³) gives a total toxicity value for each component in volume (m³) units. Individual component toxicity volumes are totaled and assigned a total toxicity value for each product based on both the amount and toxicity of the VOCs off-gassed. Based on figure 2 values, the top six materials were selected for further analysis.

Materials included: Gymnasium turf, vertical caulk, horizontal caulk, xylene solvent cleaner, metal primer, and acrylic terrazzo sealer. Each of the six was evaluated and compared to less hazardous alternatives for cost and performance.

5.3.2 Performance measures
Construction bid packages specify material quality to provide the best-quality material at a reasonable cost. Although there are various alternative products identified in bid packages for contractors to select from, some of the actual products used are not listed as approved products. It is not known if approval was given to product substitutions and on what basis the replacements were chosen. Also, since the coating industry is continuously changing, the best available technologies are not often cited in bid packages due to their limited performance history or acknowledgement as an acceptable alternative. Since different products have different performance tests, performance measures were developed for each different type of product.

Alternative products were chosen in part based on their availability of a complete MSDS, full technical data list with comparative ASTM tests, cost availability, and their ability to meet the minimum performance requirements. In the case of the acrylic resin metal primer, the product performance measures are shown in Table 3.

Metal primer cure time is the amount of time required for the product to become tacky enough for another coating application. The expected life for different paints is difficult to obtain due to the various applications of the paints. In the case of the acrylic resin metal primer, the paint is considered to have a lifetime equal to that of the product it is being applied

### Table 1. VOC components for acrylic resin metal primer.

<table>
<thead>
<tr>
<th>VOC</th>
<th>CAS #</th>
<th>VOC %</th>
<th>Density (g/L)</th>
<th>Grams VOC/ Liter</th>
<th>Total VOC (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toluene</td>
<td>108-88-3</td>
<td>5%</td>
<td>864</td>
<td>43.20</td>
<td>56</td>
</tr>
<tr>
<td>Ethylbenzene</td>
<td>100-41-4</td>
<td>2%</td>
<td>866</td>
<td>17.33</td>
<td>23</td>
</tr>
<tr>
<td>Xylene</td>
<td>1330-20-7</td>
<td>10%</td>
<td>868</td>
<td>86.75</td>
<td>113</td>
</tr>
<tr>
<td>Light Aromatic Hydrocarbons</td>
<td>64742-95-6</td>
<td>2%</td>
<td>883</td>
<td>17.66</td>
<td>23</td>
</tr>
<tr>
<td>1,3,5-Trimethylbenzene</td>
<td>108-67-8</td>
<td>2%</td>
<td>859</td>
<td>17.18</td>
<td>23</td>
</tr>
<tr>
<td>1,2,4-Trimethylbenzene</td>
<td>95-63-6</td>
<td>4%</td>
<td>876</td>
<td>35.04</td>
<td>46</td>
</tr>
<tr>
<td>Total</td>
<td>284</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 2. Toxicity volumes for acrylic resin metal primer.

<table>
<thead>
<tr>
<th>Component</th>
<th>VOC Total (mg)</th>
<th>AGC (mg/m³)</th>
<th>AGC Total (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toluene</td>
<td>5.64 x 10⁷</td>
<td>0.4</td>
<td>1.41 x 10⁸</td>
</tr>
<tr>
<td>Ethylbenzene</td>
<td>2.26 x 10⁷</td>
<td>1</td>
<td>2.26 x 10⁷</td>
</tr>
<tr>
<td>Xylene</td>
<td>1.13 x 10⁸</td>
<td>0.1</td>
<td>1.13 x 10⁹</td>
</tr>
<tr>
<td>Light Aromatic Hydrocarbons</td>
<td>2.31 x 10⁷</td>
<td>3.8</td>
<td>6.07 x 10⁶</td>
</tr>
<tr>
<td>1,3,5-Trimethylbenzene</td>
<td>2.24 x 10⁷</td>
<td>0.29</td>
<td>7.74 x 10⁷</td>
</tr>
<tr>
<td>1,2,4-Trimethylbenzene</td>
<td>4.57 x 10⁷</td>
<td>0.29</td>
<td>1.58 x 10⁸</td>
</tr>
<tr>
<td>Total</td>
<td>1.54 x 10⁹</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

ERV: 9.19.
to. This lifetime is then used to calculate the life cycle cost for the product.

5.3.3 Cost analysis

Paint failures over time will not require the full removal of the old paint down to the original substrate, but rather a recoat over a sanded and cleaned surface. Therefore, the life expectancy of metal primers is assumed to be 50 years and the cost is a one time expense over the course of the 50 year life cycle. Costs for the materials used in this study were obtained from local suppliers; costs for the alternative products were obtained from specialized painting and sealant distributors. The life expectancy for the materials analyzed was estimated based on information from technical data sheets, interviews with the manufacturers and suppliers, and warranty information. For the acrylic resin metal primer, the 50 year life cycle cost is shown in Table 4 below.

<table>
<thead>
<tr>
<th>Performance measure</th>
<th>Test</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direct impact Resistance</td>
<td>ASTM D-2794</td>
<td>7.909 Nm</td>
</tr>
<tr>
<td>Adhesion</td>
<td>ASTM D-4541</td>
<td>17.692 atm</td>
</tr>
<tr>
<td>Cure time</td>
<td>Recoat window</td>
<td>2.5 hours</td>
</tr>
<tr>
<td>Expected life</td>
<td>Manufacturer</td>
<td>50 yrs</td>
</tr>
<tr>
<td>Expected life</td>
<td>Specification</td>
<td></td>
</tr>
</tbody>
</table>

Table 3. Acrylic resin metal primer performance measures.

The ERV, performance measures, and costs were derived for each of the six products. Alternatives were evaluated for performance, cost, and ERV to assess the potential for having chosen less toxic alternatives.

5.4 Method of analysis and product selection

The method of analysis is shown for the metal primer used in the Kern Center. Three alternatives were chosen and their performance measures, ERV, and cost were compared with the acrylic metal primer used. Like cost analysis, the values obtained for all performance measures are from product technical data sheets, interviews with manufacturers and installers, and warranty information. Direct impact resistance, adhesion, cure time, and expected life were used to compare performance for the metal primer and proposed alternatives. The results of the analysis are shown in Table 5 below.

The greater the direct impact resistance number the stronger the material is against rapid impact deformation. The direct impact resistance of the urethane and acrylic metal primers performed more than...
twice as high as the acrylic resin used and the proposed alternative epoxy metal primers. Based on this performance measure, the more durable alternative is either the urethane or acrylic metal primer.

The adhesion performance of a material indicates the ability the material has to bond to a substrate. The higher the adhesion (atm), the less likely a material will be to peel in the short term and over time. The acrylic resin metal primer used on the Kern Center has the lowest adhesion value when compared with the alternatives. The urethane primer is almost four times stronger than the metal primer used.

Cure time is also important for primers due to the need for additional coatings after the primer has cured. All of the primers tested have short enough cure times for multiple coatings in a day. The acrylic and urethane metal primers have the fastest cure times and can be recoated up to four times in a single day. This advantage allows for multiple primer coats and a thinner base coat that will cure faster and more uniformly with less material applied. The expected lives for the metal primers were the same and were not considered as a performance measure. Since recoats seldom require abrasion down to the substrate for metal primers, it was assumed all four primers’ life expectancies would last the duration of the 50 year life cycle.

The next part of the product evaluation quantifies the environmental risk to employees exposed to these products. Using the MSDSs and VOC analysis previously detailed for the acrylic resin primer, the ERVs were then calculated for the alternatives.

Although the urethane metal primer performed the best, its ERV is highest of the four products evaluated. The acrylic metal primer alternative had the lowest ERV and the epoxy metal primer was less toxic than the acrylic resin metal primer used.

The final part of the product evaluation and alternatives involves the cost of the materials over a 50 year life cycle. The standard USEPA inflation of three percent was used for the analysis and the products’ expected lives were used to determine when the material would require another application. Since the metal primer is assumed to have a life longer than 50 years, the acrylic resin primer and the alternatives were figured for a one time cost during construction and the results are shown in Table 6.

The urethane metal primer is the most expensive of the four, the metal primer used in the Kern Center is the least expensive, and the other two alternatives are comparable in cost. The cost comparison for the metal primers ends the evaluation of VOC-emitting products and alternatives. This evaluation provides a numerical means by which to compare the product used in the Kern Center with three alternatives based on their performance, environmental risk, and cost.

5.4.1 Product selection flowchart

The final step for this project involves a flowchart to use in product selection. Figure 2 displays the Product Selection Flowchart (PSF) derived from the California Department of Health Services (CDHS) strategy for selecting building materials.

Using the flowchart for product selection puts first priority on performance of the product and addresses cost last. Answering yes to all three evaluation measures results in the acceptance of a product. The flowchart allows performance and cost values to be included. In the case of the metal primer, no other product substitutions are available, reformulation cannot be considered feasible, and ventilation is not known; therefore, ‘no’ responses will result in the rejection of a product.

The acrylic resin metal primer maintained high performance, but did not have an acceptable ERV, without ventilation and is therefore rejected. The urethane metal primer was also rejected. Although it has superior performance measures, it still has the highest ERV value and almost three times the cost of the Kern Center metal primer used. The epoxy metal primer performed adequately and could be considered an acceptable alternative; however, the cure time is long and could limit the ability to recoat in the same day. If this is not critical for on-time completion, the low ERV and comparable costs make this a good alternative. Finally, the acrylic metal primer has adequate performance measures, but also a long cure time. The acrylic primer has the lowest ERV value and is comparable in cost. This product would also be accepted.

<table>
<thead>
<tr>
<th>Metal primer product</th>
<th>Direct impact resistance (Nm)</th>
<th>Adhesion (atm)</th>
<th>Cure time (hrs)</th>
<th>Environmental risk</th>
<th>Total cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acrylic Resin</td>
<td>7.91</td>
<td>17.70</td>
<td>2.5</td>
<td>9.19</td>
<td>$6,551.55</td>
</tr>
<tr>
<td>Urethane</td>
<td>15.82</td>
<td>68.05</td>
<td>2</td>
<td>12.59</td>
<td>$17,232.75</td>
</tr>
<tr>
<td>Epoxy</td>
<td>3.62</td>
<td>48.65</td>
<td>5</td>
<td>8.41</td>
<td>$8,462.85</td>
</tr>
<tr>
<td>Acrylic</td>
<td>15.82</td>
<td>34.02</td>
<td>6</td>
<td>7.54</td>
<td>$7,103.55</td>
</tr>
</tbody>
</table>
By narrowing the choice to two products, the contractor has eliminated products that are most hazardous to the construction workers on site and are too costly. Taking a closer look at performance measures' impact on the quality of the specified products for their intended uses and cost comparisons will aid finding the desired product for the job. As a result of this evaluation and selection process, the acrylic metal primer would be the accepted product with higher performance standards and lower cost than the epoxy alternative.

**Selection: Acrylic metal primer.**

6 RESULTS OF PRODUCT ANALYSIS

Based on MSDSs, product specifications, and cost analysis, a contractor can evaluate VOC off-gassing products and identify alternatives based on the ERV and using the PSF. Once products have been identified according to their ERVs, performance measures and costs can be used to select a product that will conform to construction specifications. The environmental risk value can be used to compare the VOC off-gassing hazards of equivalent products before contractors bid and can be verified in the submitted stage. Historically, it has been difficult for contractors to select products based on predictions of indoor VOC concentrations. The ERV is a tool that can quantify and predict the toxicity of all the VOC off-gassing products targeted. Coupling this tool with a comparison of the product performance measures and cost analysis provides the contractor with a means to evaluate products and compare alternatives using the PSF. The four step strategy for evaluating and selecting construction projects and method of product selection

---

Figure 3. Flowchart for selecting construction materials and products (PSF).
were completed for the remaining five products. The process which was applied for the acrylic resin metal primer was used to identify the best alternatives based on performance, ERV, and cost.

6.1 Gymnasium turf

The gymnasium turf installed for the indoor basketball court and running track consisted of three installation phases for completion: Primer coat, base coat, and topcoat. The primer coat is a solvent-based, thin application that is used to seal the concrete and provide the initial bond for the additional layers. The basecoat and topcoat both use polyurethanes for the chemical base of their application. The basecoat polyurethane is mixed with rubber granules to provide the bulk and texture of the gymnasium turf. The polyurethane topcoat is applied to protect the basecoat and is tinted to give the turf uniform color [Ritchie 2004]. The gymnasium turf used two different polyurethanes for the aforementioned layers. Performance standards determined by the manufacturer to install certified gymnasium turf give flexibility in choosing alternatives based on these standards. The performance measures of the polyurethane were obtained from technical data sheets. Alternatives were assumed to be uniformly used for the basecoat and topcoat, and the spread rates were constant for all four. The primer coat was considered constant for all of the alternatives and was calculated separately from the other two layers. The primer coat’s properties do not affect the number of recoats. Existing gymnasium turf coatings are seldom taken down to bare concrete, but are rather roughly abraded and recoated at a depth determined by the amount of wear to the surface.

The product used at the Kern Center was an aromatic isocyanate prepolymer gymnasium turf. The alternatives proposed were an aliphatic polyurethane turf, a solventless elastomeric polyurethane turf, and a water-based elastomeric turf. The results of the performance measures for the polyurethane used and the proposed alternatives were analyzed. Minimum performance properties of the polyurethane require a hardness range of 55–65 Shore A. The alternatives perform at a much higher Shore A hardness than the actual material used. The solventless elastomeric polyurethane performs the best of the four with aromatic prepolymer performing at the specification minimum requirements.

The tensile strength was also tested for all of the materials. Solventless elastomeric polyurethane performed the best of the four and the aromatic prepolymer had the lowest tensile strength.

Cure time was also a performance measurement for the various gymnasium turf materials. The shorter a cure time, the sooner another coat may be applied or the sooner other construction workers may work on the floor. The solventless elastomeric polyurethane had the shortest cure time and would be at a great advantage if the process included multiple topcoats in a day. The aliphatic polyurethane had the lowest cure time and would require a full 24 hours before recoat.

The expected life before an ‘abrade and recoat’ was determined from phone interviews with installers of resilient sports flooring. There was no defined warranty, but the average time before a required recoat was 15 years. This number remains constant for all four materials; however, unlike the other materials, costs were not ascertained for the Kern Center gymnasium turf and it was not possible to compare.

An analysis of the previous performance measures indicates the solventless elastomeric polyurethane gymnasium turf is the superior product for hardness, tensile strength, and cure time. The MSDSs for all of the alternatives were used to calculate the ERVs to determine which product poses the lowest environmental risk resulting from off-gassing VOCs. The results of the ERV analysis are shown in Table 7.

Analysis of the products show, although superior in performance, the solventless elastomeric polyurethane would not be a sound alternative based on the ERV. The water-based elastomeric gymnasium turf presented the lowest risk and is considered a possible alternative. This alternative outperforms the aromatic prepolymer used and the 12 hour cure time allows for next day recoats. By applying the PSF and assuming the same restrictions as the metal primer, the ERV measure eliminates both the aromatic prepolymer and solventless elastomeric polyurethane turfs. The water-based elastomeric gymnasium turf outperformed the aliphatic polyurethane, including a shorter cure time, and also had a significantly lower ERV. Without cost analysis, the PSF defines the water-based turf as the best product alternative for the project.

Selection: Water-based elastomeric gymnasium turf.

6.2 Acrylic terrazzo sealer

The terrazzo sealer is a clear, thin coating applied as the final step of terrazzo flooring and only the terrazzo sealer will require recoating as it is assumed to be the point of impact and damage for the entire system. The

<table>
<thead>
<tr>
<th>Gymnasium turf</th>
<th>Environmental Risk Value (ERV)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aromatic prepolymer</td>
<td>12.04</td>
</tr>
<tr>
<td>Aliphatic polyurethane</td>
<td>9.29</td>
</tr>
<tr>
<td>Solventless elastomeric polyurethane</td>
<td>12.83</td>
</tr>
<tr>
<td>Water-based elastomeric</td>
<td>7.19</td>
</tr>
</tbody>
</table>
time frame by which a recoat is needed will depend on variables such as: Volume of traffic, UV exposure, chemical exposure (cleaning products used), and type of traffic. In this case, light foot traffic is assumed with various amounts of UV exposure from the large windows. Chemical exposure is not known, since the types of cleaning products used are not known. The terrazzo sealer used at the Kern Center was an acrylic terrazzo sealer. Figure 2 shows the acrylic terrazzo sealer has an ERV of 8.71 and would be identified by Task 1 as a product that requires research for alternatives. Alternatives identified included urethane, chemical resistant epoxy, and UV-cured resin terrazzo sealers. Information was available for performance, ERV, and cost measures of the terrazzo sealers. Product specifications and technical data sheets provided performance measures for the terrazzo sealer and proposed alternatives that can be evaluated. Expected life was based on the frequency of recoats required to maintain the high gloss for the terrazzo flooring. The performance measures used for the terrazzo sealers are abrasion resistance, tensile strength, cure time, and expected life. The results of the performance measures show the chemical resistant epoxy has the highest abrasion resistance and tensile strength; however, it has the slowest cure time. Since a single coat is required for the sealer, cure time is not as important as compared to the other performance measures. The abrasion resistance of the urethane and UV-cured resin terrazzo sealers are significantly lower than their alternatives, but based on the earlier assumption of light foot traffic for the area, the abrasion resistance numbers can be accepted pending contractor specifications. Tensile strength is also a measure of product durability and, like the abrasion resistance, has flexibility in determining the acceptable limit based on the light traffic assumption. The expected life of the terrazzo sealers vary primarily due to the UV exposure during use. Both the urethane and chemical resistant epoxy yellow over time from UV exposure and require frequent recoats to maintain a clear appearance. The cost of labor resulting from recoating is not known and is not considered for this evaluation; however, the increased quantity and subsequent cost increases suggest this to be a negative performance measure for terrazzo sealers. Employee exposure during these recoats is also not considered, but would be considerable since it is a closed building. The negative impact of a shorter expected life will be evaluated in the cost analysis. All four terrazzo sealers maintain high performance, pending contractor specifications, and the PSF continues to the ERV evaluation. The ERV results are shown in Table 8.

The ERV for these products show that the acrylic and urethane terrazzo sealers contain VOCs that pose a risk for construction workers on site and would be eliminated under the assumption there is no ventilation. The chemical resistant epoxy and UV-cured resin terrazzo sealers have lower ERV values. The cost analysis is shown in Table 8. The 50 year life cycle analysis for terrazzo sealers shows a large variability in costs. Based on the assumption that there is not a ventilation system in place, the less expensive of the two remaining products is the UV-cured resin terrazzo sealer. Since the selected alternative is more than four times the cost of the acrylic sealer used, and they have comparable performance measures, an increase in air exchanges could reduce the ERV number for the acrylic sealer and the cost of installing a temporary ventilation system for this phase may be a less expensive alternative. As mentioned earlier, however, for the simplicity of this study, ventilation systems are not in place and the acrylic terrazzo sealer has already been eliminated.

Selection: UV-cured resin terrazzo sealer.

6.3 Horizontal and vertical caulking

The purpose of vertical and horizontal caulking is to seal joints or cracks from the intrusion of water, air, dust, pollution, insects and noise. Traditionally, the vertical caulk will have a faster cure time to prevent the material from beading up. Although used in relatively small quantities, the ERV for the caulks used during the construction of the Kern Center are a risk for the workers nearby. The performance measures used to evaluate the caulk used and proposed alternatives include: Hardness, tensile strength, cure time, and life expectancy.

6.3.1 Vertical caulk

A polyurethane vertical caulk was used in the construction of the Kern Center and the proposed alternatives included silicone, latex, and polyurea vertical caulk.

Based on the performance measures tested for the vertical caulks, the polyurea alternative performed the best. The polyurethane caulk had the longest cure time and required a full day before the caulk could be painted. This project makes the assumption that the vertical caulk will not be painted. If a project requires

<table>
<thead>
<tr>
<th>Terrazzo sealer</th>
<th>Environmental Risk Value (ERV)</th>
<th>Total cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acrylic</td>
<td>8.71</td>
<td>$25,154</td>
</tr>
<tr>
<td>Urethane</td>
<td>9.42</td>
<td>$53,083</td>
</tr>
<tr>
<td>Chemical resistant epoxy</td>
<td>5.91</td>
<td>$150,081</td>
</tr>
<tr>
<td>UV cured resin</td>
<td>6.71</td>
<td>$109,200</td>
</tr>
</tbody>
</table>

Table 8. ERV and 50 year life-cycle total cost for terrazzo sealers.
the vertical caulk to be painted, the paintability of the caulk will need to be considered as a defining performance measure. Even with the polyurethane caulk not being painted, there is still 24 hours in which the finished product can be damaged; therefore other workers would need to keep out of areas recently caulked. Slower cure times are also coupled with slower off-gassing of VOCs and workers around the caulked areas would be exposed to VOCs for a longer period of time. The latex vertical caulk alternative performed very low on the tensile strength performance measure and is also one of two products that have a life expectancy five years shorter than their counterparts. In the case of vertical caulk, the performance measures that are of most importance are a short cure time and durability over time. Based on this statement, the polyurethane vertical caulk that was used at the Kern Center is eliminated from consideration for having a substantially longer cure time than the proposed alternatives. Since the expected lives of the remaining caulks vary by only 5 years, this measure will be further validated by the results of the cost analysis. The environmental risk values of the vertical caulks were then calculated and the results are shown in Table 9.

Although displaying superior performance measures, the polyurea vertical caulk had the same ERV as the polyurethane vertical caulk used and would be eliminated based on its risk to employees. The silicone and latex vertical caulks have lower ERV values and the cost comparison is used to determine the best product for the project. The cost comparison from the 50 year life cycle analysis is shown in Table 9.

The cost comparison of the vertical caulks assumed that all recoats used the same amount of material as the initial installation and shows the latex vertical sealant has the lowest cost of all the products analyzed. Based on earlier assumptions, short cure time, low ERV, and low cost, this product could be accepted for the project.

**Product selected: Latex vertical caulk.**

### 6.3.2 Horizontal sealant

The horizontal sealant used was a polyurethane horizontal sealer. Proposed alternatives included polysulfide rubber, silicone, and epoxy joint sealer. It was assumed that the horizontal sealant would be a one-time installation; therefore the expected life performance measure was not included for this analysis.

Like the vertical caulk, the performance measures of most importance for the horizontal sealants are the cure time and life expectancy. Horizontal sealant would also be like the metal primer as it is coated and/or covered by other materials that would prevent it from receiving direct damage or wear. Only an installation flaw would require additional work on these joints once the sealant has been installed and would be considered warranty work at no additional cost. Since it was assumed the lifetimes for the products are the same, hardness and tensile strength are used to better analyze the performance properties.

All of the products performed adequately when compared to the polyurethane sealer with the epoxy joint sealer performing the best. The cure time for the polyurethane sealer was 48 hours and would require other workers to keep off of the sealed floors for 2 days. Unless installed before a weekend, this cure time is not acceptable and the polyurethane horizontal sealant used for the Kern Center project is rejected. The ERV are then calculated and the results are shown in Table 10.

**Figure 16: ERV for horizontal sealant.**

Based on the ERV calculated from the MSDSs of the products, the epoxy joint sealer stands out as being the only product to not have an ERV. Information on MSDSs may be misleading as there are exemptions to which components are reported. The ingredients listed for the epoxy joint sealer were listed as proprietary and the true chemical composition is not known. In this case, the ERV is not a valid measure for this product; however, the polysulfide rubber joint sealer and silicone joint sealers show lower ERVs than the polyurethane product used and could be considered for product selection. The cost comparison for the horizontal sealers is shown in Table 10.

The epoxy joint sealer has the highest cost, and coupled with the lack of information regarding its risk to worker’s health, is eliminated from consideration. The polysulfide rubber joint sealer is the second most expensive and with a borderline ERV, is also eliminated. The silicone joint sealer has the lowest cure time, performed well in hardness and tensile strength, and has the lowest cost of all the products.

**Product selected: Silicone horizontal joint sealer.**

#### 6.4 Solvent cleaner

Xylene solvent cleaner at the Kern Center project was used for cleaning equipment and spills. Proposed alternatives included mineral spirits, methyl ethyl ketone, acetone, and citrus natural solvent cleaners. Performance measures were not available for various solvent cleaners and were therefore not considered in...
results are shown in Table 11.

<table>
<thead>
<tr>
<th>Solvent cleaner</th>
<th>Environmental Risk Value (ERV)</th>
<th>Total cost ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Xylene</td>
<td>9.21</td>
<td>15,201</td>
</tr>
<tr>
<td>Mineral spirits</td>
<td>8.06</td>
<td>7,010</td>
</tr>
<tr>
<td>Methyl ethyl ketone</td>
<td>7.48</td>
<td>32,720</td>
</tr>
<tr>
<td>Acetone</td>
<td>6.73</td>
<td>14,792</td>
</tr>
<tr>
<td>Citrus natural solvent</td>
<td>0.00</td>
<td>50,683</td>
</tr>
</tbody>
</table>

The ER V analysis eliminates the xylene solvent cleaner used and the citrus natural solvent cleaner becomes the obvious choice with the legitimate zero ER V. Mineral spirits, methyl ethyl ketone, and acetone solvent cleaners also present an acceptable ER V and their costs are compared with the citrus natural solvent cleaner in Table 11.

The 50 year life cycle analysis assumed that solvents would be used for all installations and recoats, so in that sense, these costs are derived from the expected lives of all the other products. Based on this usage, the volume of xylene required for any phase of installation is assumed to be unchanged and the frequency of use is derived from the life cycle scenario with the greatest number of recoats for each product. The costs of the solvent cleaners vary greatly, and since there are no performance measures to compare, the lowest priced material with an acceptable ER V is the best product for the project and subsequent recoats. Having a VOC-free product is an expensive alternative when compared to the low-VOC products available for use and therefore is eliminated from consideration.

**Product selected: Mineral spirits solvent cleaner.**

7 CONCLUSIONS

Almost 3.6 tons of VOCs were off-gassed during the construction of the Kern Center. VOCs pose a health hazard to construction workers working with and around the off-gassing materials as many VOCs are considered HAPs and, through photochemical reactions, contribute to the formation of ground level ozone. It was determined that the best method by which to eliminate VOCs from a workplace is through product replacement. In order to find which products required replacement and were significant sources of VOCs, the health risk to construction workers needed to be quantified based on their toxicity and volume used. The ER V number was developed from information obtained from MSDSs, AGC/SGC tables, and quantities given by the project manager. The ER V number is a toxicity volume that, when divided by the total volume of air in a room/facility, becomes a dimensionless toxicity value. The environmental risk of each product was quantified. Using a variation of the four-step strategy for evaluating and selecting construction products derived by the CDHS, a matrix by which to compare the products used with proposed alternatives was developed. After obtaining all information to complete the matrix, a process flow diagram was used for product selection. Using performance measures, environmental risk values, and cost comparison, best available products were derived from the materials selected and the proposed alternatives.

REFERENCES


Relative thermal performance of three test buildings – thermal mass v insulation

K.A. Heathcote & G. Moor
University of Technology, Sydney, Australia

ABSTRACT: This paper details the construction and preliminary thermal performance of three small test buildings constructed by the University of Technology Sydney in Sydney, NSW, Australia. The project involves testing the thermal performance of three buildings of differing thermal mass and resistance – one of standard brick veneer construction, one of mud brick and one of insulated AAC wall panels (Hebel) with additional board insulation. The paper details the construction of the three buildings and examines their relative thermal performance during a cool spell and a hot spell in late 2006.

1 INTRODUCTION

The present level of carbon emissions is a major concern for the world, and action is urgently needed to reduce them if we are to avoid the disastrous consequences of global warming. According to Smith (2001) “buildings in use or in the course of erection are the biggest source of carbon emissions generated by fossil fuels, accounting for over 50% of total emissions”. Air-Conditioners are a significant user of energy, and where fossil fuels are used to generate electricity, such as in Australia, any improvement in energy efficiency of buildings should lead to major reductions in CO₂ emissions.

In the past there have been many experimental studies examining the thermal performance of residential buildings, eg Richards (1954), Peleg and Robinson (1964), but it is only recently that computer programs have reached the stage that mathematical simulations of the total performance of buildings, including the effects of thermal mass, can be performed. There remains some doubt however as to the accuracy of these simulations, and the present study was initiated in order to test the accuracy of the “Accurate” program (www.hearne.com.au) in predicting the thermal performance of three buildings having varying thermal inertia.

The experimental program consisted of the construction of three small test buildings, identical except for their wall construction. One building is of heavy mass construction (Mud Brick), one a heavily insulated construction (Insulated AAC – Hebel) and the third is a control building with standard brick veneer walls. The latter is the most common form of walling in residential building in Sydney.

The mud brick building was chosen because of the bad publicity that material has received recently based on low static R value figures.

“While environmental, aesthetic and cost considerations may inspire people to build from these materials, our results show that simple rammed earth walls may not perform as well as we would like to think,” (Robin Clarke, CSIRO, 2000)

Earth walled buildings are quite common in Australia and the general folklore is that they perform very well thermally, with most people believing this is due to their high thermal mass. The Accurate program purports to take this into account.

The Insulated Hebel (AAC) building was offered as a prototype by CSR Hebel. It differs from their standard Hebel system (Stud wall with Hebel panel veneer) in that extra insulation in the form of a polystyrene panels is placed behind the Hebel panels. Combined with reflective foil insulation, this prototype system has a high R value combined with a low thermal mass.

The program reported herein was therefore set up to a) compare the relative thermal performance of identical buildings with walls of differing thermal mass and b) to see whether the computer program “Accurate” (developed by the CSIRO in Australia) can effectively model thermal mass. This paper reports on the first of these objectives.

2 DESCRIPTION OF BUILDINGS

The three test buildings are located on property owned by the University of Technology Sydney (UTS) on the outskirts of Sydney (Yarrawood). The site was once a
conference centre but is now largely derelict. The area is classified as Climate Zone 6 (cool temperate climate) by the Building Code of Australia, with winters cold enough to require significant heating and summers hot enough to require cooling. Winter temperatures typically may get down to zero degrees Celsius whilst summer temperatures can get up to the late 30’s. The use of reverse cycle air-conditioners is increasing rapidly in buildings constructed in this area.

The three buildings are each 4.5 metres square in plan and face true north. They have an internal area of 16 square metres and an internal volume of 38.4 cubic metres. There is a single glazed door on the northern side with glass louvres above. The location of the buildings on the site was determined by the need to provide sufficient spacing to avoid shading by each other and by existing buildings as well as the need for access to services. Figure 1 shows the completed three buildings.

2.1 **Floor construction**

The floor of all buildings is a conventional reinforced concrete slab on ground with a 100 mm thick slab and 300 mm by 300 mm edge thickenings. 150 mm by 86 edge set downs were used on the brick veneer and insulated Hebel buildings whilst there was no edge set down for the mud brick building. Some ground cutting was necessary to accommodate the slight fall from west to east on the site.

2.2 **Roof construction**

The roof was conventionally framed with a 22 degree pitch with gable ends on the east and west sides. The eaves are 600 mm on the north side and 300 mm on the south side. The ceiling is 10 mm plasterboard and the roof sheeting is corrugated metal deck (Colour Light Grey). Air-Cell “Glareshield”\(^1\) was placed directly under the roofing and R2 polyester insulation was placed

---

\(^1\) [www.aircell.com.au](http://www.aircell.com.au)
between the ceiling joists and against the gable ends. The gables were sheeted with fibre cement boarding.

2.3 Door details
The single light door on the north side is centrally located and consists of an 820 mm timber frame with 6.38 mm laminated glass. Above the door are two glass louvres with a glazed depth of 320 mm and width of 820 mm. These louvres were closed during this test series. The total glazed area of the door and louvres is around 1.33 m² with the overall opening including framing being around 2.11 m².

2.4 Brick Veneer walls
The walls of the brick veneer building are of typical construction, consisting of a timber stud wall with a fired clay face brick veneer. The stud walls were externally sarked and R1.5 fibreglass batts were placed between the studs.

2.5 Insulated Hebel walls
The insulated Hebel wall system consists of a normal stud wall with 50 thick SMTG Styrofoam™ (R1.79²) and 75 thick Hebel™ AAC panels (R0.51³) panels attached to the stud wall using top hat steel sections. The Hebel and Styrofoam panels were fixed to the top hat sections by screwing from the outside. Foil backed sarking was placed against the studs with the shiny side facing the 25 mm cavity between the studs and the Styrofoam.

![Figure 2. Section through roof (Source www.aircell.com.au).](image)

2.6 Mud brick walls
The mud bricks were 350 mm long by 250 mm wide and 100 mm high. They were bitumen stabilised and were laid in a mortar consisting of a mixture of sand and a clayey soil. Joints were approximately 20 mm. The walls were laid by members of the Earth Building Association of Australia with assistance from Architectural students of UTS who were undertaking an earth building elective.

3 THERMAL PROPERTIES OF BUILDING ENVELOPES
According to Van Straaten (1967) the thermal performance of buildings is determined by the product of total heat storing capacity and thermal resistance “the lower the product of total heat-storing capacity and the thermal resistance, the lower the minimum and the higher the maximum indoor air temperature and the larger the diurnal range of indoor air temperatures”
This is merely a reflection of the role Diffusivity (1/resistivity times heat capacity) plays in the fundamental differential equation of heat flow. Szokolay (2004) shows the relationship between diffusivity and the Decrement Factor (ratio of amplitude of internal to external surface temperatures). Lower values of diffusivity (higher values of resistivity times heat capacity) lead to lower values of Decrement Factor. For the purposes of this report the wall mass per square metre will be considered to be representative of the total heat storing capacity of the wall and the product of this and total thermal resistance will be termed the Thermal Performance Factor (TPF)

3.1 TPF for Brick Veneer walls
The Total R-Value for the Brick Veneer wall system can be calculated as follows
Outside Air Film 0.04 (BCA, Table 2b wind 3 m/sec)
110 Brick Skin 0.17 (BCA, Table 2a, 3.25 kg
bricks)
Insulation 1.50 (R1.5 batts)
10 mm Plasterboard 0.06 (BCA, Table 2a)
Inside Air Film 0.12 (BCA, Table 2b)
Total 1.89 m².K/W
The walls have a mass of approximately 220 kg/m², yielding a TPF of 416.

3.2 TPF for insulated Hebel walls
The Total R-Value for the insulated Hebel wall system can be calculated as follows
Outside Air Film 0.04
75 thick Hebel Panels 0.51 (CSR Hebel Tech Manual)
50 thick Styrofoam 1.79 (www.aeromfg.com.au)
Unventilated Airspace 0.17 (BCA, Table 2b)
Extra for Reflective Surface 0.48 (BCA, Table 2b)
Blanket Insulation 1.50
10 mm Plasterboard 0.06
Inside Air Film 0.12
Total 4.67 m².K/W

The walls have a mass of approximately 65 kg/m², yielding a TPF of 303.

3.3 TPF for mud brick walls

The Total R-Value for the Mud Brick wall system can be calculated as follows:
Outside Air Film 0.04 (BCA, Table 2b wind 3 m/sec)
250 Mud Bricks 0.49 (*)
Inside Air Film 0.12 (BCA, Table 2b)
Total 0.65 m².K/W

* Values for thermal conductivity vary significantly with moisture content. A value of 0.49 is consistent with NZS 4297:1998 “Engineering Design of Earth Buildings”

The walls have a mass of approximately 410 kg/m², yielding a TPF of 266.

According to Szokolay (2004, Fig 1.58) the mud brick walls should have a time-lag of around 7 hours and a decrement factor (ratio of outer to inner surface temperatures relative to average) of around 0.3.

4 MEASURED THERMAL PERFORMANCE

Single temperature data loggers (DS 19121G iButton® Thermochron® ) were hung in the centre of each building and additional data loggers were fixed to the inside and outside of the western walls. An additional data logger was placed externally in a shaded box mounted on the south side of a building directly to the north (approx 1.5 metres above ground level).

4.1 “Cool Spell” performance

The performance of the buildings on the 20th and 21st August 2006 (late winter) was taken as indicative of the “Cool Spell” performance of the buildings. Figure 4 shows the temperatures recorded in the buildings and outside over these two days.

The average external temperature during the period was 9°C with a range from approx 1°C to 17°C. The average internal temperature in the three buildings was the same (14.7°C) with minimum temperatures of around 12°C and a maximum of 18°C (17°C for mud brick building).

The occurrence of the lowest temperatures in the buildings coincided roughly with the lowest temperature outside. The maximum temperatures in the
Hebel and Brick Veneer buildings occurred at the same time as the external temperature but there was a noticeably time lag to the maximum temperature in the Mud Brick building.

Figure 5 shows the temperature differential in the western wall of the Hebel building relative to the inside wall temperature. This clearly indicates the increase in temperature that occurs when the temperature difference is positive (approx from 10 am to 5 pm) and the resultant drop in temperature during the period of negative temperature differential (heat flow out of building).

4.2 “Hot Spell” performance
A hot spell occurred at the site on the 13th and 14th October 2006 (early Spring) with maximum temperature reaching around 35°C. Average external temperature
during these two days was 24.5°C with a range from 12°C to 35°C. Figure 6 shows the temperatures recorded in the buildings and outside over these two days.

The average temperature in the Brick Veneer building was also 24.5°C (Varied from 20–29°C), with a 1.1°C lower average in the Hebel (23.4°C –Varied from 19–28°C) and a 2.4°C higher average in the Mud Brick building (26.9°C-Varied from 22–31°C).

The external range was 23°C with the range in the other buildings fairly similar at 9°C.

Figure 7 shows the temperature differential in the western wall of the Hebel building relative to the inside wall temperature. In this case the positive temperature differential continued for around 12 hours (7.30 am to 7.30 pm).

Whilst the minimum temperature differential was similar to that in August the maximum temperature difference between the inside and the outside of the western Hebel building wall reached 35°C. This is around 13 higher than the maximum temperature differential in
the Autumn and is reflected in the higher average internal temperatures recorded.

4.3 Time lag and decrement factor

Generally speaking there was little time lag for the Hebel and Brick Veneer buildings, but a significant time lag of around 4 hours for the Mud Brick building (see Figure 8). The Decrement Factor was approx 0.5. Both the time lag and the decrement were significantly different from the values predicted by Szokolay (7 hrs and 0.3).

5 THERMAL MASS v INSULATION

Figure 9 summarises the properties and performance of the three buildings.

Generally speaking there was little correlation between performance and thermal mass and resistance in the case of the brick veneer and Hebel buildings. The Hebel building has a significantly higher R value and has lower thermal mass than the Brick Veneer building yet performed fairly similarly during the cool and hot spells (except for being 1°C cooler in the hot spell). The TPF for the Hebel building is lower than that for the brick veneer building and does not account for the slightly better performance of the Hebel building.

During the cool spell the mud brick building took longer to heat up with a resulting lower maximum temperature and in the hot spell it continued to heat up after the other two buildings were starting to cool down. This is reflected in the thermal lag and it seems that the effect of thermal mass is the driver in this case.

In the case of the Brick Veneer and Hebel buildings it appears that, for the cool spell, the position of the thermal mass close to the outside and the higher R value stops the “flywheel” effect. Heat entering the building through the door is not absorbed by the inner stud wall, and is prevented from reaching the thermal mass on the outside by the higher R value, and this results in a slightly higher indoor maximum temperature.

For the Hot spell relatively more heat is entering the building through the walls, in addition to the door. In the case of the Mud Brick building this heat quickly reaches the inner surface due to the low R value and the stored heat continues to radiate after the outside temperature reaches its peak, resulting in a higher indoor temperature.
CONCLUSIONS

The recorded performance of the three buildings was not adequately predicted by either the R value, the wall mass or the TPF, although the performance of the Mud Brick building appears to reflect the effect of the high thermal mass of the walls. Further research is being carried out to see whether these trends are replicated in the mathematical model but it appears from this data that one dimensional models based on the diffusivity of the walls do not adequately predict the performance of whole buildings.

ACKNOWLEDGEMENT & DISCLAIMER

The authors would like to acknowledge the generous monetary assistance provided by CSR Hebel, without whom this project would not have been possible. The authors would like to make the point that the results presented here are for a particular configuration and method of operation of the test buildings only and should not at this stage be interpreted as conferring any particular preference for one or other form of construction.

REFERENCES

Building Code of Australia (BCA), 2006, Volume 1 - Class 2 to Class 9 Buildings, ABCB.

Hassall, D.N.H., 1973, Reflective Insulation and the Control of Thermal Environments, St Regis-ACI, Sydney.


Robertson, D.K. & Christian, J.E., 1985, Comparisons of Four Computer Models with Experimental Data from Test Buildings in Northern New Mexico, ASHRAE Transactions, Volume 91, Part 2B.


LEED-NC version 2.2 rating system applications of common structural materials

J.E. Maher
Opus Architects & Engineers, Minnetonka, MN, USA

K.W. Kramer
Department of Architectural Engineering and Construction Science, Kansas State University, Manhattan, KS, USA

ABSTRACT: Comprehensive understanding of building materials has been the basis of structural engineering. The rising environmental concern is making sustainability a crucial issue in our society. In creating a sustainable built environment, the architect usually takes the lead role with the mechanical engineer having the key responsibility for energy and water savings. Only recently have structural engineers and civil engineers begun to see the real potential of their contributions. This paper contains information pertaining to the four most common structural materials: reinforced concrete, reinforced masonry, steel, and timber. For each material, the sustainability of the material as defined by the LEED-NC Version 2.2 rating system is discussed. Information is provided on how to attain LEED points for a specific material. Whether the LEED-NC Version 2.2 rating system accurately portrays sustainability of common structural materials or needs further development is discussed in the conclusion. A comparison is provided of the four common structural materials in relation to the rating system.

1 INTRODUCTION

Edmund Burke once wrote, “Society is a partnership, not only between those who are living, but between those who are living, those who are dead, and those who are to be born” [Burke 1790]. This statement exemplifies the importance of sustainability, and why it is imperative to consider sustainability in all aspects of design and construction. Implementation of sustainability practices in the built environment is a trend that will continue as owners, along with the building and construction industries, are exposed to society’s increasing concern about the environment. Though involvement in sustainability is voluntary for some, the entire building industry is realizing its importance, as new standards are set. As of April 2005, 46 county, city, and state governments have adopted policies requiring or promoting the use of sustainability with the goal of encouraging environmentally sensitive and responsible design to create healthy communities [USGBC 2006].

Leadership in Energy and Environmental Design (LEED) was formed by the United States Green Building Council (USGBC) to address this concern. LEED created a rating system entitled LEED for New Construction and Major Renovation Projects, otherwise referred to as the LEED-NC Version 2.2 rating system which provides a comprehensive framework to assess the overall building performance and a points system for achieving certification levels.

Architects, along with mechanical engineers, are given the lead role in application of the LEED-NC rating system to the built environment. Structural engineers have had an obligation for the well-being of current and future generations, but until recently their importance in achieving sustainable design was not realized. Knowledge of the structural materials, coordination of structural systems with mechanical systems, and comprehension of alternative structural systems is essential for structural engineers. This information assists in creating innovative sustainable solutions, an integrated design, and aid in application of the LEED-NC Version 2.2 rating system. The sustainability phenomenon continues to grow necessitating the need for structural engineers to be able to fully assess common structural materials and their benefits in sustainable design.

This paper discusses how the structural engineer can achieve this. Information is provided on why the LEED-NC Version 2.2 rating system is utilized to gauge sustainability along with an explanation of how the system assesses the built environment. The focus of the paper pertains to the four most common structural

549
materials: reinforced concrete, reinforced masonry, steel, and wood. For each one, the relevancy to LEED-NC is explained.

2 IMPORTANCE OF SUSTAINABILITY

Comprehension of sustainable design is necessary before one can examine sustainability of structural materials. The World Commission on Environment and Development (WCED) defines sustainability as “meeting the needs of the present without compromising the needs of the future.” Sustainability is a word that encompasses many terms. Green design, green buildings, green engineering, green design, sustainable buildings, green architecture, ecological design, eco-effective, holistic, and environmentally friendly design are just some of the names used to refer to the issues of sustainability of structural materials. All of which encompass a similar meaning [McLennan 2004]. For the purpose of this paper, sustainable design will be used.

Sustainable design has many benefits. Considering the environmental impact, the obvious contribution is the ability to enhance and protect ecosystems and biodiversity by limiting the building’s environmental footprint. Building’s environmental footprint is the building’s use of public utilities and the waste put back into the environment. Some environmental benefits are improvement in air and water quality, reduction in solid wastes, and natural resources conserved. Economic benefits include reduced building operating costs, enhancements of the asset value and profits by improved employee satisfaction and efficiency, and optimized life-cycle economic performance. Additionally, tax credits and other incentives provided by the government are becoming common to promote sustainable design.

Benefits to people include improved air quality, increased thermal comfort, better acoustical environments, enhanced well-being and comfort of the occupant, and overall improvement in the quality of life.

2.1 The role of the construction and building industry

The functionality, aesthetics, healthfulness, safety, environmental quality, and economy of buildings are vital to the quality of life and productivity of the individuals who use these buildings. In the United States (U.S.) in 2002, new construction and renovation of buildings accounted for about 9% of the Gross Domestic Product; the value of existing buildings comprised about 48% of the fixed, reproducible, tangible wealth; buildings consume about 40% of the energy; and construction wastes were estimated to be 20–30% of the contents of landfills [U.S. Bureau of the Census 2002]. In addition, buildings account for 36% of the total energy use, 65% of the electricity consumption, 30% of greenhouse gas emissions, 30% of raw materials use, 30% of waste output (136 million tons annually), and 12% of potable water consumption [USGBC 2005b].

Clearly, there are areas of opportunity that sustainable design can address. Engineers can play a significant role in planning, designing, building and maintaining the environment. Engineers must participate with owners, suppliers, investors, regulators, community interest groups, ecologists, sociologists, and professionals from other disciplines in order to achieve sustainable strategies and solutions. Engineers provide the bridge between science and society. For example, by addressing an element in the code of Ethics of the American Society of Civil Engineers (ASCE), the engineer has a responsibility to support sustainability that requires civil engineers to strive to comply with the principles of sustainable development in the performance of their professional duties [Vanegas 2004]. Fortunately, breakthroughs in building science, technology, and operations are available to assist in advancing sustainability. Designers, builders, and owners have the ability to maximize both economic and environmental performance for the inhabitants while preserving the environment [USGBC 2005b]. Achievement of sustainability requires engineering expertise. Structural engineers can contribute to sustainability in many ways:

- Comparing embodied energy of various construction materials and systems.
- Coordinating structural systems with mechanical engineers to enhance the efficiency of Heating, Ventilation, and Air Conditioning (HVAC) systems.
- Teaming with building officials to allow both alternative materials and systems.
- Working with wind-tunnel studies of double-skin facades to create naturally ventilated buildings [Field & Hun 2006].

Structural engineers have the duty to lessen the environmental impact by using their comprehensive understanding of building materials to create integrated sustainable designs.

3 ASSESSING SUSTAINABLE DESIGN

In response to the rising concern for the environment and sustainability of the U.S., two primary approaches have emerged to gauge the environmentally friendliness of a building. One approach is Life-Cycle Analysis/Assessment (LCA) which emerged in the early 1970s. The second approach is Leadership in Energy and Environmental Design (LEED).

LCA has several interchangeable terms; these include life cycle inventory, cradle-to-grave analysis,
product life-cycle analysis, environmental profile analysis, and eco-balance. These terms all mean “measuring the total impact of a product on the environment – from when the raw materials are extracted, through the product’s life as a consumer item, to when it is disposed of or recycled” [FWPRDC 2006]. This method is much more elaborate than LEED rating system, because it involves extensive computation and documentation of data (i.e. embodied energy, life cycle cost) not readily assessable and can be challenging to accurately develop [Hewitt 2003]. Unlike the LEED system, LCA has multiple methods to rate a building varying in complexity.

In using LCA, results are often based on inadequate and incomplete data because of a lack of documentation and unavoidable assumptions about product life resulting from a lack of scientific basis [FWPRDC 2006]. Subjective questions arise, such as; how should heavy water use be compared to heavy energy demand; how should the combined impacts of landfilling of wastes be compared to pollution; and how should wastes created from burning be compared to energy production, etc. Unavoidably, the LCA approach entails subjective decisions in determining the significance of differing qualitative values [GDRC 2006]. Often, LCAs arrive at different and conflicting results. Even with similar products, assumptions are necessary. “For example, whether deliveries were made in an 8.2 ton (9 U.S. tons) truck, or a larger one, whether it used diesel or petrol (gasoline), and ran on congested city roads where fuel efficiencies are lower, or on country roads or motorways where fuel efficiencies might be better. Comparisons of products which are dissimilar in most respects can only be made by making even more judgments and assumptions” [GDRC 2006]. A further aspect to consider is obtaining accurate data while maintaining the confidentiality of commercially-sensitive data. Companies are understandably reluctant to publish information, which may indicate inferiority to competitors [GDRC 2006]. Currently, LCA is not developed enough to merit comparison of structural materials. “The system’s precision and usefulness as a comparative tool give it potential for future use, but the difficulties in obtaining accurate energy information could preclude the method’s use for some time” [Hewitt 2003].

The second approach, LEED, was developed by the United States Green Building Council (USGBC). The USGBC was formed in 1993 as “the nation's foremost coalition of leaders from across the building industry working to promote buildings that are environmentally responsible, profitable and healthy places to live and work” [USGBC 2005a]. USGBC represents the largest breadth of membership in the entire green building industry, thereby creating a strong and diverse organization. Unique perspectives combined with collective power give members the opportunity to “effect change in the way buildings are designed, built and maintained” [USGBC 2005a]. The council works together developing, managing, and forging alliances by various means to increase the support for sustainable building design. With this purpose in mind, the USGBC created 28 committees to focus on different programs of concern. “Council programs are committee-based, member-driven, and consensus-focused” [USGBC 2005a]. To remain consensus-focused, the USGBC employs decision-making that encompasses a diverse membership. “The Council’s policy for balloting LEED products helps assure that all membership comments are considered and that final decisions and rationales are conveyed to the membership prior to final votes” [USGBC 2005a]. Moreover, whereas consensus typically means that a simple majority of those voting approves a given issue, Council policies require a two-thirds approval of those voting for a standard to be approved [USGBC 2005a]. Based upon these ideals, the council members established LEED. Though neither LEED nor LCA are flawless in any context, LEED is currently the more accepted approach in the United States, because LEED is less complicated and more encompassing in comparison to the LCA.

3.1 Leadership in Energy and Environmental Design (LEED)

As previously stated, USGBC members from all segments of the building industry collaborated to continually develop LEED. LEED’s mission statement is: LEED encourages and accelerates global adoption of sustainable green building and development practices through the creation and implementation of universally understood and accepted standards, tools and performance criteria [LEED 2005]. The evolution of LEED was based on promoting integrated building design practices and raising consumer awareness of the benefits of building green while in the process generate a consensus-based standard for evolving sustainable buildings. The objective is that LEED standards would stimulate a transformation of the building market towards “green” competition and in turn LEED would recognize sustainable leadership in the building sector. Currently, LEED has six standards. The forefather standard, originally published in 1999 and now on edition three (LEED-NC Version 2.2), and the most commonly used in the green building industry is New Commercial Construction and Major Renovation Projects (LEED-NC). LEED-NC provides a comprehensive framework to assist in meeting the sustainability standards and assessing the overall building performance.

3.1.1 LEED-NC Version 2.2 rating system

LEED-NC is a sustainable building rating system designed to guide and distinguish high-performance
commercial and institutional projects, with an original focus on office buildings which has evolved to K-12 schools, multi-unit residential buildings, manufacturing plants, laboratories and many other building types. [LEED 2005]. LEED-NC Version 2.2 is the current document used for new construction and major renovations projects designed to achieve LEED certification. The four certification levels, shown in Table 1, are used to recognize achievements for building green.

LEED-NC is divided into five main categories concerned with the quality of sustainability accounting for 64 of the 69 points possible. The prerequisites must be met first in all the categories before a project can obtain any credits. Once the prerequisites have been satisfied, a project must obtain a certain number of credits to achieve the desired level of certification. The other 5 points are earned through innovation and the use of a LEED-accredited professional on the project. To aid with this process the LEED-NC Version 2.2 manual has intents, requirements, potential technologies, and strategies for acquiring each credit. Refer to Table 2 for a summarization of the categories, credits, and points possible.

The categories in the LEED-NC rating system that pertain to the sustainability of structural materials are Sustainable Sites, Energy & Atmosphere, Materials & Resources, Indoor Environmental Quality, and Innovation & Design Process. A detailed explanation of each category is available in the LEED Reference Guide. A brief description of these categories is as follows:

- Sustainable Sites: Consideration in site selection to preserve or restore the local ecosystem. Site selection also has the ability to impact credits that may not seem directly associated with the site. For instance, a site near public transportation or in a previously developed area reduces the amount of driving by the building’s occupants.
- Energy & Atmosphere: To help reduce energy use and protect the ozone layer by minimizing the amount of energy necessary and optimize the energy performance of the structure.
- Materials & Resources: To conserve raw materials and resources, for example fossil fuels. Techniques include increasing recycling and recycled content, redirecting material from landfills, and decreasing travel distances for material and product transport.
- Indoor Environmental Quality: Due to Americans spending 90% of their time inside, the indoor environment has a crucial affect on health and productivity. Related concerns encompass occupant comfort, quality of the air, thermal comfort, and daylight access.
- Innovation & Design Process: The USGBC acknowledges that LEED is a relatively new system – possibility exists that not all sustainable building aspects are covered. This category recognizes innovative solutions and having a LEED accredited professional on the project [Stern et al. 2003].

4 LEED-NC AND STRUCTURAL MATERIALS

The categories that pertain, directly or indirectly, to the four structural materials covered in this paper are Sustainable Sites, Energy & Atmosphere, Materials & Resources, Indoor Environmental Quality, and Innovation & Design Process. The following sections will explain the applicable credits based upon information obtained in the LEED-NC Version 2.2 Reference Manual. Correlations of these credits to the common structural materials will be made in order to understand the building’s environmental footprint. The pertinent credits are summarized in Table 3 located in the Appendix.

4.1 Sustainable Sites

The first category listed in Table 3 in Sustainable Sites. Destruction to local ecology is often caused by development and construction processes. Within Sustainable Sites credits 3, 5.1, 5.2, 6.1, and 7.1 apply to structural material sustainability.

4.1.1 Sustainable sites, credit 3, brownfield redevelopment

The intent of this credit is to rehabilitate sites damaged by environmental contamination thereby reducing demands on greenfield sites. To achieve this credit, one must develop on contaminated land or land
classified as a brownfield site. Documentation is mandatory indicating either site contamination or a brownfield site and the remediation implemented. In these areas, cement can be utilized to stabilize and solidify contaminated soils while reducing leachate concentrations to acceptable levels. As indicated in Table 4 located in the Appendix of the four common materials reviewed, concrete is the only structural material which can obtain this credit which is worth 1 point.

4.1.2 Sustainable sites, credit 5.1, site development – protect or restore habitat

The intent of this credits to provide habitat and promote biodiversity by preserving existing natural areas and restoring damaged regions. This credit can be obtained by two options. The requirements vary based on whether the site is a greenfield or brownfield site. Option 1 pertains to greenfield sites as follows, “On greenfield sites, limit all site disturbance to 12 m (40 feet) beyond the building perimeter; 3 m (10 feet) beyond surface walkways, patios, surface parking and utilities less than 300 mm (12 inches) in diameter; 4.5 m (15 feet) beyond primary roadway curbs and main utility branch trenches; and 7.5 m (25 feet) beyond constructed areas with permeable surfaces (such as pervious paving areas, stormwater detention facilities and playing fields) that require additional staging areas in order to limit compaction in the constructed area” [LEED-NC 2005].

Option 2 pertains to previously developed sites, often a brownfield site and is further defined as, “On previously developed or graded sites, restore or protect a minimum of 50% of the site area (excluding the building footprint) with native or adapted vegetation. Native/adapted plants are plants indigenous to a locality or cultivars of native plants that are adapted to the local climate and are not considered invasive species or noxious weeds. Projects earning SS Credit 2 and using vegetated roof surfaces may apply the vegetated roof surface to this calculation if the plants meet the definition of native/adapted” [LEED-NC 2005].

Since surface parking tends to have large impact on the site, providing a parking garage will reduce this impact. Parking structures typically constructed on reinforced concrete on the lower levels of structural systems such as cast-in-place, precast, tilt-up, reinforced masonry, steel, or wood structural systems. Another option, as used for the Kandalama Hotel located in Damulla, Sri Lanka, is to provide columns to elevate the buildings above the natural features such as boulders and to reduce cut-and-fill needs.1 This credit is worth 1 point.

4.1.3 Sustainable sites, credit 5.2, site development – maximize open space

The intent of this credit is to support biodiversity by providing a high proportion of open space to developed area. This credit is achieved by meeting the requirement of one of three options. Option 1 is as follows, “reduce the development footprint (defined as the total area of the building footprint, hardscape, access roads and parking) and/or provide vegetated open space within the project boundary to exceed the local zoning’s open space requirement for the site by 25%.

Option 2: for areas with no local zoning requirements (e.g., some university campuses, military bases), provide vegetated open space area adjacent to the building that is equal to the building footprint. Option 3 is defined as where a zoning ordinance exists, but no requirement for open space, provide vegetated open space equal to 20% of the project’s site area” [LEED-NC 2005].

The design engineer has multiple options to obtain this Sustainable Sites, Credit 5.2. Generally the development footprint can be reduced by having parking garages on the lower levels of structural systems such as cast-in-place, precast, tilt-up, reinforced masonry, steel, or wood structural systems. Another option, as used for the Kandalama Hotel located in Damulla, Sri Lanka, is to provide columns to elevate the buildings above the natural features such as boulders and to reduce cut-and-fill needs.1 This credit is worth 1 point.

4.1.4 Sustainable sites, credit 7.1, Heat Island Effect – Non-Roof

The intent of this credit is to minimize the effect on microclimates and in turn the biosphere by means of reducing heat islands from non-roofs. Credit for Heat Island Effect, Non-Roof can be obtained by fulfilling one of two options. Option 1 is listed as follows “Provide any combination of the following strategies for 50% of the site hardscape (including roads, sidewalks, courtyards and parking lots), shade (within five years of occupancy), paving materials with a Solar Reflectance Index (SRI) of at least 29 out of a possible 100, and Open grid pavement system” [LEED-NC 2005].

Option 1, of this credit, can be achieved by using concrete. This is done by using concrete instead of asphalt for a specified portion of all sidewalks, parking lots, drives, and other non-roof impervious surfaces. Concrete works well in this application, containing relatively high albedos. “Albedo is the ratio of the amount of solar radiation reflected from a material to the amount shone on the material” [Vangeem & Marceau 2002]. In general, surfaces containing high albedo are light in color absorbing less energy and thus cooler. Whereas surfaces with lower albedos absorb more solar radiation, that converts into heat, causing the surface to become hotter. Concrete’s reflective surfaces save

---

1 Additional information on case studies referenced in this manuscript may be found in “Primary Sustainability Features and LEED Applications of Common Structural Materials” by J.E. Maher 2006.
energy by reducing temperature which in turn cuts air-conditioning usage leading to less power needed and improved air quality. Concrete generally has an albedo or solar reflectance of about 0.35, although values may vary. New asphalt concrete generally has a reflectance of around 0.05 and asphalt concrete older than four years has a reflectance of approximately 0.10 to 0.15 [Vangeem & Marceau 2002]. Though not an obvious application for structural concrete the possibility exists to achieve credit.

Option 2 deals with the structure, “Place a minimum of 50% of parking spaces under cover. Any roof used to shade or cover parking must have an SRI of at least 29 out of a possible 100” [LEED-NC 2005]. Option 2, of this credit, pertains to using parking garages. All four common structural materials discussed in this report can assist in obtaining this option. Meeting the requirements of either option is worth 1 point.

4.1.5 Sustainable sites, credit 7.2, heat island effect – roof

The intent of this credit is to minimize the effect on microclimates and in turn the biosphere by means of reducing heat islands due achieve this credit one of the following three options must be fulfilled. To achieve this credit one of the following three options must be fulfilled.

Option 1 is defined as, “use roofing materials having a Solar Reflectance Index (SRI) equal to or greater than 75% of the roof surface” [LEED-NC 2005]. Option 2 is to, “install a vegetated roof for at least 50% of the roof area” [LEED-NC 2005]. Option 3 is as follows, “install high albedo and vegetated roof surfaces that, in combination, meet the following criteria: (Area of SRI Roof / 0.75) + (Area of vegetated roof / 0.5) ≥ Total Roof Area [LEED-NC 2005]. Further explanation of high albedo is found in the LEED-NC Version 2.2 Reference Guide.

A possible option for attaining this credit is to install a vegetated roof system. Though indirectly related, the type of structural material used to support this system does depend on the additional loads created from vegetation, spanning requirements, and the architectural design being expressed therefore consideration of the type of structural material used is necessary. The largest known example of a green roof system is currently the Ford Dearborn Truck Assembly located in Dearborn, Michigan. A steel superstructure supports over 4 hectares (approximately 10 1/2 acres) of green roof. A green roof regardless of the structural system is worth 1 point. All four common structural materials can obtain this point as shown in Table 4.

4.2 Energy & Atmosphere

Another category listed in Table 3 is Energy & Atmosphere. Prereq 2 and Credit 1 apply to structural materials. This category implements a number of strategies to help reduce energy use and protect the ozone layer. As pertaining to structural materials, the purpose is to minimize the amount of energy necessary and optimize the energy performance of the structure.

4.2.1 Energy & atmosphere, prereq 2, minimum energy performance

The intent of this prerequisite is to establish the minimum level of energy efficiency for the anticipated systems and buildings as required by the American Society of Heating, Refrigerating, and Air-Conditioning Engineers (ASHRAE) Standard 90.1–2004. This is done by maximizing the energy performance of the anticipated systems and building envelope. Once the minimum standard has been met i.e. Prereq 2, then points for the following credit can be obtained.

4.2.2 Energy & atmosphere, credit 1, optimize Energy Performance

The intent of this credit is to reduce the impacts (environmental and economical) associated with excessive energy use by going beyond Prereq 2 (acting as the baseline) and achieves incremental levels of energy performance above the requirements of ASHRAE 90.1–2004. The thermal mass of concrete or masonry, when combined with insulating materials, is very useful to achieve points for this credit. Concrete and masonry have the ability to absorb the heat/cold and slowly releasing it thereby reducing temperature swings and energy loads in a building. This lessens the strain on the HVAC system and reduces energy waste. Often a computer program will be used to more accurately capture the beneficial thermal properties of concrete or masonry. This is accomplished by calculating yearly energy usage based on hourly data. Points are awarded based on energy cost savings from 15 to 60% for new buildings. This credit is worth 1–10 points [PCA 2005a].

4.3 Materials & Resources

Another rating system category which applies to structural materials is Materials & Resources. This category covers a broad range of credits including Building Reuse, Construction Waste Management, Material Reuse, Recycled Content, Regional Materials, Rapidly Renewable Materials, and Certified Wood. Building materials choices are important in sustainable design because of the extensive network of extraction, processing and transportation steps required to process them. Almost all of the credits available can apply to structural materials.

4.3.1 Materials & resources, prereq 1, storage & collection of recyclables

The intent of this prerequisite is to reduce the amount of waste, anything that is transported to and disposed
of in landfills, generated by the building occupants. Once the minimum standard has been met i.e. Prereq 1, points for the following credits within the Materials & Resources category can be obtained.

4.3.2 Materials & resources, credits 1.1 and 1.2, building reuse

The intent of these credits is to extend the life cycle of existing infrastructure. For new infrastructure, the purpose is to reduce environmental impacts as it applies to the embodied energy of the material by retaining portions of the existing walls, floors, and roof. The following requirements to be met to obtain credit 1.1 are “Maintain at least 75% (based on surface area) of existing building structure (including structural floor and roof decking) and envelope (exterior skin and framing, excluding window assemblies and non-structural roofing material). Hazardous materials remediated as a part of the project scope shall be excluded from the calculation of the percentage maintained. If the project includes an addition to an existing building, this credit is not applicable if the square footage of the addition is more than 2 times the square footage of the existing building” [LEED-NC 2005].

All four common structural materials can be utilized to obtain this credit. Concrete has a long service life and with proper planning, reinforced concrete used in almost any structural system can easily be modified/retained for expansion purposes. Masonry and wood are known for being durable structural materials thus making them ideal for reuse. Steel structures are easily reused because they can be economically adapted and reinforced which permits flexibility and adaptability in the modified use of the structure. To achieve the first point, at least 75% (based on surface area) of the existing structural system must be retained. To attain credit 1.2 an additional 20% (95% total) of the building’s surface area needs to be reused.

4.3.3 Materials & resources, credits 2.1 and 2.2, construction waste management

Minimize landfill disposal and incineration of construction debris is the intent of these credits. This is to be accomplished by redirecting reusable materials to suitable sites, and divert recyclable, recovered resources to suitable locations. To fulfill the requirements for credit 2.1 the following must be met, “recycle and/or salvage at least 50% of non-hazardous construction and demolition debris. Develop and implement a construction waste management plan that, at a minimum, identifies the materials to be diverted from disposal and whether the materials will be sorted on-site or co-mingled. Excavated soil and land-clearing debris do not contribute to this credit. Calculations can be done by weight or volume, but must be consistent throughout the computation process” [LEED-NC 2005]. To obtain credit 2.2 the above requirements for credit 2.1 must be met plus an additional 25% (75% total) of non-hazardous construction and demolition debris is required to be recycled and/or salvaged.

Reinforced concrete is a construction material which is frequently being crushed and recycled into aggregate. Based on this, when any type of structure is demolished one can pursue credit in this category. The modularity of masonry creates less waste in general, because the module limits design and construction possibilities. The waste that does occur from masonry construction can be crushed and recycled. After concrete masonry waste has been crushed, it can be used for aggregate or fill. Crushed clay brick waste can be used as brick chips for landscaping. The masonry units that are intact can be used on another project or donated to a charitable organization [Subasic 2004]. Steel is one of the most recycled materials ensuring that virtually any steel on a construction site can be reused or recycled [Stern et al. 2003]. Wood is known to be a very adaptable building material meaning the ability to often cut on-site, instead of prefabricated which creates construction waste. This construction waste when managed (recycled) will leave little if any impact on the environment. This credit is worth 1 point when at least 50% of non-hazardous construction and demolition debris is recycled and/or salvaged and 2 points given for at least 75%.

4.3.4 Materials & resources, credits 3.1 and 3.2, materials reuse

The intent of both credits is to reuse building products and materials in order to decrease the use of virgin materials and reduce waste, lessening the impact associated with their extraction and processing. To achieve credit 3.1, salvaged, refurbished, or reused materials must be used, so that the sum of these products and materials is at least 5% (based on cost) of the total amount of materials on the project. Items not to be included are mechanical, electrical, plumbing, and specialty items. Only materials permanently installed in the project can be included. An extra credit can be achieved if an additional 5% (total of 10%) of the total materials have been salvaged, refurbished, and or reused [LEED-NC 2005].

Though still emerging, steel reuse is becoming a more common practice and numerous documentations, as mentioned earlier, of using salvaged steel in new, additions, or relocated structures have occurred. Wood has the ability to be salvaged from building deconstruction projects and saved for reuse. “Salvaged wood can be of higher quality than equivalent wood milled today, tending to be denser and to have fewer knots” [Larry McFarland Architects Ltd. 2003]. Another advantage is the manufacturing of heavy timbers, large dimension lumber, and glulam beams tend to be costly making salvaged wood more
advantageous. Unfortunately salvaged wood is not always easy to obtain which is why the United States Department of Agriculture (USDA) Forests Products Lab developed a Directory of Wood-Framed Building Deconstruction and Reused Building Materials Companies [Bland 2005]. Combined, these credits are worth 2 points.

Though both of these credits directly relate to steel and wood, masonry is durable and has the potential to indirectly be reused and salvaged. The Brick Industry Association does warn against their use since reused brick may not meet the requirements of present-day specifications and may not bond properly. Paver brick that are salvaged and used for interior applications on a new building meet the intent of this credit. “However, most masonry units can be reused when carefully dismantled. A good example of this is the Mountain Equipment Co-op building in Winnipeg, constructed using significant amounts of re-used brick” [Masonry Canada 2005].

### 4.3.5 Materials & resources, credits 4.1 and 4.2, recycled content

The intent of both credits is to reduce the impact from extraction and processing of raw materials while increasing the demand for incorporation of recycled material content in buildings. Credit 4.1 is awarded, if the sum of post-consumer recycled content plus one-half of the pre-consumer content constitutes at least 10% (based on cost) of the total value of the materials in the project. For the purpose of this credit, LEED has defined pre-consumer as material that has been diverted from the waste stream during manufacturing, while explaining post-consumer as all waste generated by some form of human consumption (from factory to household use) that can no longer be used for its intended purpose. Credit 4.2 is achievable by having at least an additional 10% (total of 20% of cost) of materials with recycled content [LEED-NC 2005].

Concrete, clay brick, concrete blocks, reinforcing steel, grout, and engineered lumber can all be produced from at least a portion of recycled materials. Any concrete mix can be partly composed of supplementary cementitious materials (SCMs). Since LEED considers reinforcing steel to be separate from the concrete, using recycled rebar will also help achieve credit. As previously mentioned, the recycled content of steel is one of its greatest advantages. In 2004, according to the Steel Recycling Institute, the EAF and BOF processes used 81% post-consumer recycled content and approximately 16% pre-consumer recycled content. Specifying engineered and composite lumber products that has recycled content also achieves this credit. “Engineered lumber is manufactured by combining wood fibers with plastic resins to produce high quality, structural products such as wood I-joists, laminated veneer lumber (LVL), parallel strand lumber (PSL), and glulam beams. Sheathing products manufactured in this manner, such as oriented strand board (OSB), wafer board and particle board, are made primarily of saw mill waste. Likewise, finger-jointed lumber made from wood scraps makes use of material that would otherwise be wasted” [Denver AIA 1997].

#### 4.3.6 Materials & resources, credits 5.1 and 5.2, regional materials

The intent of these credits is to support the indigenous resources while reducing transportation causing ecological impacts. To achieve 1 point, a minimum of 10% (based on cost of the total material value) of regional building materials must be used. 2 points can be awarded for at least 20%.

Concrete being cast on-site, or at a nearby facility, is a very viable option for attaining these credits. “Concrete mix plants generally use aggregate that are extracted within 80 km (50 miles) of the plant. Cement and SCMs used for buildings are also often manufactured within much less than 80 km (500 miles) of a job site. Reinforcing steel is usually manufactured within 800 km (500 miles) of a job site, and is typically made from recycled materials from the same region” [PCA 2005a]. Manufacturing facilities for masonry are located all across the nation making these credits readily attainable. Until recently, obtaining credit in the western United States for manufacturing steel within 800 km (500 miles) was difficult. This was because only four locations exist where wide flange sizes, W14x43 and larger, are rolled and all are on the eastern half of the United States. Then a credit interpretation ruling for structural steel, in February 2004, established that “the fabrication shop is considered the location where the final manufacture of the product occurred. Fabricators cut steel members to their appropriate length, punch or drill holes, weld on connection plates, and add the necessary camber to members. Fabricators may also build the steel into standard assemblies, such as trusses or frames. Steel fabricators are available within 800 km (500 miles) of most locations in the United States and the use of local fabricators fosters local economies for the product, which reinforces the intent of the credit” [Stern et al. 2003]. The extracting or harvesting of raw materials within 800 km (500 miles) of the jobsite is more difficult to achieve. For steel this includes the “location where the metal served its last useful purpose before it became scrap. Steel mills typically acquire scrap from brokers, who obtain materials from projects and products throughout the country, which are selected based upon metallurgical needs and cost. Acquiring wood products manufactured within 800 km (500 miles) is usually feasible. The most reliable way to confirm if these credits are attainable is to contact a building material supplier...
located near the future building site. All four structural materials may obtain these credits.

4.3.7 Materials & resources, credit 6, rapidly renewable materials

The intent of this credit is to utilize rapidly renewable materials, thereby reducing the need for finite raw materials and long-cycle renewable materials. A minimum of 2.5% (of the total value) needs to be from rapidly renewable building materials and products. This credit appears to be easily attainable for wood, but in actuality quite the opposite is true. LEED defines a rapidly renewable material as maximum of a 10-year harvest cycle. Generally only southern states, growing poplar or aspen, are able to meet this requirement due to longer growing seasons. “On a more positive note, OSB and other composite wood products do make extensive use of species, such as aspen or poplar, not traditionally used for lumber, many of which have a potentially much shorter harvest cycle than the species traditionally used for lumber” [Larry McFarland Architects Ltd. 2003]. An example of how this credit could apply structurally is the Solar Living Center in Hopland, California. Wood structures may be awarded 1 point for achieving this credit.

4.3.8 Materials & Resources, credit 7, certified wood

The intent of this credit is to promote environmentally conscientious forest management. To achieve this credit, a minimum of 50% of wood-based materials and products need to be used. The entire 50% needs to be certified in accordance with the Forest Stewardship Council (FSC)’s Principles and Criteria, for wood building components. Though FSC certified wood products are available, increased project cost often exists. The FSC provides a Supplier Referral Network website to assist in the process. Wood structures may be awarded 1 point for achieving this credit.

4.4 Indoor Environmental Quality

In the Indoor Environmental Quality category, Credits 4.4, 8.1, and 8.2 can apply to structural materials. The importance of this category is that Americans spend an average of 90% of their time indoors, where levels of pollutants may be two to five times higher than outdoor levels. Therefore, the cleaner the indoor environments are the healthier the individuals who use them.

4.4.1 Indoor Environmental Quality, credit 4.4, low-emitting materials

The intent of this credit is to reduce the amount of indoor air contaminants that are odorous, uncomfortable, and or dangerous to the well-being of humans. This credit refers to composite wood and agrifiber products that are manufactured using two primary ingredients: wood fibers or particles, and binders that adhere to the wood particles. Structural wood and agrifiber products include plywood, glued laminated timber (glulam), oriented strand board (OSB), laminated veneer lumber (LVL), laminated strand lumber, parallel strand lumber, and wood I-joists. Any of the aforementioned shall contain no added urea-formaldehyde resins, because urea-formaldehyde resins are carcinogenic and an irritant to most people when present in high concentrations resulting in headaches, dizziness, mental impairment, and other symptoms. Generally this credit, worth 1 point, is easily attainable when using wood products.

4.4.2 Indoor Environmental Quality, Credit 8.1 and 8.2, daylight & views

The intent of both of these credits is to introduce daylight and views into the commonly used areas of the building giving the occupants a sense of connection to the outdoors. Structural materials that are able to span longer distances based upon strength and deflection aid in attaining daylight and views in the building. Steel is able to span the longest distances based upon its strength to weight ratio. An example of steel’s possibilities is the Capitol Area East End Complex, Block 225 project in Sacramento, California. A 20 m (65 foot) steel girder was employed to span over the main lobby, creating an open space for daylight and views, to assist (among other reasons for using steel) in the project achieving a Gold rating. As a structural material, steel can assist in achieving 2 points for these credits.

4.5 Innovation & design process

4.5.1 Innovation & design process, credits 1.1, 1.2, 1.3, and 1.4, Innovation in design

The intent of these credits is for innovative green design strategies that do not correspond with any of the five LEED categories or for exceptional performance above the LEED-NC requirements. All four of the common structural materials could possibly attain up to 4 points either directly or indirectly. One possibility is to leave the structural system exposed. Not only does this create a pleasing architectural effect, but decreases the costs of finishing materials.

An aspect to consider when using reinforced concrete in sustainable design is the quantity of formwork required. Formwork is approximately 50% of the cost of reinforced concrete superstructure for a multistory commercial building. As mentioned in Reinforced Concrete, Section 5.0, several forms of reinforced concrete including cast-in-place concrete, precast, concrete, and tilt-up concrete are used in the building industry. Precast concrete uses less formwork since it is not manufactured on-site and typically uses less shoring and no reshoring. Tilt-up concrete is poured in a horizontal position, creating
Many sustainable aspects of masonry can be utilized to achieve points not accounted for in the current LEED-NC rating system. A couple advantageous ideas are the acoustical performance attainable using masonry and improved indoor air quality from masonry interior bearing wall systems creating virtually no off-gassing. Another quality to consider is masonry systems resistant to burn. “Masonry is inherently fire resistant and provides fire safety for people. Interior masonry fire partitions help stop the spread of fire. These aspects reduce the environmental impacts of fires. Passive fire protection reduces the costs of buildings” [Masonry Canada 2005]. Note that the aforementioned also applies to reinforced concrete.

Steel is known for being deconstruct-able, recyclable, and reusable. All of these factors can be used to surpass the LEED credit requirements and achieve additional points. Steel being naturally light in weight can also be advantageous in gaining points. In order to attain credits for this, the project team must clearly demonstrate significant savings using a conventional building as a baseline. An excellent example of this is the Utah Olympic Oval in Salt Lake City, Utah. The very shallow steel truss roof is supported by an innovative cable suspension system. The overall arena weighs over 865 tons (950 U.S. tons) less using this system than competing designs [Stern et al. 2003].

Wood has several possibilities for achieving points through innovation. Scraps from wood framing could be turned into wood chips onsite and used for landscaping creating virtually no waste. During the design phase, the architect and engineer need to communicate structural and aesthetical needs. If done correctly, openings for framing will coordinate with the framing module, and using exact stud height could mean a tremendous savings in lumber.

4.5.2 Innovation & design process, credit 2, LEED accredited professional

The intent of this credit is supplementary support during the application and certification. This credit is not specific to any structural material. As long as a LEED Accredited Professional is a principal participant of the project team this credit, worth 1 point, is attained.

5 CONCLUSION

As sustainability concerns increase, involvement by structural engineers is vital. The use of engineering knowledge of common and alternative structural materials, coordination of structural systems with mechanical systems, and comprehension of integrated structural systems are essential. Structural engineers having the ability to assess common structural materials and their sustainability benefits, plays a crucial role essential in sustainable design.

This paper provides the structural engineer with some of the information available to address the sustainability of the four most common structural materials: reinforced concrete, reinforced masonry, steel, and wood as they apply to the LEED-NC Version 2.2 rating system. A comparison of the sustainability of structural materials (including innovations), using the LEED-NC rating system, is provided in Table 4. Based the results in Table 4, reinforced concrete and masonry seem to be the most sustainable structural materials when evaluated by LEED-NC, though all four materials offer significant contributions.

One should consider that not all factors which make a material sustainable are properly represented using the LEED-NC rating system, thereby raising some concern. From a structural aspect, it is difficult to look at the system scoring ballot and tell which points were obtained using structural materials. Currently, the structural system seems insignificant to LEED since it is not mentioned or referred to. This should cause some concern, because the built environment would be nonexistent without the support of structural systems. The LEED-NC rating system does not directly consider energy usage (embodied energy) of a material which could increase or decrease the sustainability of the materials. For example, wood, the only naturally occurring and renewable structural material, received the lowest score as shown in Table 4.²

This seems inconsistent with the purpose behind sustainability. Further research needs to be done on how to better integrate structural materials with LEED-NC. Even with these concerns, the rating system has shown all four structural materials can make a significant impact in achieving sustainability. The adage “two minds are better than one” should be adapted to structural materials. By integrating multiple structural systems more LEED-NC points are achievable and often times a more efficient and economical design results.

Sustainability has come a long way over the past decades, as shown by the case studies mentioned in the paper. Though LEED-NC is a good tool for gauging sustainability, it must be realized that no system is perfect. Comprehension and implementation of sustainability is a continual process making it important to take the information provided in this paper, build upon it, and create a continual cycle of sustainable design for future generations. As Dietrich Bonhoeffer, a German theologian, once said, “the ultimate test of a moral society is the kind of world that it leaves to its children.”

² For further discussion refer Master’s Report entitled “Primary Sustainable Features and LEED Applications of Common Structural Materials” by J.E. Maher, 2006.
REFERENCES


### Table 3. Categories applicable to structural materials.

<table>
<thead>
<tr>
<th>Rating system categories</th>
<th>Points available</th>
<th>Applies structurally</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Sustainable Sites</strong></td>
<td>14</td>
<td></td>
</tr>
<tr>
<td>Prereq 1 Construction Activity Pollution Prevention</td>
<td>Required</td>
<td>–</td>
</tr>
<tr>
<td>Credit 1 Site Selection</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 2 Development Density &amp; Community Connectivity</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 3 Brownfield Redevelopment</td>
<td>1</td>
<td>Yes</td>
</tr>
<tr>
<td>Credit 4.1 Alternative Transportation, Public Transportation Access</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 4.2 Alternative Transportation, Bicycle Storage &amp; Changing Rooms</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 4.3 Alternative Transportation, Low-Emitting and Fuel-Efficient Vehicles</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 4.4 Alternative Transportation, Parking Capacity</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 5.1 Site Development, Protect or Restore Habitat</td>
<td>1</td>
<td>Yes</td>
</tr>
<tr>
<td>Credit 5.2 Site Development, Maximize Open Space</td>
<td>1</td>
<td>Yes</td>
</tr>
<tr>
<td>Credit 6.1 Stormwater Design, Quantity Control</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 6.2 Stormwater Design, Quality Control</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 7.1 Heat Island Effect, Non-Roof</td>
<td>1</td>
<td>Indirectly&lt;sup&gt;1&lt;/sup&gt;</td>
</tr>
<tr>
<td>Credit 7.2 Heat Island Effect, Roof</td>
<td>1</td>
<td>Indirectly&lt;sup&gt;1&lt;/sup&gt;</td>
</tr>
<tr>
<td>Credit 8 Light Pollution Reduction</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td><strong>Water Efficiency</strong></td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Credit 1.1 Water Efficient Landscaping, Reduce by 50%</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 1.2 Water Efficient Landscaping, No Potable Use or No Irrigation</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 2 Innovative Wastewater Technologies</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 3.1 Water Use Reduction, 20% Reduction</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 3.2 Water Use Reduction, 30% Reduction</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td><strong>Energy &amp; Atmosphere</strong></td>
<td>17</td>
<td></td>
</tr>
<tr>
<td>Prereq 1 Fundamental Commissioning of the Building Energy Systems</td>
<td>Required</td>
<td>–</td>
</tr>
<tr>
<td>Prereq 2 Minimum Energy Performance</td>
<td>Required</td>
<td>Required</td>
</tr>
<tr>
<td>Prereq 3 Fundamental Refrigerant Management</td>
<td>Required</td>
<td>–</td>
</tr>
<tr>
<td>Credit 1 Optimize Energy Performance</td>
<td>1–10</td>
<td>Yes</td>
</tr>
<tr>
<td>Credit 2 On-Site Renewable Energy</td>
<td>1–3</td>
<td>–</td>
</tr>
<tr>
<td>Credit 3 Enhanced Commissioning</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 4 Enhanced Refrigerant Management</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 5 Measurement &amp; Verification</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 6 Green Power</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td><strong>Materials &amp; Resources</strong></td>
<td>13</td>
<td></td>
</tr>
<tr>
<td>Prereq 1 Storage &amp; Collection of Recyclables</td>
<td>Required</td>
<td>Required</td>
</tr>
<tr>
<td>Credit 1.1 Building Reuse, Maintain 75% of Existing Walls, Floors, &amp; Roof</td>
<td>1</td>
<td>Yes</td>
</tr>
<tr>
<td>Credit 1.2 Building Reuse, Maintain 95% of Existing Walls, Floors, &amp; Roof</td>
<td>1</td>
<td>Yes</td>
</tr>
<tr>
<td>Credit 1.3 Building Reuse, Maintain 50% of Interior Non-Structural Elements</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 2.1 Construction Waste Management, Divert 50% from Disposal</td>
<td>1</td>
<td>Yes</td>
</tr>
<tr>
<td>Credit 2.2 Construction Waste Management, Divert 75% from Disposal</td>
<td>1</td>
<td>Yes</td>
</tr>
<tr>
<td>Credit 3.1 Materials Reuse, 5%</td>
<td>1</td>
<td>Yes</td>
</tr>
<tr>
<td>Credit 3.2 Materials Reuse, 10%</td>
<td>1</td>
<td>Yes</td>
</tr>
<tr>
<td>Credit 4.1 Recycled Content, 10% (post-consumer + ½ pre-consumer)</td>
<td>1</td>
<td>Yes</td>
</tr>
<tr>
<td>Credit 4.2 Recycled Content, 20% (post-consumer + ½ pre-consumer)</td>
<td>1</td>
<td>Yes</td>
</tr>
<tr>
<td>Credit 5.1 Regional Materials, 10% Extracted, Processed, &amp; Manufactured Regionally</td>
<td>1</td>
<td>Yes</td>
</tr>
<tr>
<td>Credit 5.2 Regional Materials, 20% Extracted, Processed, &amp; Manufactured Regionally</td>
<td>1</td>
<td>Yes</td>
</tr>
<tr>
<td>Credit 6 Rapidly Renewable Materials</td>
<td>1</td>
<td>Indirectly&lt;sup&gt;1&lt;/sup&gt;</td>
</tr>
<tr>
<td>Credit 7 Certified Wood</td>
<td>1</td>
<td>Yes</td>
</tr>
</tbody>
</table>

(Continued)
Table 3. (Continued).

<table>
<thead>
<tr>
<th>Rating System Categories</th>
<th>Points Available</th>
<th>Applies Structurally</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Indoor Environmental Quality</strong></td>
<td><strong>15</strong></td>
<td></td>
</tr>
<tr>
<td>Prereq 1 Minimum IAQ Performance</td>
<td>Required</td>
<td>–</td>
</tr>
<tr>
<td>Prereq 2 Environmental Tobacco Smoke (ETS) Control</td>
<td>Required</td>
<td>–</td>
</tr>
<tr>
<td>Credit 1 Outdoor Air Delivery Monitoring</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 2 Increased Ventilation</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 3.1 Construction IAQ Management Plan, During Construction</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 3.2 Construction IAQ Management Plan, Before Occupancy</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 4.1 Low-Emitting Materials, Adhesives &amp; Sealants</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 4.2 Low-Emitting Materials, Paints &amp; Coatings</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 4.3 Low-Emitting Materials, Carpet Systems</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 4.4 Low-Emitting Materials, Composite Wood &amp; Agrifiber Products</td>
<td>1</td>
<td>Yes</td>
</tr>
<tr>
<td>Credit 5 Indoor Chemical &amp; Pollutant Source Control</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 6.1 Controllability of Systems, Lighting</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 6.2 Controllability of Systems, Thermal Comfort</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 7.1 Thermal Comfort, Design</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 7.2 Thermal Comfort, Verification</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 8.1 Daylight &amp; Views, Daylight 75% of Spaces</td>
<td>1</td>
<td>Indirectly¹</td>
</tr>
<tr>
<td>Credit 8.2 Daylight &amp; Views, Views for 90% of Spaces</td>
<td>1</td>
<td>Indirectly¹</td>
</tr>
<tr>
<td><strong>Innovation &amp; Design Process</strong></td>
<td><strong>5</strong></td>
<td></td>
</tr>
<tr>
<td>Credit 1.1 Innovation in Design</td>
<td>1</td>
<td>Indirectly¹</td>
</tr>
<tr>
<td>Credit 1.2 Innovation in Design</td>
<td>1</td>
<td>Indirectly¹</td>
</tr>
<tr>
<td>Credit 1.3 Innovation in Design</td>
<td>1</td>
<td>Indirectly¹</td>
</tr>
<tr>
<td>Credit 1.4 Innovation in Design</td>
<td>1</td>
<td>Indirectly¹</td>
</tr>
<tr>
<td>Credit 2 LEED Accredited Professional</td>
<td>1</td>
<td>Indirectly¹</td>
</tr>
</tbody>
</table>

¹ Credit may be indirectly relevant to a structural material or opportunities may exist under Innovation & Design Process credits.
Table 4. LEED-NC points pertaining to common structural materials.

<table>
<thead>
<tr>
<th>Rating system categories</th>
<th>Reinforced concrete</th>
<th>Masonry</th>
<th>Steel</th>
<th>Wood</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Sustainable Sites</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Credit 3 Brownfield Redevelopment</td>
<td>1</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Credit 5.1 Site Development, Protect or Restore Habitat</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 5.2 Site Development, Maximize Open Space</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 7.1 Heat Island Effect, Non-Roof</td>
<td>1</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Credit 7.2 Heat Island Effect, Roof</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td><strong>Energy &amp; Atmosphere</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Prereq 2 Minimum Energy Performance</td>
<td>Required</td>
<td>Required</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Credit 1 Optimize Energy Performance</td>
<td>1–10</td>
<td>1–10</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td><strong>Materials &amp; Resources</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Credit 1.1 Building Reuse, Maintain 75% of Existing Walls, Floors &amp; Roof</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Credit 1.2 Building Reuse, Maintain 95% of Existing Walls, Floors &amp; Roof</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Credit 2.1 Construction Waste Management, Divert 75% from Disposal</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Credit 2.2 Construction Waste Management, Divert 50% from Disposal</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Credit 3.1 Materials Reuse, 5%</td>
<td>–</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Credit 4.1 Recycled Content, 10% (post-consumer + 1/2 pre-consumer)</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Credit 4.2 Recycled Content, 20% (post-consumer + 1/2 pre-consumer)</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Credit 5.1 Regional Materials, 10% Extracted, Processed &amp; Manufactured Regionally</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Credit 5.2 Regional Materials, 20% Extracted, Processed &amp; Manufactured Regionally</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Credit 6 Rapidly Renewable Materials</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>1</td>
</tr>
<tr>
<td>Credit 7 Certified Wood</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>1</td>
</tr>
<tr>
<td><strong>Indoor Environmental Quality</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Credit 4.4 Low-Emitting Materials, Composite Wood &amp; Agrifiber Products</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>1</td>
</tr>
<tr>
<td>Credit 8.1 Daylight &amp; Views, Daylight 75% of Spaces</td>
<td>–</td>
<td>–</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Credit 8.2 Daylight &amp; Views, Daylight 90% of Spaces</td>
<td>–</td>
<td>–</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td><strong>Innovation &amp; Design Process</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Credit 1.1 Innovation in Design</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Credit 1.2 Innovation in Design</td>
<td>1(^1)</td>
<td>1(^1)</td>
<td>1(^1)</td>
<td>1(^1)</td>
</tr>
<tr>
<td>Credit 1.3 Innovation in Design</td>
<td>1(^1)</td>
<td>1(^1)</td>
<td>1(^1)</td>
<td>1(^1)</td>
</tr>
<tr>
<td>Credit 1.4 Innovation in Design</td>
<td>1(^1)</td>
<td>1(^1)</td>
<td>1(^1)</td>
<td>1(^1)</td>
</tr>
<tr>
<td>Credit 2 LEED Accredited Professional</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td><strong>Possible Projects Totals</strong></td>
<td>25</td>
<td>25</td>
<td>17</td>
<td>16</td>
</tr>
</tbody>
</table>

\(^1\)Not included in total.
A life cycle perspective on recycling construction materials (The most sustainable materials may be the ones we already have)

T.R. Napier & D.T. McKay
U.S. Army Corps of Engineers, Engineer Research and Development Center/Construction Engineering Research Laboratory, Champaign, Illinois, USA

N.D. Mowry
School of Architecture, University of Illinois at Urbana-Champaign, Champaign, Illinois, USA

ABSTRACT: Noted sustainability authority Paul Hawken points out that nearly all production and distribution business models entirely ignore associated environmental burdens when calculating acquisition prices for goods and services; further, purchase prices rarely reflect true costs of the aftermath when remnants are disposed of, most often by landfilling. Does the current price of construction account for the damage inflicted on the environment through materials production, construction activities, and ultimately the disposal of facilities when their service life is exhausted? What are these ignored impacts and how does one begin to define and account for them? This paper acquaints the audience with the life-cycle consequences of generating and landfilling construction and demolition debris, and the benefits of salvaging, reusing, and recycling construction materials. The United States Army continuously upgrades its infrastructure to meet current mission requirements. Wood framed barracks complexes constructed in World War II are being removed. Concrete and masonry complexes built during the Cold War are being replaced by contemporary facilities. In so doing, enormous quantities of debris materials are generated. These include wood, concrete, metals, asphalt materials, fibrous materials, masonry and other construction materials. Efforts to divert construction and demolition debris from landfills, and to recycle these materials for construction applications are described. By examining available Life Cycle Assessment data, a first approximation of the value of these recovered materials is presented; however, their value is expressed in terms of the reduction in burdens to the natural environment by virtue of the process. This information provides a piece of the sustainability puzzle, which is the determination of the “true costs” of goods and services which must incorporate the value of that part of the environment which was used to create the product.

1 WASTE AND SUSTAINABILITY

1.1 Common practice

There is not much argument within the Architectural, Engineering, Construction, and Facilities Management communities that sustainability should be integral within our practices and products. In practice, consideration for sustainability, or “green building,” usually begins at a new facility’s planning & design stages. Site development, energy systems, and low-impact materials are among the signature features of a “green building.” The retrofitting of existing buildings follow similar design paths focusing first on energy and water conservation, often by using highly visible (and therefore publicly favorable) sustainable green roofs and parking lots. But the life cycle effects of managing demolition waste and recycling construction materials have held a relatively low profile when compared to more glamorous sustainable technologies.

The United States Army is removing thousands of buildings from its real property inventories (Figs 1–4). At present there remains (even after years of continuous demolition) a footprint of 39 million square feet (360 hectares) of World War II-era buildings. Over 70,000 Army Family Housing Units are being demolished. Hundreds of Korean War-era barracks, and associated buildings are being replaced with contemporary barracks complexes. In total, 23.6 gigatonne (Gt) of demolition debris will be generated within the next 15 years. Some Army installations report that construction and demolition (C&D) debris constitutes 80% of their solid waste stream.

The presence of existing buildings or structures on-site is most often seen as a liability or impediment to a property’s development and a project’s construction.
Our paradigm is to dispose of existing buildings and structures as quickly and inexpensively as possible, and get on with the “real” project. The environmental issues associated with demolition and debris disposal are often overlooked, as are the potential applications of existing materials as resources and assets. As the environmental consequences of landfilling construction materials are not widely recognized within our professions, less attention is paid to recovering construction materials for reuse or recycling than perhaps should be.

1.2 What really happens
Construction and Demolition (C&D) waste disposal triggers a sequence of adverse effects that are not always apparent to building professionals. These include the loss of useful property, wasted materials, loss of embodied energy, greenhouse gas generation, and a multitude of environmental stressors associated with producing new materials instead of using existing materials. The number of C&D landfills is declining in the United States, which means fewer disposal options, greater hauling distances, and increased fuel consumption and vehicle emissions. Capping, closing, and monitoring landfills, and cleaning up leaking or contaminated landfill sites drain public funds.

Responsibly managing waste on a construction or demolition jobsite really is a vital component of Sustainable Building.

2 CONSTRUCTION AND DEMOLITION WASTE MATERIALS

2.1 Benefits of salvaging, reusing, or recycling building materials
The most commonly cited benefits associated with reusing or recycling building materials are the reduction of waste, and the preservation of landfill capacity.
There are, however, other environmental benefits that are not necessarily intuitive or widely recognized. These include reducing fuel consumption, emissions, and other adverse effects incurred by transporting waste; reducing adverse landfill performance such as greenhouse gas emission and leachate into groundwater; preserving materials’ embodied energy; and reducing the necessity to extract raw materials, consume more energy and resources, and create more emissions when manufacturing new building materials.

There is a common perception within the United States construction industry that reuse or recycling necessitates additional costs. The value of materials and cost avoidance due to reduced hauling and landfill costs can often offset any additional effort and enable an overall project cost savings. While there are no guarantees of cost savings for any specific project, there is ample precedence that C&D materials’ salvage and recycling certainly can reduce overall project costs.

2.2 The potential

The majority of construction waste and demolition debris can be reused on site, salvaged for reuse elsewhere, or recycled. While not standard practice in all markets, diverting 90% of a construction jobsite’s waste and over 80% of demolition debris from landfill disposal is not uncommon. Reusable and recyclable materials include:

- Landscape & land clearing debris (green wood materials)
- Asphalt pavement
- Gravel and aggregate products
- Concrete
- Masonry scrap and rubble
- Metals
- Clean wood
- Plastics
- Insulation materials
- Glass
- Door and window assemblies
- Carpet and pad
- Fibrous acoustic materials
- Ceiling tiles
- Plumbing fixtures and equipment
- Mechanical equipment
- Lighting fixtures and electrical components
- Cardboard packing and packaging
- Others

Successful examples of C&D waste reduction in United States commercial markets can be found through such sources as WasteCap Wisconsin [WasteCap 1996], Triangle J Council of Governments in North Carolina [TJCOG 1995], the California Integrated Waste Management Board [CIWMB 2006], Portland Metro Regional Center in Oregon [MRC 2006], King County in Washington State [Metro KC 2006], Environmental Building News [BuildingGreen 2006], and other sources. National organizations such as the U.S. Green Building Council [USGBC 2006] have grown to promote sustainability in the building industry in general. Other organizations such as the Building Material Reuse Association [BMRA 2006] and the Construction Materials Recycling Association [CMRA 2006] have emerged specifically to support reuse and recycling practices.

Successful deconstruction, reuse, and recycling experiences have also been accomplished within the Federal arena as well. U.S. Army installations are beginning to implement C&D waste reduction practices, and are now required to divert 50% of their non-hazardous solid waste generated by their demolition and construction programs. Forts McCoy in Wisconsin, Knox and Campbell in Kentucky, Monroe and Myers in Virginia, Carson in Colorado, Gordon in Georgia, Bragg North Carolina, Lewis in Washington State, Jackson in South Carolina, and others have achieved between 60% and 95% debris diversion when removing obsolete buildings.

In 1998 the U.S. Environmental Protection Agency estimated that 148.8 Gt of building-related C&D materials are generated annually in the United States [USEPA 1998]; of that, only 20% is currently being recycled. If salvage, reuse, and recycling practices were diligently applied throughout the U.S. construction industries, this rate could easily be increased to 50% and greater.

At present, the benefits of waste reduction are usually promoted on an intuitive or subjective basis. Advocates generally rely on successful experience and passion for the subject. If the consequences of landfilling C&D debris, and the benefits of reusing and recycling building materials were more widely recognized in a tangible, objective fashion, the building industry and regulatory agencies would be more inclined to temper lowest-first-cost project economy with environmental performance.

3 A LIFE CYCLE APPROACH

3.1 Life cycle assessment (LCA)

In 1991 The Society of Environmental Toxicology and Chemistry developed A Technical Framework for Life Cycle Assessment [SETAC 1991]. This is an objective process to evaluate environmental burdens by quantifying energy and material usage and environmental releases, to assess the impact of those inputs, outputs, and energy use, and to evaluate opportunities to incorporate improvements to the process. SETAC defined the life cycle phases as raw materials
acquisition; manufacturing, processing and formulation; distribution and transportation; use/reuse/maintenance; recycle; and waste management. At each Life Cycle phase, energy and raw materials inputs are identified, as are the process outputs of airborne emissions, solid waste, other environmental releases, and useable products. Thus, a holistic approach is taken to address all environmental issues occurring at each phase throughout the material’s or process’s life cycle.

There are several sources of Life Cycle Assessment (LCA) data for building materials. These include the American Institute of Architects (AIA) Environmental Resources Guide [AIA 1998]; the U.S. Environmental Protection Agency’s (USEPA) Waste Reduction Model [USEPA 2006]; the U.S. National Institute for Standards and Technology’s (USNIST) Building for Economic and Environmental Sustainability [USNIST 2002]; from Canada, the Athena Institute’s LCA database and Environmental Impact Estimator software [Athena Institute 2006]; from the Netherlands the SimaPro LCA software [PPEC 2006]; and others.

Life Cycle Assessment data will not, in and of itself, “prove” that salvage, reuse, recycling is cheaper (or better) than traditional demolition and landfill disposal. LCA data will provide tangible, objective information about the relative effects of landflling, reusing, or recycling materials, and will reveal a magnitude-level quantities of environmental stressors associated with each approach.

The authors acknowledge there is a degree of uncertainty in LCA data. No effort was made to confirm or rebut the accuracy of the LCA analyses. Over the life cycle process, the resulting values could be greater or smaller depending on specific conditions if they were to actually be measured. However, when thinking in terms of the number of “trees saved” by recycling construction materials, this exercise can lead to a number of useful outcomes.

The authors used the following sources for life cycle assessment data when developing this paper:
- USEPA WAste Reduction Model (WARM).
- Athena Environmental Impact Estimator.

3.2 Waste reduction model

The WARM is available for use on-line, or with a Microsoft Excel spreadsheet format that can be downloaded from the USEPA website. It is designed to calculate the greenhouse gas emissions generated by landfilling various solid waste materials, and of alternative waste management strategies.

Initially, a Baseline Scenario is established by inputting the quantities of materials to be disposed of. The following disposal options are available: recycling, landfilling, combustion, or composting. Alternative Management Scenarios are then entered for the same materials and quantities. The following disposal options are available: waste reduction, recycling, landfilling, combustion, or composting. The WARM system calculates greenhouse emissions throughout materials life cycle, but does not identify or quantify other life cycle behaviors, such as emissions to water.

The WARM output consists of three values: carbon dioxide emitted into the atmosphere through transportation & landfill emissions; carbon equivalent emitted into the atmosphere through transportation & landfill emissions, and energy used throughout the material’s life cycle phases.

3.3 Athena environmental impact estimator

The Athena Environmental Impact Estimator (EIE) is a more complete LCA tool developed primarily for the building industry. The ultimate objective of the software is to assist the building community in making more informed decisions regarding the choice of design and material alternatives that will minimize a building’s life cycle environmental impact. The model and databases are North American in scope, representing average or typical manufacturing technologies and appropriate modes and distances for transportation. It covers a building’s life cycle stages from the natural resource extraction through its end-of-life, including on-site and maintenance and replacement activities. It provides a full environmental life cycle inventory as well as a set of six impact summary measures or indicators. These are as follows:

- Embodied energy includes all energy, direct and indirect, used to transform or transport raw materials into products and buildings, including inherent energy contained in raw or feedstock materials that are also used as common energy sources, and the indirect energy use associated with processing, transporting, converting and delivering fuel and energy.

- Solid waste is reported on a mass basis and is generally self-explanatory.

The four other measures are indices. They have been developed because of the difficulty of using and interpreting detailed life cycle inventory results. Emissions to air and water, and resource use encompass a relatively large number of individual substances with varying environmental impacts. However, there is no real basis for comparison from one material to another in terms of environmental impact. Therefore, related numeric results are compiled into indices that summarize the results by indicating potentials for environmental impacts.

Weighted Resource Use: The values shown for the weighted resource use are the sums of the weighted resource requirements for all products used. They can be thought of as “ecologically weighted kilograms”,
where the weights reflect relative ecological carrying capacity effects of extracting resources.

Global Warming Potential is a reference measure. Carbon dioxide is the common reference standard for global warming or greenhouse gas effects. All other greenhouse gases are referred to as having a “CO₂ equivalence effect.”

The Air Pollution Index and Water Pollution Index are similarly intended to capture the pollution or human health effects of groups of substances emitted at various life cycle stages. The Athena EIE uses the commonly recognized and accepted critical value method to estimate the volume of ambient air or water that would be required to dilute contaminants to acceptable levels, as defined by the most stringent standards applicable.

Analyzing the life cycle benefits of using or recycling materials is not the primary purpose of the Athena EIE. However, comparisons can be made using the LCA data. To analyze the Life Cycle benefits of using or recycling building materials, a simplistic approach was taken. Athena personnel were consulted to ensure the system and data were being used in an appropriate fashion for the purposes [Author’s pers. comm. 2006].

For materials being reused (lumber and steel, in this case), a baseline was established by inputting the appropriate quantities of lumber and steel. The impact reports described all substances, energy use, resources consumed, and energy consumed throughout all life cycle phases of these materials.

To represent diverting the lumber and steel materials from the landfill, the end-of-life impacts were deleted. To represent not extracting resources, not producing new products, and not transporting products to wholesale or retail distribution centers, manufacturing impacts for aggregate and the asphalt component of hot mix asphalt paving were deleted. Recycling these materials is typically a local practice, so any further transportation burdens would be negligible. The remaining impacts in construction and operation and maintenance phases will be comparable whether either new materials or salvaged materials are used. The differences represent the reduction of adverse impacts because materials were recycled.

4 FOUR DEMOLITION EXAMPLES

Four examples are described to illustrate the environmental benefits of using and recycling building materials. These are based on various U.S. Army experiences with which the authors are familiar.

4.1 Framing lumber

Seventy-eight World War II-era barracks buildings, almost 3446.7 m² in total, will be demolished on an U.S. Army installation to clear property for training facilities. These are typically one and two-story open bay buildings (Figs 5–6) built to house and train soldiers during WWII. Most have converted to office uses. The primary construction type is wood frame, on concrete pier foundations. What would be the environmental benefits if the lumber framing was salvaged for reuse, instead of crushed during demolition and disposed of in a landfill?

From characterization models of these building types, we know these buildings contain an average of approximately 4.5 board-feet of framing lumber per gross square foot (approx 10⁻¹ m²) of building¹. Altogether approximately 1751 tonne of lumber are present in these buildings.

Deconstruction experience with WWII-era wood framed buildings suggests that recovering 75% of the framing lumber for reuse is reasonable. Approximately 1315 tonne of lumber would be available for salvage and reuse.

4.2 Asphalt shingles

What would be the environmental benefits if the shingles from the same barracks buildings (Figs 7–8) were recycled into new asphalt paving for the training facilities?

Our characterization model indicates there are 1.54 kg of shingles, in two layers, per gross square foot (10⁻¹ m²) of building. Altogether there are

¹ A board-foot is one square foot of lumber one inch thick (approximately 2360 cubic centimeters).
approximately 326 tonne of shingles on the 78 buildings to be removed. A recycling rate of 95% is realistic for shingles; some loss is expected. Roughly 312 tonne of asphalt would be available for recycling.

4.3 Concrete

Eight concrete barracks buildings will be demolished to prepare a site for construction of a new barracks complex. These are three-story dormitory type buildings (Fig. 9) and the primary construction material is cast-in-place reinforced concrete. What would be the environmental benefits if the concrete were recycled into aggregate for fill and compacted base for use on-site?

The authors have developed characterization models of this type of reinforced concrete barracks building. We know there are approximately 95 kg of concrete per gross square foot \((10^{-1} \text{ m}^2)\) of building; 29 kilotonne of concrete total for all 8 buildings. A recycling rate of 95% is realistic for concrete; some loss is expected. Roughly 28 kilotonne would be available for recycling.

4.4 Steel

A 223 m² steel building (Fig. 10) needs to be removed from its present site. It is a simple pre-engineered metal building system with structural shapes and corrugated metal roofing and siding. What would be the environmental benefits if the building were disassembled for reassembly in a different location?
There are approximately 16.8 kg of steel per gross square foot (10⁻¹ m²) of building, or 40.3 tonne of steel for the building. A recycling rate of 99% of the building’s steel components is realistic; some hardware & components will be lost. Roughly 40 tonne of steel in total would be reused.

5 APPLYING THE DATA; MORE THAN INTUITIVE

Again, most would agree that not wasting materials has environmental benefits. However, this conclusion is typically based on intuition, as opposed to quantitative or objective analysis. Applying the data can verify that intuition is indeed correct, and provide some order of magnitude to the effects.

5.1 Greenhouse gas, according to WARM

If 15 kilotonne of dimensional lumber (see sample of recovered lumber in Figure 11) are reused, instead of landfilled, the environmental effects would be as follows:

5.1.1 Lumber
Carbon Equivalent emissions are reduced by 888 tonne, compared to landfilling the lumber.
Carbon Dioxide Equivalent emissions are reduced by 3,257 tonne, compared to landfilling the lumber.
Energy Use is reduced by 8,297 Million BTU, compared to landfilling the lumber. This is the equivalent of 121 passenger cars removed from the roads each year; 1,431 barrels of oil; or 251,117 liters of gasoline. (Appendix contains tabulated data.)

5.1.2 Asphalt, concrete, structural metals
Unfortunately, WARM was developed to represent general municipal waste streams. The model does not include data on most common construction materials. Therefore, one cannot perform analyses for asphalt shingles, concrete, or structural metals.

5.2 Environmental impacts according to Athena

Reuse and recycling scenarios are developed for the subject materials and quantities as, described above, and entered into the Athena EIE. Summary Results are tabulated in the appendix, highlights are as follows:

5.2.1 Lumber
If 1,206,000 board feet (equivalent of 1315 tonne of southern yellow pine framing lumber) were salvaged from 78 WWII-era wood buildings, and reused instead of being landfilled:
- Almost 10 Million Mega joules of energy would be saved.
- Approximately 430 tonne of solid waste would be saved by virtue of not manufacturing new lumber.
- The equivalent of approximately 61 Million Cubic Meters of air would not be polluted.
- The equivalent of approximately 80,000 liters of water would not be polluted.
- Approximately 237 tonne of carbon equivalent (greenhouse gas) would not be emitted to the atmosphere.
- The equivalent of approximately 93,000 liters of water would not be consumed.

The majority of environmental burdens associated with lumber construction are generated during the manufacturing processes, which include materials extraction and transportation. These include over 80% of the total life cycle energy consumption, and between 97% and 100% of the other of the other waste, emissions, and consumption burdens.
5.2.2 Concrete
If 29 kilotonne of concrete were recycled and reused (Figs 12–13) for aggregate materials on-site, instead of being landfilled:

Almost 588,000 megajoules of energy would be saved.

Approximately 127 tonne of solid waste would be saved by virtue of not quarrying and processing new aggregate.

The equivalent of approximately 1.34 Billion Cubic Meters of air would not be polluted.

The equivalent of approximately 60,000 liters of water would not be polluted.

Approximately 143 tonne of carbon equivalent (greenhouse gas) would not be emitted to the atmosphere.

The equivalent of approximately 13.2 kilotonne of material resources would not be consumed.

The majority of environmental burdens associated with aggregates used in construction are generated during the manufacturing processes, which include materials extraction and transportation. These include over 60% of the life cycle energy consumption, and between 96% and 100% of the other waste, emissions, and consumption burdens.

5.2.3 Asphalt roofing
If 354 tonne of asphalt roofing shingles were recycled and reused (Figs 14–15) for asphalt paving on-site, instead of being landfilled:

Over 47 Million Megajoules of energy would be saved.

Approximately 27 tonne of solid waste would be saved by virtue of not manufacturing new asphalt for asphalt paving.

The equivalent of almost 456 Million cubic meters of air would not be polluted.

The equivalent of approximately 90,000 liters of water would not be polluted.

Approximately 1590 tonne of carbon equivalent (greenhouse gas) would not be emitted to the atmosphere.

The equivalent of approximately 1.2 kilotonne of material resources would not be consumed.
Once again, the majority of environmental burdens associated with hot mix asphalt paving are generated during the manufacturing processes, primarily of the bitumen materials themselves. These include over 98% of the life cycle energy consumption, and between 99% and 100% of the other waste, emissions, and consumption burdens.

5.2.4 Structural metals
If 40.3 tonne of metal building components were disassembled on-site (Fig. 16) and reused at a different site instead of being landfilled:
- Over 1.28 Million megajoules of energy would be saved throughout the steel material’s life cycle.
- Approximately 10.300 tonne of solid waste would be saved by virtue of not manufacturing new steel.
- The equivalent of almost 8.2 Million cubic meters of air would not be polluted.
- The equivalent of approximately 1.6 Million liters of water would not be polluted.
- Approximately 54.700 tonne of carbon equivalent (greenhouse gas) would not be emitted to the atmosphere.
- Approximately 1.1 Million liters of water would not be consumed.
- The equivalent of approximately 143 tonne of material resources would not be consumed.

The majority of environmental burdens associated with structural steel are generated during the manufacturing processes. These include over 95% of the energy consumed throughout steel’s total life cycle, and between 99% and 100% of the other waste, emissions, and consumption burdens.

6 WHAT DOES THE DATA TELL US?

These environmental impact values are impressive. Millions of Megajoules of energy can be saved. Millions of cubic meters of air need not be polluted. Millions of kilograms of carbon dioxide equivalent, i.e. “greenhouse gas,” need not be generated. Millions of kilograms of natural resources need not be depleted. Each of the demolition and reuse or recycling scenarios described above are quite modest in their scope. Major construction projects will create far greater impacts. An a national level, the impacts of demolition and construction activities are enormous. However, these numbers are still difficult to appreciate without some benchmark or frame of reference. WARM does provide equivalents to energy use that are easily visualized by the public – that’s the purpose of the model and EPA’s mission to inform the public.

Other measures, especially volumes of air and water, or mass of carbon equivalent, are much more difficult to visualize. What does it mean to not pollute 8 million cubic meters of air, or not pollute a million liters of water, or not generate 100,000 kilograms of carbon equivalent? The scientific community has yet to develop reference points that are understandable to the public, and useful to the building professions.

Likewise, monetary values are not easy to associate with resource depletion, air and water pollution, or greenhouse gas production. But we know they’re there, especially in the United States. The existence of environmental agencies and roles they play in our daily lives; standards for clean water and air; limits to emissions; and the political controversy surrounding Global Warming and the economic effects of greenhouse gas reduction are all evidence that environmental oversight and management come at a significant monetary cost.

Regardless, any environmental burden greater than “zero” is an environmental detriment, and should be minimized or avoided altogether where possible. Even if quantitative values are still abstract, they are not trivial, and they do provide some magnitude to the effects of reusing and recycling existing materials. If nothing else, the data developed for this paper reveal that displacing manufacturing processes by re-using and recycling existing materials reduces total energy consumption by 60% to 98%, and eliminates most of the other life cycle environmental burdens.

7 CONCLUSIONS

The environmental impacts of reusing and recycling construction materials in lieu of landfilling them can be determined using available Life Cycle Assessment data. This assessment can be holistic, addressing all features of building and materials’ life cycle, and quantitative, or at least objective and measurable. While numerical values are still difficult to visualize, they do show a magnitude of effect.

Even using examples that are modest in scope, the environmental impacts of reusing and recycling
construction materials are significant. Magnitudes will be far greater with more substantial development and construction.

Note that the material extraction and manufacturing processes contribute the most to the environmental burdens associated with building materials. The greatest potential for reducing these burdens is by displacing new materials with existing materials where possible. Therefore, the most “sustainable” building materials may those that can be removed from obsolete buildings and structures, and reused or recycled in new construction.

REFERENCES


Author’s pers. comm. 2006. Personal communication; telephone discussions among the authors and Mr. Jamie Meil, Athena Sustainable Materials Institute, October 2006.


APPENDIX ATHENA ENVIRONMENTAL IMPACT ESTIMATOR AND USEPA WARM OUTPUTS

<table>
<thead>
<tr>
<th>LCA impact categories</th>
<th>Units</th>
<th>New lumber</th>
<th>Reused lumber</th>
<th>Savings</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary Energy Consumption</td>
<td>MJ</td>
<td>10,006,432</td>
<td>2,167,111</td>
<td>7,839,321</td>
</tr>
<tr>
<td>Solid Waste</td>
<td>kg</td>
<td>34,503</td>
<td>14</td>
<td>34,489</td>
</tr>
<tr>
<td>Air Pollution Index</td>
<td>10³ m³</td>
<td>49,264</td>
<td>481</td>
<td>48,783</td>
</tr>
<tr>
<td>Water Pollution Index</td>
<td>10⁶ L</td>
<td>6</td>
<td>0</td>
<td>6</td>
</tr>
<tr>
<td>Global Warming Potential</td>
<td>kg</td>
<td>190,838</td>
<td>2,699</td>
<td>188,139</td>
</tr>
<tr>
<td>Water Used</td>
<td>L</td>
<td>73,807</td>
<td>0</td>
<td>73,807</td>
</tr>
<tr>
<td>Weighted Resource Use</td>
<td>kg</td>
<td>3,938,691</td>
<td>104,759</td>
<td>3,833,932</td>
</tr>
</tbody>
</table>

572
Recycling 39,130 m² of asphalt shingles into hot asphalt paving

<table>
<thead>
<tr>
<th>LCA impact categories</th>
<th>Units</th>
<th>New hot mix asphalt paving</th>
<th>Hot asphalt paving w/ recycled shingles</th>
<th>Savings</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary Energy Consumption</td>
<td>MJ</td>
<td>7,445,754</td>
<td>350,560</td>
<td>7,095,194</td>
</tr>
<tr>
<td>Solid Waste</td>
<td>kg</td>
<td>11,193</td>
<td>2</td>
<td>11,191</td>
</tr>
<tr>
<td>Air Pollution Index</td>
<td>10⁵ m³</td>
<td>40,852</td>
<td>53</td>
<td>40,799</td>
</tr>
<tr>
<td>Water Pollution Index</td>
<td>10⁴ L</td>
<td>4</td>
<td>0</td>
<td>4</td>
</tr>
<tr>
<td>Global Warming Potential</td>
<td>kg</td>
<td>112,383</td>
<td>297</td>
<td>112,086</td>
</tr>
<tr>
<td>Water Used</td>
<td>L</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Weighted Resource Use</td>
<td>kg</td>
<td>863,817</td>
<td>3,851</td>
<td>859,966</td>
</tr>
</tbody>
</table>

Recycling 29,025 tonnes of concrete into aggregate

<table>
<thead>
<tr>
<th>LCA impact categories</th>
<th>units</th>
<th>Natural aggregate</th>
<th>Recycled aggregate</th>
<th>Savings</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary Energy Consumption</td>
<td>MJ</td>
<td>4,385,809</td>
<td>1,922,695</td>
<td>2,463,114</td>
</tr>
<tr>
<td>Solid Waste</td>
<td>kg</td>
<td>12,732</td>
<td>16</td>
<td>12,716</td>
</tr>
<tr>
<td>Air Pollution Index</td>
<td>10⁵ m³</td>
<td>1,333,489</td>
<td>556</td>
<td>1,332,933</td>
</tr>
<tr>
<td>Water Pollution Index</td>
<td>10⁴ L</td>
<td>6</td>
<td>0</td>
<td>6</td>
</tr>
<tr>
<td>Global Warming Potential</td>
<td>kg</td>
<td>146,505</td>
<td>3,117</td>
<td>143,388</td>
</tr>
<tr>
<td>Water Used</td>
<td>L</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Weighted Resource Use</td>
<td>kg</td>
<td>31,193,778</td>
<td>39,282</td>
<td>31,154,469</td>
</tr>
</tbody>
</table>

Reuseing 40.29 tonnes of steel

<table>
<thead>
<tr>
<th>LCA impact categories</th>
<th>Units</th>
<th>New steel</th>
<th>Reused steel</th>
<th>Savings</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary Energy Consumption</td>
<td>MJ</td>
<td>1,336,352</td>
<td>63,961</td>
<td>1,272,391</td>
</tr>
<tr>
<td>Solid Waste</td>
<td>kg</td>
<td>10,295</td>
<td>1</td>
<td>10,294</td>
</tr>
<tr>
<td>Air Pollution Index</td>
<td>10⁵ m³</td>
<td>8,219</td>
<td>20</td>
<td>8,199</td>
</tr>
<tr>
<td>Water Pollution Index</td>
<td>10⁴ L</td>
<td>159</td>
<td>0</td>
<td>159</td>
</tr>
<tr>
<td>Global Warming Potential</td>
<td>kg</td>
<td>54,834</td>
<td>142</td>
<td>54,692</td>
</tr>
<tr>
<td>Water Used</td>
<td>L</td>
<td>1,074,966</td>
<td>0</td>
<td>1,074,966</td>
</tr>
<tr>
<td>Weighted Resource Use</td>
<td>kg</td>
<td>144,419</td>
<td>1,494</td>
<td>142,925</td>
</tr>
</tbody>
</table>

WARM SUMMARY

Energy use from baseline waste management

<table>
<thead>
<tr>
<th>Material</th>
<th>Recycled</th>
<th>Ton landfilled</th>
<th>Ton combusted</th>
<th>Ton composted</th>
<th>Million BTU</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimensional Lumber</td>
<td>0</td>
<td>2126</td>
<td>0</td>
<td>n.a.</td>
<td>796</td>
</tr>
</tbody>
</table>

Energy use from alternative waste management scenario

<table>
<thead>
<tr>
<th>Material</th>
<th>Ton reduced</th>
<th>Ton recycled</th>
<th>Ton landfilled</th>
<th>Ton combusted</th>
<th>Million BTU</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimensional Lumber</td>
<td>2126</td>
<td>0</td>
<td>0</td>
<td>n.a.</td>
<td>–7504</td>
</tr>
</tbody>
</table>

Total Change in Energy Use 8300

Equivalent to:
121 Passenger Cars Removed from Roadways Each Year
1431 Barrels of Oil
66,388 Gallons of Gasoline
Demonstrating how plasterboard can be collected more efficiently on construction and demolition sites in the UK

S.B. Emery
Department of Geography, Durham University, Science Laboratories, Durham, (formerly Senior Environmental Specialist, Resource and Environment Group, Scott Wilson)

D.N. Smith
Head of Resource and Environment Group, Scott Wilson, Peterborough

S. Johansson
Environmental Advisor, Skanska Integrated Projects UK, Rickmansworth, Herts

J. Cope
Materials Project Manager, Plasterboard, The Waste and Resources Action Programme, The Old Academy, Banbury, Oxon

ABSTRACT: This article outlines the preliminary findings of a project commissioned by the Waste and Resources Action Programme (WRAP). The paper is based on practical experience of the collection and segregation of plasterboard waste at three construction and demolition sites between December 2005 and November 2006. The operating procedures for the reclamation of plasterboard waste on site; tonnages of plasterboard waste arising; factors affecting on-site recovery; economic constraints and opportunities, and; logistical constraints and opportunities are described and considered for each site.

In assessing the barriers to segregating and recycling plasterboard from construction and demolition sites in the UK, it is argued that too much emphasis may be placed on ‘legislative barriers’. Whilst such barriers are influential, there is, in practice, an interplay of factors governing the success of plasterboard recovery. Where there is an organisational commitment to segregate for recycling legislative barriers become less important than the site-based factors in determining the success of a particular scheme.

The three cases described demonstrate the logistical, economic and technical viability of plasterboard segregation and recycling schemes on different types of site with different material handling and waste management arrangements in place. It is hoped that the findings from these three case studies can be used to support and inform the wider adoption of similar plasterboard recovery schemes in the future.

1 INTRODUCTION

This paper presents the preliminary findings of ongoing research exploring the ways in which the barriers to the successful segregation and recycling of plasterboard waste on construction and demolition sites can be addressed. The research is funded by the Waste and Resources Action Programme (WRAP) and is based on construction sites operated by Skanska UK Ltd.

2 CONTEXT

The significant contribution that the construction industry makes to the UK’s waste arisings has long been recognised [Ferguson et al. 1995]. Of 335 million tonnes of waste produced in the UK in 2004 almost one third (109 million tonnes) is reported to arise from construction, demolition and excavation (CDE) activities (Defra 2006). Although only 10% of CDE waste is formally reported as being disposed of to landfill, a further 40% of arisings are attributed to low value ‘landfill restoration’, ‘backfilling quarry voids’ and ‘spreading on registered exempt sites’. The remaining 50% of arisings are classified as ‘recycled aggregate and soil’ [ODPM 2004, Defra, 2006].1 It is unclear

1 The figures for the destination of CDE wastes apply to England only.
from these figures what contribution recycling makes to the diversion of waste from landfill for materials that are not – or, more accurately, are not solely capable of being – recycled into aggregates (e.g. timber, plastics, plasterboard).

A recent study commissioned by WRAP has shown that the current UK production capacity of plasterboard is estimated to be 2.7 Mtpa, whilst the quantity of plasterboard waste from construction and demolition activities that is landfill is approximately 1 Mtpa [James et al. 2006]. Historically, landfill has been the principal management technique for waste plasterboard (WRAP 2006) but legislative changes to the landfill regime and increasing government attention to sustainable waste management have raised the profile of the management of plasterboard wastes.

As of 16th July 2005 the Waste Acceptance Criteria that dictate the conditions upon the landfilling of plasterboard changed. In accordance with the Landfill (England and Wales) Regulations 2002 (as amended) plasterboard waste was included in a category of high-sulphate bearing wastes. Supplementary Environment Agency guidance advises that such wastes with a content of more than 10% sulphate per load may only be disposed of in high-sulphate ‘mono-cells’ separated from biodegradable waste in non-hazardous waste landfills [Environment Agency 2005]. Mixed wastes (including mixed construction waste) containing less than 10% sulphate-bearing material (i.e. plasterboard) per load can continue to be disposed of in non-hazardous landfills. It was anticipated that a secondary effect of the new legislation would be to encourage the increased segregation for recycling of plasterboard as a result of the increased costs associated with its disposal in mono-cell landfill. However, it has been argued that the so-called ‘10% rule’ may actually have acted as a disincentive to the segregation of plasterboard waste from other wastes, with construction and demolition waste being managed such that plasterboard remains less than 10% of the total waste load and therefore eligible for disposal to general landfill at lower cost [James et al. 2006]. Indeed, the 10% rule has been referred to as a ‘loophole’ and there has been apparent surprise at the blending of plasterboard with other wastes to allow its continued disposal in non-hazardous landfill [Market Transformation Programme 2006, James et al. 2006].

Whilst the 10% rule may have provided a legislative barrier to increased segregation and recycling of plasterboard it must be borne in mind that the intention of the legislation was not primarily to reduce the quantities of sulphate-bearing wastes being landfill, but to minimise the harmful affects associated with landfilling high concentrations of such waste in association with biodegradable waste: specifically, the production of hydrogen sulphide. The issue, therefore, is not that the legislation has not served its intended purpose but that it may run contrary to political objectives aimed at increased recycling and reduced resource consumption.

3 THE BARRIERS TO PLASTERBOARD SEGREGATION AND RECYCLING

Both the plasterboard recycling industry and campaigns for sustainable resource management have been frustrated by the lack of uptake of plasterboard segregation for recycling in the construction industry. The focus of this frustration on the legislative arrangements alone, however, conceals a multiplicity of factors acting as barriers to the increased segregation and recycling of plasterboard waste. Whilst the Landfill Regulations are an easy target, and changes to the regulations (i.e. reducing or removing the 10% sulphate allowance) might induce an improvement to the management of plasterboard waste, it has also been recognised that efforts are required to address other barriers [WRAP 2005] that may be less conspicuous, but more ingrained and therefore more difficult to change.

Many of the barriers to plasterboard segregation and recycling are common to construction waste, and the construction industry in general. It has been argued that during the 1990’s construction waste management research focussed on how existing work practices, processes and technologies act as barriers to recycling, and that ‘people have been ignored in the waste equation’ [Teo & Loosemore 2001]. From the beginning of the 21st Century however, there has been a shift toward the study of organisational and individuals’ perceptions, behaviour, understanding and attitudes as barriers to waste management [Lingard et al. 2000, Teo & Loosemore 2001, Saunders and Wynn 2004]. Table 1 provides a summary of the key barriers to on-site construction waste segregation for recycling. In practice, these factors do not operate discretely but there is an interplay between them. James et al. 2006 consider, in detail, the principal barriers affecting the segregation and recycling of plasterboard. Specific barriers not covered in Table 1 include:

3.1 Legislative / Market barriers

The ‘10% rule’ has provided a disincentive for the segregation of plasterboard. It has also meant that both landfill operators (for mono-cell disposal) and
<table>
<thead>
<tr>
<th>Category</th>
<th>Barriers and influences</th>
</tr>
</thead>
</table>
| Construction Industry and organisational characteristics | – It is a very competitive industry with minimal profit margins  
– It has a culture that is traditionally resistant to change  
– Speed is seen as imperative for profitability and there is a reluctance to adopt new habits which are perceived to affect the speed of working  
– It is a highly sub-contracted industry meaning that it is easy to pass responsibility, or responsibilities are poorly defined  
– The construction environment is often hostile and unpredictable  
| Logistical                     | – It is perceived that waste segregation requires more space than traditional waste management practices and space often comes at a premium on construction sites  
– Similarly there may be limited space for the storage of products on-site, making it difficult to minimise waste caused by damage to stored goods  
– Each project is unique making it difficult to standardise procedures  
– Site conditions are often difficult  
| Economics and markets          | – There may be perceived or actual increases in cost associated with:  
  • High investment costs for establishing a waste segregation system  
  • An increase in the amount of time operatives spend training and undertaking waste segregation activities  
  • Waste contractor fees and transport costs  
– There may be uncertainty over the markets for recycled products resulting in an unwillingness to arrange long-term contracts with recyclers  
– Construction companies are more likely to invest in renewable technologies or other more visible initiatives to demonstrate their environmental performance than waste management  
– Smaller sub-contractors are unlikely to benefit from the economies of scale achieved by larger companies for the implementation of waste segregation practices  
[Tam & Tam 2006, Seydel et al. 2002, James et al. 2006] |
| Legislative                    | – There is currently no specific legislation mandating particular construction waste management practices. The competitive nature of the industry means that particular activities are unlikely to be costed-for unless legally required  
– Legislation may provide a disincentive for increased segregation and recycling by, for instance, keeping the costs of traditional disposal methods low  
[Market Transformation Programme 2006, James et al. 2006] |
| Behavioural and Educational    | – Both managerial and operational commitment are essential for successful segregation schemes  
– Behaviour is often dictated by employees perceptions of the waste management climate. Perceptions of relevance include:  
  • Perceived importance of waste management and environmental issues more generally;  
  • Perceived management commitment to waste management;  
  • Perceived ability for construction sites to effectively segregate and recycle waste; and  
  • Perceptions about the cost-effectiveness of segregation and recycling  
– It remains rare for waste management to be covered during site induction for individuals who are unlikely to be on site for a long period  
– Segregating waste on site may be regarded as a ‘hassle’  
| Contractual                    | – Site segregation and recycling is unlikely to be practiced unless there is a contractual client requirement  
– There are generally no fiscal incentives within contracts that encourage waste minimisation or segregation by sub-contractors  
[James et al. 2006, Saunders & Wynn, 2004] |
recyclers have been unwilling to invest in new infra-
structure while plasterboard waste can be legally dis-
posed of in low cost landfill sites.3

3.2 Contractual / Economic barriers
Dry-lining is usually undertaken by specialist contrac-
tors who are typically paid by the area installed. At present, material costs for plasterboard are low,
therefore the time element of a contractor’s costs are
greater than the material element and wasteful prac-
tices are accepted if they reduce the time taken to
complete a job and consequently the overall cost.
Furthermore, unused material is likely to be disposed
of rather than re-transported for use on the next job.

4 BACKGROUND TO THE PROJECT
The research project was commissioned by WRAP’s
Plasterboard Programme and is being delivered by
Scott Wilson’s Resource and Environment Group (pro-
ject management, reporting and co-ordination), two
operating units within Skanska, Skanska Integrated
Projects and Skanska Building (providing demonstra-
tion sites) and Gypsum Recycling UK (GRUK) (plas-
terboard reprocessor). The project was commission
by WRAP in order to better understand the site-based
barriers – and the interplay between them – affecting
the successful segregation and recycling of plaster-
board waste on construction and demolition sites.
Skanska are providing five demonstration sites on
which research is being conducted between December
2005 and summer 2007. The sites have been chosen
to demonstrate the issues that arise on sites with a
variety of physical characteristics and with a range of
contractual and managerial approaches in place. The
project aims to establish and demonstrate the practi-
calities of recovering and segregating plasterboard
and to provide case studies for other sites wishing to adopt
similar schemes in the future. The projects have not
been selected as models of best practice per se. Rather,
it is anticipated that through an examination of the real
issues faced on a range of sites, lessons will be learnt
that are either generally or specifically applicable to
similar projects. It is Skanska company policy to seg-
regate at least six materials on construction sites with
the intention of recycling at least 60% (2006 target is
65%) of construction and demolition waste.

5 OBJECTIVES
It is the purpose of this paper to report the prelimi-
nary findings from three of the demonstration sites
and to consider these findings in the context of the
broader literature. The findings will also be used to
direct research for the remainder of the project and
will hopefully provide some initial insights that are of
interest to the construction industry. The specific
objectives of this paper are to review:

• Operational procedures for the recovery of plaster-
board waste on-site;
• Plasterboard waste arisings;
• Site-based factors affecting recovery;
• Economic constraints and opportunities, and;
• Logistical constraints and opportunities.

6 METHOD AND SCOPE
This paper draws on plasterboard waste recovery
from three Skanska projects: Woodbridge Airfield,
Suffolk; Coventry Walsgrave Hospital; and the Home
Office, Queen Anne’s Gate, London. The reports from
both Woodbridge and Coventry examine the construc-
tion phase of the projects, whilst the report on Queen
Anne’s Gate looks at demolition phase activities.4 The
research was undertaken on the three sites between

Primary information on plasterboard waste arisings
and associated costs have been collected and provided
by Skanska and the plasterboard reprocessor. Additional
information was collected during site meetings and
site tours held intermittently throughout the research
period. Input to site meetings was provided by Skanska
site personnel, GRUK, relevant sub-contractors, trade
association representatives, WRAP and Scott Wilson.
The site meetings were supplemented with tours of the
sites to observe plasterboard handling and waste man-
agement procedures, clarify data provided by Skanska
and GRUK, interview site operatives and record photo-
graphic evidence.

Additional research was undertaken, as required,
to gather information from third parties, including the
Environment Agency, waste disposal and transfer facil-
ties, waste logistics companies, plasterboard manufac-
turers, and plasterboard reprocesors (other than
GRUK) accepting the segregated waste material.

The following sections provide an overview of the
three sites, the waste management procedures in place
and information on waste arisings and the economics

---

3 It should be noted that Northern Ireland Heritage Service
and the Scottish Environmental Protection Agency do not
currently require high sulphate wastes to be disposed of in
mono-cells as specified by the Environment Agency in
England and Wales.

4 This project is currently in construction phase, and this will
be reported at a later date.
of recovery. A consolidation and discussion of the findings is then presented in Section 10.

7 Woodbridge Airfield (Skanska Integrated Projects)

7.1 Site and project details

Skanska was contracted by the Ministry of Defence to design and construct accommodation, training and maintenance facilities at the 69-hectare Woodbridge Airfield site in Suffolk. The work involved the demolition of 92 structures (completed before this research began), the construction of 18 new low-rise buildings and ancillary structures with an approximate project value of £82 million. Work began on the site in May 2004 and was completed by May 2006. The site was spacious with the new buildings dispersed amongst the grounds of the airfield.

7.2 Plasterboard waste management procedures

Skanska’s Logistics Supervisor was designated with overall responsibility for construction waste on site, although sub-contractor responsibilities varied depending on the waste material and the nature of activities being conducted. Skanska supplied approximately 120 1,100 litre wheeled bins for the collection of source segregated construction waste. These were distributed around the site close to point of work and colour-coded for different waste materials.

Detailed site waste management procedures were outlined in a Site Waste Management Plan issued by Skanska to site operatives and sub-contractors. Furthermore, contracts between Skanska and sub-contractors included an obligation for sub-contractors to segregate waste and employed a ‘Waste Notice’ system – whereby sub-contractors were issued with warnings and financial penalties – for failure to tidy waste up appropriately. The notice and penalty system did not, however, penalise poor performance in terms of segregation. A Waste Marshall was employed to oversee the delivery (by forklift) of waste materials to a central waste skip compound, and a further two Skanska employees were responsible for, amongst other duties, implementing and monitoring waste procedures on site. Six-monthly training on waste management was provided to Skanska Site Managers who then cascaded this information down to on-site contractors.

Procedures for the management of plasterboard and plasterboard waste are summarised in Figure 1. Whilst there was provision for some re-use of off-cuts, there was no formal system in place for the collection and storage of off-cuts for future use. The logistics of doing so, and the incentive to keep the site tidy (through the Waste Notice System) were believed by Skanska to be the principal constraints on this. The 1,100 litre colour-coded bins were stored external to the building under construction. There were, however, smaller mixed waste bins stored internally, closer to the point of work and it was suspected that some plasterboard waste could have escaped capture through this route. Unlike all other waste streams, the plasterboard contractor retained responsibility for the management of plasterboard waste under contractual agreement with Skanska. This approach was adopted by Skanska due to the uncertainty over the reclassification of plasterboard waste under the amended Landfill Regulations that existed at the time the contract was awarded.

7.3 Plasterboard waste arisings and economics

It is estimated that an average of 24.7 tonnes per month of plasterboard waste was generated during the construction phase with bulk densities of 0.34–0.37 tonne/m³ achieved in off-site movement in skips. Recovered plasterboard accounted for approximately one quarter of the total construction waste stream during the project period October 2004 to July 2005. Of an estimated 898 tonnes of plasterboard delivered to the site almost 228 tonnes was ultimately segregated as waste. This is equivalent to a wastage rate of 25% and significantly higher than industry estimates which are typically in the region of 10–15% by weight.

The average cost of transporting and transferring plasterboard for reprocessing was estimated to be about £62/tonne. This compares favourably with the average £61/tonne cost of disposal of mixed waste to landfill. Segregation also avoided problems associated with the legislative requirements for the disposal of mixed wastes containing high concentrations of sulphate-bearing wastes (given that plasterboard represents 25% of the total waste stream) in monocell landfill. Even if blending or dilution of wastes did occur [James et al. 2006], to reduce the proportion of sulphate-bearing wastes in the mixed waste stream, the above figures suggest that segregation, at this location and under these conditions, is virtually cost neutral.

As well as the direct costs associated with handling and transferring the plasterboard waste there were also indirect costs associated with managing and implementing the on-site waste management scheme. It was not possible to accurately quantify the total costs of managing the plasterboard recovery programme because the responsibilities and costs for its management are combined with various other roles. On-site, the Environmental Advisor and Logistics Manager were responsible for the day-to-day management of the scheme, whilst Skanska also employed a team to oversee the collection and reporting of environmental and waste data.
8 COVENTRY WALSGRAVE HOSPITAL
(SKANSKA INTEGRATED PROJECTS)

8.1 Site and project details
The project involved the construction of a 1,250-bed, acute hospital for the University Hospital and Coventry and Warwickshire NHS Trust, a mental health unit for Coventry Healthcare NHS Trust and a clinical sciences tutorial and research building. The 27 hectare project began in 2002 with completion scheduled for 2007.

8.2 Plasterboard waste management procedures
Skanska sub-contracted responsibility for the site waste management system to a logistics company, although the Skanska Logistics Supervisor retained responsibility for overseeing on-site management of construction waste.

The logistics company was responsible for managing the skip compound on site, for issuing wheeled bins (660 litre) to contractors and for on-site management of the bins. With the exception of the plasterboard waste, which was the responsibility of a dedicated plasterboard waste carrier, the logistics company was also responsible for the transport of site-segregated wastes.

Trade contractors were responsible for the collection of the wheeled bins from the waste compound, segregation of waste at the point of generation and return of the bins to the waste compound once full.
Bins were issued to trade contractors with adhesive bar codes that identified the specific contractor and the type of waste for which the bin was designated. This allowed PC-linked monitoring of the waste arisings by type and by contractor.

To minimise the amount of operative time spent on waste management the principal site contractor designated specific employees with responsibility for transfer of the bins, and the bins were usually transferred towards the end of the working day/shift when transfer was least likely to be obstructed. The waste logistics company reported that during busy periods on site, and where waste had to be pushed manually from the upper floors of the new building, it could take up to 40 minutes for a trade contractor to make a return trip to the skip compound.

Procedures for the management of plasterboard and plasterboard waste at the Coventry Hospital site are summarised in Figure 2. Whilst plasterboard was taken to a holding area upon delivery to the site, the need to avoid delays and the nature of the build (7 storeys high) meant that plasterboard was transferred and stored adjacent to the point of use (sometimes externally) for up to a week. As at Woodbridge, it was reported that plasterboard off-cuts were used where possible but the waste logistics manager also indicated that potentially re-usable off-cuts of plasterboard frequently entered the waste stream, primarily

---

**Figure 2. Coventry hospital plasterboard material flow diagram.**
because lack of space prevented storage pending reuse.

8.3 Plasterboard waste arisings and economics

The project segregated an average of 53 tonnes of plasterboard waste each month between September 2003 and February 2006, accounting for 26% of the total waste stream. Off-site movement of plasterboard waste achieved skip bulk densities of 0.18–0.21 tonnes/m³. Because data on plasterboard use was not collected throughout the construction period it was not possible to accurately ascertain the level of plasterboard wastage for the entire project. However, between January and October 2005 1,365 tonnes of plasterboard was delivered to the site and in the same period 561 tonnes of plasterboard waste was generated. This would appear to suggest a 41% wastage rate over this period but this figure does not account for plasterboard which may have been delivered to the site prior to January 2005. The plasterboard contractor reported that wastage rates of 18–22% are more typical.

The average cost of transporting and transferring plasterboard for reprocessing was estimated at £58/tonne. This compares favourably with the average £61/tonne cost of disposal of mixed waste to landfill encountered at the site.

9 THE HOME OFFICE, QUEEN ANNE’S, GATE DEMOLITION PHASE (SKANSKA BUILDING)

9.1 Site and project details

This £100 million project includes the strip-out and refurbishment of the Home Office building in central London. The building comprises sub-basement, basement, ground floor and 16 upper floors with a gross internal floor space of approximately 51,000 m². The client requires the development, to achieve a nationally recognised environmental performance standard of ‘Excellent’ (under the Building Research Establishment Environmental Assessment Method – BREEAM). Segregating and monitoring construction waste on site is an area of environmental performance that which contributed to the achievement of this target. With the exception of a relatively small internal courtyard, there are no external grounds associated with the building.

This section reports on the ‘strip-out’ or ‘demolition’ phase of the project. The construction phase of the development will be reported at a later date.

9.2 Plasterboard waste management procedures

The Waste Management Plan for the site requires subcontractors to provide monthly figures of all wastes removed from site to Skanska site management. This includes figures on waste type, volume, tonnage, disposal cost and percentage recycled. In line with common industry practice the site demolition contractor was wholly responsible for managing site waste in accordance with Skanska’s waste management plan during the demolition phase of the project.

Procedures for the management of plasterboard waste for the site are summarised in Figure 3. Plasterboard was cut away from the frame of the internal partitions using hand tools, allowing significant capture of plasterboard waste in large sheets with minimal contamination from parts of the frame and attachments. The demolition contractor provides a pool of sixty 1,100 litre wheeled-bins for the collection of demolition wastes. The bins were labelled but not colour-coded and were used for the segregation of different waste streams as required. Due to space limitations, both within and external to the building, filled wheeled-bins were held in the basement holding area pending transfer of waste from site for as little time as possible and skips were not always stored on site but brought in as required.

Requirements to segregate waste were communicated to operatives during site induction and through focussed toolbox talks. More detailed environmental and waste training was provided by the Skanska site Environmental Coordinator to Site Managers (Skanska and contractor) as required. Skanska Site Environmental and Waste Representatives undertook frequent tours of the site to ensure that waste was correctly stored and segregated and signs were placed around the site to encourage waste segregation.

9.3 Plasterboard waste arisings and economics

Between January and February 2006 an average of 51 tonnes per month of plasterboard waste was generated, with transfer off site by skip, achieving bulk densities of 0.27–0.40 tonnes/m³. The plasterboard reprocessor reported that tonnages achieved at the upper end of this range represent very good bulk densities for segregated plasterboard waste.

The cost of recycling segregated plasterboard waste was £52–68 per tonne, depending on bulk density, with an average cost of £57/tonne. Mixed general wastes from the site were taken to a waste transfer station approximately 13 km from the site for an average cost of £50/tonne. Hence, the financial benefit of blending plasterboard with other wastes to allow disposal by landfill would have been marginal at this site. The figures also show that the bulk density of plasterboard during transfer from the site to the reprocessor could be decisive in terms of the economics of recovery. The higher bulk densities achieved during the trial resulted in costs for recycling that were comparable with those for disposal of mixed waste.

The demolition contractor also indicated that management costs for temporarily stockpiling and
segregating waste is higher than those for disposal of unsegregated waste. This observation is supported by a recent Australian study [Seydel et al. 2002], but there is currently a lack of similar evidence in the UK. It is, however, clearly important that such costs are considered when assessing the economic viability of a particular management system.

10 SUMMARY OF FINDINGS AND DISCUSSION

The key findings of the three sites are considered below using the categories outlined in Table 1 as a guide for summary and discussion. Whilst these headings have been used to guide the discussion it should be borne in mind that the principal aims, and thus findings, of the project concern the site-based factors that affect recovery of plasterboard. The concluding section of this paper considers the importance of these site-based factors within the broader context of the current climate for recycling plasterboard in the UK.

10.1 Organisational characteristics

Skanska is highly regarded for its corporate environmental commitment. In the absence of clear legislative and economic incentives for the segregation and

---

**Figure 3.** Queen Anne’s gate demolition phase plasterboard material flow diagram.
recycling of plasterboard this commitment, and specifically Skanska’s policy to segregate at least six waste streams on its projects, is perhaps the single most important factor currently driving waste management practices on their construction sites. Not only does this policy and commitment determine the operational arrangements for site waste management, it should also, if communicated effectively, improve the buy-in of staff and sub-contractors. Teo & Loosemore [2001] found that whilst operatives do not necessarily have negative attitudes towards waste management, their goodwill can be impeded by a lack of managerial commitment.

In the three projects reported, the commitment of Skanska to the environment and waste management was a key factor in the successful recovery of plasterboard for recycling. The significance of this commitment, along with the size of the company, should not be overlooked when considering the extent to which the findings of this research are transferable to the UK construction industry more generally. Without either the legislative or the organisational impetus, attempts at overcoming the more practical barriers to increased plasterboard segregation are likely to prove difficult.

10.2 Site logistics and management

It is widely recognised e.g. Craven et al. [1994], Johnston and Mincks [1995] that each construction project is unique in terms of the nature of the site and the associated logistical benefits and drawbacks. The three sites reported in this paper were selected to reflect such variability and thus expand the relevance of the project findings as far as possible. At Woodbridge Airfield the large site meant that space was not a limiting factor on the storage of waste receptacles, but the dispersed nature of the site meant that forklifts had to be employed to transfer waste bins between the point of work and the waste compound. In contrast, at the Coventry Hospital and Queen Anne’s Gate projects space was much more limited meaning that the timing of the movements of waste and waste receptacles was key. At Coventry Hospital the time spent on waste management activities was minimised by moving the wheeled-bins during quiet periods in the operational day whilst at Queen Anne’s Gate, the space constraints were overcome by a coordinated approach to the delivery, filling and transfer of waste containers.

The projects also show how logistical arrangements, or perceived logistical barriers, could result in potential abuse of the waste management systems employed. For instance at Woodbridge there was concern that plasterboard waste would be deposited in mixed waste bins that were closer to the point of work and at Coventry Hospital it was reported that sub-contractors tried to maximise the number of mixed waste bins that they were allocated in order to minimise the number of trips between the waste compound and the point of work (by taking bins for fewer waste streams).

On all three sites a combination of well-planned management and careful co-ordination between the various groups with responsibility for the waste management system minimised the logistical barriers and maximised the logistical opportunities on site. This has been referred to as ‘process optimisation’ by Wilson et al. 1998 who recognised the importance of site suitability and optimal provision of facilities. Monitoring and enforcement to mitigate against potential abuse of the waste management system were important aspects of the management approach. Skanska site managers and environmental representatives frequently reminded sub-contractors of their responsibilities during regular site-inspections and, where appropriate, ‘toolbox talks’. Training and communication also form an important component of the management system and these are discussed in Section 10.5.

One important finding arising from the Queen Anne’s Gate site is the logistical advantages of soft-strip demolition phase activities compared to construction phase activities. Both Skanska and the demolition contractor reported that successful waste segregation was facilitated by:

1. there only being one contractor on-site;
2. waste management forming part of the usual package of works for demolition contractors, and;
3. The production of large pieces of plasterboard during soft-strip demolition that are easier to segregate than (typically) smaller off-cuts generated during construction activities.

In contrast, construction sites usually involve numerous different trade contractors who traditionally have little responsibility for waste management. These factors make the training and co-ordination of waste management on construction sites a particular challenge. A single ‘waste-aware’ demolition contractor that is financially responsible for waste management, however, poses much less of a management challenge. This finding challenges, but does not necessarily contradict, that of James et al. [2006] who reported that ‘despite the potentially large arisings of plasterboard waste from the demolition sector, the construction sector has the greatest potential for the supply of suitable waste plasterboard for recycling’. Their argument is based on the fact that demolition waste is more likely to be contaminated than construction waste. However, if the demolition involves ‘soft-strip’ (or disassembly) then this is not necessarily the case. GRUK reported that the plasterboard removed from the soft-strip of Queen Anne’s Gate was of high quality, achieved good bulk densities and, in the large pieces of plasterboard that it produced, was highly suitable for their reprocessing equipment.
10.3 Economics

Whilst the cost of recovery and the cost of alternative disposal methods will differ geographically, all three sites have shown (albeit tentatively) that the cost of recycling is broadly comparable with that of disposal as mixed waste to landfill. Queen Anne’s Gate also demonstrated the significance of bulk density achieved during transfer to reprocessor on transport costs.

With only one known monocell landfill operator in the country and with landfill costs in excess of £116/tonne (inclusive of landfill tax but exclusive of VAT) [Diggle, Pers. Comm 2006], recycling plasterboard from all three sites was found to be significantly cheaper than disposal in dedicated monocell landfill for high sulphate-bearing wastes.

Evidence from the two construction sites and particularly from Woodbridge, supports the findings of Wilson et al. [1998] and James et al. [2006] who reported that the low material costs of plasterboard (relative to labour costs) means that wastage is often accepted to keep labour costs down. At Woodbridge, the plasterboard contractor retained responsibility for the waste disposal costs but this did not appear to have been effective in driving down wastage rates (with 25% wastage reported).

Insufficient evidence has been collected at this stage of the project to reflect on the management and labour costs associated with implementing a waste segregation scheme as opposed to primary disposal for all wastes as mixed waste. Contractors certainly perceived an increase in on-site waste management costs but it has, thus far, proved difficult to quantify the difference. Management responsibilities are often combined with other responsibilities and it is difficult to monitor the amount of time spent by operatives on waste management procedures specifically associated with on-site segregation.

10.4 Legislative

Because Skanska is committed to the segregation and recycling of its waste streams this study has looked beyond the purely legislative barriers. It is recognised that plasterboard segregation and recycling can be successful in spite of the legislative barriers and the principal focus of this research, therefore, has been on the site-based factors affecting plasterboard segregation and recycling. The legislative barriers are discussed in more detail by James et al. [2006].

10.5 Behavioural and educational

As outlined in Section 10.1 the environmental commitments of Skanska as a company are of utmost importance to the success of their waste management schemes. Whilst the attitudes and values of different tiers of employee have not been analysed (as for instance by Lingard et al. [2000] and Teo and Loosemore [2001]) there was certainly evidence, as would be expected, of differences in attitude between individuals. The individuals that contributed to the research ranged from those that were positively enthusiastic to the environmental cause (typically, but not always, those in environmental roles) through to those that accepted the need to segregate and recycle construction waste, to those that saw it as an unwelcome burden. It is not the purpose of this paper to analyse the extent and reasons for the individual variation (refer to Lingard et al, [2000], Teo and Loosemore [2001], Saunders and Wynn [2004]), but to understand the measures employed to successfully manage this variety of opinion.

On all three sites adequate training and communication were identified as principal factors that either ensured the success of a particular management scheme in the face of logistical difficulties, or that needed improvement to overcome observed difficulties with the management of a particular scheme. The communication of Skanska’s requirements to sub-contractors was identified as being of particular importance. Whilst Skanska employees associated with the project demonstrated a familiarity and commitment to the company’s environmental policy sub contractors (with differing organisational cultures) would receive guidance and training through site induction, tool-box talks and daily interaction with Skanska operatives on site. Successful plasterboard segregation at the Coventry Hospital site was attributed, in part at least, to the buy-in of the principal contractor to the waste management scheme and their low staff turnover which reduced the need for on-going training.

10.6 Contractual

On all three sites contractors were contractually obliged to adhere to the waste management schemes developed by Skanska. At Woodbridge and Queen Anne’s Gate the financial responsibility for waste management was passed, respectively, to the plasterboard and demolition contractors. In light of the material/labour cost dichotomy outlined in Section 10.3 it was realised that additional financial contractual obligations may be required to encourage further segregation and minimisation.

Woodbridge demonstrated how the waste notice system could provide a useful financial incentive for ensuring the site was kept clean and tidy but it may also have inadvertently inhibited waste minimisation as potentially reusable pieces of plasterboard were cleaned away. Skanska are currently looking at implementing ‘carrot and stick’ financial contractual requirements to encourage minimisation and segregation. In initial discussions with the plasterboard contractor for the construction phase at Queen Anne’s Gate the
contractor was responsive to the idea of financial incentives to improve segregation and minimisation. This observation lends tentative support to the findings of Saunders and Wynn 2004, who reported a willingness beyond what might have been expected among sub-contractors to accept some of the costs of waste reduction.

11 FURTHER WORK

The research to date suggests that the following are worthy of further investigation:

- At the three sites reported in this paper there were no formal systems in place for monitoring the amount of plasterboard entering the general waste stream. Identifying the amount of plasterboard that is entering the mixed waste stream would provide a useful measure of the success of the segregation scheme and procedures for monitoring the composition of the mixed waste stream require development.
- Operational and management costs associated with implementing on-site waste segregation schemes are currently poorly understood. Further work is required to investigate the extent to which the perceived increase in on-site management and labour costs relative to ‘traditional’ mixed-waste site waste management methods are real. The findings of such a study would be of interest within the consideration of overall waste management economics. It would also be valuable to investigate the extent to which such potential costs serve as a barrier to segregation on different sized projects and by different sized companies.
- Compaction of plasterboard prior to transport to reprocessor has the potential to improve transport economics. Further investigation is required into the costs and benefits associated with on-site compaction.
- Careful consideration is required of the potential financial contractual incentives that can be employed. A variety of carrot and stick approaches can be envisioned but care must be taken to ensure that the incentives have the desired outcomes and do not inadvertently encourage unwanted behaviour or impose an undue burden.

12 CONCLUSIONS

In assessing the barriers to segregating and recycling plasterboard from construction and demolition sites in the UK, this paper suggests that too much emphasis is frequently placed on the primacy of perceived legislative barriers. Whilst such barriers may be important, the success of plasterboard segregation for recycling on UK construction sites is governed by an interplay of factors. This paper has shown that where there is an organisational commitment to segregate for recycling, the legislative (and, to some extent, economic) climate becomes less important than the site-based factors for governing the success of a particular scheme.

Although the factors affecting successful plasterboard segregation have been reported under specific headings it is the interplay between these factors that ultimately determines material recovery success. Each construction site is unique and will have its own associated logistical problems. Successful management through process optimisation can overcome these problems and this, in turn, will depend on organisational culture, communication and adequate training. Furthermore, the imposition of contractual (possibly financial) obligations can contribute to the success of a scheme and relegate the importance of perceived legislative barriers.

In an industry in which client demands and the need to be seen to be responding to environmental issues are increasingly important the construction industry needs to understand the multiple factors affecting the success of segregation and recycling schemes. The three projects described demonstrate the logistical, economic and technical viability of plasterboard segregation and recycling schemes on different types of site with different approaches to management. The projects are not intended to represent best-practice but to illustrate the types of issues that are likely to arise in implementing such schemes and the relative benefits and drawbacks of particular management techniques used to address these issues. It is hoped that the findings from these three case studies can be used to support and inform the wider adoption of similar plasterboard recovery schemes in the future.

REFERENCES


Extending experimental data to investigate phosphogypsum use in light brick by Artificial Neural Networks

Mehmet Ali Yurdusev  
*Celal Bayar University, Civil Engineering Department, Muradiye, Manisa, Turkey*

Ahmet Ali Kumanlioglu  
*Dokuz Eylul University, Civil Engineering Department, Buca, Izmir, Turkey*

Yüksel Abali  
*Celal Bayar University, Chemistry Department, Muradiye, Manisa, Turkey*

M. Sadrettin Zeybek  
*Gazi University, Chemical Engineering Department, Corum, Turkey*

Fethullah Canpolat  
*UWM Center for By-Products Utilization, Department of Civil Engineering and Mechanics, University of Wisconsin, Milwaukee*

ABSTRACT: In this study, usability of wastes produced in phosphoric acid plants in structural brick manufacture has been investigated. A series of experiments for three different material combinations namely original waste plus normal brick clay, washed waste plus normal brick clay and normal brick clay with no additive were carried out to achieve this end. The results have shown that the industrial wastes considered improve the performance of the bricks. However, the results have also shown that further investigations are needed to explore the effects of interim or outer values on the performance of the bricks. To achieve that end, a neural experimental study is adopted. The outcome of these artificial tests have provided the appropriate values for the waste addition rate, firing speed and firing temperature.

1. INTRODUCTION

This study investigates the possible use of phosphoric acid plant by product, namely phosphogypsum, in light brick production. There are several parameters involved in using these wastes in brick production namely the rate of added waste, firing speed and firing temperature. The performance of these parameters can be measured by several criteria such as natural drying shortening, total linear shortening, water absorption and weight loss. The first three variables are preferred to be minimum whereas the last one to be largest for a quality product. Around 220 experiments for three different material combinations namely original waste plus normal brick clay, washed waste plus normal brick clay and normal brick clay with no additive were carried out to achieve this end. The results have shown that the industrial wastes considered improve the performance of the bricks in terms of the criteria mentioned above. The findings of experimental study are presented in [1]. Despite quite large number of experiments undertaken, It is observed that further investigations are needed to explore the effects of interim or outer values on the performance of the bricks. To achieve that end, an artificial neural network study is adopted to extend the data obtained through the experiments so that the interim values not caught during the experiments can be examined whether or not the suitable production values exist within them.

To explore the effects of interim design or input values on the performance of the bricks produced from the waste-added clay, the results of the experiments conducted were used to construct an Artificial Neural Network (ANN) for each type of material mixtures namely original and washed. The trained and tested network was then used to check the effects of 280 different combinations for mixtures mentioned. The outcome of these artificial tests have provided more data
2. EXTENSION OF EXPERIMENTAL DATA BY ANN

2.1 Experimental data

A series of experiments were carried out to explore the possible use of phosphogypsum in light brick production. In experiments, two different phosphogypsums were used namely: washed and original phosphogypsum referred to as YP and OP respectively. Moreover, no additive specimens were also prepared to compare the additive ones. Post firing experiments for water absorption, weight losses and total linear shortening were undertaken for the specimens. The results of the experimental study were presented in [1], to which the reader is referred for the parallel reading.

2.2 Building ANN structures by feedforward backpropagation algorithm

Although the number of specimens by which experiments were carried out is quite large, it is obvious that it is impossible to cover so many possible interim and outer values based on the four parameters considered, namely the rate of additives, firing speed, firing temperature and plasticity water. In this study an ANN approach is developed to extent the experimental data in an attempt to cover as many values as possible. This will enable to explore the effects of the interim values which were not considered during the experiments on the criteria by which the performance of the corresponding parameter set is examined.

In this study, the comparisons of traingdx and trainlm training functions employed in MATLAB have investigated in an attempt to find the most suitable algorithm for the data considered. All the ANNs transfer functions were sigmoid function. At the end of the training the ANNs output the experiments’ output were compared each other 10. Two different feedforward backpropagation artificial neural networks (FBANN) for two different phosphogypsum were used. Inputs and outputs are chosen according to the experimental parameters mentioned previously. The inputs are added waste, firing speed, firing temperature and the outputs are natural drying shortening, water absorption, and weight losses.

When undertaking ANN exercise, the training was stopped at an appropriate epoch based on the mean square error and training, validation and test squared error were drawn in figures. Regression analysis was made between the outputs of FBANNs and experiments in Matlab. The operational detail of the exercises are not included in the article.

2.3 Extending experimental data by the ANN structures

Two different training algorithm, traingdx – trainlm, were used to train the FBANNs. The traingdx training algorithm showed that it used more epochs to find the results than trainlm. The trainlm training algorithm showed that its regression results were better than traingdx training algorithm regression results. Therefore, the trainlm training algorithm is chosen to carry out artificial experiments to produce synthetic data. As mentioned earlier, the purpose for producing synthetic data by the ANN approach is to find appropriate values for the input values on which the experiments are based. The appropriate values will then be used if and when the industrial wastes are to be used in producing light structural brick. The ANN structures previously trained and tested are used, (the code is repeatedly re–run) to produce output values. For this purpose, 280 different combinations were formed for each of OP and YP types. These numbers are regarded sufficiently enough to cover the interim values not considered during the experiment.

2.4 Results and discussions

The results of the neural experiments performed are presented in Figs. 1–6. The figures give weight loss, water absorption and natural drying shortening values with respect to additive rates for both original and washed phosphogypsum. The values in the figures show the average values for the different firing speeds for a certain firing temperature. The results of the neural experiments presented in Figs. 1–6 conform the results of experimental study presented in [1].

As seen from Fig. 1–2, all values for weight loss increase with respect to the additive rate at all firing temperatures for both type of additives. However, the use of washed one gives slightly larger values even at 2% additive rates, which shows that the addition of phosphogypsum makes the brick lighter.

The water absorption values for original and washed phosphogypsum are given in Fig. 3–4. From this figure, it is seen that the more additive is used the more the brick absorbs water, which is not desirable. However, there is little increase up to 8% in washed phosphogypsum. Therefore, there will be not much problem adding phosphogypsum up to this percent.

With respect to natural drying shortening as seen from Fig. 5–6, addition of phosphogypsum greatly improves the quality of the brick by reducing the shortening values significantly, up to 1% in washed phosphogypsum.
The increase of firing temperature affects the bricks positively in terms of weight loss and water absorption for both original and washed phosphogypsum. That is, weight loss increases as temperature rises whereas water absorption decreases. This is also true for natural drying shortening in the case of washed type. For the original additive type, the lower temperatures give lower results. The aim of this study is to find appropriate additive rates and firing temperatures. The appropriate values, as mentioned previously, are those yielding the...
lowest value for natural drying shortening and water absorption and the largest value for weight loss. The appropriate additive rates and firing temperatures based on these three criteria (natural drying shortening, water absorption and weight loss) are derived from Figs. 1–6 and presented in Table 1. From Table 1 and the Figs. 1–6, it can be stated that 18% additive rate at 900–1000 firing temperatures is possible for both original and washed phosphogypsumes. In this case, weight loss and natural drying shortening values become most desirable. However, water absorption could be a little bit higher than standard values. If this is perceived more important, less additive rates (e.g., 10%) can be used.

### 3. CONCLUSIONS

An experimental study is conducted to investigate whether the industrial wastes produced by phosphoric acid plants can be used in light brick production. The experimental study is then extended to cover more input values using an ANN approach. This exercise is aimed to find more appropriate input values than those that were not covered by experiments. The results are found to be generally parallel to the those of experimental study. It seems the ANN study offers less additive (18%) than that (20%) of experimental study. However, in ANN study 20% was not used as it is in experiments. Nevertheless, it shows that the maximum additive rate is preferable as for experimental study.

### REFERENCES

Opportunities for wealth generation through small scale sustainable building materials production

S.N. Mclean, A. Williams
School of Architecture Design & the Built Environment, The Nottingham Trent University, Nottingham, England

D.R. Moore
Scott Sutherland School of Architecture and the Built Environment, The Robert Gordon University, Aberdeen, Scotland

ABSTRACT: Rural Bangladeshis face many problems; low incomes due to an inability to participate in wealth generating activities, access to clean drinking water and loss of shelter due to flooding and cyclonic conditions. More durable building materials, based on building lime, would reduce shelter losses and facilitate the harvesting of rainwater for provision of clean drinking water. Producing such materials as a complimentary activity to agriculture could provide an additional income stream for some of the world’s poorest people. The use of solar energy to process common waste products, such as shells from the fishing and Ger farming industries, could facilitate such an opportunity.

Proposals for design improvements to increase the payload capacity of a solar kiln through changes to traditional kiln operation, and methods of reducing heat losses so as to achieve efficient production of Building Lime are presented. Building lime based products would facilitate more durable shelters, in regions where supplies of traditionally harvested building materials are becoming scarce.

1 INTRODUCTION

The manufacture of more durable building materials is proposed as a sustainable means of improving the ability of housing to resist the effects of flooding and cyclones. Due to the economic and geographical nature of Bangladesh it is proposed that this production is located within the region, and undertaken by the primary users. The choice of production of a material such as building lime, which can be used to produce a number of durable building products, is seen as being able to provide benefit beyond the usage of such materials. Such production could thus help alleviate poverty by providing opportunities for generating wealth whilst remaining complimentary to primary agricultural activities.

‘Fired’ building material (such as clay bricks) production is traditionally energy intensive, causes high levels of pollution and land wastage, and is demanding of scarce natural resources. The development of sustainable technology to facilitate the small scale production of building materials, using solar energy to process waste materials is one alternative approach. The choice of building lime production is feasible in many parts of rural Bangladesh, as waste products which can be manufactured into quick lime are readily available and the methodology employed to produce lime can be made complimentary to traditional agricultural activity.

2 ECONOMIC ACTIVITY TO ALLEVIATE POVERTY

Elimination of poverty is a key objective for those involved in the development of third world nations. The Millennium Goals agreement signed by 149 countries seeks to halve poverty levels by 2015; an aim requiring a single definition of poverty, not the current plethora. One definition is simply a figure of $1 income per day, as set by the World Bank in 1985, below which an individual is classed as being in absolute poverty. This definition is convenient in so far as a monitory value can be applied to manifestations of poverty such as housing, health and welfare. Measuring access to, and participation in, the essentials of life is a non-monitory approach. One example is based on the human rights framework and measures access to clean water; sanitation; shelter; education; information; food, and health. Lack of access to one basic need defines an individual or household as being ‘deprived’. Those deprived of two or more basic needs
are defined as being in ‘absolute poverty’ (Townsend Centre, nd). It is also important to understand that there is a presupposition of a ‘normal’ position of inclusion which, for many in developing countries is simply not the case and, for some researchers (de Haan, 1994) this reduces the value of some approaches (such as the social exclusion approach). Irrespective of how poverty is defined it can only adequately be addressed by providing opportunities for the generation of additional wealth and by ensuring that full access for those in poverty is available to such opportunities. Activity targeted at producing ‘new’ wealth must not detract from the impact of existing economic activity; there must be no displacement of resources required for the production of food. Ideally, such activity would utilise an energy source not requiring fossil fuel supplies (for environmental and cost reasons), would utilise waste products as raw materials (for environmental reasons, and positive displacement of costs related to the purchase of raw materials), could utilise either non skilled labour or the existing skills of the labour resource, and produce materials to address some of the other manifestations of poverty (low grade housing, etc.). Such a scheme recognises the complexity of interactions between the causal factors of poverty and does not seek to address a single factor in isolation. Of the energy sources which do not require a ‘processed’ fuel (such as fossil fuels) for their operation, the equipment required for their harnessing is often unsustainable in third world settings due to factors such as the cost of providing equipment such as photo-voltaic cells (typically have to be purchased in hard currencies), or the difficulty of maintaining technology in regions not having the advanced engineering skills to provide spare parts or problem-solve in the context of high technology equipment. One energy source can be harnessed using comparatively low cost equipment involving only minimal mechanisation: solar energy harvested using solar gatherers. Solar technology can provide a versatile heat source, from very high temperatures generated at a point load down to moderate temperatures distributed sufficiently to cook food or season timber. Once the objectives for the use of this technology have been established, its operation is not generally labour intensive or requiring of high technology skills. In addition, Bangladesh has an abundance of solar energy throughout most of the year, suggesting that solar technologies could make a significant contribution to the alleviation of poverty in the context of Bangladesh.

3 A POSSIBLE SOLUTION FOR SOME OF THE AFFECTED PEOPLE

Economic poverty is linked to the primary industry of agriculture, as high population densities mean that most plots are small. Wealth generation is therefore most likely to be achieved from additional complementary economic activity. Short supply of traditional fuels (Nuruzzaman 2004) requires any additional energy requirements to be renewable, and gathering of any raw materials must not be at the expense of agricultural acreage. Loss of domestic shelter due to flooding could be arrested if vernacular housing was constructed from more durable materials. The typical Bangladeshi house would be built from earth, bamboo, reed and palm leaf (Muktadir & Hassan 1985). These have low resistance to floodwaters, and are unsuitable for the practice of harvesting rainwater (Khaleduzzaman 2000). The local production of durable building materials, which resist moisture, are suitable for use in rainwater harvesting, are made from waste raw materials, and are produced using renewable solar technology, could help address the problems of poverty for some of Bangladesh’s rural poor.

4 USING SOLAR KILNS TO IMPROVE DURABILITY OF MATERIALS

The use of materials such as brick, clay tiles, building lime (all kiln fired), and lime based concrete in construction could reduce housing losses due to flood waters. However, such materials are both relatively expensive and attract significant debate with regard to factors such as high levels of embodied energy. Regional alternatives (limestone, silica sand and ceramic clay are local natural resources), are available, and the production of Water Snails and Shellfish is a major Bangladeshi industry providing a waste product (shells) that can be used to produce building lime (Care International 2002). Current approaches lead to high embodied energy levels; particularly if traditional fuel fired kilns are used. There is a need to consider alternative energy sources (solar, etc), along with the design of appropriate kilns to utilise them in the production of durable building materials, thus addressing problems of both substantial shelter and clean drinking water. A further benefit would be if these objectives could be achieved without recourse to imported building materials.

5 SAND LIME: AN ALTERNATIVE MATERIAL

Focus on the production of durable building materials such as clay roofing tiles has the benefit of ‘improving’ an existing technology, but does not respond to the problem of environment degradation resulting from extraction of raw materials. An alternative approach would be to consider an abundant raw material; in
this case one possibility would be waste from the shell fishing and Ger farming industries. These can be burned in a solar kiln to produce building lime. Additional to the production of lime wash, mortar and concrete is the manufacture of sand-lime products. These may include bricks, roofing tiles and cladding panels, all produced in a process typically requiring 30% less energy than clay brick production (Contec 2002). These materials require processing in a steam autoclave. It is therefore feasible in areas where both a source of lime and silica sand are available, that solar technology could be utilised to produce alternatives to ceramic products. This possibility frees any aid scheme proposal from being restricted to areas of clay deposits and opens up other avenues for production.

6 THE USE OF SOLAR TECHNOLOGY

Solar energy technology has been developed in the region to provide heat for cooking (Popen & Weiler 2002). Additionally, work has in the past been undertaken to produce solar kilns for firing ceramics and for producing lime. This utilises the simple and robust technology of focusing sunlight using parabolic mirrors. The Laboratory for Solar Technology at the Paul Scherrier Institute (PSI) in Switzerland has successfully developed an industrial solar limekiln (lime burning requires 900°C), thus evidencing the technology’s general feasibility (Maier 2001). Heating a kiln with solar energy is technically possible. From a practical perspective, kiln capacity is a key factor. This tends to be restricted due to heat losses through the fabric of the kiln. Advances in insulation technology provide opportunities for increasing capacity, as do improvements in design capabilities.

It appears possible to achieve a capacity allowing production of sufficient quantities of quick lime or ceramic goods to enable the producers to benefit financially from the sale of some of the production, and utilise an appropriate quantity themselves. The actual product from a solar kiln depends upon natural and geological resources available (seashells, limestone, silica sand, clay, etc). Products could include building lime, roof tiles, clay pipes, domestic objects, etc. Such goods form the components of rainwater-harvesting, i.e. a suitable catchment surface, guttering, pipes and storage vessels. Additional produce could include rice husk particle board and sand-lime materials, both of which require super heated steam in the manufacturing process. In examples such as Auroville in Tamil Nadu, India solar reflectors have been used to generate 4000 kg of steam per day for use in cooking. Feeding such energy into simple autoclaves would significantly increase the range of building products possible.

7 POVERTY IN BANGLADESH

Bangladeshis face problems of low income, a lack of assets such as land and permanent housing, shortages of clean water and adequate food, and an inability to participate in commercial activity (Maxwell 1999). The average income per capita in 1998 was cited as $266 per annum (British Council 2002), well below the World Bank’s stated absolute poverty mark of $1 per day set in 1985. Population density is approximately 800 persons/Km², which when combined with, 65% of the population relying upon agriculture to support them, results in land-use problems (Asian Development Bank 1999).

Much of the country is less than 5 metres above sea level. Annual flooding covers 20% of the landmass, and in the floods of 1992 over 50% was under water (ISDWC 2002). These floods wash away crops, pollute groundwater stocks and destroy the vernacular mud-brick and palm leaf buildings (BSHF 2001). Changes in global climatic conditions are set to extend their severity (Christian Aid 2004).

Between 1980 and 2000 Bangladesh has received the equivalent of $38 billion in aid (Benson & Clay 2003), and yet poverty is actually on the increase, (Kazi 1999). Majumdar (2001) criticised the ‘dependency’ nature of aid to Bangladesh as creating a mindset of “... the more aid we receive, the more we seem to need.” If aid is to be effective it may need to be focused upon wealth generation activities. Christian Aid (2003), states that effective disaster mitigation needs to be undertaken from within the community in the form of significant self-help opportunities. Thus, an alternative approach is required.

The Overseas Development Institute (ODI) suggests that prior to any issuing of aid Participatory Poverty Assessments (PPAs) should be established (ODI 2001). PPAs include a broadening of stakeholder involvement and support. Bangladeshis need to benefit from advances in technology, thus requiring aid targeted at self-help initiatives. A historic lack of stakeholder initiative in aid has failed to maximise its effect, and created a dependency culture (Norton et al 2001). One method of engaging stakeholder activity is by encouraging activities leading to long-term quality of life improvements. The posited approach links both wealth and health improvements through improved standards of shelter (and the quality of materials used) and the supply of safe drinking water.

7.1 Poisoned drinking water

Bangladesh’s soil is enriched by waterborne silts (ISDWC 2001). These can contain levels of arsenic higher than the safe limit set by the World Health Organisation of 0.05 mg per litre. This has contaminated deep water aquifers drained by long-running...
extraction of drinking and irrigation water through tube wells. Levels exceeding the safe limit were found in two thirds of the 8000 tube wells tested in the six months up to March 1998 (Islam 1998). The potential number of Bangladeshis affected by arsenic poisoning has increased from 65 million in 1998 to 90 million in 2000 (Islam 2000). This evidences an escalating disaster causing Bangladeshis to revert to unsanitary water. The Islamic Relief Charity states that a Bangladeshi child dies from water related disease every 8 seconds thus indicating the scale of the disaster in terms of loss of life (but without explicit identification of related economic loss) and the desperate need to find a safer alternative supply of water. An alternative supply is by rainwater harvesting. However, the effectiveness of the harvesting process depends upon various factors, one being the construction form of the roof.

7.2 Insufficient housing

A typical Bangladeshi house would be a small single celled structure, mainly constructed of locally available materials such as earth, bamboo, reed, and palm leaf (Muktadir & Hassan 1985). Building commences on a plinth of rammed earth. The earth block or woven panels walls are fixed to the plinth using bamboo poles or timber. A palm leaf (Golpata) thatch on a bamboo frame achieves reasonable resistance to the ingress of rain, but has a short life span. The porous walls are protected from downpours by the overhang of the roof. Typically, there is no formal drainage, only the capability of the roof to shed water away from the base of the structure.

Much of the housing in rural Bangladesh is essentially temporary due to the inability of the materials used to withstand damage by floodwater. These materials are, however, indigenous, inexpensive and perceived to be in abundant supply. The perception of abundant supply is, however, starting to become erroneous. This is evidenced by a recently commissioned report claiming that stocks of some species of Bamboo and Rattan have almost disappeared through over-harvesting and land clearance (Nuruzzaman 2004). The transient nature of the houses acts as a disincentive to supplying fixed services to improve their basic amenities. This is evidenced by the fact that 70% of Bangladeshi families do not have access to an electricity supply (Ahsan 2004).

Houses in Bangladesh have evolved to suit popular culture, both social and religious. The implementation of different architectural practices (such as western style housing) has not been successful in rural Bangladesh (Muktadir & Hassan 1985). A more realistic approach would look to improved vernacular materials that provide better moisture resistance and greater durability. Such materials have the additional potential to contribute to the alleviation of poverty through their having a market value, additional to the provision of shelter capable of withstanding flooding. This will, in turn, reduce the reconstruction cost demand.

7.3 Rainwater harvesting

Domestic Roof Rainwater Harvesting (DRWH) is simply collecting of rainwater as it falls upon a domestic roof, prior to storing it for use as drinking water, etc. Vernacular housing has been identified as comprising neither formal rainwater goods (gutters, etc), or suitable ‘informal’ collection surfaces. Thatch roofing, for example, is an unsuitable material to provide a DRWH collection surface for harvesting rain water (UNDP 2003). This is due to the fact that contact with organic matter can colour the water. Additionally biological and chemical contaminants can not easily be removed from such surfaces even through the flushing action achievable with man-made surfaces (DTU 1998, 2001).

Roofs manufactured from tiles or corrugated steel sheeting are deemed most suitable. However lead contamination can be introduced by galvanised surfaces (UNDP 2003), which indicates that one of the most ideal surfaces for harvesting rainwater would be one constructed from clay, concrete or sand/lime tiles. These could provide a resource (clean water) that can be used directly, and thereby improve sanitation levels. It can also be used indirectly as a means of wealth generation in that clean water has a commercial value. A survey of user attitudes to DRWH cited the additional income achievable through selling water as the joint most important perceived advantage of DRWH (DTU 2002). Furthermore, one of the problems faced during flood conditions is the absence of clean water (Howard 2001).

If alternative roofing materials could be manufactured regionally from indigenous materials, the solution to the problem of providing durable shelters could also address the problem of poverty within rural areas, and finance improved shelters through ‘locking-in’ (through a rudimentary property market mechanism) what wealth there is within the region. One approach to producing improved materials would be to utilise a relatively abundant resource (solar energy) through the use of solar kiln technologies.

8 NATURAL DISASTERS

Bangladesh suffers from flooding on an increasing basis. It is an irony that the most fertile soil in Bangladesh is on the chars located in river deltas (Palakudyill & Todd 2003). However access to such fertile soil often brings with it the certainty of flooding.
In the floods of 1987 and 1988 millions of hectares of crops were lost, millions of homes were lost and an estimated 45 million people were negatively affected (Brammer 2002). These natural disasters are compounded by the fact that the region also suffers from regular cyclonic conditions, with 14 major very severe storms occurring in the last century and an annual cyclone risk level of over 10% (Benson & Clay 2003).

These problems are not exclusive to Bangladesh as figures show the number of people annually affected by natural disasters in Asia grew by a factor of thirty five in the decade 1992 to 2001, and world wide by a factor of twenty five during the same period (Christian Aid 2003). This evidences the possibility that such regional production of durable building materials could find benefit across a wider region than just Bangladesh. The reasons for the increase in natural disasters are well documented with global warming, and the El Nino/La Nina inversion phenomena being cited by many climatologists as primary reasons. It is widely believed that sea levels are set to rise globally at an increasing rate, and this will inevitably result in additional flooding in a low lying country like Bangladesh.

In Western Countries the extent of building losses following flooding may be relatively small. However, due to the nature of Bangladeshi houses, built from thatch, bamboo, mud brick and rattan; losses are much higher as the materials offer little resistance to the flood waters. This is evidenced by reports of major flooding in the UK in 2000 which caused hundreds of millions of pounds worth of damage, but provided no mention of a homeless figure. Conversely the flooding which affected Bangladesh in 1998 was cited as leaving 40 million people homeless, and the lesser flooding of 2000 was cited as having washed away 300,000 homes (Howard 2001).

8.1 Testing the relevance of technology aid

Developing countries can be characterised by a scarcity of technically skilled people, spare parts, transport infrastructure, and funds to cover maintenance. Any technology intended for long term use should be sustainable and maintainable in the region, beyond the period of the aid project. An example of a locally used technology that is not sustainable is the so-called brickfields. Bangladesh has approximately 10,000 brickfields using largely wood fuel despite government prohibition in 1996. Approximately 50% of these production units are unlicensed and around 30% of them move annually to new locations leaving behind ground too polluted to sustain agriculture. Large amounts of soil are also removed in order to extract clay. The government is now seeking to ban all clay brick production by 2008 (Rahman 2003). Aid-related technology should be seeking to reduce such adverse impacts of indigenous technologies. This paper posits that the appropriate use of solar technologies would allow the continued production of clay products whilst eliminating the high levels of pollution resulting from the current production processes.

9 ESTABLISHING PRODUCTION

Thus far this paper has proposed investigation of a scheme which could help reduce levels of poverty on three fronts, and provide disaster mitigation. Providing funding for schemes which purely provide disaster mitigation without wealth generation could not service loan funding. Regional production of simple building materials could enable the construction of shelters which will resist flood damage. These products could thus facilitate rainwater harvesting, reduce dependency on contaminated water supplies and ensure continuity of supply even during periods of high ground water levels. Finally the sale of surplus materials and clean water particularly valuable during and in the aftermath of flooding would provide additional income, which would, both provide for continuity of the development funding and put the recipients in a better economic position to recover from the effects of flooding. The latter factor could determine the success or failure of any project, and its effectiveness in disaster mitigation. The raw materials proposed are mainly waste products, and the energy source utilised has few cost-in-use implications. However, supply and maintenance of the required equipment, even if subsidised by aid monies still carries a capital charge. In Bangladesh a number of schemes operate to fund the poorest of people in setting up wealth generating activity.

10 REQUIREMENTS FOR A SUCCESSFUL KILN

For any technology to be successful it needs to be sustainable within the region. Any equipment needs to be affordable, usable with appropriate training, maintainable and economically viable. Hence development of a solar kiln needs to address the following issues.

- Any equipment must be robust and simple to manufacture and maintain in Bangladesh.
- Its operation must be simple, and possible to operate in a manner which is complimentary to the primary agricultural activities.

The starting point for research was the issue of economic viability, estimated at a daily production of not less than 50 kg of quick lime or six 25 kg bags of slaked lime. Using traditional (non-solar) kiln technology...
this would require an energy input of 6000 kJ/kg (Boynton 1966). Calculations completed at Nottingham Trent University established that such energy could be supplied in terms of solar energy. At Bangladesh’s latitude a parabolic mirror of around 15 square metres area over an eight hour daily cycle would be sufficient. For such a payload to be achieved using the proposed modest, and possibly even smaller, reflector areas, the following problems would have to be solved:

- With direct focusing of all available energy, operational heat losses and the distribution of heat throughout the kiln to be overcome.
- The solar gathering equipment must be sustainable in terms of the area of the reflector, the robustness of its design and ability to track the Bangladeshi sun.
- Design of a solar collector.

Current development of a commercial solar lime kiln by The Paul Scherrer Institute has utilised computer control and heliostat technology. Such technology would not be sustainable in Bangladesh due to initial cost, maintenance requirements, and operator skill levels. Use of simple parabolic mirror technology would produce sufficient energy, providing the dish is kept in focus with the path of the sun. During Bangladesh’s longest day the sun rises at 68.6 degrees (North East) and sets at 243 degrees (South West), (NASA 2005). To restrict heat losses through any aperture, maximum concentration of rays is required. Simulation of a 4.5 metre diameter mirror, using a Light Tools Ray Tracing Programme and allowing for navigation inaccuracies up to 0.5 degrees, established that the most efficient rim angle is 45 degrees. This allowed for an aperture diameter of less than 120 mm. Experiments using a parabolic mirror with a rim angle of 72.5 degrees confirmed a variance of as much as a factor of eight as the focus moves off centre by as little as 5 degrees.

These experiments evidence the need for utilising the most efficient rim angle in situations where the tracking system used, due to the available technology, cost of production or requirement for a robust design may not always provide the optimum focus angle. Such consideration would be of paramount importance when establishing the nature of a primary reflector, which can be used in conjunction with sustainable equipment, and might over-ride the most cost effective methods of producing parabolic mirrors.

10.1 Design of a secondary reflector

It is proposed to focus energy in to the kiln using a double solar reflector. Reflectors smaller than 500 mm diameter can suffer from optical inefficiency due to high operating temperatures, however the larger the reflector the higher the manufacturing costs, and the greater the overall design difficulty. Therefore it is essential to establish the optimum position for a secondary reflector of around 600 mm diameter, and the minimum size of aperture that such a mirror would require to focus all of the available energy inside the kiln.

Simulations were used to design the optimum size, curvature and distance from the aperture for a reflector. Optimum performance could be obtained from a 640 mm diameter secondary reflector with a radius of curvature of 588 mm and a focal point of 264 mm, located at a distance of 500 mm from the kiln (Kourentzis 2005).

10.2 Design of a solar tracking system

It was hoped that sufficiently accurate tracking could be achieved manually. However simulation of the proposed solar gathering equipment using GPS coordinates of the sun’s position, determined that due to the previously stated detriment that even a small misalignment brings, manual refocusing would be required every 30 seconds. With such short intervals, production could not be complementary to other activities, unless motorised tracking was used. For such equipment to be sustainable it must be capable of manufacture and maintenance in Bangladesh without hi-tech equipment or expertise. Choices lie between an azimuth elevation system and a polar

<table>
<thead>
<tr>
<th>Table 1. Results.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Reflector No</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
<tr>
<td>5</td>
</tr>
<tr>
<td>6</td>
</tr>
<tr>
<td>7</td>
</tr>
</tbody>
</table>
tracking system. It was decided that the electronics package for an azimuth system could be designed to be simpler. A 1:3 scale prototype was built at Nottingham Trent University where a 1.5 metre parabolic mirror was mounted upon a steel space frame. The design sought simplicity of manufacture using lightweight tubular materials and limiting the complexity of angles to 30 degrees, 45 degrees and 60 degrees, thus producing a design which is simple to fabricate and which could possibly be reproduced later using locally produced bamboo as a replacement for the tubular materials. The design process also accounted for:

- The possibility of uneven ground,
- For the structure to be able to be stowed in the event of high winds,
- And user safety.

The design of the full-size frame would weigh 700 kg, which is 400 kg less than the fixed support originally supplied to NTU with a 4.5 metre diameter parabolic mirror. The process of design included the need to allow for operation in wind speeds of up to 25 mph, and survival in the 100 mph winds that can occur in Bangladesh (Brockenborough 2005).

Sensitivity sensors, which monitor light intensity, were linked to a geared electric motor to move the reflector across a plane where the maximum azimuth angle is 222 degrees and the maximum elevation is 90 degrees. Experiments using a variety of power sources determined that the more powerful the battery the more sensitive the system. It was determined that a 24 volt power source was superior to the 12 and 5 volt options tested. The field implications of using a 24 volt source are to be examined in a subsequent stage of the research.

### 10.3 Reducing heat losses

Capacity of previous designs has been limited due to heat losses which increase as the surface area of the kiln becomes larger. This can be countered by increased amounts of insulation. However, this impacts directly upon kiln size and manufacturing costs. Solar reflective technology has been used for heating ovens. Solar reflectors are focused upon the surface of cooking vessels, with claimed internal temperatures in excess of 200°C, and claimed surface temperatures up to 480°C (Cleardome 2006). Maximum temperatures vary due to reflector design, reflective materials used, absorbent coatings and climate. It is however feasible that such technology, if used to heat the exterior of a black-coated kiln, could reduce temperature differences and thus heat losses, in a more sustainable manner than increasing insulation volumes or efficiencies.

### 10.4 Overcoming difficulties in heat distribution

In order to efficiently process an economic payload, heat needs to be distributed evenly throughout the kiln. Initial proposals were to duct super-heated carbon-dioxide created by the calcining process to distribute heat. It was realised that the proposed raw materials would not require the same exposure to maximum temperatures as limestone pieces, where heat takes time to reach the core of the individual stones. Therefore exposure to temperatures in excess of 900°C need not be so prolonged. It is proposed to restrict the heat generated to a small fusion area, and to feed the raw material in to that area using an Archimedes screw. Calcined calcium carbonate breaks down into a fine powder, which can be sieved through a grid into a removable cassette, to be replaced by further raw material. This approach whilst providing some

<table>
<thead>
<tr>
<th>Reflector No</th>
<th>Distance from kiln apperture</th>
<th>Curvature of radius</th>
<th>Focal area no tilt mm</th>
<th>Focal area tilt of 0.5 mm</th>
<th>Min diameter size of reflector mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>440</td>
<td>1414</td>
<td>151.8</td>
<td>247.6</td>
<td>1000</td>
</tr>
<tr>
<td>2</td>
<td>450</td>
<td>1203</td>
<td>151.0</td>
<td>248.4</td>
<td>110</td>
</tr>
<tr>
<td>3</td>
<td>460</td>
<td>1033</td>
<td>149.8</td>
<td>250.0</td>
<td>850</td>
</tr>
<tr>
<td>4</td>
<td>470</td>
<td>870</td>
<td>148.6</td>
<td>263.0</td>
<td>800</td>
</tr>
<tr>
<td>5</td>
<td>480</td>
<td>774</td>
<td>147.6</td>
<td>255.0</td>
<td>750</td>
</tr>
<tr>
<td>6</td>
<td>490</td>
<td>674</td>
<td>146.6</td>
<td>258.4</td>
<td>680</td>
</tr>
<tr>
<td><strong>7</strong></td>
<td><strong>500</strong></td>
<td><strong>588</strong></td>
<td><strong>146.0</strong></td>
<td><strong>264.0</strong></td>
<td><strong>640</strong></td>
</tr>
<tr>
<td>8</td>
<td>510</td>
<td>513</td>
<td>146.0</td>
<td>269.6</td>
<td>590</td>
</tr>
<tr>
<td>9</td>
<td>520</td>
<td>448</td>
<td>146.0</td>
<td>276.8</td>
<td>540</td>
</tr>
<tr>
<td>10</td>
<td>530</td>
<td>391</td>
<td>147.4</td>
<td>285.6</td>
<td>500</td>
</tr>
<tr>
<td>11</td>
<td>540</td>
<td>339</td>
<td>149.0</td>
<td>304.8</td>
<td>450</td>
</tr>
<tr>
<td>12</td>
<td>550</td>
<td>293</td>
<td>152.0</td>
<td>331.6</td>
<td>410</td>
</tr>
<tr>
<td>13</td>
<td>560</td>
<td>251</td>
<td>158.2</td>
<td>366.8</td>
<td>370</td>
</tr>
</tbody>
</table>
mechanical challenges would allow for a much greater payload.

11 CONCLUSIONS

The production of durable building materials using waste raw materials and abundant solar energy is technically feasible. Evidence would suggest that it is possible to design a solar kiln for facilitating the production of building lime using waste materials from the shell fishing and Ger farming industries in quantities which are economically viable in a rural Bangladeshi context. Bangladeshis are however amongst the world’s poorest people, and many are unable to participate in any wealth generating activity. Therefore implementation of any such scheme would require the support of aid monies. The benefits of the production of building lime from waste raw material extend beyond the ability to manufacture durable vernacular materials which will reduce housing losses due to flooding. These might include extending access to rainwater harvesting as an alternative to contaminated ground and well waters, and an ability to generate complimentary wealth through trading lime, lime products and clean water.

Using advances in materials technology and design work, particularly in the area of reduction of heat losses, undertaken at Nottingham Trent University on design of solar gathering equipment and an operational kiln has sought to test the feasibility of such technologies in terms of the sustainability of such equipment in a rural Bangladeshi setting. These involve durability of the equipment, an economic payload, regionally sustainable manufacture and maintenance and ability to operate in the regional conditions. Evidence thus far would suggest the potential feasibility of such technology.

ACKNOWLEDGEMENTS

The authors acknowledge the work of Evangelos Kourentis who undertook much of the laboratory and fabrication work as part of a thesis for MSc Engineering award.

REFERENCES

Contec (2002). Contec is AAC. Technical Specification Notes on Autoclaved Aerated Concrete Products, Contec Inc. Texas, USA


Electrokinetic treatment for freezing and thawing damage mitigation within limestone

H.E. Cardenas, P. Paturi, & P. Dubasi
Louisiana Tech University, Ruston, Louisiana, USA

ABSTRACT: Advances in sustainability can be achieved by extending durability. Freeze and thaw damage is a major source of degradation in limestone structures that causes cracking and increased porosity. The increase in porosity tends to compromise durability further. Organic sealants used to curb the moisture intrusion are associated with ground level ozone production. In addition, the limited penetration of these compounds tends to cause freeze and thaw damage to occur deeper within the wall. This study investigated the application of reactive electrokinetic treatment to improve the strength and durability of limestone. The reactants in this case were sodium silicate and calcium hydroxide. Treatments were applied to vertical surfaces using novel re-circulating flow electrodes. A given treatment was applied for 12 days, during which sodium silicate was driven deeply into the limestone block from one electrode and calcium hydroxide from the other. The reactants met inside the stone and formed pore-blocking phases that increased the strength while reducing porosity. Freeze and thaw damage was induced within specimens in accordance with ASTM D 5312. The treatments prevented deterioration of porosity and strength. In other cases, the treatments appeared to reverse the effects of freeze and thaw damage. Some treatments yielded strength increases of 100% and higher.

1 INTRODUCTION

This work deals with the damage that develops in limestone due to freeze and thaw cycles in temperate climates. Upon freezing, pore fluid expands and induces tensile stress. This stress tends to cause cracking and changes in porosity that can accelerate deterioration. Organic coatings used to seal these surfaces tend to be high in volatile organic content (VOC). In addition, these surface treatments can permit water to reside more deeply in the wall where it can still freeze and cause damage. For a more sustainable remedy, this work examined the use of charged reactive agents that could fill these pores and cracks deeply in order to reduce porosity, heal microcracks, and increase strength. Reducing the porosity was critical since this was needed to inhibit the transport of moisture that leads to cracking. The treatment used in this work involved the application of an electric field. This field induced electrophoresis and ionic conduction that carried pore-blocking particles and ions throughout the pore structure of the limestone. Applying electrokinetic treatments to vertical surfaces required the development of a sponge electrode assembly that is shown in Figure 1.

2 BACKGROUND

Limestone is a porous material consisting primarily of calcium carbonate. One of the main reasons for its degradation is expansive freezing of absorbed moisture. On a visual basis, resultant cracking exhibits the
appearance of spalled sections and popouts. The interior sections can also suffer cracking induced by freezing water. This damage can increase porosity and degrade strength in the stone structure (Viklander and Eigenbrod, 2002). During a period of thaw, resident water can migrate to other parts of the pore system where it can refreeze. These processes can thus cause widespread damage throughout the material regions that are subject to freeze and thaw cycling.

Surface treatments are used to seal outer surfaces of masonry walls. These include polymeric as well as ceramic species that can penetrate the pores to some extent and even provide some strengthening of the outer layer of the stone (Brus and Kotlik, 1996 and Kumar and Ginell, 1997). Unfortunately, the depth of penetration of these surface treatments tends to be limited. Since no mechanically applied surface barrier is completely without defect, water can migrate past this layer through the process of capillary draw. Even in the case of a perfect coating, water can still occupy pores located on the interior side of the coating boundary but still close enough to the wall surface to permit freeze and thaw damage. For organic sealants, high VOC content creates disposal restrictions. In addition, the simple use of these products directly contributes to the production of ground level ozone which itself is associated with respiratory health risks.

A more sustainable manner in which porosity and permeability has been reduced in concrete is by electrokinetic agent transport (Otsuki and Ryu, 2001 and 2002). There are several transport phenomena through which the ions and particles can migrate inside a porous material such as stone, concrete, or soil. These include capillary adsorption, ionic conduction, liquid diffusion, electromigration, electrophoresis, and electroosmosis (Hearn, N., Hooton R.D, Mills, R.H., 1994). The electrokinetic transport of nanoparticles is largely influenced by electrophoresis, electroosmosis and electromigration (Cardenas, 2002). The motion of charged ions is termed electromigration (Wieczorek, S., Weigand, H., 2004). The motion of charged particles in the size range of colloids is termed electrophoresis (Budhu, M., Rutherford, M., 1997). The motion of fluid contained in pores when an electric field is applied is referred to as electroosmosis (Devasenathipathy, S., Santiago, J. G., 2003).

In prior work applied to concrete, a voltage potential was used to transport charged ions and suspended particles such as calcium and silicate ions and silica particles into the pores of hardened cement paste (Cardenas, 2002 and Cardenas and Struble, 2006). These ions and particles reacted with available calcium inside the concrete to form precipitates of calcium silicate hydrate (C-S-H), the primary binder in portland cement (Mindess et al., 1996, and Taylor, p.296, 1997). The precipitate (C-S-H) forms when calcium hydroxide is permitted to react with sodium silicate (Taylor H.F.W., 1997). This precipitate tends to block pores and cracks while increasing material strength. The compression strength and porosity was examined to characterize the restorative impact of a given treatment.

2 PROCEDURE

12.7 cm square cubes and 5.1-cm square cube specimens of Cordova Cream limestone were obtained from Texas Quarries, Cedar Park, TX. Freeze and thaw cycling was carried out according to ASTM D 5312. Mass loss was monitored during the thermocycling period. During the freezing cycle, each specimen was maintained inside individual containers that were partially filled with water. They were placed in a freezer and maintained at a temperature in the range of −18 to −20°C for 12 hours. Following this period the specimens were removed from the freezer in order to commence a thawing cycle. During the thaw portion of the cycle, the specimens were maintained at 32°C for a period of 8 hours. 10 freeze and thaw cycles were run for each specimen in accordance with ASTM requirements.

The specimens were electrokinetically treated using sodium silicate and saturated calcium hydroxide solution for 12 days. The sodium silicate was a 44 wt % product manufactured by Oxychem, Dallas, TX. The calcium hydroxide was a reagent grade material that was combined with deionized water to yield a saturated solution that exceeded 0.17 g/l of dissolved calcium hydroxide. Each stone was sandwiched between two sponge electrodes as shown Figure 1. The sponges were manufactured by Armaly Brands, Walled Lake, MI. The electrodes were manufactured by Corrpro. Calcium hydroxide solution was pumped to the top side of one sponge electrode and sodium silicate was pumped to the top of the other sponge electrode. The fluids were permitted to flow down through each electrode. A voltage of approximately 15–30 V was used to provide a treatment of a constant current density of 1 A per square meter. The electric field drew the reactive agents out of each sponge and into the stone. A return port located at the bottom of each sponge was used to capture the exiting flow and recirculate it back to the top. 4 liters of each treatment fluid was circulated over the 12-day treatment period using 800 liter per hour submersible pumps.

Several treatment cases were considered. Each case was replicated a total of three times for each of two specimen sizes, the 5.1-cm cube, and the 12.7-cm cube. The cases are summarized as follows.

Case F-E: Freeze and thaw cycling followed by 12-day electrokinetic treatment,

Case E-F: 12-day electrokinetic treatment followed by freeze and thaw cycling.
Compressive strengths were obtained for all specimens in accordance with ASTM C170. Components of fractured specimens were used for microstructural characterization using a Field Emission Scanning Electron Microscope. Energy Dispersive Spectroscopy was also conducted to assess chemical changes to microstructures. Samples for these post-treatment analyses were removed from the geometric center of each cube specimen.

For porosity testing, a 5-gram sample was acquired from each fractured specimen. These samples were immersed in lime water for approximately 20 days until the specimen mass was stable. Immersion was not required in cases involving electrokinetic treatment since these specimens were already saturated. The porosity was taken as mass of water lost (due to drying at 105°C) divided by the original mass of the specimen.

3 RESULTS

Figure 2 shows an image of the limestone microstructure that contains an apparent microcrack. This specimen was removed from the geometric center of a specimen that was subjected to 10 freeze and thaw cycles. The image shows a broad range of grain sizes. These cracks appeared to propagate in between the larger and smaller grains.

Figure 3 contains an energy dispersive spectroscopy (EDS) spectrum obtained from the specimen shown in Figure 2. This spectrum indicates the presence of carbon, oxygen and calcium. Trace amounts of sulfur and chlorine were also detected.

Figure 4 contains an SEM image of a limestone specimen that was subjected to electrokinetic treatment with sodium silicate and calcium hydroxide. This treatment was conducted over a 12-day period. The micrograph indicates a densified structure with no evidence of microcracks. This sample was also drawn from the geometric center of the treated specimen.

Figure 5 contains the EDS spectrum from the electrokinetically treated sample shown in Figure 4. This spectrum indicates the presence of carbon, oxygen, calcium, and silicon. This spectrum also indicates trace amounts of sulfur and chlorine.

Table 1 contains the compressive strength and porosity values obtained in this study. The undamaged and untreated control specimens exhibited an average strength of 1,300 psi and a porosity of 10%. For undamaged control specimens, electrokinetic treatment yielded values of 2,800 psi and 6%. When these specimens were subjected to 10 freeze and thaw cycles, the resultant strength was 880 psi and the porosity climbed to 15%. When electrokinetic treatment preceded freeze...
and thaw exposure, the strength achieved was 2,600 psi and the porosity was 7%. Applying the electrokinetic treatment after freeze and thaw exposure resulted in a lower strength value of 1,600 psi. This approach also produced and a higher porosity of 9%.

In addition to the cracks in the microstructure, freezing and thawing is generally expected to result in weight loss. Table 2 shows the % of weight loss that has occurred in the specimens in every case where they were subjected to freezing and thawing.

Several specimens were also subjected to chemical analysis to determine calcium carbonate content. Table 3 contains the results of this analysis conducted by the Bowser-Morner Laboratories of Dayton, OH. For the cases, F-E, E-F and E, the calcium carbonate content was less than compared to the other cases.

### Table 1. Strength and porosity results.

<table>
<thead>
<tr>
<th>Treatment Case</th>
<th>Compressive strength (psi)</th>
<th>Porosity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(F-E) Freeze and thaw exposure followed by electrokinetic treatment</td>
<td>1,600</td>
<td>9</td>
</tr>
<tr>
<td>(E-F) Electrokinetic treatment followed by freeze and thaw exposure</td>
<td>2,600</td>
<td>7</td>
</tr>
<tr>
<td>(F) Freeze and thaw exposure only</td>
<td>880</td>
<td>15</td>
</tr>
<tr>
<td>(E) Electrokinetic treatment only</td>
<td>2,800</td>
<td>6</td>
</tr>
<tr>
<td>(C) Control, no freeze and thaw and cycling and no treatment</td>
<td>1,300</td>
<td>10</td>
</tr>
</tbody>
</table>

### Table 2. Weight loss.

<table>
<thead>
<tr>
<th>Specimen size</th>
<th>Case F</th>
<th>Case F-E</th>
<th>Case E-F</th>
</tr>
</thead>
<tbody>
<tr>
<td>13 cm × 13 cm cube</td>
<td>1.81</td>
<td>0.55</td>
<td>0.12</td>
</tr>
<tr>
<td>5 cm × 5 cm cube</td>
<td>1.65</td>
<td>1.04</td>
<td>0.76</td>
</tr>
</tbody>
</table>

### Table 3. Calcium carbonate content.

<table>
<thead>
<tr>
<th>Treatment case</th>
<th>Calcium carbonate (Weight %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-E</td>
<td>96.58</td>
</tr>
<tr>
<td>E-F</td>
<td>95.99</td>
</tr>
<tr>
<td>F</td>
<td>99.20</td>
</tr>
<tr>
<td>E</td>
<td>95.90</td>
</tr>
<tr>
<td>C</td>
<td>99.11</td>
</tr>
</tbody>
</table>

### 4 DISCUSSION

When compared to the unexposed specimens (Table 1), the specimens exposed to freeze and thaw cycling exhibited a 32% drop in strength. At the same time these specimens exhibited a significant increase in porosity (from 10 to 15%). These changes suggest that damage in the form of microcracks (Figure 2) may have accumulated as a result of this exposure. Based on these observations it was found that the freeze and thaw cycling conducted in this study caused significant damage to the limestone.

Figure 6 contains a plot of strength versus apparent porosity as observed in all the damaged and undamaged cases considered in this study. In general, strength was expected to go down as the porosity of a material increased (Cardenas, H., 2002 and Salem, N., 2003). Such a trend was clearly observed among all the cases considered in this work. From this stand point the damage simply looks like an increase in porosity. In contrast, the distinction between porosity and micro-cracking are geometrically very different since we are comparing an infinitely sharp crack that could raise stress by a factor of 14 (or more) to a circular defect that would influence stress by only a factor of 3. Presumably the path of microcrack extension is expected to follow a relatively porous path since this would be a propagation path of least resistance. The most likely place for a crack to arrest is when the stress raiser at the tip drops below the stress intensity needed to propagate. This is most likely to occur at pores. The end result of microcrack propagation is then just a better connected pore network. It is conceivable that the damage accumulated from freeze and thaw cycling could simply proceed as the progressive inter-connection of existing regions of porosity that are undergoing tensile cracking as pore fluids expand. The classic porosity versus strength trend exhibited in Figure 6 appears to fit with this notion. It is thus conceivable that freeze and thaw damage may be viewed as a damage-related...
increase in porosity. When it comes time to heal damage, this distinction is very important because filling a circular pore with a restorative deposit is much easier to attain than it is to fill in the tip of an infinitely sharp crack. It seems unlikely that the radical increases in strength achieved in this work would be realized if large numbers of infinitely sharp cracks were in need of repair.

It is conceivable that damage repair from microcracking may largely manifest as a reduction in porosity. The pathways that allow access to water and that become interconnected through freeze and thaw damage may also provide passage for restorative particles (Figure 1). This behavior was observed in the undamaged porous path when the porosity of the undamaged specimens was found to decrease from 10% to 6% as a result of treatment. Similarly among the damaged specimens, post-treatment reduced the apparent porosity from 15% to 9%. In both the damaged and undamaged cases, the resultant strength increases were approximately 100%. Since the samples came from the middle interior of the specimens, these observations appeared to show that the original porous path as well as the newly created paths and microcracks could be penetrated deeply and effectively with these restorative treatments. This protection is more sustainable than could be achieved with a surface sealant since the depth of the surface seal can continue to permit sub-surface water to freeze.

Figure 7 contains a bar chart comparing the strengths and porosities achieved in various cases involving both controls and treated specimens. As mentioned earlier, the freeze and thaw damaged specimens (F) exhibited significantly lower strength compared to the undamaged controls (880 psi versus 1,300 psi) and higher porosity (15% versus 10%). When electrokinetic treatment was applied after freeze and thaw exposure (Case F-E), the damage-related porosity appeared to recover significantly from 15% back to 9% which was comparable to the original porosity of the limestone (10%). The resultant strength also recovered with a value of 1,600 psi that is 23% higher than the undamaged controls. Based on this work, it is concluded that the electrokinetic treatment not only restored but actually enhanced the properties of the damaged limestone, leaving it stronger and less porous than the pre-damaged state. This was achieved without the use of environmentally damaging organic sealants.

The case in which electrokinetic treatment preceded freeze and thaw exposure (Case E-F) was also examined. In this case the intent was to prevent damage by occupying the pores with phases that prohibited transport of moisture that could then help propagate damage. It was found that these E-F cases yielded strength values that exceeded the damaged material by a factor of 2.95 and the undamaged controls by a factor of 2. Case E-F yielded a porosity of 7% as compared to the 10% porosity of the control and the 15% porosity of the damaged stone. These changes are manifest in the SEM images when the damaged microstructure of Figure 2 is compared to the significantly densified microstructure of Figure 4. The EDS spectra presented in Figures 3 and 5 also indicate the presence of silicon in the densified cases that was not observed in the damaged cases. These observations appeared to indicate that the treatment

Figure 6. Relationship between altered porosity and strength.
significantly reduced the susceptibility to freeze and thaw damage by blocking the pores that enabled this degradation.

The E-case (no damage, only treatment) produced the lowest porosity and the highest strength of all cases considered. It is important to note that a given treatment was linearly oriented through the material. This means that the discontinuous branches of a porous network would not receive treatment because they would not be conducting much if any electric current. In contrast, freeze and thaw damage is not expected to be oriented in any particular direction. The higher porosity and lower strength of the restored (Case F-E) and pretreated (Case E-F) as compared to the treatment-only (Case E), suggests that the single-direction treatments may have bi-passed some pore space regions that were not well connected to the rest of the material. Less porosity means fewer possibilities of missing these regions. Future work will examine the best combination of treatment directions needed to maximize treatment effectiveness along these side corridors.

A key feature in this work is the large difference in performance observed between the pre-treated (E-F case) (2600 psi and 7% porosity) and the restored (F-E case) (1600 psi and 9% porosity). The preventative treatment engaged a much smaller pore volume (10%) as compared to the restorative treatment (15%). Based on previous discussion, this means that the pre-treatment will be less likely to bi-pass some pore space regions. Based on these observations it is concluded that a preventative treatment may be significantly more effective at reducing freeze and thaw damage as compared to a restorative treatment.

The key sustainability features of this work is the avoidance of VOC containing surface sealants that lead to the development of ground level ozone. The other key feature is the deep penetration of protective inorganic sealants that avoids the possibility of sub-surface freezing and thawing attack.

5 CONCLUSIONS

The freeze and thaw cycling conducted in this study caused significant damage to the limestone. This work showed that the original porous paths of the stone as well as the damage-related paths (formed by microcracks) could be penetrated effectively with the restorative electrokinetic treatment that enabled the reactants (calcium hydroxide and sodium silicate) to meet and react deep within the material. This provided a repair that is more sustainable than a simple surface treatment that would permit sub-surface water to continue to freeze and thaw. The electrokinetic treatment not only restored but actually enhanced the properties of the damaged limestone, leaving it stronger and less porous than the pre-damaged state. The treatments significantly reduced the susceptibility to freeze and thaw damage by blocking the pores that enabled this degradation to occur. This work also showed that freeze and thaw damage may be viewed as a damage-related increase in porosity. When it comes to healing damage, this distinction is very important because filling a circular pore with a restorative deposit is much easier to do than it is to fill in the tip of an infinitely sharp crack. It therefore seemed unlikely that the radical increases in strength achieved in this work could be realized if large numbers of infinitely sharp cracks were present. This work also showed that a preventative treatment may be significantly more effective at reducing freeze and thaw damage as compared to a restorative treatment. This may be due to the single-direction in which the treatments were oriented. This singular orientation may have bi-passed some pore space regions that were not well connected to the rest of the material. When there was relatively little porosity to engage, this meant that there were fewer possibilities of missing such regions. The key sustainability feature of this work is the avoidance of VOC-containing surface sealants that lead to the development of ground level ozone. The other key feature is the deep penetration of protective inorganic sealants that avoids the possibility of sub-surface freezing and thawing attack. Future work will examine the best combination of treatment directions needed to maximize treatment effectiveness along side corridors of the pore network.

REFERENCES


1 INTRODUCTION

The use of artificial pozzolana as a partial replacement with cement in mortar and concrete has been considered recently in order to produce durable, low-cost and sustainable mortar and concrete. Alternative cement blends containing waste materials as artificial pozzolana such as fly ash, silica fume, blast furnace slag and rice husk ash have the advantage of energy saving, cost reduction, recycling of the wastes and reduction of the carbon dioxide emission during cement production. Such cement blends are known to have promising results for the production of high performance concrete.

It is crucial to fully understand the hardening mechanism of cement-pozzolana blends since the presence of the pozzolana influences the progress of cement hydration and its compounds to some extent. The main hardening mechanism is a combined reaction composed of cement hydration and pozzolanic reaction, which is rather complicated as each reaction follow different reaction processes at different rates. Additional hydrated phases are formed upon pozzolanic reaction between the pozzolana and calcium hydroxide formed during the hydration of cement. The pozzolanic reaction, which proceeds much slower than the fast hydration of the cement, generally becomes effective between 3–14 days. This is mainly after 70–80% of the alite content in the cement has hydrated. This period can be shorter for the pozzolana with relatively high reactivity and specific surface area [Massazza 1998, Taylor 1990, Ogawa et al. 1980]. Hydration of alite and belite has been reported to be accelerated with the presence of pozzolana [Massazza 1998, Ogawa et al. 1980, Wu and Young 1984, Sharara et al. 1994]. This is due to the fact that fine pozzolana grains act as a precipitation site for additional hydrated phases and dissolution of alite and belite is accelerated as a result of the decrease in the calcium hydroxide content due to their consumption through the pozzolanic reaction [Ogawa et al. 1980].

Formation of additional hydrated phases by the pozzolanic reaction modifies the microstructure of the cement matrix and results in a pore size refinement effect causing a decrease in the large pores and increase in the fine pores [Mehta 1981, Mehta and Gjørv 1982, Chengzhi et al. 1996]. Strength increase of the
cement mortars and concrete blended with pozzolana is, therefore, correlated with the pore size refinement effect. Binary and ternary blends can also be used to produce low-cost and sustainable binders. While the former includes cement-lime binders, the latter refers to a ternary composition of cement, pozzolana and lime in varying ratios. Literature concerning the hardening reactions of these blends is rather limited. It has been reported that hydration of tricalcium aluminate was slightly retarded in the presence of additional lime and hexagonal hydro-aluminate crystals were formed [Collepardi et al. 1978]. Carbonation of the binary and ternary blends should be also considered due to the presence of lime from the beginning but this is generally neglected in the literature.

In this study, hardening of cement blends with pozzolana and lime has been studied with mortars using rice husk ash (RHA) as a highly reactive artificial pozzoana when produced under controlled burning of abundant agricultural waste material of rice husks. Its reactivity derives from its high content of amorphous silica (around 90%-wt) and high specific surface area governed by the porous structure of the ashes. Cement blends with RHA have proven promising results for the production of high performance concrete [Zhang et al. 1996, Bui 2001, Isia et al. 2003, Sousa Coutinho 2003, Nehdi et al. 2003, Bui et al. 2005, Rodriguez de Sensale 2006]. It is also used for producing cheap binders, such as RHA-lime and RHA-cement, for mortars and plasters in rural areas of developing countries where the portland cement is scarce and expensive for construction practices [Stroeven et al. 1999]. Considering these aspects, a research has been conducted in the framework of collaboration between the K.U.Leuven in Belgium and a local NGO (Mwanza Rural Housing Programme) in Tanzania in order to rationalize the use of cement with locally available RHA for the production of sustainable and low-cost binders. This study summarizes the results on the hardening of cement blends with RHA and lime in various compositions. The emphasis is given on understanding of the combined hardening reactions of these blended binders and their consequences on the long-term strength development and modification of the microstructure of the mortars.

2 RESEARCH PROGRAMME

2.1 Materials

Portland cement (CEM I 52.5 N), RHA and commercial lime hydrate were used for the production of supplementary cementitious binders for mortars. The lime hydrate is composed of 83.22% calcium hydroxide content with a Blaine specific surface area of 19,464 cm²/g and a mean value of its particle size distribution of 4.40 µm.

Table 1. Chemical composition of the RHA.

<table>
<thead>
<tr>
<th>Composition</th>
<th>(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SO₃</td>
<td>0.25</td>
</tr>
<tr>
<td>CaO</td>
<td>0.78</td>
</tr>
<tr>
<td>MgO</td>
<td>0.53</td>
</tr>
<tr>
<td>SiO₂</td>
<td>90.86</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>0.24</td>
</tr>
<tr>
<td>K₂O</td>
<td>1.13</td>
</tr>
<tr>
<td>Loss on ignition at 975°C</td>
<td>2.49</td>
</tr>
</tbody>
</table>

Figure 1. XRD pattern of the RHA.

2.2 Characteristics of the RHA

RHA was provided from Tanzania where the ash samples were obtained by burning rice husks in a local kiln under poorly controlled conditions. The RHA is composed of high contents of silica (90.86%-wt) and low contents of carbon (2.49%-wt) (Table 1). XRD analysis indicated the amorphous nature of the silica due to the broad band between 15–30° and certain amounts of cristobalite and tridymite as crystalline silica (Figure 1). The latter gives evidence to high burning temperatures (800–1000°C) leading to the crystallization of the amorphous silica. This is the result of the poorly controlled burning conditions in the kiln, creating locally high temperatures [Shinohara and Kohyama 2004].

Reactivity of the RHA with lime significantly depends on its specific surface area which is controlled by unburnt carbon particles that are very porous. Recent studies have reported a significant influence of grinding time on the specific surface area of the RHA [Bui 2001]. Therefore, RHA was ground to various degrees of fineness using a laboratory vibrating ring mill in order to determine the optimum grinding time for achieving adequate pozzolanic activity. Upon initial grinding until 10 minutes, the specific surface area increased as a result of a decrease in the particle diameter of the ashes (Figure 2). However, further grinding caused a decrease in the specific surface area of the RHA even though the fineness of the particles remained mostly the same. This decrease was reported...
to be due to the collapse of the porous structure of the ashes upon further grinding [Bui 2001]. Decrease in the specific surface area led to a decrease in the pozzolanic activity of the RHA (Figure 3). Considering these results, total batch of the RHA was ground by means of a laboratory batch ball mill until certain fineness and specific surface area (13,047 cm²/g) were reached. The ground RHA grains are mostly angular edged and spherical grains with varying particle sizes less than 50 µm.

Pozzolanic activity of the ground RHA was studied with RHA-lime paste prepared in 1:1 RHA-lime ratio by weight and 1.25 water/solid by weight. The paste was stored in closed plastic vessels and then hydrated for 28 days. Consumption of lime by pozzolanic reaction and formation of hydrated compounds were followed by thermogravimetric analysis (TG-DTG) at 3, 7, 14, 21 and 28 days of hydration. The results revealed that RHA is quite reactive since all available lime was consumed very rapidly within 21 days as a result of the pozzolanic reaction (Figs. 4, 5). Hydrated compound formed is calcium silicate hydrate (C-S-H) phase giving a dehydration peak at around 110°C. This phase can be already observed at 3 days, revealing that the pozzolanic reaction started just before.

2.3 Mortar compositions

Three types of mortars were prepared as RHA-cement mortars, RHA-cement-lime mortars and RHA-lime mortars (Table 2). The mortars were prepared using standard quartz sand with maximum grain size of 2 mm [EN 196-1]. Binder/sand ratio of these mortars was kept 1:3 by weight. Reference cement mortar and three types of RHA-cement mortars were studied for
concerning the latter, cement was replaced at 30%-wt, 50%-wt and 70%-wt with RHA (Table 2).

Two types of ternary blended mortars composed of RHA, cement and lime were prepared. The ratio of the cement was kept 10%-wt for both compositions with varying lime and RHA ratios (Table 2). Due to the higher porosity and specific surface area of the RHA and lime than the cement, they have higher water absorption. This led to the use of much more water for the blended mortars to reach similar workability with the RHA-cement mortars, resulting in different water/binder. For practical reasons in Tanzania, no superplasticizer was used to reduce the amount of water.

RHA-lime mortars were studied in 1:3 binder-sand ratio both by volume and by weight for the comparison of the data (Table 3).

### Table 2. Compositions of the cement mortars blended with RHA and lime.

<table>
<thead>
<tr>
<th>Mortar</th>
<th>RHA (%)</th>
<th>Cement (%)</th>
<th>Lime (%)</th>
<th>Water/binder (by weight)</th>
<th>Flow (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cref</td>
<td>–</td>
<td>100</td>
<td>–</td>
<td>0.48</td>
<td>144</td>
</tr>
<tr>
<td>RHA-C.3-7</td>
<td>30</td>
<td>70</td>
<td>–</td>
<td>0.60</td>
<td>150</td>
</tr>
<tr>
<td>RHA-C.5-5</td>
<td>50</td>
<td>50</td>
<td>–</td>
<td>0.65</td>
<td>146</td>
</tr>
<tr>
<td>RHA-C.7-3</td>
<td>70</td>
<td>30</td>
<td>–</td>
<td>0.70</td>
<td>147</td>
</tr>
<tr>
<td>RHA-C-L.7-1-2</td>
<td>70</td>
<td>10</td>
<td>20</td>
<td>0.80</td>
<td>151</td>
</tr>
<tr>
<td>RHA-C-L.5-1-4</td>
<td>50</td>
<td>10</td>
<td>40</td>
<td>0.82</td>
<td>148</td>
</tr>
</tbody>
</table>


### Table 3. Compositions of the RHA-lime mortars.

<table>
<thead>
<tr>
<th>Mortar</th>
<th>Lime (g)</th>
<th>RHA (g)</th>
<th>Sand (g)</th>
<th>Binder/sand (by volume)</th>
<th>Binder/sand (by weight)</th>
<th>Water/binder (by weight)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RHA-Lwt</td>
<td>225</td>
<td>225</td>
<td>1350</td>
<td>–</td>
<td>1:3</td>
<td>0.80</td>
</tr>
<tr>
<td>RHA-Lvol</td>
<td>87.5</td>
<td>87.5</td>
<td>1350</td>
<td>1:3</td>
<td>1:7.7</td>
<td>1.25</td>
</tr>
</tbody>
</table>

the comparison of the data. Concerning the latter, cement was replaced at 30%-wt, 50%-wt and 70%-wt with RHA (Table 2).

Two types of ternary blended mortars composed of RHA, cement and lime were prepared. The ratio of the cement was kept 10%-wt for both compositions with varying lime and RHA ratios (Table 2). Due to the higher porosity and specific surface area of the RHA and lime than the cement, they have higher water absorption. This led to the use of much more water for the blended mortars to reach similar workability with the RHA-cement mortars, resulting in different water/binder. For practical reasons in Tanzania, no superplasticizer was used to reduce the amount of water.

RHA-lime mortars were studied in 1:3 binder-sand ratio both by volume and by weight for the comparison of the data (Table 3).

2.4 Methods

Hardening of the mortars was studied using standard mortar beams (40 × 40 × 160 mm) which were prepared in accordance with the European standards [EN 196-1]. Mortars were cured at the standard laboratory conditions (20°C, 60% R.H.) for 360 days. The progress of hardening was studied by means of mechanical strength test and thermal analysis. Pore size distribution and modification of the microstructure was studied using respectively Mercury Intrusion Porosimetry (Micromeritics AutoPore IV) and Philips XL 30S FG Scanning Electron Microscope (SEM) coupled with X-Ray Energy Dispersive System. Mechanical strength tests were carried out by three-point bending test using the standard mortar beams at 7, 28, 60, 90, 120, 180 and 360 days of hardening [EN 196-1]. Thermal analysis was performed on finely ground samples after they were vacuum dried at 0.025 mbar. The analysis was carried out using a Netzsch STA 409 PC DSC-TGA system in static nitrogen atmosphere at a temperature range between 20-1000°C with a controlled heating rate 10°C/min. SEM analysis was carried out on the fresh fractured surfaces coated with gold after the samples were dried at 40°C.

3 RESULTS

3.1 Thermal analysis

Progress of hardening of the mortars was followed by thermal analysis using the first derivative of the thermogravimetry (DTG). Results for the Cref, RHA-C.3-7 and RHA-C.5-5 mortars are given in Figure 6 and those for RHA-cement-lime mortars are given in Figure 7. Peak at around 110°C is due to the dehydration of the C-S-H phase and at around 160°C due to the dehydration of the AFm phase. The weight loss at around 450°C indicates the decomposition of calcium hydroxide. Weight loss recorded between 500°C and 800°C derives from the decomposition of the carbonated phases. Two decomposition peaks appeared at around 600°C and 700°C for the blended mortars. Partial replacement of cement with RHA led to a decrease in the calcium hydroxide content and an increase in the carbonated phases. Further decrease in the cement content resulted
in a decrease in the hydrated phases. In the RHA-cement-lime mortars and RHA-lime, carbonated phases are much more pronounced than the hydrated phases (Figs. 7, 8).

### 3.2 Mechanical strength

Progress of flexural strength of reference cement mortar and RHA-cement mortars is given in Figure 9. While the reference cement mortar yielded the highest flexural strength values at all stages, these values became lower as cement was partially replaced with RHA at 30%, 50% and 70%-wt respectively. RHA-C.3-7 and RHA-C.5-5 mortars yielded an increasing flexural strength development until 360 days whereas no strength increase was recorded for the RHA-C.7-3 mortar after 120 days. Early strength development of the RHA-C.3-7 mortar between 7 and 28 days was...
much more pronounced than that of the RHA-C.5-5 and RHA-C.7-3 mortars having lower cement content than the RHA-C.3-7 mortar.

RHA-cement-lime mortars yielded relatively lower flexural strength values than those of the RHA-cement mortars (Figure 10). Reduction in the flexural strength was recorded at the very early stage after 28 days of hardening. Flexural strength of RHA-C-L.5-1-4 mortar decreased between 28 and 60 days but after that it stayed almost at the same value until 120 days followed by a decrease at 180 days. Strength reduction was much more pronounced for RHA-C-L.7-1-2 mortar between 28 and 360 days even though its initial strength was higher than that of the other. The decrease in the flexural strength is recorded to be from 0.68 N/mm² to 0.21 N/mm².

3.3 **Pore size distribution**

Pore size distribution of the blended cement mortars was determined at 90 days of hardening (Figure 12). RHA-cement mortars had higher volume of smaller and larger pores when compared with the reference cement mortar. RHA-cement-lime mortars containing only 10%-wt cement yielded the highest porosity with their higher volume of large and small pores than the rest.

3.4 **Microstructure of the mortars**

Use of RHA and lime together with cement led to modification in the microstructure of the mortars. The microstructure of the reference cement mortar was composed of needle-like and reticular C-S-H phase, and calcium hydroxide crystals formed inside the pores and in the cement matrix (Figure 13). RHA-cement mortars were characterized by the microstructure with flocs-like and fibrous-like C-S-H phase that were well connected to each other (Figure 14). Needle-like C-S-H
phase were not observed that much in the matrix of these mortars.

In the matrix of the RHA-cement mortars, plate-like, semi-circle and spherical RHA particles remained substantially unreacted or partially reacted. In some cases, shrinkage cracks were observed between the RHA particles and cement matrix, which may have occurred during the chemical shrinkage caused by the cement hydration (Figure 15). It is evident that these RHA grains acted as a surface for the precipitation of additional C-S-H phase through pozzolanic reaction with calcium hydroxide formed during the cement hydration (Figure 16). When totally reacted, flocs-like C-S-H phase was formed, which played a bridging role between hydrated and unhydrated cement compounds by growing from the grain surface towards the surrounding matrix (Figure 17).

Figure 13. SEM micrograph of needle-like and reticular C-S-H phases formed inside the pore of the Cref mortar at 120 days.

Figure 14. SEM micrograph of flocs-like C-S-H phases formed in the matrix of RHA-C.5-5 mortar at 120 days.

Figure 15. SEM micrograph of shrinkage cracks observed between the RHA particle and cement matrix of the RHA-C.3-7 mortar at 120 days.

Figure 16. SEM micrograph showing the precipitation of the C-S-H phases on the surface of a RHA grain.

Figure 17. SEM micrograph showing a totally reacted RHA grain, forming flocs-like amorphous C-S-H connected to the surrounding matrix.
RHA-cement-lime mortars exhibited a totally different microstructure when compared with the RHA-cement mortars. The early-stage matrix was mostly characterized by amorphous C-S-H phase with large pores in-between (Figure 18) while this microstructure was modified upon hardening. The matrix at 60 and 120 days was composed of silica-rich and carbonated dense phases which agglomerated with rather large pores in-between (Figure 19). Clusters of needle-like crystals and hexagonal plate-like crystals were observed in the matrix of these mortars at all stages of hardening, mostly at 120 days (Figure 20).

Microstructure of the RHA-L mortars revealed some similarities with that of the RHA-cement-lime mortars with the morphology of the carbonated phases (Figure 21). However, needle-like crystals and hexagonal plate-like crystals were not observed that much.

4 DISCUSSION

Blended binders composed of RHA-cement, RHA-cement-lime and RHA-lime revealed different hardening reactions, strength development and microstructure. Thermal analysis results indicated that while hardening of the reference cement mortar was governed by the cement hydration, that of RHA-cement mortars occurred as a result of early-stage cement hydration combined with pozzolanic reaction and carbonation at later stage (Figure 6). Decrease in the calcium hydroxide content in the RHA-cement mortars upon
hardening gives evidence to its consumption by the pozzolanic reaction and carbonation. The carbonation reaction is much more pronounced with decreasing cement content and increasing porosity which favours the diffusion of the carbon dioxide into the core where calcium hydroxide is still present to carbonate [Van Balen and Van Gemert 1994]. The reaction started to appear much more after 60 days for RHA-C.3-7 and after 28 days for RHA-C.5-5 and RHA-C.7-3. It is not that clear exactly when the pozzolanic reaction has started but it has probably started before 7 days and contributed to the formation of hydrated phases upon hardening as RHA is quite reactive. Unreacted or partially reacted RHA grains could be detected in the matrix of the RHA-cement mortars even at the age of 120 days, indicating the ongoing pozzolanic reaction (Figure 16). The mechanism of the pozzolanic reaction could be estimated as well. The reaction started on the surface of the RHA grains, leading to the precipitation of amorphous C-S-H phase. When totally reacted, they became flocs-like C-S-H phase playing a bridging role between hydrated and unhydrated cement particles by growing from the grain surface towards the surrounding cement matrix (Figure 17). This could help improving the microstructure at the interfacial transition zone between sand and the cement matrix.

RHA-cement mortars reached high strength values at 7 days, confirming that early-stage strength gain was mainly controlled by the rapid hydration of cement as well as pozzolanic reaction. Contribution of the pozzolanic reaction and carbonation to the strength development of these mortars is also evident as the strength values increased gradually until 360 days while no considerable increase was recorded in the strength of the reference cement mortar after 28 days (Figure 9). RHA-C.3-7 and RHA-C.5-5 mortars yielded an increasing long-term strength development whereas no strength increase was recorded for the RHA-C.7-3 mortar after 120 days as it contained lower amounts of cement (30%-wt) than the other blended mortars. All the RHA-cement mortars indicated a long-term strength development even though their strength values were lower than that of the reference cement mortar. Strength values became lower as the cement was partially replaced by RHA respectively at 30%-wt, 50%-wt and 70%-wt. This was ascribed to their higher porosity than the reference cement mortar, which derived from high W/B of the mortars due to the high water demand of porous RHA particles. This resulted in an increase in the volume of large pores (>0.1µm) (Figure 12). Increase in the volume of small pores (<0.1µm) was also recorded due to the formation of additional C-S-H phases by the pozzolanic reaction. This can be verified with SEM micrographs as the matrix of the RHA-cement mortars was mostly composed of well-connected flocs-like C-S-H phase with much more small pores in-between while that of the reference cement mortar was characterized by large pores containing needle-like and reticular C-S-H phases (Figs. 13,14).

RHA-cement-lime mortars revealed totally different hardening mechanism and relatively lower strength values when compared with those of the RHA-cement mortars. Strength reduction was recorded at the very early stage of 28 days and continued until 360 days of hardening (Figure 10). This reduction was also reported in the literature for the same type of mortar composition after 28 weeks [Stroeven et al 1999] and for ternary grouts containing 10%-wt cement after 60 days or more [Van Rickstal et al. 2003]. As the cement content was relatively low (10%-wt) in the composition of the RHA-cement-lime mortars, its hydration would be at the very early stage together with the pozzolanic reaction, and carbonation would be expected to be much more effective at later stage. This has been verified by thermal analysis results revealing that hydration and pozzolanic reaction were effective until 28 days when the highest strength values were recorded. Carbonation was much more pronounced at 60 days when strength reduction was recorded. This could indicate that calcium carbonate phases formed afterwards could have a destructive effect on the initially hydrated cement phases. In addition, decalcification of the C-S-H phase is possible due to the slight decrease in the intensity of the C-S-H peak and increase in the intensity of the peak of carbonated phases at 60 days. The overall effect of this would be to weaken rather than to strengthen the existing C-S-H phase. Dry curing conditions (60% R.H.) should be considered for the progress of the hardening reactions as it is more ideal for the carbonation rather than for the cement hydration and pozzolanic reaction.

Strength reduction of the RHA-cement-lime mortars can be related with their microstructure that has modified upon hardening. Amorphous C-S-H phase can be observed at the 28 days (Figure 18) while the matrix is quite poor at 60 and 120 days with the silica-rich carbonated dense phases that were agglomerated with rather large pores in-between (Figs. 19,20). This can strengthen the finding on the decalcification of the initially hydrated phases, which was determined by thermal analysis. This verifies that the carbonation has negative consequences on the microstructure of the initially hydrated phases, leading to a decrease in the strength of the mortar. In agreement with the earlier research, it can be concluded that 10%-wt cement was not sufficient for the long-term strength development of these ternary blended mortars [Van Rickstal et al. 2003].

Concerning the results on the RHA-lime mortars, hardening occurs as a result of rapid pozzolanic reaction within 28 days and carbonation mostly after that period (Figure 8). The rapid consumption of lime by highly reactive RHA resulted in the formation of C-S-H phase. Strength reduction recorded after 60 and 90 days of hardening can be explained with the phenomenon.
given for the strength reduction of the RHA-cement-lime mortars. This decrease in both mortar compositions can also be related with the presence of excess amount of the unreacted RHA in the matrix as all of the lime was consumed mostly by highly reactive RHA and carbonation reaction. It is reported in the literature that when all lime is consumed, the excess silica can react with the initially formed C-S-H phase, which will lead to the formation of highly polymerized C-S-H phase [Wu and Young 1984]. The overall effect of this was reported to be a decrease in the Ca/Si (~1.0) of the existing C-S-H. In this case, it seems that 1:1 RHA:lime is not suitable for the binder composition of the lime mortar. Decrease in the RHA amount and increase in the lime content can help in maintaining the C-S-H structure, promoting the carbonation reaction and contributing the long-term strength development of the lime mortar.

5 CONCLUSIONS

Hardening of cement in combination with RHA and lime occurs as a result of combined cement hydration, pozzolanic reaction and carbonation. Initial strength development of the RHA-cement mortars was mostly governed by the cement hydration while pozzolanic reaction and carbonation contributed to the long-term strength development. Carbonation was favoured at the later stage with the increase in the porosity of the mortars due to high W/B. High porosity caused relatively lower strength values when compared to the reference cement mortar. Flexural strength of each RHA-cement mortar increased gradually, indicating their long-term strength development. However, strength reduction was recorded at the very early stage for the RHA-cement-lime mortars and RHA lime mortars. This was explained with the insufficient cement content in the RHA-cement-lime mortars, destructive effect of the calcium carbonate phases on the initially hydrated phases and partial carbonation of the hydrated phases. This decrease can also be related with the presence of excess amount of the unreacted RHA in the matrix as excess silica can react with the initially formed C-S-H phase, which can lead to the formation of highly polymerized C-S-H phase. For such reasons, mortar compositions and hardening reactions can be important for strength development when producing cement blends using pozzolana and lime. Rural construction in Tanzania can be benefited by recycling the rice husk waste with its use in RHA-cement mortars with given compositions. However, 10%-wt cement in the composition of the ternary blended mortars is not sufficient for long-term strength development of the mortars. The lowest cement content leading to long-term strength development for the ternary blended mortars will be further investigated.

ACKNOWLEDGEMENT

This study is the part of an ongoing Research Project (OT/04 3E030765) conducted at the Department of Civil Engineering and a Master thesis research “Performance Evaluation of Pozzolanic Rice Husk Ash Binder in Tanzania” by Jan Campforts at the K.U.Leuven. The research grant offered to Ozlem Cizer by the OT/04 project funded by the K.U.Leuven is gratefully acknowledged.

REFERENCES

Biostore: towards sustainable composite waste reuse

B.K.C. Chan & A.W.L. Dudeney

Imperial College, London, United Kingdom

ABSTRACT: The paper describes the construction and monitoring of pilot scale composite emplacements at two sites in Yorkshire, in which designed combinations of coarse mineral (limestone, coal shales or construction & demolition waste) were compacted together with fines (stabilised sewage sludge with fly ash and lime or sodium hydrogen sulphate) within lined systems. Results after 18–24 months indicated that the composites were mechanically and microbiologically stable, and effectively isolated from the surrounding environment. The emplacements – termed Biostore – showed the potential for sustainable waste management by permanent co-utilisation of organic and mineral wastes in civil engineering groundwork for brown field site restoration to amenities or the light built environment.

1 INTRODUCTION

Large masses of waste bulk mineral are generated each year in the UK, e.g., some $8 \times 10^6$ tonnes coal spoil produced in 2003 [DEFRA 2004], $94 \times 10^6$ tonnes demolition rubble in 2001 [DEFRA 2003] and $5.1 \times 10^6$ tonnes pulverised fuel ash (fly ash) in 1997 [EEA 2002]. Most is stockpiled or landfilled in the vicinity of densely populated industrialised areas, thus adding to historic deposition. Similarly large quantities of bulk biodegradable waste are produced in the community, much of which has no market and represents an environmental risk, e.g., through adding to the burden of carbon dioxide in the atmosphere. Thus, $1.1 \times 10^6$ tonnes [DEFRA 2002] and $5 \times 10^6$ tonnes [DETR 2000] pa sewage sludge (dry solid) and green waste respectively, are typically surplus to requirements. Much mineral and organic waste is either associated with, or generated near to, brown field land – which, despite much recent development, still accounts for up to 1% of the total land area [UK Parliamentary Office of Science & Technology 1998].

Owing to the large volumes, low intrinsic values, variable compositions and negative environmental and social impact of most bulk wastes, special planning constraints are increasingly imposed, particularly regarding landfilling, lagooning, spreading and incineration [Commission of the European Communities 2000 & 1991]. For instance, the European Landfill Directive (1999/31/EC) [Commission of the European Communities 1999], and amendments which came into force on 16 July 2004, requires all waste going to landfill to be pre-treated, sets substantive targets for reducing municipal disposal and ends the practice of indiscriminate co-disposal. As a result, disposal costs will continue to increase. These will fuel renewed research into component recycling or regulated engineering application in the environment. The restoration and regeneration of brown field land, which represents a key plank of government policy on sustainable communities, is likely to be prominent in this activity.

Biostore is a new approach to the restoration of contaminated land, particularly former colliery and demolition sites, through incorporation of un-marketable bulk materials in engineering groundwork. It integrates two main technical objectives: permanent utilisation of selected wastes (both ‘in-situ’ and ‘pipe-end’) in processed co-products and provision of novel composite substrata for emplacement beneath surface foundations before site development to open amenities and light built environments. This new technique is based on an earlier idea to restore coal tips using artificial soil formulations at the previously restored Reden tip (Saarland, Germany) [Neu 1997]. It described the dimensions, composition and construction of these emplacements during 1993–4: 2 m depth over several ha of mixed – 20 cm coarse shale (80%), – 5 cm wood chips (10%) and weathered sewage sludge (10%) constructed by layering and compaction with a heavy vibratory roller. Some six years later, the emplacements were found to be largely unaltered, although vegetated. Thus, beneath a thin oxidized surface layer, the main mass was moist, anaerobic, low in permeability and mechanically hard. In addition to unaltered shale, the material clearly still contained wood and...
other organic detritus (which had been squeezed in between the shale particles). It was evident that much of the original carbon remained in place. Such an observation should not be unexpected as lignite, peat and other organic residues remain stable indefinitely under oxygen-free conditions [Andriesse 1988]. The emplacements were mechanically much harder than would be required for foundations in light building construction and were apparently continuous over large areas, without obvious discontinuities or cracks. This gave rise to the Biostore concept outlined below.

The central idea is that inter-particle voids (normally 25–50 vol%) within a packed coarse mineral mass of narrow particle size range can be filled with stabilised organic and mineral fines, mixed and compacted within a lining (isolated from the outside environment), to form mechanically competent, inert, and largely impermeable ground distinct from conventional landfill. The resulting uniform and stable stratum is analogous to an undisturbed geological formation, but with the added security of engineered drainage. It should provide a competent base on which to place soil, foundations and buildings. Although the literature of research on bulk mixing of organics with mineral waste as soils for surface restoration are well established [Poon & Boost 1996] [Lim et al. 2002], integration of Biostore procedures for permanent utilisation and isolation of organic/mineral emplacements in engineering foundations is novel.

In a preceding fundamental study [Tyrologou 2005], laboratory experiments were carried out on compacted cylindrical composite specimens (50 × 20 mm diameter) typically containing 60 vol% coarse matrix (75 g of quartz in the size range +5 – 10 mm) mixed with 40 vol% sludge phase (25 g mixture containing digested sewage sludge (15–25% solids), fly ash and lime in the mass ratio 17:5:3, respectively). The specimens exhibited unconfined compressive strength (UCS) in the range 220–250 kPa, similar to stiff soil or soft rock, after 28 days of curing. They showed hydraulic conductivity and porosity of 0.5–5 × 10⁻⁷ m/s and 11–14%, respectively. The values of UCS were satisfactory for use of these materials in engineering foundations while the hydraulic conductivity indicated the need for lining systems to protect the environment. Figure 1 shows ettringite growth on fly ash spheres believed to be partly responsible for the observed strength development.

2 METHODOLOGY

Two site emplacements were employed as below: small scale (~1 tonne) ‘pipe’ field testing of selected compacts at the Bradley site of Yorkshire Water Services; larger scale (~100 tonnes) ‘area’ emplacement with lining containment at Skelton Grange Landfill (Biffa Waste Management).

2.1 ‘Pipe’ emplacement

The ‘pipe’ emplacements modelled the behaviour of cores within possible full-scale formulations of Biostore. They employed four vertical uPVC water pipes of 2–3 m length and 0.45 m id, waterproofed and immobilised with silicon sealant and concrete (Figure 2). The pipes were provided with tap connections at the base for leachate collection and at 150 cm from the base for temperature and pressure measurement. Each pipe had a 30 cm deep composite drainage
and bottom-lining layer, consisting of gravel (~10 cm) in between two layers of clay (each ~10 cm). The clay was thoroughly mixed with water to achieve a compactable mass (approximately 10% water). The coarse matrix (50–70% by volume) was 2 or 10 cm limestone or burnt coal shale (Table 1). The fine matrix was: Pipe 1, digested sewage sludge (DSS, 280 kg, 15% dry solids) and sodium hydrogen sulphate (20 kg) (w/w ratio DSS:NaHSO4, 1:0.1, at pH 3); Pipes 2 and 3, DSS and treated conditioned sewage sludge (TCSS, 25% dry solids), respectively, (75L) plus UKQAA/Drax fly ash (17L) and Lhoist quicklime (7 L) (vol/vol ratio DSS or TCSS:fly ash:CaO approximately 1.0:0.2:0.1 at pH 10–12); and Pipe 4, TCSS, (neutral control). The fine matrix was prepared on site by coning and quartering together the calculated quantities of the requisite materials. The mixture required for Pipe 1 evolved hydrogen sulphide gas in small quantity and produced a cool relatively plastic sludge. In the cases of Pipes 2 and 3, heating occurred with evolution of steam and ammonia to leave a grainy sludge. The cooled mixture was emplaced with suitable quantities of aggregate and compacted with the aid of a 3 m length (5 x 10 cm cross section) of timber. Two separate methods were tested using successive layers of fine matrix and coarse aggregate. In Method (A) – used in Pipe 1: the Biostore was emplaced with burnt shale by embedding the shale in 30 cm thickness of sludge and compacting with the aid of the timber. In Method (B) – used in Pipes 2, 3 and 4: thin layers of aggregate (approximately 10 cm depth) were compacted with thin layers (10 cm) of sludge. Water was added to all the pipes to achieve designed degrees of saturation. Pipes 1 and 2 were partly saturated (25 L and 62 L water added) whereas Pipes 3 and 4 were fully saturated (100 L water added to each pipe). Three pipes (Pipes 2, 3 and 4) were sealed at the top with a 10 cm layer of packed clay. The Biostore in all pipes was covered with a 5 cm head of water and a loose polythene sheet (to limit evaporation and contamination). Straw bales were used for insulation during winter time. A pressure probe and thermocouple were provided for insertion into the sampling points as required. Leachate, where present, was collected from the bottom tap of each pipe for on-site analyses (for dissolved oxygen, pH, temperature and conductivity). Samples were acidified with nitric acid and stored in 4°C for standard laboratory analyses, including ICP (Inductive Coupled Plasma spectrometry) for metals concentration, Hach UV/vis spectrophotometry for ammonia and Shimadzu Total Organic Carbon Analyzer (Model TOC-5050) for total organic carbon. After one year, a 320 kg load (approximately 2 tonnes/m²) was applied to the top of the clay cap in Pipe 2 to observe any consolidation of the Biostore. Pipe 1 was cut (1.35 m from the top) by drilling and sawing to expose the Biostore for examination and sampling (Figs 2–3).

2.2 ‘Area’ emplacement

The design concept was based on Pipe 3 with a coarse-fine ratio of 1:1 using YWS composted sewage. Demolition rubble (screened to remove ~10 cm material), fly ash, gravel, clay and water (available on site) were employed together with composted sludge and Lhoist lime; and emplaced with the aid of 20 tonne Biffa machinery (CAT320C) to form the structure illustrated in Figure 4. Thus, the structure was an inverted square pyramid some 36 m² in

<table>
<thead>
<tr>
<th>Table 1. Masses of materials (kg) used in ‘pipe’ emplacements.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Materials</strong></td>
</tr>
<tr>
<td>Limestone 0.5–2 cm</td>
</tr>
<tr>
<td>Limestone 5–10 cm</td>
</tr>
<tr>
<td>Burned shale 5–20 cm</td>
</tr>
<tr>
<td>TCSS mixture</td>
</tr>
<tr>
<td>TCSS compacted</td>
</tr>
<tr>
<td>Fly ash</td>
</tr>
<tr>
<td>CaO</td>
</tr>
<tr>
<td>DSS mixture</td>
</tr>
<tr>
<td>DSS compacted</td>
</tr>
<tr>
<td>Fly ash</td>
</tr>
<tr>
<td>CaO</td>
</tr>
<tr>
<td>NaHSO4</td>
</tr>
<tr>
<td>Total biostore mass</td>
</tr>
<tr>
<td>Clay &amp; gravel layers</td>
</tr>
<tr>
<td>Water added</td>
</tr>
<tr>
<td>Weight of pipe</td>
</tr>
<tr>
<td>Total weight</td>
</tr>
</tbody>
</table>
area at the surface and containing Biostore to a maximum depth of 3 m, approximately the minimum dimensions required to achieve results relevant to practical ground surface. It adopted the materials and proportions derived from the pipe systems and, with the exception of Yorkshire Water treated conditioned sewage sludge, Lhoist lime and Wyoming bentonite, was constructed entirely with materials and equipment provided by Biffa Waste Management, which were already on site.

The structure was set in the mixed fly ash/spoil horizon typical of that part of the former Oxbow Colliery, later used as a fly ash lagoon for the Skelton Grange power station (also now closed and dismantled). It was therefore subject in principle to movement as a whole, a fact taken into account by surveying to blocks within the structure to assess Biostore stability rather than to external reference points. Ideally the structure should have a conventional profile (slopes <1:2) for security and be much

larger in scale, e.g. 1 ha area and 6 m depth, so that consistent emplacement and compaction could be employed and surface development with vehicular access, light building construction and open amenity satisfactorily demonstrated. The generally small size and steep slopes of the actual emplacement were constrained by economic and temporal considerations, and gave rise to the novel construction method employed, aimed at providing at maximum surface area and Biostore depth, together with an imposed surface load, within funding limits.

Materials used are listed in Table 2. An initial excavation of $14 \times 14$ m² area and 3 m depth (nominal slope 1:2) was made within the fly ash/spoil horizon near the northern perimeter of the site, where landfill would not commence for at least 2 years. A second (rectangular box) excavation of dimension $3 \times 3 \times 1.5$ m depth was made in the base of the first excavation and filled with clay, which was compacted. A $2 \times 2 \times 1$ m scraping was taken from the centre of the clay mass. This scraping was filled with gravel (0.5 m) and then clay (0.5 m), both of which were compacted in place. A heavy-walled plastic water pipe (5.5 m length and 0.3 m id), with its lower 0.3 m liberally perforated was set upright in the centre of the gravel (with a concrete block placed immediately beneath the pipe for surveying purposes) before adding the top layer of clay, and supported in place. The pipe was sealed with bentonite against the clay in the upper layer. The sides of the emplacement were formed by successive clay bunds (approximately 0.5–1 m width and 30 cm depth) built upward and outwards to give the required slope. The inner volume created by each 30 cm bund was filled with Biostore while surrounding spoil was drawn level with the bund. The combined surface area was formed as a unit. Two additional pipes (2 and 4.5 m in length) were set on concrete blocks at the top and bottom of the Biostore, respectively. The sludge fraction was a

Table 2. Materials used in ‘area’ emplacement.

<table>
<thead>
<tr>
<th>Material</th>
<th>Volume m³</th>
<th>Quantities tonne</th>
</tr>
</thead>
<tbody>
<tr>
<td>BOTTOM LAYERS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Drainage layer</td>
<td>13</td>
<td>3</td>
</tr>
<tr>
<td>Clay layer</td>
<td>26</td>
<td></td>
</tr>
<tr>
<td>BIOSTORE LAYERS</td>
<td>52</td>
<td></td>
</tr>
<tr>
<td>Fly ash</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>Quicklime</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>TCSS</td>
<td>27</td>
<td></td>
</tr>
<tr>
<td>Construction &amp; demolition rubble</td>
<td>62</td>
<td></td>
</tr>
<tr>
<td>SIDES</td>
<td>27</td>
<td>54</td>
</tr>
<tr>
<td>Clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top layers (capping)</td>
<td>24</td>
<td>48</td>
</tr>
<tr>
<td>Clay and spoil</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3. Interior of cut-through Pipe 1 after test.

Figure 4. Diagrammatic cross-section through the Skelton Grange emplacement.

626
mixture of TCSS, fly ash and lime in the ratio mentioned above, mixed by machinery on site. Heat and steam were evolved during mixing. For emplacement, a layer of about 30 cm sludge was covered by a layer of 30 cm C&D material. The sludge was wetted from a bowser to increase its plasticity and compacted with the aid of the excavator.

Nominally 10 layers were so prepared and compacted, and the final surface was similarly covered with clay (0.5 m) and spoil (0.5 m). A 5 tonne load (C&D material plus a large rock), equivalent to approximately 2 tonnes/m² (the same as in the Bradley pipe trial), was applied after three months. Regular monitoring (monthly) was carried out to check for any gas emanation and collect leachate for analysis. Any relative movement either vertical or horizontal of the blocks beneath the three pipes was measured by standard surveying techniques (spirit levelling and electronic distance measurement). This ‘area’ emplacement was regulated by means of an Environment Agency waste management licence.

3 RESULTS AND DISCUSSION

3.1 ‘Pipe’ emplacement

The pipe system employed proved to be a suitably space-saving, protected and inexpensive (though labour intensive) means of screening different environmentally isolated conditions relevant to Biostore. Thus, the four pipes, weighing a total of less than four tonnes and occupying no more than 10–15 m² floor area, effectively modelled different cylindrical cores of depth 2–3 m through clay-lined field emplacements, which would in reality require at least 5000 tonnes per ha. The pipes were structurally stable and leak-proof, while facilitating assessment of consolidation, internal T and P, overall permeability, and liquid or gaseous products. The system enabled preliminary comparative testing of a number of emplacement variables. Thus, two aggregate/sludge ratios (3:1 and 1:1), two methods of material packing (A and B), three pH regimes (3–5, 6–8 and 9–10), two levels of water saturation (saturated and unsaturated), three aggregate particle size ranges (0.5–2, 5–10 and 5–20 cm), two types of coarse aggregate (burnt shale and limestone) and two types of sludge (DSS and TCSS) were considered. Because of this wide range of variables, the results were not intended to have quantitative statistical significance. The system also had inherent drawbacks tending to reduce quantitative reliability, particularly pipe surface/edge effects resulting from the limited diameter of the pipes and temperature variation (in the range 4–15°C) greater than expected in an underground emplacement (10–12°C). However, as will be seen, the results nonetheless provided sufficient information for larger scale design.

The contents of Pipe 1 (Fig. 2, Table 1) were impermeable: no leachate was obtained during the first year, even though a clay lining was not used between the standing head of water and the Biostore. This indicated that Method A (burying coarse aggregate in a thick layer of sludge) obviated all effective liquid pathways, both within the Biostore and between the Biostore and internal pipe surfaces. A relatively large sludge-aggregate volume ratio (1:1) was necessary for this method – and was actually an advantage in that it would utilise the maximum proportion of sludge without leading to significant consolidation. The Biostore in Pipe 1 was designed to be acidic to prevent effective anaerobic bacterial activity. Under these conditions, reaction with residual sulphide phases in the sludge caused development of a small internal pressure (4.3–12.3 kN/m²) containing hydrogen sulphide. The pH also increased slowly (from 3–5.5) to levels at which microbial activity could resume. Thus, the cut through Pipe 1 after one year depicted in Figure 3 showed small black areas in the Biostore believed to result from incipient methanogenesis or sulphate reduction. Samples analysed from this cut gave an organic carbon content of 18–19% compared to that 17% in the original mix. Thus, taking account of experimental error, changes in carbon content were negligible and bacterial activity was therefore small. However, evolution of hydrogen sulphide at any odorous level was clearly a potential health hazard and obviated further consideration of acidic conditions for Biostore stabilisation.

In contrast, the Biostore in Pipes 2–4 exhibited substantial hydraulic conductivity, although the clay lining (which had a permeability of approximately 10⁻⁹ m/s) restricted leachate yields to 1–20 L over the first year of trials. This high conductivity was due to the poor packing characteristics of the sludge-aggregate matrix, which were observed during construction by Method B (compaction of sequential layers of the materials). They resulted primarily from the low sludge-aggregate ratio employed (1:3) and – in the cases of Pipes 2 and 3 – from the relatively low plasticity of sludge containing fly ash and lime. Thus, although such a ratio is theoretically ideal for a close packed system, in practice it prevents complete filling of voids between randomly packed aggregate particles, particularly when the sludge phase resists flow under compaction. Separate tests showed that this lack of plasticity could be largely offset with use of a larger proportion of water during sludge mixing.

The Biostore in Pipes 2 and 3 (unsaturated and saturated, respectively) was alkaline to prevent microbiological activity. Unlike Pipe 1, pH change did not give rise gaseous evolution or pressure increase, although ammonia was formed and dissolved in pore
Cuts through pipes 2 and 3 after test work facilitated direct observations of internal structures: solid and leachate analysis, indicated the maintenance of high pH and significant pozzolanic hardening. It had been predicted that the mechanical integrity of the alkaline Biostore would increase through pozzolanic re-crystallization with time [Mindness & Young 1981]. Conditions were anoxic but without evidence of anaerobic bacterial activity. Recovered pore water had moderate conductivity (6–8 mS/cm) but little free water remained. Thus, chemical reaction had resulted in consumption of oxygen and partial hydrolysis of the sludge materials. Figure 5 shows dissolved ammonia and total organic carbon determined over one year for Pipe 3, which, owing to its relatively large hydraulic head, yielded leachate more reliably from the bottom tap. This leachate yield was the main apparent difference between Pipes 2 and 3. As can be seen in Figure 5, ammonia was fairly stable in the range 0.5–0.65 g/L while organic carbon varied more erratically between 1.5–3.0 g/L, although with most readings close to the average value of about 2.0 g/L. These concentrations were as might be expected for an alkaline leachate from sewage sludge [Bagchi 1994] [Reinhart & Grosh 1998].

Figure 6 gives typical examples of heavy metal concentrations leached from the Biostore in Pipe 3. The values were, as expected, low and approximately constant, although slightly elevated with respect to trigger levels given in the groundwater Dutch list [Netherlands Ministére van Volkshuisvesting Roe 1994].

The Biostore in Pipe 4 was a neutral control. Analysis of leachate indicated increased conductivity (9–14 mS/cm), probably the result of greater reactivity in the presence of active bacteria. The total organic carbon was slightly lower than in Pipes 2 and 3, possibly owing to re-use of carbon by bacteria. An absence of significant pressure increase suggested a slow process toward fully anaerobic conditions and, possibly, greater sulphate reduction than methanogenesis. This was consistent with observed low, but not zero, dissolved oxygen contents.

Additional loading of the Biostore in Pipe 2 did not induce measurable consolidation. Although wall effects in all the pipes would reduce any tendency to consolidate, the general mechanical stability of the systems appeared to be at least adequate, as would be anticipated for hard aggregates particles in mutual contact and fully contained. Thus, observation of the Biostore surface against marks on the inner pipe walls and spikes pushed through the sampling ports and drillings into the centre of the Biostore mass indicated a complete absence of movement. Particles of three distinct size ranges were used in the different pipes: 5–20 cm (Pipe 1), 0.5–2 cm (Pipes 2 and 3) and 0.5–2 cm (lower half)/5–10 cm (upper half) (Pipe 4). However, there was no evidence that different particle size had a significant impact on Biostore properties, provided the particles were reasonably graded and did not contain substantial quantities of ‘fines’. In other words, the simple one-step screening of aggregate particles at sizes far larger (e.g. >0.2 mm) typical of the sludge phase would suffice. Of course, sludge particles may be deformable under compaction. Two types of aggregate mineralogy were also used (burnt shale and limestone). Neither phase would be readily available as waste aggregate for application in Biostore. However, they reasonably represented coarse shale and C&D waste which are more generally discarded and are clearly mechanically hard enough for the Biostore purpose. Of course, acid-consuming materials like limestone could not suit conditions established in Pipe 1, but shale (provided it is not too softened by weathering) or C&D materials should be of general applicability in the range pH 3–10 of interest.

Pipes 1 and 2 contained digested sewage sludge while Pipes 3 and 4 contained treated conditioned
sewage sludge, i.e., compost containing green waste. The two types had quantitatively different compositions, but, when combined in a composite sludge phase with coarse material, apparently behaved quite similarly. Previous laboratory work [Tyrologou 2005] had shown that the characteristics of digested sludge or composted sludge mixed with fly ash and lime in the ratio used in this work were probably optimal, although the system was quite robust to moderate changes. Thus, moderate differences in hydraulic conductivity (greater for compost because of its more open structure) were unimportant when compared to the much larger effects of varying compaction efficiency. Moreover, proportions leached as total organic carbon were different but insignificant in terms of the total contained carbon, which (as mentioned above) remained essentially unchanged throughout the trial.

From the results obtained it was thus deduced for scale-up purposes that the following conditions should be adopted: an alkaline system containing a 1:1 sludge-aggregate ratio, a 1:0.2:0.1 sludge/fly ash/lime ratio and sufficient water to induce plasticity.

3.2 ‘Area’ emplacement

Construction was carried out largely according to plan, although the limited surface area led to restricted machine access, and, in particular, to relatively poor compaction between the sampling pipes. The clay lining, sample pipes and drainage layer satisfactorily facilitated gas monitoring, leachate control and sample collection (as they would in a larger structure). The construction also facilitated conventional surveying for three-dimensional stability, as outlined below.

During 18 months of monitoring, the emplacement was observed to be mechanically and microbiologically stable. The 2 m and 5.5 m pipes initially remained dry while the 4.5 m pipe contained a small volume of anoxic ammoniacal liquor. Figure 7 shows a view of the surface features of the structure after essentially dry weather six months after completion. Following heavy rain in April-May 2006, the emplacement became surrounded by water and infiltration occurred around the monitoring pipes (where full compaction of the Biostore had not been possible). However, final excavation of the emplacement in July 2006 indicated that the main mass of the emplacement remained essentially dry (and had characteristics similar to those observed in the alkaline pipes at Bradley).

For instance, the recovered water had conductivity and pH of 7–8 mS/cm and 9–10, respectively. Dissolved ammonia increased at first and stabilised at 0.45 g/L, a level similar to that observed in Pipes 2 and 3 at Bradley. Total organic carbon was in the range 1.5–3.0 g/L. Landfill gas was not detected in any of the pipes. Survey levelling and electronic distance measurement indicated <1–2 cm 3D movement of the concrete blocks (beneath each pipe) with respect to each other, even after the emplacement of a surface load of 5 tonnes of rock/rubble (Figure 7).

4 CONCLUSIONS

The field work indicated the technical potential of the Biostore concept and conditions likely to be required for full scale emplacements in the range $0.5–1 \times 10^5$ tonnes to be securely constructed, stabilised and isolated within the near surface environment. Large scale sludge mixing with quicklime should be undertaken in enclosed rotary equipment so that water content and heating effects can be optimised and ammonia recovered for re-use. During emplacement (within a lining/drainage system), each sludge layer of approximately 300 mm depth should be covered with an equal volume of screened aggregate and compacted by means of a heavy vibratory roller to achieve low permeability. A lining/drainage system, including a surface cap, of layered clay and gravel (or synthetic analogues) should allow rainwater to flow around, rather than through, the Biostore mass. The whole construction should be covered with subsoil and soil (or synthetic analogue) to a depth of 1 m. Proposals are being developed to provide a full-scale demonstration project to substantiate the practicality and economics of these pilot scale observations and to include a suitable regulatory and planning framework.

More generally, the field results should contribute to national strategies for waste management by providing an alternative method for the management of sewage sludge, ash and other materials. The engineered composite sub-strata envisaged will incorporate near-surface contaminants present at a particular site, with
brought-in materials, and should become mechanically competent, stable and effectively isolated from the outside environment. Costly separation and disposal will be obviated. Because of the simple physical processing base, expensive additives or heat treatments will also be avoided. Moreover, on account of immediate stability and permanence, relaxation of the landfill tax requirements may be appropriate. Thus, several current material management problems are addressed. Not only will ‘pipe end’ waste producers gain access to this management route, but also site owners and construction companies will benefit from an additional re-development methodology. A large potential application exists. Much of the contaminated land in industrial and urban areas suited in principle to this proposed technology over the short to medium term. In the longer term application is possible as part of general urban regeneration and development. Each hectare emplaced to a depth of 10 m could in principle utilise some 35,000 tonnes of judiciously mixed materials – equivalent to the annual raw sludge output of many sewage treatment works. A wide variety of shapes, sizes and materials – mineral and organic, coarse and fine, ‘in-situ’ or local import – would be potentially applicable. If this technology is successfully developed and authorised, significant reductions in requirements for landfill, incineration and disposal can be anticipated.

ACKNOWLEDGEMENTS

Project staff at Imperial College, British Geological Survey and Clean Rivers Trust would like to acknowledge support from Biffaward, Biffa Waste Management, Yorkshire Water Services and the Institution of Civil Engineers. Particular thanks are due to Ian Fairless and Tom Taylor (Yorkshire Water Services) and Mike Winterbottom (Biffa Waste Management) for advice, materials supply and site access and to Lindon Sear (Biffa Waste Management) and James Ng (Lhoist UK) for provision of fly ash and quicklime, respectively. They also appreciate the assistance of John Harrison, Irina Tarasova, Graham Nash, Pavlos Tyrologou, Steve Bouzalakos, Kimberley Neville, Barry Coles and Ines Baptista.

REFERENCES


Review of composting and anaerobic digestion of municipal solid waste and a methodological proposal for a mid-size city

M.R.Q. Silva & T.R. Naik
UWM Center for By-Products Utilization, University of Wisconsin-Milwaukee, Milwaukee, WI, USA

ABSTRACT: Composting industry is a progressive and innovative industry that has been growing worldwide, especially in the last 20 years. Firstly, in this paper, an overview of fundamentals and processes on composting and anaerobic digestion are compiled, showing the versatility and multivariable profile of these processes. Secondly, a methodological proposal for a mid-size city is presented. It is proposed that the biodegradable part of the municipal solid waste, called BSW, can be efficiently composted with wastewater sludge, wood ash, coal ash, lime-kiln dust, and/or limestone quarry dust to improve the profile of the compost. In addition, anaerobic decomposition followed by vermicomposting is pointed as one of the best and efficient MSW treatment system, since it may reduce total time of the composting process. Furthermore, it generates liquid fertilizer and biogas, which provides energy to supply the composting plant. This makes the plant auto-sustainable in energy. Finally, an outline using real data from the City of Milwaukee, Wisconsin, USA, is being presented to illustrate an example of this proposed methodology of efficient composting.

1 INTRODUCTION

The “Garbage” is one of the major challenges in the world, especially in the cities and other population centers. Municipal solid waste (MSW) management is being considered a luxury item due to a variable quality of the service as a consequence of a combination of lack of resources, lack of expertise, lack of political will, and inadequate legislation [Fourie 2006]. In the last two decades, MSW management is one of the main public subjects under discussion. In the literature, researchers have been sharing case-studies about waste management practices in many different countries [Magrinho et al. 2006]. Many cities in developing Asian countries have difficulties for managing their solid waste. More than 90% of MSW in India is disposed of on the land unsatisfactorily [Sharholy et al. 2006]. In North America, the cost of MSW represents a multibillion-dollar industry [Huang et al. 2004]. Incineration and landfiling are some possible ways of dealing with Municipal Solid Waste; however, there is a better option, composting of MSW. This is a recent field in the U.S. In 2000, it was reported that there were 16 operating facilities in the U.S. that compost Municipal Solid Waste; all these are aerobic systems [Block & Goldstein 2000]. Anaerobic composting is not well used in the U.S until now [Goldstein 2004]; in contrast, anaerobic composting has been underway in Europe for food residuals for over 30 years especially as a solution for dwindling landfill space [Nichols 2004].

In 2003, MSW generation was estimated to be 4.5 pounds (2.04 kg) per person per day. Approximately 55% of the 240 million tons ($2.4 \times 10^8$ kg) of MSW generated per year were landfilled, 14.0% were combusted, and 31% were recycled or composted [USEPA 2003]. Composting industry has grown in the USA in the past 20 years [Coker & Goldstein 2004]. Recycling of MSW increased 9% and composting of MSW increased 5% from 1990 to 2003, totaling 7% for composting [USEPA 2003]. Just to have an idea how much of an impact MSW has, residential waste of City of New York is approximately 28% composed of leaves, grass, yard, food, and other organic waste. New York could save $12 million per year by restoring and expanding composting programs [Goldstein & Goldstein 2004].

Therefore, sustainability is the key-word of this proposal. “Garbage” should not be thrown away because it is possible to generate compost from MSW, while producing energy from one of the by-products of the process, the methane gas (“cleaner energy”) and harvesting materials from MSW to either recycle or reuse in cement industry. For instance, broken glass can be used as a finely ground mineral additive (FGMA) in cement, and up to 70% of portland cement
cinder can be replaced with broken glass [Sobolev et al. 2006]. In addition, it is proposed to use residues from other industries, such as wastewater sludge, wood ash, coal ash, lime-kiln dust, and/or limestone quarry dust to co-compost with BSW.

2 OVERVIEW OF COMPOSTING AND ANAEROBIC DIGESTION– FUNDAMENTALS AND PROCESSES

There are three major biological management of MSW: biological processes occurring in landfills, anaerobic digesters, and composting facilities [Palmisano & Barlaz 1996]. In this paper, landfills will not be covered since there is a tendency of diverting biodegradable components from MSW from landfills to other alternatives, such as composting [Braber 1995, Harrison & Richard 1992, Komilis 2006, New Mexico State University 2004] and anaerobic digestion [Braber 1995, Shin et al. 2000]. This tendency is not only due to dwindling landfill space [Nichols 2004, Shin et al. 2000, Song & Greenway 2005, Tinnaz & Demir 2006], but also due to certain challenges, such as emanating odors, attracting vermin, emitting toxic gases, and potentially contaminating groundwater [Shin et al. 2000]. However, it is important to note that there are some studies that have proposed improvements in the challenges associated with landfills [Münnich et al. 2006].

Composting is the biological decomposition and stabilization of organic substrates that involves aerobic respiration [Palmisano & Barlaz 1996] and produces a final product that is stable, and free of pathogens and plant seeds that can be beneficially applied to land [Haug 1993]. During the process of aerobic composting, it generates carbon dioxide, water, and heat. On the other hand, anaerobic digestion is the biological decomposition of organic substrates in the absence of oxygen. During the process of anaerobic digestion, it generates methane, carbon dioxide, and numerous low-molecular weight intermediates such as organic acids and alcohols [Haug 1993] and humus [Chynoweth & Pullammanappallil 1996]. This implies that significantly less energy is required per mass of organic decomposed during anaerobic digestion compared to aerobic composting. Anaerobic digestion of MSW has developed more rapidly in Europe in the last decade than in the U.S. [Palmisano & Barlaz 1996]. Anaerobic digestion is also preferred in other countries, such as Korea, to manage food residuals [Shin et al. 2000]. Within the projects reported in the U.S. that turn methane from anaerobic digestion into electricity, there is a very good example of the Straus Family Creamery in California that generates up 600 000 kWh/year from anaerobic digester, saving $6,000/month in energy costs [Goldstein 2004].

2.1 Biological fundamentals

In anaerobic digestion, four different types of microorganisms are responsible for the degradation of MSW: hydrolytic, fermentative, acetogenic, and methanogenic [Braber 1995]. In the first step of anaerobic digestion (depolymerization), the hydrolytic bacteria are responsible for depolymerization of polymeric solid substrates into smaller molecules such as organic acids, alcohols, and the methanogenic substrates. Some bacteria involved in this step are: Bacteroides succinogenes, Clostridium lactobacillus, Clostridium cellulosporum, Ruminococcus flavefaciens, Ruminococcus albus, Butyrivibrio fibrisolvens, Clostridium thermocellum, Clostridium stercorarium, and Micromonospora bispora. In anaerobic digesters with MSW, the predominant bacteria are Clostridia. In this stage, typically only 50% of the organic matter is degraded. Also, some intermediate reactions occur: for example, products of depolymerization reactions are converted to fermentation products [Chynoweth & Pullammanappallil 1996].

The last step is the methanogenesis. Methanogenic bacteria are slow-growing anaerobes that degrade acetate, methanol, carbon dioxide, formate, carbon monoxide, methylamines, methyl mercaptans, and reduced metals [Chynoweth & Pullammanappallil 1996].

A variety of microorganisms are present in a composting system. Even though the microbial population may be vast, the main microorganisms that affect the composting system are fungi, actinomycetes, and bacteria. Also present are protozoa and algae [Stoffela & Kahn 2001]. Bacteria and fungi are the most important microorganisms for composting [Haug 1993].

A composting system may contain three classes of microorganisms: cryophiles or psychrophiles, mesophiles, and thermophiles [Stoffela & Kahn 2001]. Cryophiles bacteria do their best work at about 13°C, although they work even until −20°C. Between 0°C to 40°C, mesophilic bacteria predominate. Above this temperature, thermophilic bacteria work faster. When the temperature of the compost comes down, mesophilic bacteria again predominate [Wassenaar 2003].

Considering a windrow system, mesophiles and thermophiles are the most common microorganisms in composting, contributing to composting at different times of the process. This has four different stages. In the first stage, there are abundance of substrate and mesophiles that are predominant and very active. During this stage, large quantities of heat energy are generated, which increases the temperature of the compost pile. The favorable temperatures to this kind of microorganism are between 35°C and 45°C [Miller 1996, Stofella & Kahn 2001].

As the temperature rises, it becomes more favorable to thermophiles, for which the best temperature
is higher than 45°C. The compost pile reaches about 65°C to 70°C. At this stage, the food sources decreases for microorganisms and the temperature falls, resulting in one more mesophilic stage. In the last stage, the temperature falls to ambient temperature again [Miller 1996, Stofella & Kahn 2001]. About 80 to 90% of the microbial activity during composting is due to bacteria [Haug 1993, Stofella & Kahn 2001]. In composting, species were found belonging to Bacillus, Pseudomonas, Arthrobacter, Alcaligenes [Stofella & Kahn 2001], as well as Staphiloccoci [Hassen et al. 2001] in the mesophilic stage. In thermophilic stage, bacteria are predominantly of the Bacillus sp., as B. subtilis, B. stearothermophilic, and B. licheniformis. Above 65°C, compost populations are often reduced to pure cultures of B. stearothermophilus [Miller 1996]. The optimum starting temperature for decomposition was found to be 40°C, considering the amount of TOC reduction [Hamoda et al. 1998].

Actinomycetes usually can be observed after five or seven days from the beginning of the composting. They are easily detected because of their grayish appearance in the compost. They are responsible for the “earthy” smell that the compost emits under favorable conditions. Micromonospora, Streptomyces, and Actinomyces are some species that can be regularly found in composting materials. Actinomycetes play an important role in composting once they have conditions to degrade cellulosic components such as bark, newspaper, and woody stems [Stoffela & Kahn 2001, Wassenaar 2003]. Actinomycetes tend to be common in the later stages of composting. They prefer moist, highly aerobic conditions and a neutral or slightly alkaline pH [Miller 1996].

Fungi are important because they can break down tough debris and organic residues that are too dry (fungi have a lower moisture requirement), acidic (fungi can live in a broad range of pH), or low in nitrogen, enabling bacteria to continue the decomposition process. Fungi are most common in mesophilic and thermophilic stages of composting. Species more common are Mucon, Aspergillus, and Humicola [Epstein 1997, Miller 1996, Wassenaar 2003].

Ants, beetles, centipedes, green fruit beetle larvae, millipedes, mites, redworms, sowbugs, springtails, and redworms can be found in a compost pile [Wassenaar 2003]. They are important not just for decomposition of the organic matter, but as source of nutrients and biomass to the soil [Epstein 1997].

2.2 Chemical fundamentals

Carbon (C) and nitrogen (N) are essential to the composting process. Carbon provides the primary energy source and it is utilized for cellular growth. Nitrogen is also essential to the growth of microorganisms in order to synthesize new cellular material [Diaz et al. 1993, Epstein 1997, Stofella & Kahn 2001]. C/N ratio is a very import parameter and expresses the effect of raw waste quality [Hamoda et al. 1998]. The optimum C/N ratio was found to be in the range of 25–30 [Diaz et al. 1993, Epstein 1997, Hamoda et al. 1998, Huang et al. 2004, Stofella & Kahn 2001].

Phosphorus (P) and potassium (K) are also important to the growth of the plants. Substrates such as biosolids, yard debris, and animal manure may have sufficient P, but Municipal Solid Waste (MSW) may not have enough P if it is high in cellulose. Cysteine and methionine – both aminoacids found in protein materials – are the main sources of sulfur (S) in substrates, which in sufficient quantities may generate volatile and odorous compounds (detected by people even at a low-level of concentrations). In well-aerated process, sulfites are oxidized into the sulfates by combining with oxygen in well-aerated zones. However, in anaerobic conditions, volatile organic sulfides and H₂S vaporize into the atmosphere, being responsible for many of the malodor associated with composting. In regard to heavy metals, some countries have established compost quality standards that limit the concentration of heavy metals (arsenic, cadmium, chromium, cobalt, copper, lead, mercury, molybdenum, nickel, and zinc). This refers to the heavy metal concentration of the substrate and the final compost [Stofella & Kahn 2001].

Many organic substances, including MSW, sewage sludge, and other industrial waste are being introduced to composting, which increases the level of heavy metals when compared to the traditional “green” waste compost [Song & Greenway 2005]. However, heavy metals do not seem to be an obstacle to composting. For example, the US Environmental Protection Agency (US-EPA) considers that the levels of trace elements found in MSW compost do not represent a significant risk in terms of human health and impacts to the environment. In addition, the separation of MSW into compostable and non-compostable fractions reduces the level of heavy metals. Another concern about the presence of heavy metals in compost is the possibility of their leaching to groundwater or runoff to surface waters. It has been reported that the leaching of heavy metals increases with application rate of the compost in the soil. However, the concentration of some heavy metals has been reported to be lower than that by the US-EPA drinking water limits [Epstein 1997].

Composting process is relatively insensitive to pH because it has a wide range of microorganisms that work in different pH [Diaz et al. 1993, Stofella & Kahn 2001]. For example, bacteria acidophiles need pH 5 or less for maximal growth; bacteria alkalophiles need a range of pH between 7 and 12; and bacteria neutrophiles need pH about 7.0 [Epstein 1997].
However, the optimum pH generally is considered to be between 6.5 and 8.5. In aerobic composting an oxidation process occurs where $\text{O}_2$ is consumed and $\text{CO}_2$ is produced. Thus, controlling the level of these two gases during the composting process can provide an indication of composting activity. For an efficient composting, an adequate aeration is required, which is based on temperature, free air space, and moisture [Stoffella & Kahn 2001]. Aeration is important to obviously satisfy the $\text{O}_2$ level, to remove water from wet substrates, and to remove the heat generated by organic decomposition in order to control the process temperature [Haug 1993].

2.3 Physical fundamentals

Moisture content is an important variable in composting process in order for the bacteria to assimilate their nutrients that could be dissolved in water. Moisture comes from the original moisture of substrates and from the biological oxidation of organic matter. It is lost through evaporation. Diaz et al. [1993] have stated that the microbial activity ceases when the moisture is less than 8% to 15%. It is reported that it may start to occur when the moisture content is about 12% to 15% [Stoffella & Kahn 2001]. The desirable moisture content is between 50% and 60% [Epstein 1997, Hamoda et al. 1998, Stoffella & Kahn 2001]; although, for domestic waste, the optimum moisture is between 52% and 58% and for food waste it is 60%. Lower moisture levels can make composting facility operators prematurely conclude that their compost process has stabilized. In addition, microbial activity decreases when moisture is below 40%. On the other hand, high moisture levels (above 60%) can block the air passageways, enabling anaerobic conditions with the potential for odor formation. Composting containing materials with structural strength, with a degree of resistance of individual particles to consolidate under compression, (e.g., woodchips, straw, hay (dried grass), rice hulls, and corn stover) can tolerate high moisture of about 75% to 80%. On the other hand, if particles are structurally weak, they are deformed and compressed upon compression, and the volume of interstices is proportionally reduced, which means that it reduces the available space for air and water; so, the permissible water content is lower. Fruit wastes, caning process wastes, sludge, and manures are example of materials that have little or no structural strength. In this case, it is necessary to add a bulking material, which will improve the strength of the compost material. Any material can be added with a high degree of structural strength or even finished compost material (which is usual), or some amorphous material can be added after undergoing a specific process [Diaz et al. 1993].

Particle size affects moisture retention, free air space, and porosity of the compost mixture. Large particle size materials imply more free air space and high porosity. On the other hand, small particle size materials increase the surface contact between $\text{O}_2$ and compost material, increasing the aerobic composting activity. Hamoda et al. [1998] showed that the rate of organic matter decomposition is higher for material of 40 mm particle size due to the fact that the voids between such larger waste particles are bigger than with smaller sizes (5, 10, and 20 mm size), where the oxygen may not access particles easily.

The control of the temperature in composting process is important to maximize the decomposition rate and to inactivate pathogenic organisms [Epstein 1997, Martin 1991, Stofella & Kahn 2001]. For substrates that do not contain human pathogens, a certain desirable temperature is necessary to inactivate plant pathogen or to destroy weed seed. Temperature also affects the moisture content, which affects microbiology activity [Epstein 1997]. The best temperature varies according to the type of substrate that is being used and according to the microorganism population that is desired; for example, there is different optimum temperature to mesophilic bacteria and to thermophilic bacteria [Stoffella & Kahn, 2001].

2.4 Thermodynamics fundamentals

The designer of a composting plant needs to understand the microbes involved, reactions they mediate to obtain energy, environmental conditions required for growth and metabolism, and, in certain cases, conditions required to kill organisms such as pathogens [Haug 1993].

The First Law of Thermodynamics is the law of the conservation of energy. The concepts of heat, work, internal energy, and enthalpy are related to the First Law. The Second Law of Thermodynamics resulted from a search to explain the direction in which spontaneous process would occur. The concept of free energy was developed from the First and Second Laws. Free energy gives the useful work, which can be derived from a chemical reaction that occurs under constant pressure and temperature conditions (most microbial reactions occur under such conditions). The water and solid fractions of a composting material can be treated as separate components from a thermodynamic standpoint, which allows calculating the energy balance of a composting process [Haug 1993].

Interesting studies were done in this area very recently. For instance, mass and energy balances of four alternatives strategies were examined for energy recovery from MSW by dedicated waste-to-energy plants generating electricity through a steam cycle. The largest energy savings were achieved by combusting materials recovery without treatment in large scale plants [Consonni et al. 2005]. In another study, energy conservation and carbon dioxide emission
reductions due to recycling in Brazil were estimated, given the amount and the composition of waste recycled and the scenario for the energy generation expansion in that country [Pimenteira et al. 2004].

2.5 Composting

Composting may be performed in various ways. Currently, the leading concepts are: non-reactor systems (windrows and static pile) and reactor systems (vertical flow, horizontal and inclined flow processes, and nonflow processes).

Windrow is the most common composting method practiced [Diaz et al. 1993, Haug 1993, Stofella & Kahn 2001] and is a potential treatment technique for MSW [Komilis 2006]. Mixed feedstocks are placed into long narrow piles and turned periodically, usually by mechanical equipment.

Height, width, and shape of the windrows vary depending on the nature of the feed material and the type of equipment used for turning [Haug 1993]. Windrow process can be naturally aerated. In this case, the oxygen is supplied primarily by natural ventilation resulting from the buoyancy of hot gases in the windrow system and, to a lesser extent, by gas exchange during turning. Another variation of the windrow process is the one with forced aeration. In this case, oxygen transfer into the windrow is aided by forced or induced aeration from blowers [Haug 1993, Manser & Keeling 1996].

Turning feedstocks aerates the windrow, although not for a long time because microorganisms consume the oxygen within hours [Epstein 1997, Stofella & Kahn, 2001]. Another effect of turning is the loss of water. It can be good, if the substrate has high moisture content. On the other hand, because the turning process makes it easier to add water, it is a good opportunity if the moisture level of the feedstock is low. The frequency of turning is indicated by the ratio of oxygen availability to oxygen demand and it depends on many factors: nature of the material, its structural strength and moisture content, pathogen kill, uniformity of decomposition, and the rapidity of the composition desired by the operator. Turned windrow systems have some limitations. The first is that temperatures lethal to pathogens do not prevail throughout a windrow. Another disadvantage is that the turning procedure can contaminate the stabilized material with nonsterile material in the outer layers of the windrow in which bactericidal temperatures did not develop. Repeated turnings can reduce the pathogens populations to less than affective concentrations. In addition, turning improperly or insufficiently leads to malodors. Reduced frequency, increased space requirement, intensity and frequency of turnings, have also been listed as disadvantages [Diaz et al. 1993].

Static piles may be passively aerated, assisted passive aerated, and aerated. The passively aerated process is based on passive aeration, natural decomposition, to produce compost. It is normally used to slowly decompose cellulosic feedstocks, such as leaves, brush, bark, wood chips, and some agriculture residues. Feedstocks may be combined and mixed to adjust moisture, porosity, density, and/or C/N ratio. After the pile is formed, this is turned with a bucket loader every one to three month, resulting in one or two turnings during the composting cycle [Stoffela & Kahn 2001].

For the static pile process, it is difficult to maintain the aerobic condition; usually, aerobic condition exists only within one to three meters from the pile surface and perhaps along air channels that form within the pile. However, oxygen conditions can be improved with well-mixed feedstocks and with the increasing of porosity.

One of the most common assisted passive aerated methods is the Passively Aerated Windrow System (PAWS). This method involves short windrows with one to three meters height and they are not turned. There are some characteristics usually common for any variation of this method: feedstocks have homogeneous and relatively porous mixture; a delivery system for passive air flow; a base layer of stable absorbent material like straw or compost; and an exterior layer (approximately 150 mm thick) of stable coarse material that retains heat, moisture, odor, and NH₃.

In the aerate static piles method, fans aerate the composting materials. It improves aeration, reduces the processing time, and reduces the potential to produce odors, while providing oxygen, cooling the pile and removing water vapor, carbon dioxide, and other products. There is no intentional turning or agitation; it occurs just when materials are moved [Stoffella & Kahn 2001].

Some advantages of this method are that it is simple, requires less area than passively aerated methods and turned windrows, while aerating is better. In addition, it can be enclosed within a building because it is a more compact process and require less space [Stofella & Kahn 2001]. Additionally, static piles are not required to be turned regularly [Manser & Keeling 1996]. This method, however, requires attention due to compaction, short circuiting of air, and inconsistent decomposition within a batch of compost [Stoffela & Kahn 2001]. Moreover this system needs a power supply for the fans, and a control system to alternate the fans between suction and blowing. Another problem is that the air pipe could be in the way of handling equipment inside a plant, interfering in the process [Manser & Keeling 1996].

Within the vertical processes, there are agitated solids bed and packed bed. Agitated solids beds have multiple hearths; solids are agitated during movement down the reactor, with forced aeration. Feeding can be continuous or intermittent and there is some mixing.
in reactor. A very good example of this type of reactor is the NGK reactor that began operation in 1989 in Japan. In this system, two reactors are provided, each with six cells arranged vertically. The facility composes slightly over two dry tons/day (2,000 kg/day) of lime and ferric conditioned, raw sewage sludge, filtered press cakes. Packed bed (silo reactors) are usually used for composting of sludge cake amended with sawdust and other materials [Haug 1993]. Periodic transfers of solids from the bottom to the top of the reactor occur [Haug 1993, Stofella & Kahn 2001], allowing agitation of the solids. Feedstocks must be mixed before loading. The time that the material remains in the silo ranges from 10 to 30 days, depending on the application [Stofella & Kahn 2001].

Within the horizontal and inclined solids flow, there are the tumbling solids bed, agitated solids bed, and static solids bed. Tumbling solids bed (rotary drums or kilns) have drums usually slightly inclined, rotating slowly (continuously or intermittently), tumbling the material inside. With this movement, the material is mixed, agitated, and moved through the drum or kiln. Feedstocks are fed at the top of the drum and the compost is removed at the opposite side [Stoffel & Kahn 2001]. Agitated solids beds have two variations (agitated bins or open channels). One has cylindrical tanks with a set of screws that allows air to be discharged into composting material. If retention time is less than two or three weeks, the material must be finished in windrows [Diaz et al. 1993]. The other has rectangular bins. Feedstocks are placed into designated cells along the bin length, and they are turned about once a week. They remain in the system for about six days and the material is allowed to mature in windrows for one to two months. This system combines forced aeration with tumbling; the air is forced through the composting mass at least once each day [Diaz et al. 1993, Stofella & Kahn 2001]. For the static solids bed there are two types: tunnel reactor and conveyor type. This system has been applied to a variety of feedstocks, for example: sludge, manure, and MSW fractions. The vessel is essentially a plug flow, tubular reactor or tunnel of rectangular cross section. One design uses a pusher plate with forward and backward movement which is situated at the feed end of the tunnel. Air can be supplied and gases can be removed along the length of the reactor [Haug 1993].

Nonflow reactors (compost boxes) are very simple systems in which materials are loaded in the box reactor and remain there for 7 to 14 days. Aeration is usually controlled and curing is usually conducted in windrows for several months afterwards [Haug 1993].

2.6 Anaerobic digestion

Anaerobic digestion has been used for food residuals in Europe for over 30 years [Nichols 2004]. In the U.S., this technology has been growing since 1990 due to increased technical reliability, growing concern of facility owners about environmental quality, increased number of federal and state funds available for cost share, and emergence of new state energy policies [Mattocks & Wilson 2005]. Digestion has been even moved from “unthinkable” to “Best Management Practice” in the West Coast [Mattocks 2004]. This process is used mostly for treatment of municipal sludges and industrial wastes. Some countries such as China and India use the biogas generated for cooking, lighting, and operation of small engines, using the residues as soil amendment [Chynoweth & Pullamanappallil 1996]. Currently, the leading concepts of anaerobic digestion are dry continuous system, dry batch systems, wet continuous systems, and co-digestion [Braber 1995]. In a continuous system, the dry matter varies between 20% and 40% and fresh substrate is added continuously and an equivalent amount is withdrawn once each day or two. In the dry batch system, material is not added or withdrawn from the digester till the digester process has been completed [Diaz et al. 1993]. In addition, each batch lasts for two to three weeks and percolate is recirculated to stimulate mixing and digestion [Braber 1995].

The wet continuous system is subdivided into conventional slurry systems and anaerobic filters. Conventional slurry system usually works with codigestion of animal manure, MSW and other wastes, and a dry matter of about 10%. Anaerobic digesters usually have two- or multi-phases systems. In the first reactor, hydrolysis and acidification of the substrate occur [Braber 1995]. Organisms decompose lipids, cellulose, and protein and generate soluble compounds. Acid bacteria work on theses compounds, producing organic acids. This is known as the acid-former phase [Diaz et al. 1993]. Methane fermentation occurs in the second reactor. Bacteria either work on organic acid producing methane and carbon dioxide or reduce carbon dioxide into methane by using hydrogen or formate produced by other bacteria. The very final products of this stage are methane, carbon dioxide, trace gases, and a stable residue. This is known as the methanogenic phase [Diaz et al. 1993]. About 90% of the facilities in Europe use the one-stage reactor [Vandevivere et al. 2002].

The advantages of wet processes over dry processes are: higher homogeneity of wastes and possibility of separating out contaminants, floating matter, and sediments. On the other hand, the disadvantages are: greater reactor capacity; large material flows to be transported, treated, and heated; higher cost for water supply and dewatering; and slightly lower gas yield since heavy inerts and scum layer need to be removed [Nichols 2004].

The digestion procedures are plugflow, completely mixing, or anaerobic filters. Plugflow is a very simple
system: it is lined in an excavated pit with a wall of native materials; the interior of this wall is lined with an impervious material such as a plastic sheet. Suitable provision must be made for capping the pit and collecting and storing the gas. Availability of a suitable lining material may be an issue for some situations. For example, the durability and degree and permanence of impermeability of the plastic sheet are difficult [Diaz et al. 1993]. The mixing can be done either mechanically or by gas injection [Braber 1995]. Mixing is important to achieve an efficient operation of a digester, avoiding sedimentation, and breaking up the scum layer.

Anaerobic digesters may be run at mesophilic or thermophilic temperatures. Mesophilic process has higher operational reliability as the microorganisms are less sensitive to temperature and concentration fluctuations; less energy required for heating up wastes; and higher percentage of methane in biogas. However, the residence time is higher, the volume of gas produced is lower, and the sanitation is not certain [Nichols 2004]. Thermophilic process has been suggested for anaerobic co-digestion of substrates such as mixed municipal and industrial waste [Oleskiewicz & Poggi-Varaldo 1997], anaerobic co-digestion of coffee waste and sewage sludge [Neves et al. 2006], co-digestion of paper mill sludge, biosolids, and MSW [Poggi-Varaldo et al. 1997], and co-digestion of paper mill and MSW [Poggi-Varaldo et al. 1999].

2.7 Other processes

For the home composting process, a bin or some type of container is used to keep the composting material within a confined space. The feedstocks that can be used are: garden trimmings, manures, garbage, vegetable trimmings, paper and cardboard, and various absorbent materials. Any decomposable organic material can be used, except human feces, diseased animals, and plant debris heavily dosed with pesticides and other toxic material in general, for reasons of public health [Diaz et al. 1993].

The maximum C/N requirement should be 25:1 to 30:1. For home composting, this ratio can be adjusted considering the relation of “green” debris or garbage to “dry” garden debris. Green debris, e.g., lawn clippings, fresh leaves or plants, green plant stems, roots, flowers are rich in nitrogen. Dry debris, e.g., dried grass (no longer green), matured flowers stalks, branches (excluding leaves), straw, and fall leaves are rich in carbon [Diaz et al. 1993, Hanson 1997].

Another composting process is the vermicomposting, which is an aerated composting system where the redworms process the feedstock, and they excrete vermicasts at the soil surface. Compost worms do best under moist conditions, but are very sensitive to elevated temperatures [Dominguez et al. 1997]. The best temperature for worms is between 13°C and 29°C [Pittaway 2001], although maintaining moderate bed temperatures can significantly increase performance of the process [Frederickson & Howell 2003]. This system can be processed in bed or in windrows, indoor or outdoor.

Vermicomposting supplies mineral balance, improves nutrient availability, and could act as complex-fertilizer granules. In addition, vermicomposting allows a great reduction of pathogenic microorganisms. Vermicomposting may also bring a decrease of bioavailable heavy metals than in the composting process, and there is evidence that the final product may contain hormone like compounds that accelerate plant growth [Dominguez et al. 1997].

3 METHODOLOGICAL PROPOSAL: A MSW TREATMENT FOR MILWAUKEE

Although Milwaukee has recycling plants for newspaper, office paper and cardboard, aluminum and steel/tin cans, glass bottles and jars, and #1 and 2 plastic containers, and has already plants for composting yard trimmings, in this paper a new methodological proposal is being advocated for Milwaukee (or, any other mid-size cities in USA) that would cover the recycling of the whole amount of MSW generated by the city.

Wisconsin's Recycling Law requires everyone in the state whether they are at home, at work, or at a special event to recycle newspaper, office paper and cardboard, aluminum and steel/tin cans, glass bottles and jars, and #1 and 2 plastic containers. [WDNR 2003].

The City of Milwaukee has approximately 940,000 inhabitants [DHFS 2004], which is about 17% of the population of Wisconsin, according to the Census 2004. Therefore, the MSW generation from Milwaukee can be extrapolated, considering 17% of the total MSW generated by Wisconsin. It means that Milwaukee produces about 2,000 metric tons/day of MSW. From this total amount, about 18% consists of organics (yard waste, food, diapers, animal waste/kitty litter, and bottom fines/dirt). In addition, the DNR report showed that food waste is a material found in sufficient quantity (10%) to offer significant opportunities for increased diversion for composting.

A flow chart for the proposed MSW treatment plant is shown in Figure 1. The methodology of recycling MSW consists of a combination of various unit operations. The proposed unit operations in this paper are generally known in the world of recycling. The design of the cited unitary operations is simple most of the time, but extremely important. The key to success
Figure 1. Flow chart for the proposed MSW treatment plant.
as proposed in this paper is the selection of the better combination of the unit operations in this process in order to obtain the best results for recycling matter and energy in this complex process.

The methodological proposal involve three main steps: (1) picking up station for removing bulky material and recyclable material; (2) anaerobic digestion, and (3) vermicomposting.

Considering the proposed plant, a city the size of City of Milwaukee would need about 70 garbage trucks managing two routes a day, assuming that a garbage truck has the capacity of 15 metric tons.

In order for each silo to receive one truck load, and considering the average density of mixed MSW as 0.44 metric ton/m³ (26 lb/ft³) in the garbage-truck and 0.24 metric ton/m³ (14.8 lb/ft³) after dumping from garbage-truck [Diaz et al. 1993], it will be necessary to have the silo with the capacity of 65 m³.

In addition, considering that the new composting plant would work 24 hours a day, seven days a week, it means that the plant is expected to process about 85 metric tons/hour.

Furthermore, a conveyor belt is proposed under each silo, and the unloading of the silo controls the velocity of each conveyor belt. It means that it would not be necessary to have a valve in the exit of the silo, which means a reduction on the cost of the process and one less variable in the process.

It is suggested that the silo has one meter height extension on their upper part. The objective is to avoid MSW falling outside the silo during the unloading of the garbage-truck. It is suggested that each silo has an angle of 66°, since it avoids build up in silo walls [ASTEC 2004].

3.1 Processing plant

The plant would have six conveyor belts, one for each of the six silos.

The volume (V) of the silo is one m³, considering dimensions of 0.5 m × 0.5 m × 4 m. The mass (m) of the material of the silo is 0.24 metric tons.

Considering that

\[ v = \frac{QL}{m} \]  

where v is velocity of the conveyor belt, Q is flow of the material of the silo, L is length of the silo and m is mass of the material of the silo, the velocity of each conveyor belt should be 24 m/min. As 24 m/min is so fast, it is suggested to have 2 silos and two conveyor belts (one for each silo) in the plant, in order to reduce the velocity of each conveyor belt to 12 m/min.

In regard to the sorting station, it is known that nowadays there are a considerable number of studies against the sorting of all material only manually, since it increases costs with employees and reduces the quality of the feedstock to composting and the quality itself of the recyclable material. For example, it is recommended to have screens or a series of screens in the beginning of the process to separate paper from the other materials, since paper and cardboard correspond to major products to be recyclable. In addition, there are machines that can sort the materials automatically, guaranteeing the quality of sorting. However, for the recycling process of MSW, it was chosen to work with a very common situation such as hand picking station. In addition, it is known that Milwaukee has curbside residential recyclables collection program, in which non-organic materials, such as glass, metals, and newspaper are removed from the MSW feedstock. Therefore, in this process, the MSW generated by the City of Milwaukee can be readily and efficiently sorted at the plant.

The number of employees in the new composting plant (in the MSW recycling area, which is the area that most demand employees) is estimated from some data from other MSW recycling plants. The Del Norte Regional Recycling and Transfer Station in the City of Oxnard, California process about 1,300 metric tons of waste and recyclable material a day and have nearly 100 employees. The Columbia County, Wisc., solid waste operation process 100 tons of waste and recyclable material a day and have 16 employees [Tilton 2000] Therefore, in the California plant the productivity is 13 metric tons/employee/day and in the Wisconsin plant is 6.25 metric tons/employee/day.

Therefore, it is estimated that the proposed plant will have a productivity of about 10 metric tons/employee/day, which is the average of the productivity of the two plants cited above. This is a reference number since the plants work 24 hours a day and it is not exactly known what the productivity per employee would be. Therefore, it is estimated that the proposed plant will need about 200 employees.

Considering three work shift and considering that each work shift has the same demand, each work shift will about 70 employees. It means that approximately 35 employees per sorting line would be needed. The number of employees per each sorting line might be too high because they were based on the number of employees reported for the two plants already referred (California and Wisconsin). It is not clear the exact number of employees working specifically on sorting line. However, probably the plant may still need this number of employees to work in the new proposed composting plant, considering all departments, including the MSW recycling department.

After each sorting line of the proposed plant, there would be two screens: the primary screen and the secondary screen.

It is specified that the primary screen would have 127 mm (5") openings [Hecht 1983] with capacity of 50 metric tons/h; the secondary screen would have
13 mm (1/2 inch) openings [Hecht 1983] with capacity of 30 metric tons/h.

3.2 Composting plant features

To harvest worms it is indicated 6 mm (¼ inch) screen [Codner 2000] be used for the first pass, with capacity of 0.79 metric tons/h.

Experience shows that for a city the size of City of Milwaukee, primary hammermill should have a capacity of 50 metric tons/h and the secondary should have a capacity of 25 metric tons/h. The magnetic separator should have a capacity to process 32 metric tons feedstock per hour, separating 0.58 metric tons/h of ferrous material. The air classifier should have a capacity to process 30 metric tons/h.

The density of food waste ranges between 0.35 metric tons/m$^3$ to 0.40 metric tons/m$^3$ [Hecht 1983], and yard waste shredded and mixed ranges between 0.14 metric tons/m$^3$ and 0.19 metric tons/m$^3$ [Hickman Jr. 1999]. The density of the water/moisture is one metric ton/m$^3$. In addition, if the mass of feedstock is 563 metric tons, of which 47% is food waste, 20% yard waste, and 33% moisture, then the minimum volume of the storage tank should be 2,700 m$^3$ (considering a safety factor of 1.5), since

\[ d = \frac{m}{V} \]  

(2)

where \( d \) is density, \( m \) is mass of the feedstock, and \( V \) is the volume of the feedstock.

The proposed composting plant should have seven storage tanks of capacity of 2,700 m$^3$ each, considering that each tank receives the daily demand of 1,754 m$^3$ and considering that the residence time of feedstock in the next step – mixing tank – is seven days.

The proposed compost plant should have seven mixing tanks at 37°C (mesophilic temperature), since the residence time in each tank is 7 days. Therefore, it will have one mixing tank to receive the feedstock from each storage tank. The volume in each of these tanks should be higher than the storage tank, since in this step other materials are added as sewage sludge, wood ash, coal ash, and/or lime kiln dust. The quantity of these materials is not yet determined, since this subject requires additional work and empirical proof. The volume of each mixing tank should be 3,500 m$^3$, since a safety factor of 2.0 is more appropriate to estimate the volume of each of the seven mixing tanks.

In the mixing tank biogas is also generated. However, in order to calculate the production of biogas, it is assumed that the total production would come from anaerobic digester only.

3.3 Anaerobic digester

Considering that the recommended retention for Rotalller model anaerobic digesters tanks time is between 2 to 15 days [RISE-AT 1998], the proposed composting plant should have anaerobic digesters with 7 days of retention time at 55°C or higher. It is estimated that the volume of an anaerobic digester is three times higher than a mixing tank, since the generation of gas is higher too, based on data from an anaerobic digester plant from Anyang City, Korea, which process MSW [RISE-AT 1998]. Then, the volume of each anaerobic digester should be approximately 10,500 m$^3$. However, this is a large volume, the proposed plant would have two anaerobic digester tanks with 5,000 m$^3$ for each mixing tank. The total amount of anaerobic tanks required is 15.

Usually the volume of biogas produced varies between 0.6 m$^3$/kg solids to 0.9 m$^3$/kg solids [Hickman Jr. 1999]. In addition, considering that the solids entering in the anaerobic digester are 450 metric tons of feedstock (it is being considered just the mass of BSW), and that usually between 40% and 60% of the organic matter present is converted to biogas [European Commission 2004], the volume of biogas generated in the proposed plant is 135,000 m$^3$, since:

\[ (225,000 \text{ kg BSW}) (0.6 \text{ m}^3/\text{kg}) = 135,000 \text{ m}^3 \text{ biogas/day}, \text{ considering 50% of the BSW.} \]

Considering that the calorific value of biogas is 20.5 MJ/m$^3$ [Hickman Jr. 1999], and the lowest reported generation rate is 0.6 m$^3$/kg, then the resulting potential calorific value of the waste is 12.3 MJ/kg.

Therefore the power generation of the proposed plant is 768,750 kWh, since:

\[ (12.3 \text{ MJ/kg})(225,000 \text{ kg BSW}) = 2,767,500 \text{ MJ/day} = 768,750 \text{ kWh/day}. \]

However, assuming that the efficiency of the generator is 25% [European Commission 2004]; so, about 190,000 kWh is produced per day.

In the proposed composting plant, part of the biogas would be stored in spherical tanks of 5,000 m$^3$, to satisfy the demand of energy and steam of the composting plant. Remaining may be sold to an electricity power company by the City of Milwaukee.

It is considered that 50% of mass of the feedstock would be used to generate biogas and the other 50% would generate compost and liquid fertilizer. Therefore, the proposed composting plant will generate about nine metric tons of compost per day and 190 metric tons of liquid fertilizer.

In the Anyang City, Korea, the anaerobic digester plant produces 230 m$^3$ of biogas per day, 100 kg compost, and two metric tons of liquid fertilizer from three metric tons of food waste [RISE-AT 1998].

If three metric tons of waste generates 230 m$^3$ biogas, the 563 metric tons of waste from the proposed Milwaukee plant would generate 43,000 m$^3$ biogas, if the process was exactly the same. However, it would probably generate 135,000 m$^3$, because the feedstock has different composition. Therefore, in the Korean
plant the generated volume is proportional to only 9% of the feedstock of the proposed Milwaukee plant.

In order to estimate the generation for the proposed plant, the proportion of the generation of each product (compost, biogas, and liquid fertilizer) was calculated assuming that in the proposed plant 50% feedstock would be used to generate biogas.

3.4 Vermicomposting

It is necessary to have 45 kg of worms to process 34 kg of feedstock. Therefore, the proposed plant would require 6.8 tons of worms.

It is reported that the residence time in a vermicomposting system is about 30 days if this is the only composting step in the process [Dominguez et al. 2000]. However, considering that when the product leaves the anaerobic digester, it is already a soil conditioner [RISE-AT 1998], the vermicomposting reactor will just refine the compost. Therefore, it is estimated that only 10 days of residence time should be enough.

Because Milwaukee has a cold winter, reaching temperatures of about minus 20°C ± 5°C, it is necessary to insulate the vermicomposting tanks with polyurethane, in order to keep the temperature of the tank between 13°C [Holmstrup 2004] and 29°C [Holmstrup 2004, Pittaway 2001], which is the best temperature for worms. In addition, the reactors should be preferably in a building (indoor system) insulated too.

The supernatant storage tank should store at least the production of one day. Therefore, the storage tank should have the capacity of about 200 metric tons of liquid fertilizer. Considering the density of liquid fertilizer as the density of water (1 metric ton/m³), the minimum volume of the tank is 200 m³. This is assuming that the part of the liquid fertilizer be fed back to the mixing tank.

This proposed process would allow obtaining a faster composting process since the residence time in the mixing tank and in the anaerobic digester is seven days, and, therefore, the total time is 14 days. At this point, the resulting product is a soil conditioner. It means that probably only 10 days of residence in the vermicomposting reactor (instead of the usual 30 days) will generate the finished compost.

4 CONCLUDING REMARKS

Composting and anaerobic digestion are multivariable processes, based on microbiological, chemical, physical, and thermodynamics fundamentals. They have several variations; they may be operated in open or closed reactors. They represent intelligent options for processing MSW, converting organic matter into soil amendment, managing dwindling landfill space, and be a potential source of renewable energy.

As a solution to efficiently treat MSW and obtain compost and energy at the same time, it is proposed to have a plant with anaerobic digester followed by vermicomposting. The compost obtained by anaerobic digester has the same quality of the compost obtained by aerobic digester, with the advantage that the process generates liquid fertilizer (which can be marketed as a soil conditioner) and biogas (which can be used to provide energy and steam to the plant and also sell the gas).

A vermicomposting reactor would follow an anaerobic digester reactor. It is probable that it is not necessary to blend the compost with other minerals to complement its profile. In addition, the combination of these two unit operations would allow reducing significantly the total process time.

It is proposed to co-compost MSW, which is a major concern worldwide, with sewage sludge, adding wood ash, fly ash, lime-kiln dust, and/or limestone quarry dust to reduce the generation of organic sulfur from the anaerobic digester.

REFERENCES


Innovative use of clay backfill at the new Wembley Stadium, UK

S.W. Carley, A.S. O’Brien, F.A. Loveridge & Y.S. Hsu
Mott MacDonald, Croydon, Surrey, United Kingdom

ABSTRACT: Granular materials, sourced from quarried natural aggregates, are usually specified as backfill to retaining walls. These natural materials are becoming a scarce and expensive resource, particularly in Southern England. If used for urban projects, they will inevitably lead to additional construction traffic, with associated environmental damage. At Wembley Stadium, the use of clay backfill, derived from the on-site excavations, provided significant environmental benefits and a sustainable solution. Approximately 130,000 cubic metres of clay backfill was placed behind different types of retaining wall, varying from large embedded bored pile walls up to ten metres high (both propped and anchored) to small L-shaped walls. In order to verify the acceptability of the use of clay backfill, which can lead to large swelling pressures developing on the back of retaining walls, novel investigation and analysis techniques were necessary. This included specialist laboratory testing, an earthworks trial, and non-linear numerical analyses (using the FLAC code). The use of clay backfill is compared to the use of granular backfill in the context of sustainability and a whole life cost approach to engineering solutions. Social, environmental and economic issues are considered in order to provide a quantitative assessment rather than a simple qualitative assessment of the sustainable nature of the resulting scheme, taking into account any alterations or additional work required to accommodate the use of the clay backfill.

1 INTRODUCTION

1.1 Traditional construction

Granular materials, sourced from quarried natural aggregates, are usually specified as backfill to retaining walls. These natural materials are becoming a scarce and expensive resource, particularly in southern England. If used for urban projects, they will inevitably lead to additional construction traffic, with associated environmental damage.

However, there is both limited experience and research with regards to the alternative, using clay as backfill to retaining walls. Previous research undertaken by the Transport Research Laboratory (TRL) in the United Kingdom indicated that the key issues with the use of clay fill are swelling pressures developed within the backfill and ground movements, Clayton et al. (1989). Further work undertaken by the TRL on the swelling pressures of clay fill confirmed that the swelling pressures generated are sensitive to the moisture content of the clay; O’Connor & Taylor (1994). Therefore, due to the limited research in this area, in order to use clay as backfill to the retaining walls, additional investigation and more advanced analysis is required than if a granular material is used.

1.2 Wembley stadium

The new Wembley Stadium required the use of approximately 130,000 cubic metres of backfill to large retaining walls, which were necessary due to the larger footprint of the new stadium. There are no local sources of granular fill. In addition the local road network already suffered from congestion, even without the additional traffic generated during construction.

The excavation work required at Wembley would produce large volumes of natural clay (weathered London Clay), and therefore it was decided that the preferred option would be to use this material as the backfill to the retaining walls. This fill was placed behind different types of retaining wall, varying from large embedded bored pile walls up to ten metres high (both propped and anchored) to small L-shaped walls. The use of this clay required additional investigation and analysis, as described in section 2. This paper describes a quantitative assessment of the use of clay fill, rather than conventional granular fill.
2 INVESTIGATION AND ANALYSIS

2.1 Investigation

In order to assess the feasibility of using the site won clay material as backfill to the retaining walls, additional investigation was required. A trial embankment was constructed to obtain site-specific data for Wembley. From this trial the following information was obtained:

- Identify practical and cost-effective compaction procedures and equipment
- Enable block samples to be taken in order to carry out specialist triaxial stress path tests on the clay fill
- Enable in-situ suction tests to be undertaken to assess the influence of compaction on the clay’s stress state

In addition routine acceptability index testing was undertaken including tests such as moisture content, grading and chemical testing. This was to determine both the suitability of the material for backfill, and to assist with the additional numerical modelling required. The weathered London Clay’s moisture content was higher than the plastic limit, which means that the potential swelling pressures are far smaller than ‘drier’ unweathered London Clay, Clayton et al. (1989). This means it is more suitable for reuse as retaining wall backfill than the over-consolidated clays that underlie many parts of south-eastern England.

During placement of fill further quality control testing was specified to verify the expected behaviour and properties of the fill. Advanced testing and a compaction trial would not have been necessary for the granular fill option, however it is likely that a substantial amount of routine testing would have been carried out on the first few layers to verify the fill placement.

2.2 Numerical modelling

Using the data obtained from the trial embankment and associated testing, numerical modelling was carried out using the FLAC software. Two types of wall were initially considered: an L-shaped reinforced concrete wall and a contiguous bored pile embedded retaining wall supported by dead man anchors. At conceptual stage, other forms of wall were considered, however these all required some form of propping, increasing the wall stiffness. It is known that this causes difficulties in design with regards to the swelling pressures generated, and therefore further analysis of these wall types was not undertaken. Typical cross-sections through the two types of wall are shown in Figures 1 and 2. Analyses were also undertaken for larger retaining walls with several levels of dead man anchors.

The clay backfill stress-strain data used for the analyses is based upon the laboratory data shown in Figure 3.

The horizontal total stress (swelling pressures) acting on the two types of wall are quite different, with the L wall swelling pressures being typically about 50% higher than for the anchored wall (varying...
between maximum values of 100 kN/m² and 65 kN/m² for the L wall and anchored wall respectively). This is due to the restraint provided by the L wall heel being greater than that provided by the dead man anchors, which are relatively flexible. Additional analyses for a ‘rigid’ wall indicated a maximum swelling pressure equivalent to the initial soil suction (or mean effective stress), in this case equal to 120 kN/m². Hence, wall flexibility and movement is highly beneficial, by reducing the swelling pressures acting on the wall and associated wall bending moments and shear forces.

Some variability in clay backfill swelling behaviour is inevitable due to variations in compaction and the ‘parent’ natural clay leading to variations in normalised stiffness and suction. Therefore, it is essential to verify that the movement and forces on a retaining structure are not unduly sensitive to variations in clay fill behaviour. The analysis showed that for flexible anchored walls the maximum wall displacement only varied by about ±15% for changes in suction of ±50%. In contrast, a propped wall would be overly sensitive to changes in clay behaviour. The props act as local ‘hard’ points on the wall leading to the development of locally very high swelling pressures, which would vary in direct proportion to the initial suctions.

From this analysis, it was determined that the use of clay as backfill to the retaining walls at Wembley Stadium was feasible. However, for the anchored contiguous bored pile wall approximately a third more anchors were required than would have been expected for a granular backfill design.

3 QUANTIFYING SUSTAINABILITY

3.1 Introduction

There are over 200 definitions of sustainable development, Parkin et al. (2003), however there is currently no consensus with regards to a definition applicable to the built environment, Bartlett & Guthrie (2005). The definition suggested by Brundtland (1987) is still widely used, which states that sustainable development ‘meets the needs of the present without compromising the ability of future generations to meet their own needs’. An alternative definition is provided by the UK government, which outlines four key objectives, DETR (1999):

1 Prudent use of natural resources
2 Effective protection of the environment
3 Social progress that recognises the needs of all individuals

However, sustainable development is normally broken down into three categories: economic, environmental and social impact. These three criteria form the triangle of eco-economics, presented by Barbier et al. (1992), or the ‘triple bottom line’.

There are various discussions within the literature as to how to assess the sustainability of any given project. However, a definitive methodology has yet to be developed. This is partly due to the project-specific nature of many of the variables that are to be quantified. Furthermore, a procedure for combining the comparisons for all three criteria is even more difficult to define due to the lack of a common variable to use.

Many of the documents currently available in the public domain which address the implementation and assessment of sustainable development focus on actions which are predominantly concerned with environmental issues, Bartlett & Guthrie (2005). The guidelines provided by the BRE (1999) provide a methodology for assessing the environmental impact of a project in terms of both embodied energy and emissions. These are both important variables to consider. It is estimated that the energy embodied in new construction and renovation each year accounts for approximately 10% of the UK energy consumption, Sustainable Homes (1999). However, even considering a variable such as embodied energy which is widely used when considering sustainability, there is a lack of consensus on values within the literature due to differing methods of calculation.

The social impact of civil engineering projects can be considered as being issues such as health and safety, traffic, employment, noise and vibration. The majority of these are difficult to quantify as shown by the lack of guidelines in current literature. More research into this area is therefore required before one overall methodology for quantifying sustainability is agreed.

There is therefore no current methodology providing a unified weighting system across all three criteria associated with sustainable development. For this particular case study, in order to quantify the sustainability of the clay backfill scheme, it has been compared to the more traditional use of a granular backfill using the three basic criteria of the ‘triple bottom line’, which in this instance is defined as the cost of the scheme, the social impact on local residents in terms of lorry movements and the environmental impact of the scheme.

3.2 Backfill options

3.2.1 Option 1: granular backfill

It is likely that this option would have consisted of a core of Class 1A material, with Class 6N being used in the active wedge zone adjacent to the walls, as specified in HA (1998). The material would have been imported fill. In terms of placement of the granular material, following the Highways Agency Specification (Series 600), HA (1998), Class 1A material is placed to a layer thickness and number of
passes dependent on the type of plant used, whereas Class 6N is placed with an end-product specification. For ease of construction it is likely that both the Class 6N and 1A would be placed in the same layer thickness to the same specification.

For comparison purposes it has been assumed that the granular fill would have been placed in 450 mm thick layers (as for the clay option), requiring 8 passes of a Bomag BW 226 DH-4 single drum vibratory roller, which has equivalent engine power to the roller used for compacting the clay.

3.2.2 Option 2: clay backfill
The general construction sequence for the cut and fill operation is outlined below. The principle is to avoid stockpiling of the clay and move the clay direct from the cut to the fill operation, which will minimise any potential for wetting of the fill.

1 Excavation using a 20 tonne 360° hydraulic excavator.
2 ‘Processed material’ loaded into 30 tonne dumptrucks using a 40 tonne 360° hydraulic excavator.
3 Material transported to fill area in the 30 tonne dumptrucks.
4 Fill spread across filling surface using laser controlled blade.
5 Compaction with Bomag BW 225 PD-3 padfoot.

Once the cut face had advanced far enough, the processed material was moved directly by excavators, removing the need for the use of the 30 tonne dumptrucks.

A specification for the placement of the clay backfill was produced by the earthworks contractor, following the compaction trial in December 2002. The compaction method adopted was 450 mm (compacted) thick layers, compacted using 12 passes of the Bomag BW 225 PD-3 roller, in deadweight mode.

This option requires 1.5 times as many passes as the granular fill option.

3.3 Economic impact
This criterion is relatively easy to assess in that the variable for comparison is defined. However, although costs of schemes can be readily estimated, in order to compare them not just with each other on one particular scheme, but with other projects, a normalisation factor needs to be determined. This has been considered in section 4.

In order to fully quantify the cost of each option, the whole life cost must be considered. This includes any costs prior to construction such as investigation and analysis, construction costs and any costs subsequent to construction. These latter costs will include measures such as monitoring of the walls to verify their behaviour and any anticipated maintenance measures during the lifetime of the structure. It should be noted that decommissioning of the stadium has not been considered.

3.3.1 Investigation costs
In order to verify the suitability of clay as a backfill material, additional investigation was undertaken prior to analysis and construction. The cost of the compaction trial and advanced acceptability testing has been estimated using data from the earthworks report produced by the earthworks contractor.

It has been assumed for comparison purposes that the cost of testing during placement for both types of material will have been very similar and these have therefore not been considered.

3.3.2 Analysis costs
Due to the more complex nature of the modelling undertaken for a scheme using clay backfill as opposed to granular backfill, the cost of analysis was twice as much as what would have been required using traditional materials. Smaller projects would probably require a disproportionate increase in this ratio for investigation and analysis.

3.3.3 Construction costs
There are several costs that need to be considered during the construction phase of the project.

3.3.3.1 Material cost
In April 2002, the Government introduced an aggregates levy of £1.60 per whole tonne in order to promote the use of sustainable construction materials, HM Customs and Excise (2004a). As the backfill to the retaining walls at Wembley Stadium occurred during March to June 2003, this levy would have had an impact on the cost of Option 1. Based on information from the British Aggregates Society (2001) and SPONS (2004), the cost of the two types of granular material in 2003 are assumed to be as follows:

- Class 6N: £22/m³
- Class 1A: £18/m³

It should be noted that these figures include for transportation to site and the aggregates levy.

As discussed in section 3.1.1, Class 6N fill will only be placed in the active wedge zone behind the walls. Over the whole scheme this has been calculated to be approximately 30% of the total backfill required.

The cost of the clay backfill is simply due to the manpower and plant hire required to excavate the material and move to the point of fill. Figures from SPONS (2004) have been used to estimate the cost of these items, and the cost for the clay backfill has been estimated to be approximately 10% of the cost of granular backfill.
3.3.2 Labour and plant costs
Due to the increased length of time required to compact the clay fill as opposed to the granular fill, there are greater labour and plant costs associated with the use of clay fill. There will also be resulting fuel costs. From the site records it took 13 weeks for the complete cut and fill operation, of which approximately 50% was spent placing the fill. Assuming the use of four rollers to compact the fill (filling undertaken in two areas concurrently), the costs of placing the fill can be calculated. As the granular option requires two thirds of the number of passes per layer of fill placed, the granular option costs would be about two thirds of the clay option.

3.3.3 Total construction cost
It should be noted that these costs do not currently include those associated with the retaining walls themselves. The cost of the piles and pre-cast walls would be similar for each option. However, where a piled wall was used, anchors were required, and the use of clay backfill resulted in approximately a third more anchors. This additional cost, and environmental and social impact associated with it should be analysed to provide a more complete appraisal of the sustainable nature of the chosen option.

Using the methodology outlined in previous sections, the construction cost associated with the placing of the backfill can be compared for the two schemes. The cost of the clay backfill option is approximately 15% of the granular backfill option. Disposal of material and the costs associated with this operation are considered in section 3.3.3.

3.3.4 Post-construction costs
There are also costs associated with the clay backfill option in particular after construction has been completed.

3.3.4.1 Monitoring costs
Due to the innovative approach regarding the use of clay as backfill to retaining walls, additional monitoring post-construction was commissioned to verify the performance of the walls. This cost is relatively small (less than 5%) compared to the actual cost of construction itself, and therefore hasn’t been considered any further.

3.3.4.2 Maintenance costs
It is anticipated that some settlement of the clay backfill will occur throughout the life of Wembley Stadium. The design life for the project was 60 years. From the results of the numerical modelling, it is estimated that there may be about 100 mm of settlement behind the retaining walls. Hence, some additional maintenance to pavements, etc, may be required. In addition to the cost of these, there will also be associated environmental and social impacts related to the maintenance.

3.4 Environmental impact
To assess and compare the environmental aspects of both schemes, there are various issues to consider.

3.4.1 Embodied energy
The embodied energy of a material or product is not the energy available or inherent, but can be defined as the ‘Cumulative Energy Demand’ i.e. it is the sum of all the energy inputs into a product system from all included stages of the life cycle. The system boundaries used to determine the exact nature of the life cycle stages can vary from analysis to analysis, making it difficult to compare studies. The uncertainties and variations in embodied energy analysis are often due to the methodology used. Examples of the variations and uncertainties are:

- The energy used to transport a material is difficult to evaluate, and is site-specific
- A consistent methodology to assess the effect of recycling on embodied energy has yet to be established
- Evaluating the energy associated with installing substructures is generally uncertain due to the fact that it relies on factors such as ground conditions and site practices that are site-specific
Different countries will have differing energy values for the same material, dependent on the natural resources available. Values may even vary considerably within a country.

In the context of the two schemes considered at Wembley Stadium, the embodied energy of the construction materials, the fuel consumption on site and the fuel consumption associated with transport to site will be considered.

3.4.1.1 Construction materials
The embodied energy associated with a granular fill material has been determined following a literature review. As can be seen in Table 1, values ranged from 0.03 to 0.3 GJ/tonne. It is thought that this large range of values is partly a result of earlier, more rudimentary, attempts at quantifying the embodied energy of construction materials. Based on more recent data, for the purposes of this study a value of 0.1 GJ/tonne has been assumed. It should be noted that this value does not include any embodied energy associated with transport to site, and this will need to be calculated separately, as discussed in section 3.3.1.2.

In order to assess the embodied energy of a clay fill material, a site-specific study has been undertaken. The energy associated with excavating the material has been determined based on site records. This was found to be 0.0011 GJ/tonne, i.e. about 1% of that of the granular option. This is largely due to the fact that there is no processing or storage associated with the clay fill option.

3.4.1.2 Fuel consumption during transportation to and from site
It is important to have a method of calculating fuel use for each scheme. Embodied energy values are generally derived using system boundaries that mean that transport of raw materials to point of processing is included. They do not generally include transport from the final point of processing or production, to the site itself. The transport methodology that will be used to calculate fuel use is outlined in the following sections and is based on the guidelines in BRE (1999).

It is likely that the aggregate would be transported largely by road, although this is dependent on the source of the aggregate. The following formula can be used to calculate the fuel used to transport materials by road:

\[
\text{Fuel used} = \left( \text{no. of deliveries} \times (\text{distance travelled}) \times (\text{part Load} \%) \times (\text{fuel consumption}) \right)
\]

The number of deliveries can be calculated from data provided. The distance travelled is dependent on whether the return trip is full or empty. If empty, the total distance travelled per trip is twice the delivery distance. From given data it can be estimated if there will be any part loads being transported. The fuel consumption can be taken from the DETR UK fuel statistics for each class of vehicle and converted to litres/km, BRE (1999).

Once the litres of fuel required have been calculated, the embodied energy associated with this can be calculated using published data. From the figures for embodied energy of diesel gathered during research, the value ranged from 37.5 to 45.8 GJ/tonne, see Table 2. The higher value of 45.8 GJ/tonne has been taken, as this was the only data source available within the United Kingdom.

(1) Granular Option
The transport considered for this option is that from the quarry to the site. There are various possible sources of appropriate material in the UK, which based on the ESRSA tool, AggRegain (2005), for a site in London are:

1. Blast furnace slag (BFS)
2. Land won: London
3. Igneous rock: East Midlands
4. Limestone/dolomite: South West
5. Steel slag, electric arc furnace (EAF) and basic oxygen furnace (BOF)
6. Land won: South East
7. Marine dredged: London

### Table 1. Embodied energy of granular material.

<table>
<thead>
<tr>
<th>Material</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural aggregate</td>
<td>0.03</td>
<td>0.12</td>
<td>0.29</td>
<td>0.1</td>
<td>0.3</td>
</tr>
<tr>
<td>Sand</td>
<td>0.006</td>
<td>0.1</td>
<td>0.04</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crushed stone</td>
<td>0.054</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*References for each embodied energy value are as follows:
1. Ecosite (1994)
2. Glover (2001)

### Table 2. Embodied energy of diesel.

<table>
<thead>
<tr>
<th>Material</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diesel</td>
<td>37.5</td>
<td>39.2</td>
<td>45.8</td>
</tr>
</tbody>
</table>

*References for each figure are as follows:
2. BRE (1999)
All of these sources will have very different transportation requirements. Therefore as a base case the transport to site for this option has been calculated assuming a quarry which is 50 miles from site, and that all transport is by road. Due to the large volumes of material required for this project, this is likely to be a minimum distance travelled, and the actual distance could have been up to double this figure. For the volumes of material required it is considered that recycled products and land won/marine dredged in London and the South East would not have been available.

Conservatively it has been assumed that the return trip is empty, however it is possible that this trip could have been used to take waste to a landfill (the unused clay). In addition it has been assumed that a 33 tonne truck has been used, and the relevant fuel consumption rate utilised in calculations.

(2) Clay Option
The transport associated with this option is simply from the point of winning the material to the point of filling. As noted in the construction sequence listed in 3.1.2 not all fill will be transported using the 30 tonne dumptrucks. An average distance travelled is assumed to be about 125 m. It is assumed that the return trip is empty and so total distance travelled per trip is 250 m. It is assumed all loads are full. The amount of fuel used per week and therefore the amount of embodied energy can then be calculated.

3.4.1.3 Fuel consumption during construction
During construction for both schemes, the use of the rollers to compact the fill will require the use of diesel, and therefore there will be an associated embodied energy. Using data for the fuel consumption from SPONS (2004), and based on the compaction requirements outlined in section 3.1, the embodied energy during construction can be calculated.

3.4.1.4 Total embodied energy
Using the methodology outlined in previous sections, the embodied energy associated with the placement of the backfill can be compared for the two schemes, as shown in Figure 5. The embodied energy of the clay backfill option is less than 2% of the granular backfill option. Disposal of material and the embodied energy associated with this operation are considered in section 3.3.3.

The exact figures shown in the chart are for comparison purposes only, and indicate a relative ranking.

3.4.2 Emissions
There are emissions associated with all stages of construction, from extraction of raw materials to the transportation to site and installation of these materials. This needs to be quantified for each of the backfill options. It is important to consider emissions for a number of reasons. Manufacture and transportation of construction materials releases CO₂, CH₄, CO, Volatile Organic Compounds (VOCs), Non-Methane Volatile Organic Compounds (NMVOCs), NOₓ, SO₂ and PM₁₀ (particulates that are less than 10 microns in size). It is possible for the VOCs and NOₓ to react in sunlight and produce ozone at ground level where it is toxic to humans, plants and animals. Also, NOₓ and SO₂ are acidic in combination with rainwater. The particulates released are extremely harmful to the lungs, causing diseases such as silicosis and other respiratory problems. Also, particulates can combine with SO₂ in affecting the lungs.

Pollution is also important to consider as it crosses borders. The effects of water pollution cross frontiers, and air pollution is very much a trans-boundary phenomena. The effects of acid deposition are felt many thousands of miles from their country of origin. Gaseous emissions have global effects.

Using figures from various published sources, it is possible to calculate the amount in kg of each gas and particulate emitted due to particular processes such as transport, quarrying of material and placement of backfill. However, simply calculating the amount of each particular gas emitted for each system is not necessarily the best way to compare the two backfill options, and to assess their affect on the environment. Suggested categories of comparison that take into account environmental impact are listed below:

1 Global Warming Potential (GWP): This concept compares the ability of each greenhouse gas to trap heat in the atmosphere relative to another gas. IPCC
guidelines have chosen CO₂ to be the reference gas. Relevant gases that have a GWP value are CH₄, CO₂ and N₂O. There are no figures for the criteria pollutants such as CO, NMVOCs and SO₂ as there is no agreed method to estimate the contribution of gases that have only indirect effects on the greenhouse effect. GWP values have been found to be consistent across a wide variety of sources.

2 Acid Deposition: This concept has been developed so that it is possible to compare the contribution of various emissions to acid deposition. The chosen reference gas is SO₂. Relevant factors that have an effect are SO₂, NO₂ and NOₓ.

3 Human Toxicity: This concept assesses the danger to humans of the various emissions, and is measured in kg toxicity.

4 Photochemical Ozone Creation Potential: This concept quantifies the likelihood of various emissions to creating photochemical ozone in the atmosphere. Relevant emissions that have an effect are VOCs and NMVOCs. The reference chemical for this concept is ethane i.e. it is measured in kg of ethane equivalent.

5 Eutrophication: This quantifies the effect of various emissions on water. Eutrophication (or algal bloom) of water supplies leads to a loss of biodiversity through over-enrichment, increasing mortality of aquatic fauna and flora. The reference chemical for this measure of environmental harm is phosphate (PO₄), and as such conversion factors exist for equating the relevant chemicals to kg phosphate equivalent.

All conversion factors are based on those suggested in the BRE Methodology, BRE (1999). The exact figures shown in the following charts are for comparison purposes only, and indicate a relative ranking.

3.4.2.1 Emissions due to transportation
From the values of embodied energy associated with transportation (see section 3.3.1.2), upstream and combustion emissions due to the fuel used can be calculated using figures from the NETCEN National Atmospheric Emissions Inventory. The comparison between the two options is shown in Table 3 and on Figure 6.

3.4.2.2 Emissions due to material production
There will also be emissions associated with production of the materials themselves. Those associated with excavation of the clay material at the Wembley site would have been produced regardless of the scheme chosen, and could be estimated from the project data. Emissions associated with the production of the granular material are more difficult to quantify, as they are dependent on the method used for extraction. This comparison has therefore currently not been considered any further.

Table 3. Emissions of CO₂ equivalent.

<table>
<thead>
<tr>
<th>Emission</th>
<th>Backfill option</th>
</tr>
</thead>
<tbody>
<tr>
<td>grams</td>
<td>Option 1:</td>
</tr>
<tr>
<td></td>
<td>granular backfill</td>
</tr>
<tr>
<td>CO₂ eq.</td>
<td>1428</td>
</tr>
</tbody>
</table>

Figure 6. Emissions generated during transportation to point of fill.

3.4.3 Waste
Waste is becoming increasingly important to consider, particularly following recent governmental legislation such as the Landfill Tax, which was introduced in 1996. As well as considering the economic impact of waste, the environmental impact needs to be considered. The UK construction industry produces approximately 60 million tonnes of waste a year, Griffiths et al. (2002).

Due to the granular backfill option requiring backfill to be imported to site, all the clay material ‘cut’ and used in Option 2 would need to be disposed of in Option 1. There are opportunities for the clay material to be sold to industries such as the landfill industry to be used as the capping layer on landfill. However, it is extremely unlikely that this volume of material would have been able to be disposed of in this manner. Therefore a large proportion of it, if not all, would
have had to be treated as waste. For the purposes of this comparison, it has been assumed that 80% of the waste will need to be disposed of.

As well as the production of waste being an environmental concern, there are all the associated side effects that need to be considered such as transportation to landfill and the emissions this causes. In turn this has social consequences due to lorries passing through residential areas and the effect on the environment local to the landfill, and the knock-on effect of filling up existing landfill leading to the need for new landfill.

### 3.4.3.1 Cost of waste disposal
This cost has been estimated assuming a distance of 18 miles to the nearest landfill site, Environment Agency (2005) and the use of 25 tonne lorries. Landfill tax for inactive material has been applied. The cost for disposal is calculated as being approximately £2.5 million (to the nearest hundred thousand).

### 3.4.3.2 Embodied energy of waste disposal
The embodied energy associated with waste disposal is as a result of the fuel used to transport the material. The methodology outlined in section 3.3.1.2 has been followed. The embodied energy was calculated to be 7500 GJ.

### 3.4.3.3 Emissions of waste disposal
As in section 3.3.3.2, the emissions associated with waste disposal are as a result of the fuel used to transport the unwanted clay to landfill. The methodology outlined in section 3.3.2 has been followed. The associated emissions are shown in Figure 7, with the exception of the CO2 equivalent, which was found to be 429 grams.

### 3.5 Social impact
The social impact of sustainable development can be described as how the development serves the needs and quality of life of people and their communities. It is perhaps the most difficult of the three elements of the triangle of eco-economics to quantify. In this comparison, the main social impact is the number of deliveries to site. As discussed in section 1.2, this was a key consideration when assessing the backfill options at Wembley Stadium.

An additional impact is the length of programme for the construction work. This can be compared considering just the length of time associated with the backfill operation, however as this operation was just a small part of a much larger project, the programme implications are considered relatively small for this particular case study.

#### 3.5.1 Delivery of imported granular fill
In Option 1, a total of 127,400 m$^3$ of granular fill is required to be delivered to site. Assuming delivery is in 33 tonne lorries, as for previous analysis, this equates to a total of 7722 lorries of fill.

#### 3.5.2 Disposal of waste
In Option 1, a total of 127,400 m$^3$ of clay material needs to be transported to landfill. Assuming the use of 25 tonne lorries as in the previous analysis, this equates to 10192 deliveries. If larger lorries are used as for the importing of fill, then 7722 lorries would be required.

#### 3.5.3 Construction programme
As noted in section 3.1, the length of time taken to place the fill for the granular option would have been about two thirds of that required for the clay option. This would have resulted in a construction programme of approximately 9 weeks, as opposed to 13 weeks.

It should also be noted that the weather is an important factor in the placement of clay material as backfill. The backfill was placed during the spring and summer of 2003 at Wembley Stadium, which is the optimum time to carry out the operation based on the amount of rainfall generally expected in these months. If the backfill operation had been scheduled to take place during the winter, it is likely that there would have been delays to the anticipated programme due to heavy rainfall. The length of time taken to place the fill could easily double in such circumstances. The impact of the weather on the placement of granular fill would not be significant.

#### 4 Comparison Summary
In section 3 the two options for backfill to the retaining walls have been compared using numerous variables: cost, embodied energy, emissions, number of deliveries and programme. This is summarised in Table 4. The cost figures do not include maintenance or monitoring costs. With the exception of programme, the use of the
clay material scores significantly better than the granular fill option. Therefore it is relatively straightforward to conclude that the clay option is a more sustainable method of construction than the use of imported granular fill.

However, when undertaking attempts to quantify the sustainability of schemes, it may not always be as clear which option is considered to be the most sustainable. If one option scores better than the other in one category, but is less favourable in the other two, how do we assess which scheme overall best promotes sustainable development? In order to do this there needs to be a weighting scheme and a common variable across all three criteria, as previously noted in section 3.1.

One available method for comparing the environmental impacts on a single scale or score is that of UK Ecopoints, Dickie & Howard (2000). A UK Ecopoint is a single unit measurement of environmental impact, which is available across thirteen categories such as climate change and minerals extraction. Impacts are then related to the environmental impact of an average UK citizen over one year, and Ecopoints awarded. However, this system is only valid for environmental impacts, and therefore cannot be used to compare two schemes across all three criteria, as previously noted in section 3.1.

One available method for comparing the environmental impacts on a single scale or score is that of UK Ecopoints, Dickie & Howard (2000). A UK Ecopoint is a single unit measurement of environmental impact, which is available across thirteen categories such as climate change and minerals extraction. Impacts are then related to the environmental impact of an average UK citizen over one year, and Ecopoints awarded. However, this system is only valid for environmental impacts, and therefore cannot be used to compare two schemes across all three criteria.

5 GENERAL OBSERVATIONS

As discussed in section 4, when considering cost, embodied energy and emissions, the clay backfill option has been shown to be more sustainable than the use of granular material at the new Wembley Stadium. Therefore, although variations would be expected between different sites it would normally be anticipated that reuse of a readily available clay fill would be a more sustainable option than imported granular fill. However, there are significant technical constraints and risks which need to be carefully managed, before clay fills are used as structural backfill:

1 Clay backfill should only be used behind relatively flexible retaining walls, e.g. rigid props or stiff shear keys, large diameter piles, etc, should be avoided

2 The clay fill behaviour will be sensitive to placement and compaction procedures, as well as its moisture content. Good site control and uniform compaction is necessary. Backfilling in small confined areas would be inappropriate

3 Swelling pressures will be sensitive to both the state of the clay fill and soil structure interaction

6 CONCLUSIONS

The use of the site won weathered London Clay, rather than importing granular fill, produced major environmental benefits and benefits to the local community. This innovation introduced additional project risks, which were minimised by carrying out earthworks trials and the design of ‘flexible’ retaining structures. Sophisticated laboratory testing and non-linear numerical modelling played an important role in checking that the conceptual design was appropriate.

In terms of sustainable development, the analysis undertaken conclusively shows that the use of a clay backfill material was a more sustainable solution than the use of a granular backfill. This was evident across the three criteria of the triangle of eco-economics. The over-riding reason for this being the site-won nature of the clay material, significantly reducing the number of deliveries required to site and therefore the embodied energy and cost of the solution.

However, there is still the need for additional research and thought with regards to the objective comparison of such case studies. Without this it is not possible to compare different civil engineering projects, and it may be more difficult to assess different options on the same project where the comparisons are not as straightforward as they have been for this particular case study.

However, one conclusion that can be made is that one of the most important criteria with regards to the sustainable nature of a civil engineering scheme will be the number of deliveries required, and the distance required for these deliveries. These have a direct impact on both energy and emissions, two of the measurable criteria with regards to environmental impact. They are also the biggest influence on many projects when considering the social impact. Therefore, it would normally be anticipated that reducing the required number of deliveries to site will lead to a more sustainable development.

REFERENCES


Glover, J. 2001. Which is better? Steel, concrete or wood: A comparison on three building materials in the housing sector. *Department of Chemical Engineering University of Sydney Fourth Year Thesis*.


A rationale for the production of Devon Cob to ensure performance

K.A. Coventry  
School of the Built Environment, Northumbria University, UK

J. Griffiths  
Plymouth University, UK

ABSTRACT: Construction using material made from raw earth may seem to lie in the domain of the historic or vernacular. However, using Devon Cob as an example of a viable earthen building material, this paper shows that raw earth offers a real and sustainable alternative to today’s more conventional construction materials. The viability of Devon Cob is shown through a clearly defined rationale which guides material selection, mix, and compaction methods. The adoption of this rationale within a rigorous laboratory test program produces a material of consistent load bearing capacity for a given sample of raw earth. This paper addresses the relationship between raw earth samples and highlights the importance of the earthen matrix to material performance. The pressure membrane test is introduced as a means of assessing the earthen matrix and classifying the fabric of Devon Cobs at a microstructural level. The ability to classify at this level offers a clear insight into the durability of this sustainable building material which has been proven against the historical background of its use. In exploring Devon Cob as a contemporary material, this paper concludes by explaining why material selection has determined those structures that endure today.

1 INTRODUCTION

Construction utilising materials such as bamboo, straw, natural stone and earth may seem to the majority of people to lie in the domain of the historic and vernacular. However, to some of the future constructors of our built environment these materials will offer obvious, more sustainable alternatives to the conventionally less sustainable options posed by fired clay bricks, concrete and steel as their potential is explored through the tertiary education system (Little and Morton, 2001; Walker, 2002).

Earthen construction is a generic description for one natural, sustainable, building technology encompassing many differing methods (Houben and Guillard, 1994). This paper is particularly concerned with one earthen construction material, namely Devon cob. Implementing natural building technologies within mainstream construction, using materials like Devon cob, can only be achieved by establishing standards and specification in order to allay concerns of proposers, procurers and financiers regarding suitability, quality and performance (Little and Morton, 2001). However, the durability and longevity of many of the existing traditional cob buildings cannot fail to impress a generation that discusses the design-life of buildings in time scales of but a few decades.

2 DEVON COB

Cob construction (or mud walling) is a monolithic building technique utilising earth mixed with straw and water. This matrix is stacked in layers with each layer compacted prior to the formation of the next and thus the process continues until a wall of the desired height is achieved.

However, on referring to traditional ‘Devon cob’ a very particular process of cob construction is suggested which identifies soil selection, formation strata, labour, resources and process. It is a tradition which has spanned between the fourteenth and nineteenth centuries (Beacham ibid. Keefe, 1998) and has been predominantly used within the county of Devon, in areas where the limited availability of suitable building stone forced the use of alternative building materials. Consequently half of the United Kingdom’s earthen buildings are believed to found in Devon (Gillilan, 1995).

The formation strata for Devon cob is traditionally a stone plinth (or pinning) of approximately 600 mm in height and 350 mm in depth. Once the plinth was built, preparation of the cob mix began adjacent to the selected site (Egeland, 1988). Thus soil selection for Devon cob was defined by the landscape of Devon and more specifically the citing of the new structure.
Operating in a gang of four men, sub-soil was removed from the ground and the larger stones were picked out. The earth would be placed in a heap and regularly turned-over by two men with picks or trodden by horses or cattle (Williams-Ellis & Eastwick-Field, 1947; McCann, 1983). Another man would be regularly adding water as the material was worked, while the fourth member of the gang was responsible for the addition of straw. Williams-Ellis and Eastwick-Field (1947), Brown (1979) and Egeland (1988) all testify to the use of barley straw.

On preparation of the cob matrix building can begin. An eloquent account of cob construction activity is given by the Reverend Coperinger Hill (McCann, 1983), “one man gets upon the pinning with a small three-tined fork; his partner throws up to him small lumps of clay, the size of a double fist, which he adroitly catches on the fork, and deposits smartly on the wall, walking backwards.” Once deposited, the cob matrix is treaded into place with any surplus material projecting from the sides, eventually pared off. Each layer (or perch) was constructed in this fashion and left, covered with straw, to dry whereupon subsequent layers could be constructed. According to Williams-Ellis and Eastwick-Field (1947), construction activity occurred between March to September to facilitate drying.

3 THE REVIVAL OF EARTHEN BUILDING CONSTRUCTION, THE REVIVAL OF COB

Talk of revival in earthen building techniques has been documented in Britain, since 1919, when St. Loe Strachey (1920) wrote of the building crisis faced by Britain due to a shortage of materials coupled by considerable demand for new housing after the First World War. Weller (1922), in his introduction to a special report commissioned by the Building Research Board into cob and pise de terre, cited the virtues of this form of construction, but viewed the suitability of earthen building techniques to address a social housing problem as implausible. L’isle D’Abeau, a sixty unit social housing project, has since been built near Lyon in France during the mid-nineteen eighties (Sinha & Schumann, 1994) representing a showcase of earth building technologies.

In 1947, St. Loe Strachey’s comments of 1920 re-accompanied the introduction to the re-print of “Building in Cob, Pise and Stabilised Earth”, the publication in which they first appeared (Williams Ellis et al, 1947). In this post- Second World War period, Britain was to face a similar situation to that experienced post World War I and earthen building techniques were again discussed as a potential solution. However, opportunities to promote cob construction were to remain stifled until 1985, for two reasons: the lack of a British Standard Code of Practice applicable to cob and the nature of the Bylaws and Building Regulations imposed by Local Government since 1858, to control construction activity (Ley and Widgery, 1997). Fortunately changes to these Bylaws and Regulations in 1985, paved the way for a potential revival of cob construction.

The changes to the Building Regulations were particularly timely given the rising concerns of energy consumption attributed to the development of our built environment (Brundtland Report, 1987; CERF Report, 1996). The total energy consumed in development considers the energy used in the extraction of raw materials for product production, the energy spent in product production and all associated transportation costs. The analysis and summation of these energies defines an ‘embodied energy assessment’. On consideration of the cob building process, the embodied energies associated with cob construction are unquestionably negligible when compared with more common forms of construction. Needless to say, the energy associated with cob building conservation is similarly low.

International efforts to conserve earthen architecture had gathered apace during the nineteen eighties (ICCRROM, 1987). Interest groups in the conservation of vernacular architecture and the conservation and repair of cob buildings, in particular, had formed (Keefe and Child, 2000). During the early nineteen nineties these groups, namely the Devon Earth Building Group and Plymouth University Centre for Earthen Architecture, focused on the dissemination of good practice and appropriate repair techniques in hope of arresting the escalating demise, in the structural integrity of existing cob buildings. A retired Devon cob building mason was also investing time disseminating the practice of cob construction to an apprentice, thus assuring the sustainability of this traditional, low-energy building technique for the future (Harrison, 1992).

Coincident to this period of local focus on cob, The United Nations Earth Summit in Rio de Janeiro, 1992, was being held. One outcome from this summit was a programme of action – Agenda 21- to support the implementation of international sustainability measures (United Nations, 1993). Agenda 21, outlined a holistic approach to creating a more sustainable future by encouraging a ‘global partnership’ to assume collective responsibility. Local Agenda 21 called for communities to engage at a local level in the sustainability debate and actively promote a climate of change. More specifically, the scientific and technological community was encouraged to develop codes of practice and guidelines in the pursuit of research and implementation focused towards low energy impact development while educators were asked to improve dissemination and skill transfer (United Nations, 1993).

It is much to the credit of Devon Earth Building Group and Plymouth University Centre for Earthen...
Architecture that their own objectives, focused towards training, dissemination and research, echoed the ideals and philosophy that underpinned international developments at that time. Fortunately work into other earthen building technologies is also progressing to this end (Walker, 1999; Mesbah et al 1999; Minke, 2000; Morel et al; 2000). This work is necessitated by a lack of standardisation in performance criteria (Houben and Guillard, 1994; Walker, 1999). Without this, loan institutions and investors are reluctant to solicit services readily extended to purchasers of developments built from contemporary building materials, due to anxieties concerning the recuperation of monies beyond the loan period. The density of historic cob construction in the South-West is testament to its viability and while new cob building work now progresses (Keefe and Child, 2000), reluctance to approve its use could be dissipated by definitive documentation on use and specification (Ley and Widgery, 1997). This documentation would not only promote sustainability for the developments to come, but would also ensure the sustainability of Devon’s cultural heritage of cob buildings.

4 THE STRUCTURAL UTILISATION OF DEVON COB

The use of Devon Cob as a building material is evidenced throughout the County of Devon and the longevity of many of the existing traditional cob buildings, as documented by Forde (2002), cannot fail to impress a generation of constructors that discusses the design life of buildings in terms of but a few decades. The primary mode of load transfer through a cob structure is in the form of compression forces through the structural walls. However the drying-out of cob also induces lateral stressing in the matrix as the soil shrinks but remains bound with the straw.

Practitioners (designers, engineers, and architects) utilise material capacity in order to achieve appropriate design. Thus quantification of the compressive strength of traditional Devon cob, qualified by material definition, is required to inform design at both the time of construction and subsequent stages of drying of the cob matrix. In the absence of a standard test specification for cob, the formation of an appropriate test methodology becomes an integral step towards realising the ultimate objective, to determine the quality of the cob mix.

4.1 Test methodology

The establishment of a rigorous test-methodology has been discussed, in detail by Coventry (2004). The ‘seven- blow Proctor’ (Coventry and Griffiths (in preparation)) is presented as a method specification for the production of the test cylinders achieving repeatable densities which reflect those attained by means of traditional and contemporary cob manufacture techniques.

In consideration of the loading environment used in cob construction, it was determined that unconfined compression testing would offer the most appropriate loading simulation within the laboratory; Figure 1 illustrates the test apparatus adopted.

Within the testing programme undertaken recom pacted soil and cob samples were formed into cylinders of 100 mm diameter and 200 mm in height. These were subject to uniaxial compression until failure occurred. Failure was defined as either the sample failed in shear and collapsed or strain deformation reached 10%. In addition to shear failure the strength of the sample at yield was also identified.

4.1.1 Mix proportions

The determination of how much straw should be added to the soil samples to form the cob matrix was informed by the availability of relevant literature. The Technical Panel of Devon Historic Building Trust (1992) recommends a straw content of 1.5% to 2.5% by weight. Goodhew (1993) deconstructed a single original cob sample to obtain a fibre content of 1.25% by weight of sample analysed. While this offers the best insight into the actual straw amounts originally

---

![Figure 1. Compressive test apparatus.](image-url)
adopted, consideration of the mixing methods employed in traditional cob construction does present the possibility of large variations in straw dispersal through the mix.

The potential for large variations of straw content within the cob matrix was highlighted by Greer (1996) through a compositional analysis of eight new cob bricks selected from a large quantity of similar bricks manufactured for use in a repair scheme to a traditional cob house. Skilled craftsmen, appointed to execute these repairs, adopted traditional building methods and techniques. The analysis of the cob bricks found that the straw content of the cob matrix, within these bricks, ranged from 1% to 2.5% of the total weight.

To facilitate comparisons between cob matrixes produced from different soils, the results from the cob samples produced in this paper consider the recommendations given by Saxton (1997). Saxton suggests that an optimal straw content for cob lies between 1.0% to 1.5%. The cob matrices presented in this paper are based on mix incorporating 1% straw, by weight.

4.1.2 Straw type

The larger portion of the available literature testifies to the traditional use of barley straw within the cob matrix. However it is highly probable that the addition of straw, with respect to the selection of straw-type was traditionally a detail resolved by the individual practices of cob masons between villages.

Straw is essentially composed of cellulose (a crystalline linear glucose polymer) fibres. These discrete fibres are naturally embedded and bonded together by a continuous organic matrix known as lignin. Lignin, an amorphous polymer of benzene rings, cements these fibres together, Swamy (1988). The difference between barley and wheat straws will be defined by the difference in the percentage composition of these two chemical components.

Wheat straw with its higher percentage of cellulose possesses more structural rigidity. This enhanced rigidity can make it less malleable when worked into a cob matrix, possessing a tenacity to spring-out randomly, when formed for laboratory testing. The higher fibre content of wheat straw may indicate a higher tensile strength than that of barley straw, and therefore it may be assumed to accommodate the lateral stressing induced in the cob matrix during the drying and compressive loading stages of Devon cob construction. However Coventry (2004) has shown that if these benefits are exhibited by the straw itself, they are not transferred to the cob matrix and all data displayed herein pertains to barley cobs.

4.1.3 Soil selection

The adoption of sites for sampling was determined out in the field where potentially suitable sites were identified by virtue of the existence of an historic cob building constructed from the land on which it was founded. This testified to durability characteristics of the material.

Greer (1996) has previously pursued the idea of manipulating cob matrices by optimising compressive strength characteristics through varying the percentage component of specific soil particle sizes. This approach may sit comfortably with materials manufactured from traditional process industries such as the concrete industry. However in order both to support the preservation of traditional cob buildings and promote this construction method as a viable construction technology, this work pursues the effects of natural matrix variation on the structural behaviour of earthen building materials.

Natural matrix variation is afforded by consideration of sampling from a catena sequence. Selecting soils from areas overlying the same parent rock but of progressively altering topographic formation, permits the engineering capacities of these soils, when used in cob construction, to be assessed with respect to soil weathering.

Clayden (1971) has shown three Devon soils, the ‘Dunland soils’, pedologically known as Dunsford, Halstow and Tedburn to form part of one Catena sequence, see Figure 2. The redland soils of ‘Crediton’ and Bridgnorth also derived from the same parent rock but occurred on two differing areas of topographical boundaries, one of the Breccia Hills and the other of the Sandstone lowlands respectively (Clayden, 1971), see Figure 3.

These soils have been classified, in accordance with BS1377 (1990) with respect to their particle size distributions and the fractional percentages for each soil is presented in Table 1.

Thus in accordance with BS 1377 we can classify the Crediton and Bridgenorth soils as clayey, slightly silty sandy gravels: and the Dunsford and Tedburn soils as gravelly, sandy gravelly clayey silts.

5 THE UNCONFINED COMPRESSIVE STRENGTH OF DEVON COBS

Tables 2 and 3 illustrate the unconfined compressive strength obtained at manufacture of the test cylinder.
and on air-drying of the test cylinder respectively. These tables highlight the general enhancement of the straw to the engineering characteristics of the cob matrix over that of the soil, giving an indication of the macroscopic response of the structure to the load. However they also indicate the importance of moisture content at the time of test as the air-dried samples exhibit much higher strength values over those of the post-manufactured samples.

Coventry (2004) and Coventry and Griffiths (in preparation) offer further insight to the role of moisture content in the realisation of the compressive strength of Devon cob at the point of mix design and test sample manufacture. Water is shown to be highly influential to both the macro and micro material response to the applied load by influencing the frictional response of the soil at a macro-level and the bond strength in the soil at a micro-level. In order to explain this response the Pressure membrane apparatus has been employed to offer some means of classifying the cob matrixes at the microstructure level.

6 PRESSURE MEMBRANE APPARATUS

The pressure membrane apparatus (shown in Figure 4) consists of a chamber into which the samples tested are placed. Once inside the chamber, the samples are subjected to air pressure, used to force out water from the samples’ pore spaces. Increasing air pressure forces water out of smaller and smaller pore spaces. The amount of water extracted under a given pressure is monitored daily until water extraction fails to exceed 3 mg over successive days. At this point the sample is said to have equilibrated under the air pressure applied, Hall et al (1977). Given that each pressure application can be linked to a specific pore diameter using the approximation from Landon (1991):

\[
\text{diameter of pore (cm)} = 0.3/h, \quad \text{where } h \text{ is the cm of water applied to the system}
\]

pore volumes can be equated to the volume of water released between each successive pressure application.

---

**Table 1. Soil fraction percentages for selected soils.**

<table>
<thead>
<tr>
<th>Soil series</th>
<th>Gravel</th>
<th>Sand</th>
<th>Silt</th>
<th>Clay %</th>
</tr>
</thead>
<tbody>
<tr>
<td>60 mm–2 mm</td>
<td>%</td>
<td>%</td>
<td>%</td>
<td>%</td>
</tr>
<tr>
<td>Crediton</td>
<td>42.4 ±</td>
<td>48</td>
<td>2.1 ±</td>
<td>2.9 ±</td>
</tr>
<tr>
<td></td>
<td>6.9</td>
<td>5</td>
<td>2.8</td>
<td>3.11</td>
</tr>
<tr>
<td>Dunsford</td>
<td>13.2</td>
<td>4.69</td>
<td>21.74</td>
<td>5.13</td>
</tr>
<tr>
<td>Tedburn</td>
<td>35.69 ±</td>
<td>13.33</td>
<td>25.42</td>
<td>26.36 ±</td>
</tr>
<tr>
<td></td>
<td>6.62</td>
<td>2.26</td>
<td>7.91</td>
<td>4.2</td>
</tr>
<tr>
<td>Halstow</td>
<td>0.66 ±</td>
<td>4.41</td>
<td>45.17</td>
<td>49.35 ±</td>
</tr>
<tr>
<td></td>
<td>0.225</td>
<td>1.67</td>
<td>3.41</td>
<td>0.98</td>
</tr>
<tr>
<td>Bridgnorth</td>
<td>26.50 ±</td>
<td>66.48</td>
<td>4.50</td>
<td>2.53 ±</td>
</tr>
<tr>
<td></td>
<td>9.53</td>
<td>2.80</td>
<td>8.76</td>
<td>3.35</td>
</tr>
</tbody>
</table>

**Table 2. Characteristic unconfined compressive strength values of air-dried soil/cob cylinders.**

<table>
<thead>
<tr>
<th>Test sample group</th>
<th>Average peak unconfined strength (kN/m²)</th>
<th>Approx. yield strength (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crediton soil cylinders</td>
<td>360.75 ± 17.55</td>
<td>160</td>
</tr>
<tr>
<td>Crediton cob cylinders</td>
<td>721.34 ± 24.43</td>
<td>300</td>
</tr>
<tr>
<td>Dunsford soil cylinders</td>
<td>564.25 ± 27.5</td>
<td>300</td>
</tr>
<tr>
<td>Dunsford cob cylinders</td>
<td>709.44 ± 31.68</td>
<td>400</td>
</tr>
<tr>
<td>Tedburn soil cylinders</td>
<td>372.98 ± 21.7</td>
<td>200</td>
</tr>
<tr>
<td>Tedburn cob cylinders</td>
<td>478.92 ± 43.91</td>
<td>300</td>
</tr>
<tr>
<td>Halstow soil cylinders</td>
<td>1234.84 ± 118.87</td>
<td>600</td>
</tr>
<tr>
<td>Halstow cob cylinders</td>
<td>1185.52 ± 95.49</td>
<td>650</td>
</tr>
<tr>
<td>Bridgnorth soil cylinders</td>
<td>1030.76 ± 125.14</td>
<td>500</td>
</tr>
<tr>
<td>Bridgnorth cob cylinders</td>
<td>1188 ± 134.4</td>
<td>550</td>
</tr>
</tbody>
</table>

**Table 3. Characteristic unconfined compressive strength values of cylinders tested immediately post manufacture.**

<table>
<thead>
<tr>
<th>Test sample group</th>
<th>Average peak unconfined strength (kN/m²)</th>
<th>Approx. yield strength (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crediton soil cylinders</td>
<td>81.89 ± 4.20</td>
<td>30–40</td>
</tr>
<tr>
<td>Crediton cob cylinders</td>
<td>274.09 ± 41.0</td>
<td>150</td>
</tr>
<tr>
<td>Dunsford soil cylinders</td>
<td>100.76 ± 9.21</td>
<td>40</td>
</tr>
<tr>
<td>Dunsford cob cylinders</td>
<td>179.84 ± 29.26</td>
<td>75–100</td>
</tr>
<tr>
<td>Tedburn soil cylinders</td>
<td>96.39 ± 14.82</td>
<td>20–40</td>
</tr>
<tr>
<td>Tedburn cob cylinders</td>
<td>138.39 ± 11.82</td>
<td>30–40</td>
</tr>
<tr>
<td>Halstow soil cylinders</td>
<td>96.20 ± 4.51</td>
<td>80</td>
</tr>
<tr>
<td>Halstow cob cylinders</td>
<td>159.83 ± 11.9</td>
<td>80</td>
</tr>
<tr>
<td>Bridgnorth soil cylinders</td>
<td>122.87 ± 9.97</td>
<td>60</td>
</tr>
<tr>
<td>Bridgnorth cob cylinders</td>
<td>312.17 ± 4.01</td>
<td>200</td>
</tr>
</tbody>
</table>
and this provides an insight into the microstructure of the fabric which in turn facilitates an interpretation of the compressive load characteristics of the tested matrix. Coventry and Griffiths (in preparation) give a full account of the application and interpretation of this method to Devon cob samples.

7 DEVON COB MATRICES

Confined material selection has hitherto resulted in erroneous extrapolations regarding the beneficial contribution of straw reinforcement to the unconfined compressive strength of Devon cob, this is not supported by the Halstow cob results shown in Table 2. Classification of these matrices at macro and micro levels offers a clear insight into the performance and durability of this sustainable building material which has been proven against the historical background of its use and the role of straw inclusion, Coventry (2004). Thus in the case of Devon Cob performance must be defined by material selection and qualified by test methodology.

8 CONCLUSIONS

Earthen buildings and particularly Cob construction offers a realistic and alternative choice in sustainable building technologies. Through understanding of our historic cob building stock arose an opportunity to learn from the past, to inform conservation while influencing the future, by developing specification and an awareness of appropriate testing methods which would remove the vernacular image of earthen building and replace it with comprehensive technical credibility, while meeting the challenges posed by the sustainability debate.

REFERENCES


662


Mixture proportioning and strength prediction technique for soil mortar made using soil generated at construction sites

T. Yoshida & Y. Kitamoto

Kajima Technical Research Institute, Kajima Corporation, Tokyo, Japan

ABSTRACT: There has been a sharp increase in Japan over the past ten years in the application of so-called soil mortars, made by mixing soil, cement, water, and admixtures. Among soil mortars, ERS mortars (soil mortars made of excavated recycled soils) are produced by mixing soil with cement slurry. To produce ERS mortar of controlled quality, it is necessary to continually modify the mixture proportions carefully throughout the construction period because of the nonuniformity of the soil. The authors have devised a practical method for determining mixture proportions which involves referencing a diagram in which the optimum mixture proportions are expressed as a function of the fine fracture content of the soil. Furthermore, they have found that the compressive strength of a soil mortar, including ERS mortar, at a material age of 28 days can be predicted from its compressive strength at 7 days and its water-cement ratio. This information will enable mixture proportions to be determined even more efficiently.

1 INTRODUCTION

In recent years there has been growing public demand in Japan for the reduction of by-products generated by construction, including excavated surplus soils. This demand has been fuelled not only by environmental considerations but also by economic ones, which has resulted in the use of soil as a construction material, often mixing it with cement.

Techniques for mixing soil and cement can be broadly categorized into two types. One involves mixing cement into soil in situ. Cement shallow mixing and cement deep mixing, which include both mechanical and jet mixing, are examples of this type. The other involves the use of soils generated by construction and mixing plants above the ground, and is divided into two groups: fluid mixtures, or soil mortars; and nonfluid mixtures.

One commonly used method of preparing soil mortar is to add cement to a mud slurry made beforehand from soil and water. However, this method, known as the liquefied stabilized soil method, is not necessarily appropriate for sandy or gravelly soils, because sand and gravel particles settle quickly, and therefore the quality of the slurry is apt to be unstable. Another commonly used method of preparing soil mortar, which is discussed later, is to throw soil directly (without making a mud slurry) into a cement slurry, to produce high-density soil mortar. This method is suitable for almost all soils, including sandy and gravelly soils.

High-density soil mortar made of excavated recycled soil (hereafter called ‘ERS mortar’) has become widely used for diaphragm walls and backfills in Japan in the past ten years [Funabasama & Yoshida 2006].

This paper first presents an overview of the production process and the properties required of ERS mortars, then introduces a practical mixture proportioning technique. The hardening characteristics also are discussed, focusing on the prediction of the unconfined compressive strength at a material age of 28 days.

2 OVERVIEW OF ERS MORTAR

2.1 Mixing plant and process

In general, special mixing plants are used exclusively for producing ERS mortar to ensure stable quality regardless of the soil type. Figure 1 shows an example of a major mixing plant in Japan. In this plant, wet soil is put on a vibrating screen to remove gravel particles having a diameter greater than, for example, 30 mm and is then carried by belt conveyor to a measuring container. A measured amount of soil is then thrown into the mixing room (mixer) and stirred together with the cement slurry. In the case in Figure 1, a mineral admixture (powdered clay) was added. When completely mixed, the ERS mortar flows down into a
stock container and is then pumped into an agitator truck.

2.2 Applications

The amount of ERS mortar applied in Japan has been growing year by year (about 180,000 m³ in 2004). The major application targets are backfills of underground structures and diaphragm walls (cutoff walls or earth-retaining walls). When used as backfill, ERS mortar is applied from above and is self-levelling. In diaphragm walls, including column walls made using an overall casing method, the ERS mortar is usually applied through a tremie pipe into drilling slurry or a clay suspension.

2.3 Quality control

Table 1 shows examples of the properties required of ERS mortar to be used in diaphragm walls. In Japan the design properties of hardened ERS mortar (for example, compressive strength, modulus of deformation, and permeability) are usually provided as those at a material age of 28 days. These properties are confirmed using specimens obtained by boring vertically into the wall, although specimens sampled before hardening and cured at 20°C are also used sometimes. The compressive strengths of ERS mortars are generally designed to be higher than those of ordinary liquefied stabilized soils, namely, 0.5 ~ 0.6 N/mm² in most cases.

However, certain properties are required of fresh ERS mortar to guarantee smooth placement and its quality after hardening. A relatively high density is required especially for diaphragm walls in order to replace drilling slurry or dense clay suspensions with the mortar. While an adequate fluidity is essential, excessive fluidity conflicts with the requirement for density and bleeding rate. If steel H-beams for reinforcement are to be inserted into the wall after filling with ERS mortar, fluidity must be provided for at least several hours, so a retarder in the form of a chemical admixture is used. The fluidity index is determined from the slump flow using a cylindrical mould with a diameter of 80 mm and a height of 80 mm (Fig. 2), as defined in test standard JHS 313 of the (defunct) Japan Highway Public Corporation. This index (hereafter called ‘table flow’ or just ‘flow’) has been commonly used for the liquefied stabilized soils. Gravel particles over a certain diameter, for example,
10 mm, should be removed before measuring the flow. The bleeding rate can be determined using JSCE-F 522.

3 Mixture Proportioning

3.1 Objective
As both soil properties and target quality of the ERS mortar vary with the site, a blending test is usually performed at each site to determine a standard (recommended) mixture proportion that is economical and satisfies the quality requirements. Although a production test using a mixing plant is necessary, a unique standard mixture proportion can be used during the entire construction period with only minor modifications if the soil properties are more or less constant. However, it is often the case that the basic properties of the soil, for example, gradation or water content, change significantly. In such cases, the mixture proportions must be changed considerably to maintain an adequate fluidity of the ERS mortar, as pointed out by Manabe et al. [2000]. In this chapter, a practical method of determining the mixture proportion in accordance with the gradation and the water content of the soil is proposed using a case study.

3.2 Application target
The ERS mortar is to be applied in a diaphragm column wall using an overall casing method. The properties required of the hardened ERS mortar were an unconfined compressive strength of not less than 0.6 N/mm² and a permeability not exceeding 1 × 10⁻¹⁰ cm/s. Besides, a density of not less than 1.5 t/m³, a table flow with a range of 180–260 mm, and a bleeding rate not exceeding 1% were required of the fresh ERS mortar. These are listed in Table 1 as examples of target values of ERS mortars.

3.3 Soil, cement, and admixture
The soil used was gravelly surplus soil generated from the excavation of another diaphragm wall close to the site. The properties of the soils sampled at the site are listed in Table 2. Judging from their values of ignition loss, Ca content, and pH, these soils were considered to contain fractures of the wall bodies (self-solidifying drilling slurry) constructed beforehand.

For the blending tests in the laboratory, two other soils with different gradations were used in addition to the two soils shown in Table 2 to allow for variations in the fine fracture content (Fₚ) of the soils, namely, soil C (Fₚ = 32.6%), prepared by adding powdered clay to soil B (Fₚ = 21.5%); and soil D (Fₚ = 7.6%), prepared by adding gravel (coarse aggregate for concrete with a grain diameter of 2–19 mm) to soil A (Fₚ = 15.0%). The four soils were applied in a wet condition with a water content of around 40% for soils A, B, and C and about 15% for soil D. The cement used was type B portland blast-furnace slag cement (JIS R 5211). To ensure the target permeability value, powdered clay for shield tunnelling and sold under the trade name SCP-A in Japan (the same material used in the preparation of soil C) was used as a mineral admixture.

3.4 Blending test in the laboratory
In consideration of the variation in the properties of the ERS mortar at the site and the differences in mixing methods between the laboratory and the site, in the blending test higher target values were provided for some of the properties, as shown in Table 3. Among these properties, the target unconfined compressive strength was set much higher than for the site, assuming a reduction rate at the site and a coefficient of variation to be 25% and 30%, respectively, from authors’ experience. A handy-type mixer with a rotation speed of 600 rpm was used. The volume of ERS mortar when mixed was 2 l. The duration of agitation was 15 s. The proportions of cement and clay were changed as parameters into a few levels each to investigate the

<table>
<thead>
<tr>
<th>Table 2. Soil properties.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Property</td>
</tr>
<tr>
<td>Density of soil particles (g/cm³)</td>
</tr>
<tr>
<td>Natural water content (%)</td>
</tr>
<tr>
<td>Liquid limit (%)</td>
</tr>
<tr>
<td>Plastic limit (%)</td>
</tr>
<tr>
<td>Plasticity index, I_p (%)</td>
</tr>
<tr>
<td>Maximum particle size (mm)</td>
</tr>
<tr>
<td>Gravel particle content (%)</td>
</tr>
<tr>
<td>Sand particle content (%)</td>
</tr>
<tr>
<td>Fine fracture content, F_p (%)</td>
</tr>
<tr>
<td>Ignition loss (%)</td>
</tr>
<tr>
<td>Ca content (%)</td>
</tr>
<tr>
<td>pH</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 3. Target properties of ERS mortar for the blending test.</th>
</tr>
</thead>
<tbody>
<tr>
<td>State of ERS mortar</td>
</tr>
<tr>
<td>Hardened Permeability</td>
</tr>
<tr>
<td>Unconfined compressive strength</td>
</tr>
<tr>
<td>Fresh Density</td>
</tr>
<tr>
<td>Table flow (JHS 313)</td>
</tr>
<tr>
<td>Bleeding rate (JSCE-F 522)</td>
</tr>
</tbody>
</table>
optimum quantities, while those of soil and water were adjusted to provide the ERS mortar with a flow value within the target range of 240~260 mm. In this way, the standard mixture proportions were determined for each of soils A to D so as to best satisfy the target values listed in Table 3.

In Figure 3, the standard proportions of soil (in dry mass), water (including that in the wet soil), cement, and clay were plotted against the fine fracture content, $F_c$. The proportions of cement and clay were held constant, at 150 and 120 kg/m$^3$, respectively, because the ideal proportions of these were not very sensitive to $F_c$. The fitting curve for the quantity of water, $W$, in Figure 3 is the logarithmic function:

$$W = c_1 \ln F_c + c_2$$  \hspace{1cm} (1)

where $c_1 = 112$, $c_2 = 276$, and $F_c$ is expressed as a percentage. Another curve in Figure 3 shows the recommended quantity of soil back-calculated from that of water given by equation (1), assuming the particle densities of the soil, the cement, and the clay to be 2.68, 3.04, and 2.70 t/m$^3$, respectively. Regardless of the variation in $F_c$, these curves indicate mixture proportions with almost unique fluidity, which is primarily essential for smooth work on site. Because the quantity of water shown in Figure 3 includes the water contained in the soil, the latter must be subtracted from the amount of water to be added during mixing.

### 3.5 Production tests using the plant

The mixing plant shown in Figure 1 was used for actual placement. The volume of ERS mortar produced in one go was 1.5 m$^3$, and the mixing duration was 1 minute. A retarder was applied as a chemical admixture because of the requirement to insert steel H-beams as reinforcements into the ERS mortar column, although this was not planned at the time of the blending test. All of the differences between actual placement and the blending tests were thought to affect the fluidity of the ERS mortar. Therefore, full-scale production tests of the ERS mortar were performed repeatedly to check the reliability of the standard mixture proportions indicated by Figure 3, prior to actual placement. A retarder (GEORETAR 10) was added at a ratio of 4% of the cement by mass (6 kg/m$^3$) and it was considered part of the water in the mixture proportions.

In Figure 3, the mixture proportions providing a successful flow value within the target range (180~260 mm) were plotted (denoted as ‘plant’ in the legend) in contrast with the standard proportion curves. It can be seen that the standard mixture proportions obtained using the blending tests result for actual placement in an excessive proportion of water and an insufficient proportion of soil. In Figure 4, the apparent shortage of water as compared to the standard curve (that is, a surplus of water brought by the curve) is plotted against the water content of the soil. It should be reminded that all sample soils used in the blending test had the almost same water content, except for soil D. Figure 4 includes unsuccessful data with an excessive flow over 260 mm. The straight line in the figure indicates the averaged relationship for successful data only. It can be seen that unsuccessful data with excessive fluidity were obtained by using mixture proportions near to the standard curve (that is, the apparent shortage of water is closer to 0), while the successful data with a proper flow were obtained by using less water. Furthermore, it is particularly evident that the apparent shortage of water decreases as the water content of the soil increases. Such behaviour, also reported by Sasakura et al. [2003], is thought to be caused by fractures of the wall bodies (self-solidifying drilling slurry). The absorbed water within such porous particles does not

![Figure 3. Diagram indicating standard mixture proportions to satisfy the target values listed in Table 3.](image)

![Figure 4. Relationship between water content of soil and apparent shortage of water in accordance with the flow.](image)
contribute to the fluidity of the mortars, although it does count in the water content of the soil. Figure 4 suggests that the standard mixture proportion indicated for arbitrary $F_c$ by Figure 3 can be modified according to the water content of the soil.

It is assumed that the apparent shortage (or surplus, possibly) of water is independent of the water content of the soil, if the soil is free of porous particles. In such cases, for actual placement it may be sufficient to uniformly modify the amount of water recommended by the blending test, regardless of the water content of the soil. However, the modification in accordance with the water content of the soil would be significant if an ERS mortar of proper quality is to be produced consistently, since excavated recycled soils often include porous particles such as fractures of solidified soils or wall bodies.

4 HARDENING CHARACTERISTICS

4.1 Objective

The mixture proportioning method described above enables the stable production of ERS mortar of the required quality. However, some problems remain, the most notable of which is that the blending tests require considerable time and effort. Furthermore, if there are any defective parts in ERS mortar, time will be wasted if this is not detected before the material has an age of 28 days.

This chapter focuses on the unconfined compressive strength of ERS mortars, among other properties, and proposes a new method of predicting the strength at a material age of 28 days (cured at 20°C) directly from specimens cured at 20°C at a young material age of 7 days [Yoshida & Kitamoto 2006].

4.2 Past approaches

Kurihara et al. [1994] and Kitamoto et al. [2006] studied the long-term (up to 22 years) hardening characteristics of artificial soft rock (a kind of high-density soil mortar) made from sand and mudstone (Nos. 12 and 13 in Table 4). They showed that at a unique curing temperature the unconfined compressive strength $\sigma_7$ (at 7 days) of 7-day-old mortar, $\sigma_7$, both mortar specimens having been cured at 20°C. Most of these materials are ordinary ERS mortars made from one or more kinds of natural or surplus soils. However, to obtain results that are as universally applicable as possible, other materials similar to ERS mortar were also investigated, including soil mortar made from waste sludge in the form of slurry generated from shield tunnelling (No. 1), high-density soil mortar using only commercialized mineral materials (No. 8), low-strength concrete (No. 9), artificial soft rocks (Nos. 12 & 13) containing wet-milled mudstone and having a compressive strength somewhat higher than ordinary ERS mortars, and so-called cement-bentonite mortars (No. 14). In the table, where a unit quantity has a range, two or more mixture proportions for that mortar were investigated. Nos. 6 & 13 were plant mixed at the site, while the others were subjected to blending tests in the laboratory. In this paper, only data from specimens cured at a constant temperature of 20°C are discussed.

4.3 Materials investigated

Tables 4 & 5 list the 14 soil mortars investigated by the authors to determine the relationship between the unconfined compressive strength of 28-day-old mortar, $\sigma_{28}$, and that of 7-day-old mortar, $\sigma_7$, both mortar specimens having been cured at 20°C. Most of these materials are ordinary ERS mortars made from one or more kinds of natural or surplus soils. However, to obtain results that are as universally applicable as possible, other materials similar to ERS mortar were also investigated, including soil mortar made from waste sludge in the form of slurry generated from shield tunnelling (No. 1), high-density soil mortar using only commercialized mineral materials (No. 8), low-strength concrete (No. 9), artificial soft rocks (Nos. 12 & 13) containing wet-milled mudstone and having a compressive strength somewhat higher than ordinary ERS mortars, and so-called cement-bentonite mortars (No. 14). In the table, where a unit quantity has a range, two or more mixture proportions for that mortar were investigated. Nos. 6 & 13 were plant mixed at the site, while the others were subjected to blending tests in the laboratory. In this paper, only data from specimens cured at a constant temperature of 20°C are discussed.

4.4 Prediction of compressive strength

In Figures 5 & 6, $\sigma_{28}$ and the ratio $\sigma_{28}/\sigma_7$ are respectively plotted against $\sigma_7$ for the materials listed in Tables 4 & 5. The data points in Figure 6 are classified into four groups ((i) to (iv)) in accordance with the water-cement ratio (W/C), as defined in Table 6. Figures 5 & 6 show that $\sigma_{28}/\sigma_7$ ranges from almost 1.5 to 3 and tends to decrease as $\sigma_7$ increases. A higher W/C yields a smaller $\sigma_{28}/\sigma_7$ for arbitrary values of $\sigma_7$.

To formulate these tendencies, the representative relationship:

$$\frac{\sigma_{28}}{\sigma_7} = c \cdot e^{-A \cdot \sigma_7} + 1$$  \hspace{1cm} (2)

was assumed between $\sigma_7$ and $\sigma_{28}/\sigma_7$ for arbitrary values of W/C, where $c$ is a constant independent of W/C, whereas A depends on W/C, and $e$ is the base of natural logarithms. In equation (2), the lower limit of $\sigma_{28}/\sigma_7$ is 1, because $\sigma_{28}$ is always larger than $\sigma_7$. The
Table 4. List of mixtures.

<table>
<thead>
<tr>
<th>No.</th>
<th>Type</th>
<th>Target</th>
<th>Place of mixing</th>
<th>Cement*3</th>
<th>Soil or aggregate (dry mass)</th>
<th>Mineral admixture (dry mass)</th>
<th>Chemical admixture*6</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Soil mortar</td>
<td>Diaphragm wall</td>
<td>Laboratory</td>
<td>BB 160~200 kg</td>
<td>Construction waste sludge About 410~450 kg*5</td>
<td>Solidified and milled sludge 200<del>300 kg Sand 0</del>100 kg</td>
<td>Retarder (GEORETAR 10) 3% to C</td>
</tr>
<tr>
<td>2</td>
<td>ERS mortar</td>
<td>Diaphragm wall</td>
<td>Laboratory</td>
<td>BB 50~150 kg</td>
<td>Silt 262~346 kg</td>
<td>None</td>
<td>Retarder (SHO-ACE) 0~6% to C</td>
</tr>
<tr>
<td>3</td>
<td>ERS mortar</td>
<td>Diaphragm wall</td>
<td>Laboratory</td>
<td>BB 50~200 kg</td>
<td>Sandy soil 824~1329 kg</td>
<td>None</td>
<td>Retarder (SHO-ACE) 0~6% to C</td>
</tr>
<tr>
<td>4</td>
<td>ERS mortar</td>
<td>Diaphragm wall</td>
<td>Laboratory</td>
<td>BB 100~200 kg</td>
<td>Gravelly soil 1289~1374 kg</td>
<td>Clay (SCP-A) 100 kg</td>
<td>None</td>
</tr>
<tr>
<td>5</td>
<td>ERS mortar</td>
<td>Diaphragm wall</td>
<td>Laboratory</td>
<td>BB 100~200 kg</td>
<td>Sandy or gravelly soils 974~1080 kg</td>
<td>Clay (SCP-A) 100 kg</td>
<td>None</td>
</tr>
<tr>
<td>6</td>
<td>ERS mortar</td>
<td>Plant</td>
<td>BB 150 kg</td>
<td>BB 162 kg</td>
<td>Gravelly soil About 800~1100 kg</td>
<td>Clay (SCP-A) 100 kg</td>
<td>Retarder (GEORETAR 10) 4% to C</td>
</tr>
<tr>
<td>7</td>
<td>ERS mortar*1</td>
<td>Not specified</td>
<td>Laboratory</td>
<td>BB 201 kg</td>
<td>Siliceous sand #5 538 kg</td>
<td>Clay 172 kg</td>
<td>Retarder (GEORETAR 10) 0~4% to C</td>
</tr>
<tr>
<td>8</td>
<td>Soil mortar</td>
<td>Diaphragm wall</td>
<td>Laboratory</td>
<td>BB 36~68 kg</td>
<td>Fine aggregate (sand)</td>
<td>Bentonite</td>
<td>Superplasticizer</td>
</tr>
<tr>
<td>9</td>
<td>Low-strength concrete</td>
<td>Diaphragm wall</td>
<td>Laboratory</td>
<td>BB 60~180 kg</td>
<td>Coarse aggregate 0~912 kg</td>
<td>Bentonite</td>
<td>(MIGHTY 150SC) 1.5% ot P</td>
</tr>
<tr>
<td>10</td>
<td>ERS mortar</td>
<td>Diaphragm wall</td>
<td>Laboratory</td>
<td>Blended cement*4 120~180 kg</td>
<td>Silty sand 1157~1230 kg</td>
<td>None</td>
<td>Air-entraining superplasticizer (MIGHTY 300S) 1~ 3.25% to P*7</td>
</tr>
<tr>
<td>11</td>
<td>ERS mortar</td>
<td>Diaphragm wall</td>
<td>Laboratory</td>
<td>Blended cement*4 60~210 kg</td>
<td>Silty sand 190~1191 kg</td>
<td>Bentonite 0~80 kg</td>
<td>None</td>
</tr>
<tr>
<td>12</td>
<td>Artificial soft rock*2</td>
<td>Backfill</td>
<td>Laboratory</td>
<td>Blended cement*4 180 kg</td>
<td>Sand 700~1110 kg</td>
<td>Mudstone wet milled 185~230 kg</td>
<td>None</td>
</tr>
<tr>
<td>13</td>
<td>Artificial soft rock*2</td>
<td>Backfill</td>
<td>Plant</td>
<td>Blended cement*4 60~210 kg</td>
<td>Sand 700 kg</td>
<td>Mudstone wet milled 230 kg</td>
<td>None</td>
</tr>
<tr>
<td>14</td>
<td>Cement bentonite</td>
<td>Backfill</td>
<td>Laboratory</td>
<td>Portland cement type I (Taiwan)</td>
<td>None</td>
<td>Bentonite Wyoming 50 kg</td>
<td>None</td>
</tr>
</tbody>
</table>

*1 Data by Flowric, Co., Ltd.
*2 Kurihara et al. [1994].
*3 BB is portland blast-furnace slag cement type B (JIS R 5211).
*4 Clinker, blast-furnace slag, and gypsum blended at 62%, 20%, and 18%, respectively.
*5 Used in form of slurry with density of 1.33 t/m³.
*6 C and P indicate content of cement and powders (i.e. total of cement, clay, bentonite, and milled limestone), respectively.
*7 Defoaming agent and viscous agent also used.
values $c = 2.01$ and $A = 0.67$ for group (ii) $(3 \leq W/C < 5)$, which has the largest number of data among the four groups in Figure 6, were obtained using the least-square method. For convenience in practical use, the value of $c$ was modified and equation (2) was rewritten as:

$$
\sigma_{28} = \sigma_7 \left( 2e^{-A \cdot \sigma_7} + 1 \right)
$$

Figure 5. Relationship between $\sigma_7$ and $\sigma_{28}$ for a variety of materials (See Table 4 for material numbers).

Figure 6. Relationship between $\sigma_7$ and $\sigma_{28}/\sigma_7$. 

Table 5. Properties of mixtures listed in Table 4.

<table>
<thead>
<tr>
<th>No.</th>
<th>Max. grain size (mm)*1</th>
<th>Fine fracture content, $F_r$ (%)</th>
<th>Density (t/m$^3$)*2</th>
<th>Table flow (mm)*3</th>
<th>Bleeding rate (%)*4</th>
<th>Water-cement ratio, W/C (See Table 6)</th>
<th>W/C class*5</th>
<th>No. of data*5</th>
<th>No. of specimens*6</th>
<th>Specimen size*7</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>–</td>
<td>–</td>
<td>1.48−1.53</td>
<td>224−279</td>
<td>0−1.5</td>
<td>3.13−4.29</td>
<td>(ii)</td>
<td>9</td>
<td>3</td>
<td>Small</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>96.3</td>
<td>(1.25−1.26)</td>
<td>150−384</td>
<td>0.5−2.5</td>
<td>5.67−17.0</td>
<td>(iii)−(iv)</td>
<td>4</td>
<td>2</td>
<td>Small</td>
</tr>
<tr>
<td>3</td>
<td>4.75</td>
<td>14.0−28.8</td>
<td>(1.61−1.89)</td>
<td>251−346</td>
<td>1.1−2.0</td>
<td>3.12−13.0</td>
<td>(ii)−(iv)</td>
<td>7</td>
<td>2</td>
<td>Small</td>
</tr>
<tr>
<td>4</td>
<td>30/15</td>
<td>7.7</td>
<td>2.05−2.10</td>
<td>185−200</td>
<td>0</td>
<td>2.07−4.15</td>
<td>(i)−(ii)</td>
<td>3</td>
<td>3</td>
<td>Large</td>
</tr>
<tr>
<td>5</td>
<td>30/15</td>
<td>19.5−30.3</td>
<td>1.76−1.97</td>
<td>120−200</td>
<td>0</td>
<td>2.63−5.32</td>
<td>(i)−(iii)</td>
<td>6</td>
<td>3</td>
<td>Large</td>
</tr>
<tr>
<td>6</td>
<td>30/9.5</td>
<td>About 20</td>
<td>1.5−1.73</td>
<td>170−230</td>
<td>0</td>
<td>About 4.0</td>
<td>(ii)</td>
<td>51</td>
<td>3</td>
<td>Small</td>
</tr>
<tr>
<td>7</td>
<td>2</td>
<td>64.1</td>
<td>(1.78)</td>
<td>–</td>
<td>–</td>
<td>3.42</td>
<td>(ii)</td>
<td>9</td>
<td>1</td>
<td>Small</td>
</tr>
<tr>
<td>8</td>
<td>2.4</td>
<td>–</td>
<td>1.75</td>
<td>270</td>
<td>0</td>
<td>3.00</td>
<td>(ii)</td>
<td>1</td>
<td>3</td>
<td>Small</td>
</tr>
<tr>
<td>9</td>
<td>5 or 25</td>
<td>–</td>
<td>2.08−2.34</td>
<td>245−668</td>
<td>0</td>
<td>4.00−4.80</td>
<td>(ii)</td>
<td>6</td>
<td>3</td>
<td>Small</td>
</tr>
<tr>
<td>10</td>
<td>–</td>
<td>–</td>
<td>1.77−1.83</td>
<td>170−181</td>
<td>0−0.5</td>
<td>2.82−8.72</td>
<td>(i)−(iv)</td>
<td>5</td>
<td>3</td>
<td>Small</td>
</tr>
<tr>
<td>11</td>
<td>–</td>
<td>–</td>
<td>1.28−1.81</td>
<td>172−240</td>
<td>0−0.1</td>
<td>2.82−5.94</td>
<td>(i)−(iii)</td>
<td>10</td>
<td>3</td>
<td>Small</td>
</tr>
<tr>
<td>12</td>
<td>–</td>
<td>–</td>
<td>1.71−1.88</td>
<td>180−200</td>
<td>Less than 3</td>
<td>2.62−8.27</td>
<td>(i)−(iv)</td>
<td>7</td>
<td>–</td>
<td>Small</td>
</tr>
<tr>
<td>13</td>
<td>–</td>
<td>–</td>
<td>1.71</td>
<td>189</td>
<td>Less than 0.5</td>
<td>3.32</td>
<td>(ii)</td>
<td>1</td>
<td>–</td>
<td>Small</td>
</tr>
<tr>
<td>14</td>
<td>–</td>
<td>–</td>
<td>1.16−1.30</td>
<td>199−263</td>
<td>1.3−3.1</td>
<td>2.13−4.58</td>
<td>(i)−(ii)</td>
<td>5</td>
<td>2</td>
<td>Small</td>
</tr>
</tbody>
</table>

Blank (−) means uncertain.

*1 At mixing. Value after an oblique is for specimens for compression tests.

*2 Values between parentheses are assumed from mix proportions, while others were directly measured.

*3 Tested just after mixing. Particles over 9.5 mm were removed before test. (Test method for air mortar and air milk: JHS 313).

*4 At material age of 24 hours (JSCE-F 522).

*5 Number of points plotted in Figures 5, 6, & 8.

*6 Average of this number of data is plotted in Figures 5, 6, & 8.

*7 Cylinder of diameter 5 cm and height 10 cm (small) or of diameter 10 cm and height 20 cm (large). In No. 9, “large” was used for maximum grain size of 25 mm.
where \( A \) is again a coefficient dependent on W/C. The curves in Figure 6 were drawn by obtaining values of \( A \) for each group ((i) to (iv)) using the least-square method. These curves successfully represent an approximate relationship between \( \sigma_7 \) and \( \sigma_{28}/\sigma_7 \) for each group.

Figure 7 shows the relationship between W/C and coefficient \( A \), where median values for each of the groups were used for W/C. For group (i), the minimum value of W/C was considered as the lower boundary of the section. Similarly, the maximum value of W/C was considered as the upper boundary of the section for group (iv). The relationship between W/C and coefficient \( A \) is well expressed by a parabola, namely:

\[
A = 0.0281(W / C)^2 - 0.003(W / C) + 0.24 \quad (4)
\]

where W/C is not expressed as a percentage. For a soil mortar with an arbitrary mixture proportion, coefficient \( A \) is obtained from W/C using equation (4), and accordingly \( \sigma_{28} \) can be predicted from \( A \) and \( \sigma_7 \) using equation (3).

In Figure 8, the data shown in Figure 5 are plotted again using the same classification (namely, groups (i) to (iv)) as in Figure 6, accompanied by \( \sigma_7 - \sigma_{28} \) curves obtained using equation (3) for each group.

5 CONCLUSIONS

Mixture proportioning and hardening characteristics of the ERS mortar (high-density soil mortar made using excavated recycled soil) have been investigated. In chapter 3, a practical method of determining the mixture proportions for ERS mortars of controlled quality has been derived from a case study. In chapter 4, the hardening characteristics of ERS mortars were discussed. The results are summarized below.

5.1 Mixture proportioning

By performing laboratory blending tests using some soil samples with different gradations, the standard mixture proportions for the required properties of fresh and hardened ERS mortars (fluidity, especially) were obtained as a function of the fine fracture content, \( F_{c2} \), of the soil, where the quantity of water increases while the soil decrease, as \( F_{c2} \) increases. These relationships were summarized in a mixture proportioning diagram as Figure 3.

To determine a mixture proportion of ERS mortar referencing such a diagram at actual placement, only two property index are to be measured: namely, the fine fracture content, \( F_{c2} \); and the water content of the soil. Since the quantity of soil in the diagram is in dry mass, the water contained in the soil must be subtracted from the amount of water and added to the

<table>
<thead>
<tr>
<th>Group</th>
<th>W/C Section</th>
<th>Median Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>(i)</td>
<td>Less than 3</td>
<td>2.535</td>
</tr>
<tr>
<td>(ii)</td>
<td>3~5</td>
<td>4</td>
</tr>
<tr>
<td>(iii)</td>
<td>5~8</td>
<td>6.5</td>
</tr>
<tr>
<td>(iv)</td>
<td>More than 8</td>
<td>12.5</td>
</tr>
</tbody>
</table>

Table 6. Classification of materials by W/C.
amount of soil indicated by the diagram, in accordance with the water content of the soil, to know the actual quantities of water and soil to be thrown into the mixing plant.

The above mixture proportions should be modified to adjust the fluidity through full-scale production tests, reflecting the difference in mixing conditions from those of the laboratory tests. It is considered that there are cases, as the case presented in this paper, where it is essential to modify the standard mixture proportions by adjusting the amount of water in accordance with the water content of the soil rather than using a uniform modification. Such requirement derives, the authors consider, from the porous particles such as fractures of solidified soils or wall bodies, which are often contained within the excavated recycled soils.

5.2 Hardening characteristics

The authors investigated the relationships between the unconfined compressive strength at material ages of 7 days, \( \sigma_7 \), and of 28 days, \( \sigma_{28} \), using specimens of many kinds of ERS mortars and other soil mortars cured at 20°C with \( \sigma_{28} \) of up to about 4 N/mm\(^2\).

The value of \( \sigma_{28}/\sigma_7 \) ranges from almost 1.5 to 3 and tends to decrease as \( \sigma_7 \) increases. The higher the water-cement ratio (W/C), the smaller \( \sigma_{28}/\sigma_7 \) is for arbitrary values of \( \sigma_7 \). Using this relationship, shown by equations (3) and (4), the value of \( \sigma_{28} \) can be predicted, albeit approximately, from \( \sigma_7 \) and W/C. The value of \( \sigma_7 \) is obtained by performing compression tests at a material age of 7 days using specimens cured at 20°C, while W/C is known from the mixture proportion. This prediction method will be useful because of its simplicity.

REFERENCES


Utilization of olive oil liquid waste as an additive to roadway construction

O.A. Abaza,
Associate Professor, An-Najah National University, Nablus, Palestine

ABSTRACT: Several attempts were done in the last three decades to utilize waste materials through recycling, treatment, or reuse. One of the successful attempts to utilize Zebar, (a liquid by-product of olive oil, which consists mainly of oil residue) to be used in portland cement concrete (PCC) as an additive. The objective of this study is to investigate the effects of adding olive oil liquid waste (Zebar) to basecourse to improve its properties and attempt to benefit from such discarded materials.

This study was carried out to investigate the physical properties of the basecourse like optimum moisture content, California Bearing Ratio (CBR), density, permeability, shrinkage and swelling, and control fine particulates when olive oil liquid waste is used as a replacement for fresh water in roadway construction. This investigation considered properties such as optimum moisture content, dry density, penetration, swelling, dust control, and shrinkage of basecourse. A number of laboratory measurements were undertaken such as California Bearing Ratio test, compaction test, shrinkage test, dust control test, etc.

Results showed that using olive oil liquid waste as a blending additive to basecourse reduces the optimum liquid (moisture) content and increase the maximum dry density. In addition, olive oil liquid waste reduced the amount of dust to zero percent at a replacement by liquid waste of 7 % which makes it an ideal blending material for unpaved agriculture roads. The permeability of basecourse decreased with increase in the percentage of olive oil liquid waste in the basecourse providing a water tight layer. A significant improvement in CBR values were noticed with an increase in olive oil liquid waste content in roadway basecourse material.

Key words: olive oil liquid waste, basecourse, physical properties, replacement, CBR, permeability, dust control, swelling, shrinkage.
waste and 1001 of liquid per 100 kg of treated olives (Alcaide and Neffzou, 1996). Olive husk-based carbon seems to have special features such as low sulphur content, low ash content, high surface areas, and uptake values (Martinez et al., 2006; Stavropoulos and Zabaniotis, 2005).

2 BACKGROUND

Olive oil production industry is characterized by relevant amounts of liquid and solid by-products and by economical, technical, and organizational constraints that make it difficult to adopt environmentally sustainable disposal approaches. Olive seeds are usually crushed and disposed of after processing the olive oil. The olive oil seeds after production of oil contains 6–8% oil and when dumped, such oil reacts to yield hazardous chemicals such phenols and other aromatic (type of organic material that causes some scent in the material) (Caputo et al., 2003). In a few cases, the solid waste is pressed using a costly high pressure system to recover lower quality oil, usually useful for the production of soap. For many years, the solid waste has been partly used as inefficient polluting heating fuel (Cichelli and Solinas, 1984; Garcia et al., 1982; Amat et al., 1999).

An additional by-product of olive oil is the aqueous liquid wastes (Zebar) produced by olive mills which is usually drained in the form of sewage or sprayed on agricultural land and unpaved roads. Several attempts were done to either provide treatment for this liquid waste or make use of waste in other construction products.

Waste treatment technologies arrived at energy recovery represents an interesting alternatives. The technical and economic analysis of thermal disposal plant which allow the combined treatment of solid and liquid waste, although penalizes the energy recovery, may prove to be feasible and profitable in future legislative scenarios. This may be more critical when stricter limitations on liquid waste disposal force oil producers to bear some high disposal costs (Caputo et al., 2003). Since the liquid olive oil mill waste has significant oil content, before extracting this oil, moisture of the liquid waste has to be reduced from approximately 65% to about 8% (Arjona et al., 1999). One alternative (for example in producing energy or disposing in uncontrolled manner etc) is either utilization as a source of nutrients for animals which may also contribute to the development of efficient and environmentally conservative extensive animal production systems within Mediterranean semi-arid ecosystem (Alcaide and Neffzou, 1996). In recent years both physicochemical and biological treatment methods for liquid waste have been employed through the potential use as an additive for the development of construction materials through cement mixed with an improver/additive containing a mixture of natural and synthetic zeolites, alkaline elements, and oxides (Hytiris et al., 2004).

Many attempts were made for the treatment of olive oil liquid waste through physical, biological, and management techniques (Canepa et al., 1988; Vitolo et al., 1999; Azbar, et al., 2004, Armesto et al. 2002, Gernjak et al., 2004, Ntdre et al., 2005; El-Hamous et al., 2006, Putun et al., 2005).

Application of olive oil waste water to cultivated soil was explored to solve the disposal problem and at the same time, to improve the physiochemical characteristics of intensely cultivated soils. The results of this experiment confirmed that the spreading of olive oil waste water on cultivated soil could be an eco-compatible practice (Piovanelli et al., 2005).

3 SIGNIFICANCE AND OBJECTIVES

The objective of this study is to investigate the effects of adding olive oil liquid waste (Zebar) to basecourse to improve its properties and attempt to benefit from such discarded materials. The lack of information related to the effect of adding this waste material on the physical and chemical properties of natural basecourse makes this research of special interest.
4 OLIVE OIL LIQUID WASTE CHARACTERISTICS

Olive oil liquid waste may be defined as a by-product material resulting from the production of oil from olives. This waste contains materials of concentration that varies from site to site due to the quantity of water added to olives during the process of oil production. The olive oil waste used in this work was taken from an olive oil mill and was kept in a controlled environment to ensure the uniform properties. The average value of the specific gravity of the liquid waste has been tested with an average equal to 1.005 compared with .998 for drinkable water. The properties and chemical analysis of olive oil liquid waste used in this research are given in table 1. The analysis by Gas Chromolography MS (GC-MS) showed that the major organic compound is Benzenedicarboxylic acid, which was proved by PH-measurements.

The olive oil liquid waste used for the purpose of this research were extracted fresh from a local olive mill and kept refrigerated before use to keep uniformity in the physical, chemical, and biological characteristic (in case of variability) of this waste material.

5 EXPERIMENTAL

In the analysis of the physical effects of olive oil liquid waste, several tests were performed on natural basecourse material conforming to the ASTM D1559 Specification for basecourse material for the construction of highway basecourses. Each of the proposed tests were carried out for five different ratios of olive oil liquid waste mainly 0%, 1%, 3%, 4%, 5% and 7% by weight of basecourse material.

5.1 Compaction

Compaction test were done in accordance to ASTM: D1557 to explore the effect of the liquid waste on the maximum dry unite weight and the optimum moisture content. Figure 1 gives a summary of the trends of the changes in the maximum dry density with an increase in the percentage of olive oil liquid waste. An increase in the maximum dry density was recorded with an optimum value at an olive oil liquid waste of 7%. Table 2 gives the relation with respect to the optimum moisture content with a drop in the amounts of water required for optimum compaction reaching a full replacement of mixing water with olive oil liquid waste at approximately the same optimum percentage of moisture content by the liquid waste. This implies that olive oil liquid waste can fully replace fresh water for the purpose of compaction of highway basecourses which considered an ideal solution for the water scarce region of the Middle East.

5.2 California bearing ratio (CBR)

The main input parameter in the design of pavement structure is the indicators of material strength, mainly the California Bearing Ratio (CBR), applicable for the subgrade, basecourse, and subbase. Utilizing olive oil liquid waste for compaction purposes might have some implication with regard to other physical characteristics of basecourse as construction material. Several percentages of this liquid waste was used in the same manner as water in the process of compaction of the base layer in testing the strength of basecourse based on the CBR test (ASTM: D 1883–99). Samples were tested on both faces for consideration of any variability in the compaction efforts in the molds.

Table 3 gives the laboratory results for CBR with values ranging between 81.2 using pure water to 92.7 using the different percentages of olive liquid waste. Figure 2 gives a representation of the effect of the liquid waste on the strength of basecourse showing a drastic increase in the value of CBR of 12–13 % with increase of the percentage of 6–7 % of liquid waste replacing the optimum water content derived from the compaction test.

Improvement in the physical characteristics of the compacted basecourse material through an increase in the density in the compaction test was reflected in the strength of the basecourse in the CBR test. This means that the olive oil liquid waste works well as a
lubricant material between the aggregate particles to form a dense aggregate mixture, at the same time; it doesn’t leave the greasy effect after a short time period, which might develop lower shear resistance during loading.

This means that a change was going on right after the application of the liquid waste and/or the greasy material are absorbed by the aggregate forming a dry adhesive aggregate surface.

In quantifying the improvement in strength of basecourse through the CBR test, an increase up to 10 and 11 percent for the top and bottom of the mold respectively was recoded with an average of 10.5 percent. This increase is associated with a replacement of olive oil liquid waste of 5.5 and 6.0 percent for the top and bottom cases with an approximate overall value of 5.8 percent. This indicates that an approximately full replacement of fresh water can be possible to achieve optimum strength for basecourse in the highway construction. Furthermore, this will not only utilize the olive oil liquid waste, but it will reserve fresh water and saves more money for blending fresh water with this waste material.

6.1 Swelling and shrinkage

The volume of the soils tends to change as the moisture content change, even though no change occurs in

---

Table 2. Values of optimum moisture content and maximum dry unit for several percentages of olive oil liquid waste.

<table>
<thead>
<tr>
<th>Olive oil liquid waste percentage</th>
<th>Max dry density t/m³</th>
<th>Optimum moisture content</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2.17</td>
<td>7.75</td>
</tr>
<tr>
<td>1</td>
<td>2.18</td>
<td>7.72</td>
</tr>
<tr>
<td>3</td>
<td>2.19</td>
<td>7.60</td>
</tr>
<tr>
<td>4</td>
<td>2.19</td>
<td>7.30</td>
</tr>
<tr>
<td>5</td>
<td>2.19</td>
<td>7.25</td>
</tr>
<tr>
<td>7</td>
<td>2.20</td>
<td>7.20</td>
</tr>
</tbody>
</table>

Table 3. CBR values for basecourse with several percentage of olive oil liquid waste content.

<table>
<thead>
<tr>
<th>Olive oil liquid waste content</th>
<th>CBR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top face</td>
<td>81.2</td>
</tr>
<tr>
<td>Bottom face</td>
<td>85.2</td>
</tr>
<tr>
<td>1% Olive oil liquid waste content</td>
<td>Top face</td>
</tr>
<tr>
<td></td>
<td>Bottom face</td>
</tr>
<tr>
<td>2% Olive oil liquid waste content</td>
<td>Top face</td>
</tr>
<tr>
<td></td>
<td>Bottom face</td>
</tr>
<tr>
<td>3% Olive oil liquid waste content</td>
<td>Top face</td>
</tr>
<tr>
<td></td>
<td>Bottom face</td>
</tr>
<tr>
<td>4% Olive oil liquid waste content</td>
<td>Top face</td>
</tr>
<tr>
<td></td>
<td>Bottom face</td>
</tr>
</tbody>
</table>
the external load. This volume change may be a swelling or shrinkage. This phenomenon usually causes serious detrimental effect to roadway pavements through either surface cracks in the asphalt layer in the case of swelling or rutting and/or cracks in the pavement structure in case of shrinkage. For that purpose, several samples of basecourse material were tested for shrinkage in a cubical metal mold for a period of three days to allow for chemical or physical change (if any) to take place. Samples were tested in place by plates measuring device. Figure 3 shows the percentage change in swelling for different percentages of olive oil liquid waste. Generally, the swelling increases as the percentage of olive oil liquid waste increases. This increase of a maximum of 0.2 percent is considered insignificant compared with the specification limits set by most engineering projects for volume change because it has no significant effect on uplift force that usually causes cracking in the pavement structure (PECDAR, 1997). This is also applicable to the swelling case tested through the methodology adopted by ASTM: D 1883–99, in which the percentage of swelling is less than one percent as shown in figure 4.

6.2 Permeability test

Controlling water seepage through the pavement structure is considered a key element in the successful performance of its function. Saturated subbases and basecourses have detrimental effect on the load carrying capacity of those layers. In addition, water also increases the rate of oxidation and polymerization of the asphaltic pavement layer. Most highway engineers especially in wet regions prefer to take measures to limit the existence of water or moisture in the pavement structure to improve durability and sustainability (AASHTO, 1993). High permeability usually caused by existence of gap or poor graded aggregates in the basecourse material, lake of protection of the pavement structure from capillary rise and high water table.

To explore the effect of olive oil liquid waste on permeability in basecourse, several tests were performed on the basecourse with different replacements by olive oil liquid waste to measure the coefficient of permeability using the falling head method ASTM: D 2434-68. The coefficient of the permeability is a constant of proportionality relating to the ease with which a fluid passes through a porous medium. Figure 5 shows the change the coefficient of permeability as the percentage replacement by olive oil liquid waste with drastic drop reaching a value of .003 or about 92%. This will ensure a water titer layer preventing any water seepage into the pavement structure and eventually the layer will keep its sustainable strength characteristics. The drop in the coefficient of permeability was predictable considering the improvement in maximum dry density a achieved in the compaction test as well as the drastic increase in the CBR value.
Figure 3. Percentage of shrinkage for different replacement by live oil liquid waste.

Figure 4. Percentage swelling of basecourse for different replacements by olive oil liquid waste.

6.3 Dust control test

Wind erosion of soils, roads, and other bare surfaces is a major source of dust. Amount of dust which is emitted depends on the physical characteristics of the material under consideration. Adhesion or cohesion between the material particles determines the amount...
of dust emitted into the air. In the areas of low precipitation or arid regions, dust control is considered a major cause of air pollution especially for rural highways and farm access roads. Local farmers use water spraying to control dust on frequent basis helping to exhaust an already scarce water supply.

To explore the effect of replacement of olive oil liquid waste on basecourse material, a locally made mechanism was manufactured to measure the amount of dust emitted into the air. A cone with openings from both sides having a dimension of one metre long and diameters of 10 and 45 centimetres for both sides were assembled horizontally. The basecourse sample was placed on one end of the cone (the bigger diameter end) with an air jet polling device directed on the sample in the direction of the cone collecting the dust generated on a filter placed on the other end of the cone. The weight of dust collected on both, the inside of the cone and the filter are measured. The several cases of olive oil liquid waste replacements were tested using this mechanism to explore the effect of the waste on dust emitted. Figure 6 shows the change in the amount of dust emitted versus the percentage replacements by olive oil liquid waste. A drop of about 95% in the amount of dust emitted is achieved at 5% replacement reaching zero dust emission at a replacement less that 6%. It is evident that the replacement of fresh water by olive oil liquid waste will have positive impact on the physical characteristics of basecourse material and it will provide an efficient, practical, and cost effective method of utilizing this waste material. Full replacement of fresh water will provide an optimum benefit with respect to the physical characteristic tested as apart of this research.

7 ENVIRONMENTAL IMPACT

The use of olive oil liquid waste in the construction of highway courses for subgrade, subbase, and basecourse might have an impact on the vegetation and water quality in the vicinity. Olive oil waste water has a high content of organic matter and mineral elements such as potassium and phosphorus. Those elements have generally a stimulating effect on soil micro flora activities and biomass content. It can increase the availability of organic matter, thus helping to overcome the problems associated with chemical and physical degradation caused by the progressive decrease in the organic matter content of the soil subjected to intensive cultivation. At the same time, the waste contains polyphenols, toxic compounds having bacteriostatic activity, which can be harmful to the aerobic process of the soil micro flora, particularly to the nitrification process (Gamba et al 2005). The work of Gamba showed that the spreading of olive oil liquid waste on cultivated agricultural soil, not containing xenobiotic products, could represent an
ecologically agronomical practice. The biological parameters indicated that the olive oil waste water did not have a toxic effect on soil micro flora but stimulated its growth and cellular reproduction. The only activity inhibited by the presents of polyphenols was the potential nitrification activity, with a consequence reduction of nitrite and nitrite contents. Nevertheless, this reduction did not affect the alfalfa production. In the case of the use of olive oil liquid waste in highway construction in almost negligible amounts relative to the direct application to soil, no significant impact is expected. This is clear from the mixture behavior during mixing and compaction in the laboratory, showing very little sticky behavior during mixing and no appearance of liquid waste on aggregate surfaces after compaction except the yellow darkish colour of the basecourse. The basic function of basecourse layer is load carrying and load transferring with protection from asphalitic cement surface eliminating any chance of leaching that might wash out traces of this waste material (If any left after mixing and drying) to sub-surface layers. Based on the work of Gamba et al, (2005), and the limited amounts of olive oil liquid waste utilized in the basecourse material, there is no detrimental effect of this waste material on either cultivated soil or fresh water.

8 CONCLUSIONS

Based on the outcome of this research, the following are concluded:

1. Olive oil liquid waste can partially or fully replace fresh water in the compaction of basecourse material in highway construction with reduction in the required optimum liquid content and increase in optimum dry density.
2. Replacement by olive oil liquid waste gives a significant increase in the load carrying capacity of the basecourse through CBR test.
3. Compaction of basecourse material mixed with olive oil liquid waste provides a water tight layer reflected through the significant reduction in the coefficient of permeability.
4. Utilizing olive oil liquid waste will have insignificant effect on the swelling and shrinkage of the basecourse.
5. Mixture of olive oil liquid waste and basecourse material provides dust free environment limiting the emitted basecourse particulates into the atmosphere.
FURTHER WORK

Further work in the utilization of solid waste extracted from olive oil can be done in other fields of construction materials production. Olive husk can be tested in the production of light weight concrete for non-structural purposes as well as in the production of light weight concrete blocks. Utilization of olive oil liquid waste can be used in agricultural use in hot climates in its dry state through the utilization of solar energy, which will help in getting red of the negative effects of relatively high PH value.

REFERENCES

Investigation into the use of cement stabilised sand and silty sand in road pavement construction in Bangladesh

Waliur Rahman, Richard Freer-Hewish & Gurmel Ghataora

Department of Civil Engineering, The University of Birmingham

ABSTRACT: This paper describes an investigation into the use of cement stabilised fine to medium sand and silt in the construction of roads undertaken entirely in Bangladesh. Normally brick aggregates are used in sub-base and sometimes roadbase layers in Bangladesh as the stones are scarce.

Agricultural clay is burnt in kilns to make bricks, furthermore, wood and coal are used as energy source. Bangladesh is also a flood prone country, therefore, cementitious materials are expected to provide durability as well as minimize carbon emission. The laboratory tests were carried out with only about 8 to 15% cement.

The results indicated that the unconfined compressive strength, durability (soaked tests) and constructibility of sand-cement and sand-silt-cement mixtures were acceptable for road pavement layers. The sand-cement material with 8 to 10% cement proved to be adequate for subbases replacement layer for heavily trafficked roads.

It was also found that if the silt content of the sand-silt-cement mixes was increased, strength increased significantly. Mixes containing sand with 10% silt and 15% cement could be used in roadbases for roads that carried heavy traffic. The tensile strength properties of these materials were proven to be acceptable. A pilot field trial showed the constructability of a road pavement using these materials. The unsurfaced trial road only experienced a few minor cracks after six months of construction. The soil-cement materials also proved to be cost-effective.

1 INTRODUCTION

In regions of high rainfall (like Bangladesh) the performance of unbound pavement layers and also the behaviour of subgrade soils on soaking have long been points of concern for the engineers. During the monsoon period, moisture enters into the pavements from the flooded roadside drainage system. Subsurface layers weaken due to inundation and can result in shear failure manifested by rutting in the upper layers. This also results in the punching of granular upper layers into the weakened subgrades. The hot and humid climate in the countries like Bangladesh can result in weathering of black top surface layers in the case of sealed roads. The weathered asphaltic layer cracks permitting water to enter from upper pavement layers from above. Under wheel load hydrodynamic stresses develop and consequently lead to creation of potholes, raveling, alligator cracking, rutting. According to Dempsey [1982], the hydrodynamic pore water pressures exerted by heavy wheel load may reach at least 20.7 KPa, which is high enough to cause ejection of materials up to 10 mm sieve size. Furthermore, brick manufacturing process in Bangladesh is an environmental hazard. One way to counteract these problems are to use natural sands or silts and a binder to bind near surface pavement layers.

This paper reports on the work carried out to investigate the potential of using cement to stabilise fine to medium sand found in estuarine deposits in Bangladesh. In order to get stronger materials fine sieved (0.075 mm) silt was used in the sand-cement mixes. It was anticipated that the fine silt particles would fill the voids in the mixes and would be able to generate well-graded stronger cemented materials.

2 BRICK MANUFACTURING – AN ENVIRONMENTAL HAZARD

In the subbase courses and occasionally in the roadbases locally produced brick aggregates are used in Bangladesh. Local bricks are manufactured by burning soil in kilns using timber, coal or diesel or natural gas in very few cases. There are more than ten thousand brick manufacturing plants, usually called brickfields, all over Bangladesh and more than 25 billion pieces of bricks are produced in these brick-fields every year (Source: Brick-field owners association of Bangladesh). The production is increasing very fast.
due to the demand of bricks. As a result, kilns are increasing so as the brick-fields. The enormous amount of oxides of carbon and sulphur as well as particulate emission from these kilns is polluting the environment tremendously.

Brick kilns are mostly situated on good agricultural land as brick manufacturing needs silty clay loam to silty clay soils with good drainage conditions, which is turning good agricultural land into unproductive lands. Brick kilns are spread all over the country, and are degrading lands. Large population of Bangladesh, about 140 million, mainly depend on agriculture. The country is losing valuable agricultural land. Furthermore, the environmentally unfriendly way of brick manufacturing is playing a big role in decreasing the crop production as well.

The Ministry of Environment of the Government of Bangladesh is seriously considering controlling brick production. In a circular dated 27 September 2003 from the Prime Ministers office, the road construction agencies were advised to find out the alternative way of road construction, which can exclude the use of bricks in roads. This paper provides some alternative techniques.

3 MATERIALS

3.1 Soils

River sand from the central part of Bangladesh (near Dhaka) was used in this investigation. The fine to medium grained sand used was uniformly graded with fineness modulus of 0.9 to 1.1. It was sub-rounded to sub-triangular with relatively high number of rounded grains and it was whitish grey to greenish black. Fine to medium silt from the western part of Dhaka city was also used in this investigation.

Particle size distribution of sand was carried out in accordance with Section 9.3 of BS 1377: Part 2: 1990 [British Standard, 2002a]. For silt and clayey silt, hydrometer method was performed for analysing the particle sizes in accordance with Section 9.5 of BS 1377: Part 2: 1990 [British Standard, 2002a]. The particle size distributions of the soils are shown in Figure 1.

The percentages of sand and silt fraction present in the soils are listed below:

a) Fine to medium sand: sand = 96 to 99% and silt = 1 to 4%

b) Local fine to medium silt: sand = 30% and silt = 70%.

The grading of the fine to medium sand mixed with 10 percent silt passing 0.075 mm sieve (#200) is also shown in Figures 1.

3.2 Portland cement (PC)

Portland cement was supplied by Cemex Bangladesh and it was stored in airtight containers to prevent hydration and carbonation. The set times of the cements and 3, 7, 28 days crushing strength tests were determined using 70 mm cube specimens. The tests were carried out for three bags of cement randomly at different times. The results of the specimens from three different bags were not more than 5 percent apart.

![Particle size distribution of sands and soils used in the investigation.](image-url)
The set times of PC were measured using the Vicat apparatus [Roads and Highways Department, 2001]. The initial set time and the final set time were 2 hours 27 minutes and 3 hours 35 minutes respectively. Both the set times showed acceptable results as initial set time should be more than or equal to 45 minutes and the final set time should be less than or equal to 6 hours and 15 minutes [Roads and Highways Department, 2001].

Compressive strength tests on 3, 7 and 28 days cured cube specimens were carried out using the concrete cylinder (or cube) compression-testing machine. The ratio of cement and Ottawa sand was 1:3 and water-cement ratio was 0.4 in the mix for preparing the specimens according to BS 12 1996 [British Cement Association, 1998]. As the test results were satisfactory, the cements from the same manufacturer, which were collected from the same supplier, were used in the entire investigation.

4 STRENGTH OF SAND-CEMENT MIXES

Unconfined compressive strength (UCS), flexural strength (FS), indirect tensile strength (ITS) and California Bearing Ratio (CBR) of the mixes were determined. The choice of these parameters, particularly the UCS and CBR, for quality assessment was largely based on the fact that these have been widely used by previous researchers and also by several road-controlling authorities. The FS and ITS were mainly chosen because cement stabilised layers can be subjected to flexural loading due to traffic.

Fine to medium sands were mixed with a range of cement and water contents and compacted for preparing unsoaked and soaked UCS, FS, ITS and CBR specimens. Mixes investigated are shown in Table 1.

4.1 Moisture sensitivity and UCS values of the sand-cement mixes

The uniformly graded fine to medium sand and Portland cement of different proportion with a range of percentages of water were thoroughly mixed together for preparing cylindrical specimens (height 76 mm, diameter 38 mm) for the UCS tests.

<table>
<thead>
<tr>
<th>The mixtures</th>
<th>UCS for 7,14 and 28 days curing</th>
<th>Durability test</th>
<th>Flexural strength for 7, 14 and 28 days curing</th>
<th>CBR of 4 day soaked specimens</th>
<th>Indirect tensile strength for 7 days curing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand + 6% cement</td>
<td>X</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Sand + 8% cement</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Sand + 10% cement</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Sand + 12% cement</td>
<td>X</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Sand + 15% cement</td>
<td>X</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Sand + 20% cement</td>
<td>X</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

Fine to medium sand with 20% cement by weight of the dry sand and water above and below 2% water content above and below optimum moisture content (OMC) were mixed together and compacted to prepare the UCS samples (OMC = 13.5%). The 7-day cured strength of the specimen below OMC showed much higher strength (3.34 MPa) in compared to the specimen above OMC (1.97 MPa). The same tests with 8 and 12% cement showed similar behaviour.

Specimens for the sand-cement mix with 8% cement (OMC = 12.25%) and 10% cement (OMC = 12.5%) of the dry weight of sand (FM = 0.9) were tested for UCS at moisture content 2 and 4% below optimum moisture content (OMC). Interestingly the specimens at 4 percent below OMC gave slightly higher strength than those at 2 percent below OMC. It proved very difficult to prepare specimens at moisture content further below OMC (6% below), as these tended to break during extraction. The UCS values are presented in Table 2.

4.2 Durability

The soaked tests were performed on selected specimens in accordance with Test 12 in BS 1924 [British Standard, 2002b]. The compressive strengths of 14-day cured specimens were compared with 7-day cured and 7-day soaked specimens. The selected sand-cement mixes for soaked test were fine to medium sand mixed with 8 and 10% cement (2% below OMC). These mixes were selected because they might provide acceptable compressive strength and expected to be cost-effective mixes. The strength reduction on soaking for specimens with 8 and 10% cement were 9.4 and 8.95% respectively as shown in Figure 2.

Table 2. Comparisons of UCS below 2% and 4% OMC.

<table>
<thead>
<tr>
<th>Moisture content</th>
<th>Sand + 8% cement (7-day cured UCS) (MPa)</th>
<th>Sand + 10% cement (7-day cured UCS) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Below 2% OMC</td>
<td>1.62</td>
<td>2.27</td>
</tr>
<tr>
<td>Below 4% OMC</td>
<td>1.76</td>
<td>2.44</td>
</tr>
</tbody>
</table>
According to the British Standards the strength reduction should not be more than 20 percent [British Cement Association, 1998]. These soaked tests also showed negligible changes in both the volumes and weights of the specimens.

### 4.3 Discussion of the results

The UCS specifications for soil-cement materials of TRL Overseas Road Note 31 [1993] are listed in Table 3. UCS for the above mixes and the comparisons with the specifications are shown in Figure 3.

<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
<th>Unconfined compressive strength of 150 mm cubical specimens (a) (MPa)</th>
<th>UCS (psi) of 2:1 (height:dia) cylindrical specimens (a/1.25) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB1</td>
<td>Stabilised roadbase for heavy traffic</td>
<td>3.0–6.0</td>
<td>2.4–4.8</td>
</tr>
<tr>
<td>CB2</td>
<td>Stabilised roadbase for light to medium traffic</td>
<td>1.5–3.0</td>
<td>1.21–2.4</td>
</tr>
<tr>
<td>CS</td>
<td>Stabilised subbase</td>
<td>0.75–1.5</td>
<td>0.6–1.21</td>
</tr>
</tbody>
</table>

Figure 2. Durability test results of sand-cement mixes.

Table 3. Standard UCS results (7-day cured-7-day soaked) (after TRL Road Note 31, 1993).

Figure 3. UCS values of sand-cement specimens with a range of moisture and cement content.

688
Based on the soaked UCS results, according to TRL Overseas Road Note 31 [1993], the fine to medium sand mixed with 8 and 10% cement (1.64 and 2.35 MPa) was acceptable for roadbases (CB2). Other specifications provided by Davidson [1960] and Maclean and Lewis [1963] require 7-day UCS of 1.72 MPa for roadbases of lightly trafficked roads and Department of Transport [1986], used in the United Kingdom, specifies a minimum compressive strength of 2.76 MPa for cylindrical samples (approximately equal to 3.45 MPa for cube samples) in soil-cement roadbases of heavily trafficked roads. Therefore, the sand-cement materials with 8 and 10% cement would be acceptable for roadbases of light traffic considering the UCS and durability tests. However, due to adverse field conditions, possibility of improper mixing, lack of quality control and lack of legal enforcement of maximum axle load it would be better to use the mix (fine to medium sand mixed with 8 to 10% cement) in the subbase courses of heavily trafficked roads in the context of Bangladesh. More than 6 million Equivalent Standard Axle (ESA) could be considered here as heavy traffic [TRL Overseas Road Note 31, 1993].

5 FLEXURAL STRENGTH (FS) AND INDIRECT TENSILE STRENGTH (ITS) TESTS

Pavement layers are subjected to flexural stresses to a greater or lesser extent depending on their position. Thus it was deemed necessary to determine flexural strength of selected mixes. The mixes acceptable in terms of UCS and durability (fine to medium sand, 8, 10% cement) for subbase layers were selected for static flexural tests. The flexural strengths of the specimens containing 8 and 10% cement and cured for 7 days were 0.41 MPa and 0.64 MPa respectively. The flexural modulus of elasticity at 7, 14 and 28 days cured flexural strength test specimens were also determined and are shown in Table 4.

The same materials as those used in static flexural strength tests were investigated for indirect tensile strength. This test was carried out in accordance with National Institute for Transport and Road Research [1986]. The 7-day cured tensile strength values for fine to medium sand mixed with 85 and 10% cement was 0.22 MPa and 0.28 MPa respectively. According to Williams [1978], flexural strength should be about one-fifth of the uni-axial compressive strength measured on cylinders to rely upon in most cases in the soil-cement pavement layers. According to National Institute for Transport and Road Research [1996], the flexural strength of cemented materials is about one-third of the compressive strength for low-strength materials and about one-fifth of the compressive strength for high strength materials. In this study, in the case of sand mixed with 8% and 10% cement the 7-day cured flexural strength was about one-forth and one-third of the 7-day cured unconfined compressive strength respectively. The flexural strengths and specifications are presented in Figure 4. Both the mixes are likely to display allowable load-deformation characteristics and also likely to prevent unwanted fatigue cracking if they are used in the structural layers of pavements.

Normally the tensile strength of a cemented material should be minimum one-tenth of the unconfined compressive strength value of the same mix compacted in a similar way [Thom, 2004]. According to National Institute for Transport and Road Research [1996], the tensile strength is about one-eighth of unconfined compressive strength. Dunlop [1980] pointed out that a cemented soil needs to have a 7-day cured tensile strength of greater than 0.08 MPa for using in a subbase course. Even though there is no recommended value for indirect tensile strength for soil-cement to be considered for a particular pavement layer, the tests allowed predicting the behaviour of the mixes under tension. The indirect tensile strength values of selected mixes are also presented in Figure 4.

6 CALIFORNIA BEARING RATIO

The four-day soaked CBR tests of fine to medium sand mixed with 8% and 10% cement were performed in the laboratory. Four-day soaked specimens were selected because in Bangladesh normally the critical CBR values are measured in this way. The average of three values was taken. The average CBR values for 8% and 10% cement mixes became 275.3% and 347.8% respectively as shown in Figure 5.

According to Metcalf [1977], 4 day soaked specimens’ CBR value for subbase and also base courses (roadbases) of light traffic should be within 50% to 150%. For base courses (roadbases) of heavy traffic the CBR value should be within 200 to 600%. The CBR values of the materials investigated could be used in roadbase layers of heavily trafficked roads. The CBR values are incorporated in Figure 5.

<table>
<thead>
<tr>
<th>Mixes</th>
<th>Flexural modulus of elasticity (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand + 8% cement</td>
<td>624.91</td>
</tr>
<tr>
<td>Sand + 10% cement</td>
<td>1298.19</td>
</tr>
</tbody>
</table>

Table 4. Flexural modulus elasticity values for 7, 14, 28 days cured flexural test specimens.
Many researchers have suggested due to the possibility of reflection cracking that the soil-cement materials could perform better in the subbase layers rather than in the roadbases. Arnold [2000] said that the stabilized materials should not be accepted as roadbase materials unless the tensile strength and shrinkage are within the allowable values. This requirement will probably limit the amount of cement added to a maximum of 2%. However, higher cement contents could be considered if there are provisions for additional maintenance requirements to seal cracks when comparing alternatives. Therefore, an “upside-down” pavement could be acceptable. The use of granular roadbases above the stabilised layer could arrest the shrinkage and traffic associated cracks to propagate in the surface courses. Illustrating the problems of reflection cracking Williams [1978] depicted,

“In the case of cement bound material and soil-cement, restrictions on their use in roadbases for heavily trafficked roads have effectively designated them as subbase materials in either flexible or rigid pavements”.

7 POSSIBLE APPLICABILITY

Figure 4. Flexural and indirect tensile strength of sand-cement mixes.

Figure 5. CBR values of sand-cement mixes.
Djakfar [1999] tested several pavements using different combinations of crushed stone and soil-cement bases and subbases with the same wearing course. As many as nine test sections were constructed. From the analysis he found that the combination of crushed stone base over a soil-cement subbase, known as an inverted pavement, provided the best performance in resisting rutting and retarding the occurrence of reflection cracking. Furthermore the stabilized sub-base layer stiffens the overall pavement system, thus reducing the likelihood of physical degradation occurring in the unbound base [Dunlop, 1980].

According to National Institute for Transport and Road Research [1996], cracks in bituminous surface have been observed mostly in pavements where the cement stabilized layer has been used in the base courses. However, the post-cracked phase of a cement-stabilized subbase adds substantially to the useful life of a pavement. The suitable material for the subbase layers have been obtained by performing the standard tests as discussed above. However, it was predicted that filling the voids in the sand-cement mixes for obtaining well-graded mixtures could enhance strengths of the mixes. Therefore, silts were added in the sand-cement mixes for obtaining materials for using in the roadbases.

8 SAND-SILT-CEMENT MIX

In this investigation the fine to medium sands were mixed with a range of silt and cement content and compacted for preparing unsoaked and soaked UCS, ITS specimens. A slightly coarser fine to medium sand (FM = 1.1) and silt particles were used in this investigation. This sand was used because the coarseness of available fine to medium sands varies from FM = 0.8 to 1.2. Sand and 10, 25 and 50 percent silt by weight of the sand were investigated with 10, 12 and 15 percent cement content by weight of the mix. All the mixes in this investigation gave very high compressive strength varying from 2.52 MPa to 6.46 MPa, as shown in Table 5.

8.1 Discussion of the results

UCS values and the specifications are shown in Figure 6.

The 7-day cured strengths of the above specimens except 10% and 25% silt and 10% cement were higher than the mix of fine to medium sand with 15% cement only (2% below OMC) (2.79 MPa, 7-day cured). This is mainly because the silt particles filled the voids within the uniform sand particles and formed a well-graded mixture. The mixes with 15% cement showed greater 7-day cured strength than the sand-cement mix with 20% cement. 10 and 25% silt content did not provide a great deal higher compressive strengths with 10% cement than only sand with 10% cement. This is probably because 10% cement was not sufficient to increase the strengths appreciably though the voids are partially filled by the silt particles. However, the mixes gave good strengths due to nature of the well-graded mixtures. 50% silt with 10% cement gave high strength because most of the voids were filled by the silt particles and cement paste was free to make bonds among the soil particles. 10, 25 and 50% silt content gave high strengths with 12% cement due to the same reasons as above.

According to the specifications provided by Maclean and Lewis [1963], Sherwood [1968] and Department of Transport [1986], soil-cement bases of heavily trafficked roads the minimum compressive strength of cylindrical specimens should be 2.75 MPa. Therefore, all the mixes discussed above gave acceptable compressive strength to be used in roadbases for heavily trafficked roads except the mixes with 10%, 25% silt and 10% cement.

9 DURABILITY TEST

As far as compressive strength, adverse field conditions, possibility of improper mixing, lack of quality control, lack of maximum axle load legal enforcement and cost are concern the mix fine to medium sand, 10% sieved silt and 15% cement was considered to be the best among the above materials for roadbases of heavily trafficked roads [TRL Road Note 31, 1993]. Therefore, soaked tests were carried out with this mix. The result showed strength reduction on soaking only about 8.4% (see Figure 7). Durability tests of fine to medium sand mixed with 50% silt and 12% cement were also carried out because the UCS of this mix was high and acceptable for roadbases of heavy traffic. The strength reduction on soaking was only about 4%. However, the cost of preparing such a mix may be prohibitive in practice.
10 INDIRECT TENSILE STRENGTH TEST

The indirect tensile strength test with the above sand-silt-cement mix was also carried out. The 7-day cured tensile strength was 0.49 MPa. The value is more than one-ninth of the compressive strength value. Therefore, the mix can probably be considered for roadbases in tensile strength consideration, as normally the strength ratio is one-tenth. The tensile strength value is shown in Figure 8 below.
11 POSSIBLE APPLICATION AND PERFORMANCE OF THE STABILIZED SILTY SAND

The sand-silt-cement roadbase material could be used in road construction and would probably be cost-effective. However, it needs to be proven. Reflection cracking consideration a thick surfacing of more than 100 mm has to be provided in this case. Another problem with this material is that the sieved silt would be difficult to obtain in the field. However, if the commercial importance of this material can be established the local cement companies in Bangladesh may become interested to produce the sieved silts commercially.

12 TRIAL CONSTRUCTION

Two materials showed encouraging laboratory results were fine to medium sand mixed with 10% cement for subbase layers and fine to medium sand mixed with 10 percent silt passing 0.075 mm sieve and 15% cement for roadbase layers. Both the materials showed satisfactory unconfined compressive strength, durability, flexural strength, tensile strength and CBR. However, the diversified field conditions in terms of material variability and construction quality needed to be considered to assess whether the materials could perform well in the field. Pilot field trials were therefore considered to prove the materials.

Owing to the limited funds and time only one field trial with fine to medium sand mixed with 10% cement was constructed as a subbase layer. A small section was constructed to observe the primary surface erosion and cracking of the surface and durability of the material.

12.1 Trial section

A 3.6 m by 3 m section was constructed with the selected material. The fine to medium sand was spread into 0.13 m deep excavation and all the materials were mixed manually as would be the case in practice.

Prior to starting the construction the moisture in the sand was adjusted in the field to provide moisture content to final 1% below optimum moisture content of the mix. Laboratory results showed better performance for mixes at lower moisture content; however, it was felt that some water would be lost due to evaporation during mixing. When final mixing was completed on a half of the strip (1.8 meter by 1.5 meter), an eight-ton smooth wheeled roller was passed for 2 times. After that another half was prepared and compacted in the same way. Finally total section was compacted by passing the roller 4 times over the surface until the surface looked hard and dense.

The dry density of the compacted material was measured from the 7 days cured core specimens, the field dry density measurement should be made at least 4 hours after the completion of the compaction work and preferably within a period of 24 hours [Williams 1978]. However, it was not possible to measure field dry density using the sand replacement method after 4 hours of compaction, as the material was stiff. This would have cracked the pavement. The dry density achieved was approximately 96% of the laboratory density according to the measured dry densities of 7-day cured cores.

12.2 Core strength and laboratory results

The core strengths for 2:1 cylindrical samples were 2.52 MPa, 2.77 MPa and 3.02 MPa for 7, 14 and 28...
days curing time respectively. Core strengths were slightly higher than the strengths found in the laboratory (see Figure 8).

This may have been because the fine to medium sand used in the trial had a better grading with an FM of 1.1 whereas in the laboratory during sand-cement strength tests sand of FM 0.9 was used. The other reason might be due to presence of warmer curing condition in the field. Mixing in the field may not have been as thorough as in the laboratory and the cores taken from the places where cement content might have been a little higher due to variable mixing. In the trial construction the weight of the sands was determined by loosely pouring the materials in a container and after that the weight of further sands were assumed by pouring the same container loosely. As the weight was assumed during the construction the cement content might have been a little higher.

For labour intensive construction of the stabilised layer it would always be safer to use 10% cement with the sand though 6% and 8% cement gave acceptable results in the laboratory to be used in subbase layers. For equipment based construction with close quality control 8% cement (mixed with fine to medium sand) should be sufficient for subbase layers.

13 ECONOMIC BENEFIT

Cost saving of about 17 and 28% for subbase and road-base can be made by using the above mentioned soil-cement in pavement structural layers. See Figure 9.

Stabilised layers certainly provide additional benefits in terms of cost savings and longer life when compared with the conventional unbound construction. With the limited availability of high quality aggregates and the increasing cost of carrying them to the construction site, use of stabilised local materials in the pavement structure must continue to increase.

14 CONCLUSIONS

The following conclusion are drawn from this study.

14.1 Sand-cement mixes

- Mixes with 6% cement gave adequate strength to be used in the subbases. Mixes with both 8% and 10% cement showed sufficient unsoaked and soaked strengths for using in roadbase layers. However, these mixes may need to be used in subbase layers if the possibility of variable mixing in field conditions and also variable compaction are anticipated.
- British Standard soaked UCS tests of the sand-cement specimens with 8% and 10% cement showed only marginal reduction (less than 10%) in compressive strengths, therefore, the materials are expected to perform satisfactorily on inundation and under traffic.
- The trial section showed good core strengths and also in-situ labour intensive mixing was good enough to show very little variation among the individual cores. After more than six months of construction the trial has not shown any cracks or rutting.

14.2 Sand, silt and cement mixes

- The addition of the fine silt in the sand-cement mix improved strength values further. The specimens with 12 and 15% cement gave high enough strength to be used in roadbases for heavily trafficked roads. However, minimum silt content in the mix has to be considered, as it would be very difficult to get large amounts of silt. In this regard 10 percent silt and 15% cement would be more beneficial to be used in roadbases among these mixes.
REFERENCES


Thom, N. [2004]. Lecture Note of the Professor of Nottingham University. Department of Highway Management and Engineering, University of Birmingham, UK.


Monitoring of strawbale and non-food-crop based walling systems

S. Goodhew, R. Griffiths, P. de Wilde
*Environmental Building Group, School of Engineering, University of Plymouth, Devon, UK*

T. Simmons
*Genesis Centre, Somerset College of Arts and Technology, Taunton, Somerset, UK*

ABSTRACT: This paper describes the rationale for using non-food-crop materials in buildings and briefly reviews the possible uses of such materials as construction materials. The transfer of strawbale building techniques in Western Europe, from established technologies in the North America’s is outlined. Climatic factors, combined with the major concerns related to the deterioration of strawbale walls through the action of moisture are discussed. A commonly used monitoring technique is described, and case studies provided. Adaptations to monitoring systems to allow appropriate measurements of new and emerging straw bale building techniques are described. Two new case study buildings are introduced and the preliminary results of a proposed monitoring strategy are briefly analysed.

1 RATIONALE FOR THE USE OF NON-FOOD-CROP MATERIALS IN BUILDINGS

1.1 Type area

The choice of the type of materials used by the construction industry has a direct impact upon the performance of buildings in terms of their sustainability. Sustainable issues and impacts are closely associated with material extraction, collection, embodied energy content, transportation and growing processes. Other issues influence the performance of the building such as energy efficiency, any end of life or disposal issues related to the demolition or reuse of any building components. These concerns influence the decisions taken by individuals and institutions responsible for the specification of such materials.

A raft of different agreements have emanated from world environmental meetings such as those held in Rio, Kyoto, Johannesburg over the past 14 years. From these meetings, a number of agreements have lead to the introduction of targets and guidance, much of which have been directly related to reducing the harmful emissions and other impacts of buildings. One of the many methods of complying with these guidance and targets is to use building materials that are more benign, both in use and also in their manufacture and sourcing. A range of benchmarking, guidance and indicator systems are now available that allow the building professional to select appropriate materials and quantify the level of potential harm in a variety of categories. Some guides focus upon the relationship between health and the environment. [Curwell & March, 2002]. Others have a more environmental steer [Woolley et al, 2001]. The environmental preference method [Anink et al, 1996] compares materials and products currently on the market and ranks them according to their environmental attributes. Normally such methods do not consider cost or aesthetics, but mainly focus upon issues such as;

- harmful emissions, both local and global in nature
- energy consumption at all stages
- ecological damage caused by the extraction of raw or component materials
- embodies water content
- health related issues
- maintenance
- waste related issues including reuse.

Using the above criteria, or a very similar version, it can be quite clearly seen that using waste derived non food crops, such as straw, will inevitably be a good way to score well in most categories.

Straw bales are just one of a number of grown materials that are being advocated for use in the UK construction Industry. Tim Yates of the Centre for Construction Consultancy [Yates, 2006] has reviewed the utilisation of such non-food crops within buildings in the UK and identified the following current and possible uses in table 1.

Although Yates identifies a wide range of current and possible uses for non-food crops within buildings,
this paper will concentrate on the use of straw bale walling systems and investigate the moisture performance of such walling systems.

2 STRAW, DURABILITY AND MOISTURE

Properties of straw bales have been investigated by a number of authors. Several diverse pieces of research have investigated the structural properties of walling systems built from straw bales, [Faine and Zhang, 2002] and [Taylor et al, 2005]. Mas and Everbach have looked into the acoustics of plastered straw bale walls, including the transmission characteristics of such walls. The thermal properties of straw based insulation have also been measured [Christian et al, 1998]. However if these straw based products are to be seriously considered for widespread use in buildings, the long term performance needs also to be investigated; in particular long term durability. In this paper it is intended to provide the reader with the preliminary results of a system for monitoring one of the major perceived issues connected with straw bale walling systems, that of long term durability.

Straw and other non-food crop building products, due to their organic origin, can suffer from degradation in connection with specific moisture contents which are discussed later in this paper. With the increasing transfer of straw bale building techniques to Western Europe, the established technologies mostly developed over many years in the North America’s are being used in strawbale buildings in the UK.

The climate of traditional strawbale building areas of North America, such as Nebraska and Quebec, are quite different to the maritime nature of the UK climate. This factor combined with the major concern of deterioration of strawbale walls through the action of moisture, invariably means that strawbale walls and other non-food-crop based materials may not behave in the same manner as those in non-maritime climates.

The possible degradation of straw bale walls and other non-food crop materials is a potential barrier to lenders, buyers and occupiers of these types of buildings. Within the UK, the local authority is responsible for policing the building regulations normally through internal building control officers or external approved inspectors. These officers/inspectors are often wary of new building techniques and the ability for the occupier/owner to monitor the condition of a straw bale/non food crop based building will normally instill certain amount of confidence. To improve the saleability of such buildings will also involve the owner/occupier/contractor taking out a form of insurance system or warranty such as the NHBC system in the UK. A monitoring system will again, be likely to improve the chance of a warranty on the property being successfully enacted. A method of monitoring the moisture content, and therefore the potential for degradation of the walls is one method to establish confidence in these materials/technologies.

The generally accepted danger moisture content in connection with increased risk of the degradation of the straw is 20% [Still, 1997]. It has been suggested that the ideal moisture content of a straw-bale wall is 14%, as this is below the level that is believed to allow biological activity to begin [Steen et al, 1994]. The main difficulty for a building owner/occupier is to assess the moisture content of the wall once the internal and external coverings/render are in place. This is further compounded by the normal thickness of such walling systems which hampers the access to the central parts of the wall, even if the walls were still bare. A solution to this monitoring problem was originally proposed by Habib Gonzalez a Straw builder from British Columbia, Canada, [Gonzalez, H. J., 1998] and pioneered in eastern Canada by Fugler, (2000).

3 THE MONITORING SYSTEM AND PAST CASE STUDIES

The monitoring system relies upon the ability for the moisture content and timber to be measured using a relatively simple electrical resistance meter. There is a further reliance, that of the assumption that a thin wood
buildings solid earth floor, probes were inserted into on a straw clay light earth building. Because of the probes at the lower levels. high up in the wall were considerably drier than the and at its face closest to the exterior. The upper probes, moisture contents were highest at the foot of the wall of measurements included plenty of obvious lessons; effects/influences. The lessons learned from this series ture from the external climate, excluding other that would still be attributable to the ingress of mois- vide evidence of a pattern of moisture content readings the same unoccupied building. The monitoring did pro- inside a fully occupied building when compared with moisture variations and temperatures normally found inside a fully occupied building when compared with the same unoccupied building. The monitoring did pro- vide evidence of a pattern of moisture content readings that would still be attributable to the ingress of mois- ture from the external climate, excluding other effects/influences. The lessons learned from this series of measurements included plenty of obvious lessons; moisture contents were highest at the foot of the wall and at its face closest to the exterior. The upper probes, high up in the wall were considerably drier than the probes at the lower levels.

A second series of measurements was undertaken on a straw clay light earth building. Because of the buildings solid earth floor, probes were inserted into the walls horizontally. The probes were inserted, 200 mm and 400 mm above finished floor level, with a final probe at high level, often 300 mm below the top of the wall. The walls were constructed in the wettest autumn for 10 years and for a period of time exhibited higher moisture content readings than were anticipated in walls of this type. This was further compounded by some failures of the lime render of the wall facing the prevailing wind. Readings above the advised 20%, moisture content were noted in several parts of the wall, but after a number of months, the readings did drop to acceptable levels. The pattern of the readings when analysed against the location/position of the probes showed a pattern that was hinted at by the first case study. The two lower probes showed higher moisture content readings, with the probe often 3–5% higher than the 400 mm probe. A third case study [Goodhew et al, 2005], built from light earth and clay wood chip walls, replicated the monitoring system placement, of case study 2 as closely as possible. This was hindered because of a novel 'in-wall' piped heating system, embedding narrow gauge flexible piped in the internal earth plaster coating. This aspect of case study 3 did highlight the effect of heating upon moisture migration across walling systems made from organic materials. Again the pattern of higher moisture contents in the lower reaches of the walls was, in part repli- cated. Confidence in the technique, both through the laboratory calibration and the patterns exhibited by the case study buildings; persuaded the team that the undertaking of a more comprehensive study of two further buildings was warranted. As these two build- ings exemplified new factors of construction, occupa- tion and use the monitoring systems were adjusted to account for these factors. Initial monitoring of the Genesis Centre, Taunton commenced in early 2006 and the Totnes home, Totnes commenced in late 2006. The two buildings are very different, both in construction, use and occupancy.

4 GENESIS CENTRE, STRAW BALE PAVILION

The straw bale pavilion is approximately a quarter section of the Genesis Centre, Somerset College, Taunton Somerset. The Genesis Centre is a collection of 4 different pavilions, each constructed as a showcase of a unique sustainable construction method ort material. Each pavilion has a different use connected with education at the College and the Straw Bale pavilion serves as a group of 3 seminar rooms as can be seen in the figures 2 & 3. The pavilion uses a load bearing rendered straw construction technique (figure 2) and endeavors to provide the college with a modern and unique design, shying away from traditional straw building techniques and aesthetics exemplified in straw bale buildings elsewhere in the UK. The occupancy of
the pavilions will be transient, only evident during the
daytime and provide little obvious sources of internal
moisture. However, the unique design features, particu-
larly the differences from traditional practice, includ-
ing a small eaves overhang and sections of straw bale
end walls that protrude into areas that are perceived to
more vulnerable to weather. The design warrants fur-
ther interest due to effect that the under-floor heating
system may have upon the moisture content of the
lowers bales nearest to the heated areas. The design
warrants further interest due to effect that the under-floor heating
system may have upon the moisture content of the
lowers bales nearest to the heated areas. The novel tim-
ber beam combined DPC will offer the research team
another new form of technology to assess. It is hoped
that the sheer number of novel features will still enable
the results to isolate both cause and effect as far as the
moisture content of the bale walls.

The majority of the moisture probes were placed in
the outer exposed walls of the pavilion with a smaller
number placed in the central internal bale walls to act
as comparators (figure 3). The main development of
the monitoring system involved a number of extra-
long probes that were produced in an attempt to allow
appropriate access to the buttress walls. These walls
extended the outer ends of the bale walls beyond the
external glazing and were too deep in section to allow
the normal 300 mm long probes to measure the outer
volume of straw. Previously no sensors/probes of this
length had been used and so the research team is keen
to know how the sensors perform.

5 THE TOTNES HOUSE

The Totnes house is very different; both in construc-
tion and occupancy. The Totnes house is located on a
north west facing slope of a hill in Totnes, Devon. The
homes, as its name implies offers accommodation for
a family of four with all the associated occupancy
rates and sources of moisture. The house is two sto-
ries high, constructed from a load bearing timber
frame with the straw bales used on edge as an exter-
nal cladding. The design is, in some ways, more tradi-
tional than the Genesis Centre, providing quite large
eaves, overhangs, tending to protect the wall from any
wind driven rain. The bales differ from the Genesis
centre bale placement as the Totnes bales are placed
on edge and because of this some extra structural sup-
port was deemed necessary. A timber ladder system
has been used horizontally through the centre of the
bale walls to enhance its structural stability (figure 4).
However, as the walls in the Genesis Centre are load
bearing and do not rely upon a frame for support,
interesting comparisons are possible.

A new probe, a half length version has been used for
the first time as a part of the monitoring system with
the Totnes house. The intention is that the volume of
straw closest to the internal wall faces adjacent to the
anticipated higher moisture areas, can be monitored. These are situated in any areas used in either cooking or bathing spaces. This property is the first to use these shorter probes and the research team are hoping that they will be able to detect any interior %RH influence as opposed to exterior moisture ingress.

A series of internal and external humidity and air temperature measurement devices have been installed in both case study buildings in an attempt to link outside and inside influences upon the moisture measurements in the wall. The users/owners of each building will be recording the data, initially for 2 weekly intervals, then monthly.

6 INITIAL MONITORING RESULTS

From previous case studies it was found that the results from moisture monitoring using wood disc sensors, although reasonably accurate and repeatable, a time lag of up to several months was encountered between conditional external to the wall and the effect of those a proposed monitoring strategy are briefly analysed conditions being measured. This is in part due to the slow diffusion of moisture through relatively dense straw bales and the delay in the wood block sensor/probe reacting to the increase/decrease of the moisture content of the straw surrounding the sensor/probe. Therefore the first measurements taken by any moisture monitoring system can be unreliable. In the light of this the reader at this point might quite rightly question the worth of the following results. However, the relative magnitude of the measurements and any general trends are likely to be informative concerning the ‘health’ of the straw bale walls.

The table shown in table 2, shows the first two series of measurements taken for the Genesis straw bale pavilion. LT1 and LT2 represent the Lecture Theatre 1 and 2 respectively. E represents East, W, west, I inner and O outer sensors/probes. 200, 400, 1800 represent the heights in mm of each sensor/probe above finished floor level.

As can be seen, the results for all elevations and heights of sensors are stable between the months of October and November and well within the accepted norms of around 15%.

The table below, Table 3, shows the first two series of measurements taken for the Totnes straw bale house. The table shows the different types of probes including the shortened versions and the type of external cladding applied to the area closest to the position of the relevant probes.

The results from the Totnes moisture probes mimic the Genesis centre results in that they are stable and well within the acceptable boundaries for wall moisture content. However, the lower probes do show a higher moisture content reading than those of the higher probes. This pattern is further coloured by the fact that the shorter probes are constantly reading a small percentage higher than the long probes, hinting that the moisture content of the external parts of the walls is more of an influence at this moment in time, than the interior conditions.

7 CONCLUSIONS

Although, as previously stated, little can be extrapolated from a small series of moisture readings from a straw bale walling system, some pieces of information are beginning to form a pattern. The stability of both buildings is far better than any of the previous case studies that have been monitored. This is further marked as the monitoring system was installed with the previous knowledge of the past monitoring and therefore the sensor/probe positioning is likely to have a better chance of picking up any areas/volumes of higher moisture. The overall drier readings from the sensors/probes in the Genesis pavilion may be due to the more sealed modern design, or perhaps the

<table>
<thead>
<tr>
<th>Room</th>
<th>Date</th>
<th>EO 200</th>
<th>EO 400</th>
<th>EO 1800</th>
<th>EI 200</th>
<th>EI 400</th>
<th>EI 1800</th>
</tr>
</thead>
<tbody>
<tr>
<td>LT 1</td>
<td>19/10/2006</td>
<td>11</td>
<td>11</td>
<td>12</td>
<td>12</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>LT 2</td>
<td>19/10/2006</td>
<td>12</td>
<td>14</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>LT 1</td>
<td>20/11/2006</td>
<td>9</td>
<td>11</td>
<td>12</td>
<td>12</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>LT 2</td>
<td>20/11/2006</td>
<td>11</td>
<td>12</td>
<td>12</td>
<td>11</td>
<td>11</td>
<td>11</td>
</tr>
</tbody>
</table>

Table 2. Initial monitoring results for the Genesis straw bale pavilion, (LT1 = Lecture Theatre 1, LT2 = Lecture Theatre 2, W = west, E = east, O = outer, I = inner, numbers = height above finished floor level in mm).
novel timber beam combined DPC. Only the next stage of monitoring will give the research team the evidence to establish any correlations.

Since the two sets of walls are being monitored after a relatively dry and quite long summer, it may not be surprising that the bale walls are starting at a low value of moisture content. However, some quite heavy periods of rain have occurred during the monitoring period and this appears not to have heavily influenced any of the readings yet. The internal lowest short sensors in the Totnes house have been showing a higher outer reading, typically just over 13% than the interior readings typically just over 12% from the short sensor/probe. It will be during the next 6 months, when the average rain fall for West Somerset and South Devon are in the upper range for the UK, that the research team hypothesize that the sensors/probes will start recording.

As the heating season has not been underway long enough it is not possible to compare the under-floor heating system of the Genesis centre with any effects that might be noticeable from the traditional wet radiator system installed in the Totnes house.

ACKNOWLEDGEMENTS

The Authors wish to thank the National Non-Food-Crop Centre, York and the Ecology Building Society for their kind support.

REFERENCES


Table 3. Initial monitoring results for the Totnes house.

<table>
<thead>
<tr>
<th>Room</th>
<th>Height of probe</th>
<th>Length of probe</th>
<th>1st or ground floor</th>
<th>Orientation</th>
<th>external wall finish</th>
<th>14th Nov reading</th>
<th>24th Nov reading</th>
</tr>
</thead>
<tbody>
<tr>
<td>shower</td>
<td>200 mm short</td>
<td></td>
<td>ground</td>
<td>northwest</td>
<td>25 mm lime</td>
<td>12.9</td>
<td>12.6</td>
</tr>
<tr>
<td>shower</td>
<td>200 mm long</td>
<td></td>
<td>ground</td>
<td>northwest</td>
<td>25 mm lime</td>
<td>13.5</td>
<td>13.4</td>
</tr>
<tr>
<td>shower</td>
<td>2200 mm short</td>
<td></td>
<td>ground</td>
<td>northwest</td>
<td>25 mm lime</td>
<td>10.2</td>
<td>10.0</td>
</tr>
<tr>
<td>shower</td>
<td>2200 mm long</td>
<td></td>
<td>ground</td>
<td>northwest</td>
<td>25 mm lime</td>
<td>12.8</td>
<td>12.7</td>
</tr>
<tr>
<td>bath</td>
<td>200 mm short</td>
<td></td>
<td>ground</td>
<td>southwest</td>
<td>25 mm lime</td>
<td>12.7</td>
<td>12.3</td>
</tr>
<tr>
<td>bath</td>
<td>200 mm long</td>
<td></td>
<td>ground</td>
<td>southwest</td>
<td>25 mm lime</td>
<td>13.5</td>
<td>13.1</td>
</tr>
<tr>
<td>bath</td>
<td>2200 mm short</td>
<td></td>
<td>ground</td>
<td>southwest</td>
<td>25 mm lime</td>
<td>11.9</td>
<td>11.6</td>
</tr>
<tr>
<td>bath</td>
<td>2200 mm long</td>
<td></td>
<td>ground</td>
<td>southwest</td>
<td>25 mm lime</td>
<td>/</td>
<td>/</td>
</tr>
<tr>
<td>hall</td>
<td>200 mm short</td>
<td></td>
<td>ground</td>
<td>southwest</td>
<td>25 mm lime</td>
<td>12.8</td>
<td>12.6</td>
</tr>
<tr>
<td>hall</td>
<td>200 mm long</td>
<td></td>
<td>ground</td>
<td>southwest</td>
<td>25 mm lime</td>
<td>13.2</td>
<td>13.0</td>
</tr>
<tr>
<td>hall</td>
<td>200 mm short</td>
<td></td>
<td>first</td>
<td>southwest</td>
<td>timber cladding</td>
<td>10.9</td>
<td>10.6</td>
</tr>
<tr>
<td>hall</td>
<td>200 mm long</td>
<td></td>
<td>first</td>
<td>southwest</td>
<td>timber cladding</td>
<td>11.8</td>
<td>11.6</td>
</tr>
<tr>
<td>larder</td>
<td>200 mm short</td>
<td></td>
<td>first</td>
<td>northwest</td>
<td>timber cladding</td>
<td>10.7</td>
<td>11.5</td>
</tr>
<tr>
<td>larder</td>
<td>200 mm long</td>
<td></td>
<td>first</td>
<td>northwest</td>
<td>timber cladding</td>
<td>/</td>
<td>/</td>
</tr>
<tr>
<td>larder</td>
<td>2020 mm short</td>
<td></td>
<td>first</td>
<td>northwest</td>
<td>timber cladding</td>
<td>/</td>
<td>/</td>
</tr>
<tr>
<td>larder</td>
<td>2200 mm long</td>
<td></td>
<td>first</td>
<td>northwest</td>
<td>timber cladding</td>
<td>/</td>
<td>/</td>
</tr>
<tr>
<td>kitchen</td>
<td>200 mm short</td>
<td></td>
<td>first</td>
<td>northwest</td>
<td>timber cladding</td>
<td>11.5</td>
<td>11.4</td>
</tr>
<tr>
<td>kitchen</td>
<td>200 mm long</td>
<td></td>
<td>first</td>
<td>northwest</td>
<td>timber cladding</td>
<td>11.4</td>
<td>11.4</td>
</tr>
<tr>
<td>kitchen</td>
<td>2200 mm short</td>
<td></td>
<td>first</td>
<td>northwest</td>
<td>timber cladding</td>
<td>11.8</td>
<td>11.3</td>
</tr>
<tr>
<td>kitchen</td>
<td>2200 mm long</td>
<td></td>
<td>first</td>
<td>northwest</td>
<td>timber cladding</td>
<td>11.4</td>
<td>11.3</td>
</tr>
</tbody>
</table>

Development of sustainable forms of construction

A.E. Long & S.E. Taylor
Queen's University Belfast, Northern Ireland

R.K. Venables
Crane Environmental Ltd. UK

J. Kirkpatrick
Owen Williams Consulting Ltd

ABSTRACT: Initially in this paper, a brief overview of a recently established COST Action “Sustainability of Constructions – an Integrated Approach.” is given. Secondly, details are given of recent research at Queen’s University Belfast on two innovative forms of construction, both of which have the potential to be highly sustainable. The first is on un-reinforced concrete bridge desk slabs and the second is the development of a flexible concrete arch system for short span bridges with no steel reinforcement.

1 INTRODUCTION

The built and natural environments are inseparably linked. Energy, materials, water and land are all consumed in the construction and operation of buildings and infrastructure to such an extent that sustainable development can be said to depend on the built environment. The world’s cities have a major impact on emissions of “green house gases” and global warming: they take up around 2% of the earth’s surface but account for nearly 80% of the carbon emissions from human activities. The urban environment influences our living conditions, social well-being and health. Thus the performance characteristics and quality of our infrastructure are of fundamental importance to urban sustainability and the well-being of our environment. The significance of this should not be underestimated especially if it is borne in mind that our infrastructure accounts for at least 50% of our national wealth.

Sustainability issues in construction are characterised by their complexity, the diversity of those involved and the need for innovative and special solutions. As the largest and most fragmented industry, the construction sector faces huge challenges in the pursuit of sustainability. Sustainable construction is a way for the industry to move forward, taking into account environmental, socio-economic and cultural issues.

Enhanced sustainability can be achieved through an integrated approach and by adopting innovations in technologies as well as by avoiding over-complex and high-energy-consuming solutions. A science-based approach is essential for transforming the potential of the so called “enabling technologies” into practice. Thus information and communication technology, biotechnology and advanced industrial materials represent some of the opportunities to move towards more sustainable construction processes and products.

Recognising the need for an integrated approach to advance sustainability within construction a COST Action C25 was initiated by the European Commission in 2006. This action is supported by over twenty European countries and its main objective is to “support the science-based development of sustainable construction in Europe through the exchange of scientific results concerning design tools, assessment methods, advanced materials and technologies as well as construction processes, both for new construction and the rehabilitation of existing”. More detailed information on this Action is provided in this paper complemented by details of bridge research at Queen’s University on two technological innovations which could lead to much more durable and more-sustainable forms of construction.

2 COST ACTION C25: SUSTAINABILITY OF CONSTRUCTIONS – AN INTEGRATED APPROACH

2.1 Background and concept

COST (European Co-operation in the field of Scientific and Technical Research) represents a bottom-up...
approach to research, and trans-national committees have been involved in various research themes (Actions) in difficult fields since the early 1970s. In 1991 COST Urban Civil Engineering Technical Committee was established and COST C25 is the 25th Action undertaken under the umbrella which has representatives from Architecture, Planning and Civil Engineering and can include participants from up to 34 member states largely from within Europe. This action follows on from a number of earlier actions which directly or indirectly had their focus on sustainability issues.

2.2 Issues being addressed in COST action C25

The development of the concept of “Sustainable construction” is aimed at ensuring more-economical use of finite raw materials and reducing, or mitigating against, the accumulation of pollutants and waste. The complete cycle of sustainable construction activities comprise the ways in which built structures and facilities are procured and erected, used and operated, maintained and repaired, modernised and rehabilitated, and finally dismantled for reuse or recycling. Compared with other industrial products, those in construction are long-lasting. This fact emphasizes the need to incorporate sustainability principles at an early stage in the design and development of construction projects. More specifically the following aspects need to be considered.

2.2.1 Environmental assessment methods and tools

Major civil engineering and development projects are subject to Environmental Impact Assessments as part of the development of their design and to secure planning approval. Yet the assessment of environmental impacts over the lifetime of built facilities (life-cycle environmental assessment) is fraught with uncertainty. Ideally, such life cycle assessments, in addition to estimates of life cycle costs, should be made available to clients before construction work begins, and many clients, architects and consulting engineers already take detailed account of environmental aspects in their designs, e.g. consideration of service life in combination with structural design. However, much better tools for life-cycle environmental and cost assessments need to be developed and integrated with each other. This would enable different strategies for infrastructure projects to be evaluated during design e.g. long life vs design so that repairs and rehabilitation can be carried out without prohibitively expensive disruption (eg. Tinsley Viaduct in the UK). What such improved tools need are, for example, much better information on the life cycle performance and environmental impacts of materials and components used in civil engineering and major construction projects, including their embodied energy.

Performance assessment tools and award schemes such as BREEAM and CEEQUAL need further adoption in industry as targets and rewards to project teams for improved performance – see www.breeam.co.uk and www.ceequal.com.

2.2.2 New materials, products and technologies

New materials, products and technologies offer longer-term opportunities for the reduction in environmental impacts. For example, the latest construction products such as lighting, heating and cooling systems can play a major role in improving the energy efficiency of buildings. It is also important that life cycle approaches to improving environmental performance are adopted not only for products but for construction works as well. Construction performances need to be viewed in terms of functional units, how they perform throughout the life-time of a built facility and what happens to them when deconstruction or demolition takes place. Focusing on an integrated and holistic approach, research is necessary as the associated problems are interrelated and cover a wide spectrum – typically a single building can consist of many basic materials and thousands of separate products. The challenge is how to measure and manage the impacts of construction projects. Generic performance based design and product development technologies offer tools for the management of research and development work.

2.2.3 Reuse and recycling of materials

Reuse and recycling of materials and components achieve a rate of over 80% in some Organisation for Economic Co-operation and Development (OECD) countries, but it should be noted that much of the material is used in low-value-added forms. Increasing use of recycled waste for structural applications is one way of positively addressing such sustainability impacts.

2.2.4 Environmental management

Environmental management of the construction process, for example for a new building, a bridge or for a renovation project, is now a very well-developed process, used by the majority of major UK construction firms. It necessitates an integrated and performance-based approach for project management so as to optimise the overall function of the completed project. Improved methods of integrating environmental and fiscal analyses would be helpful so as to take into account the different phases of the life-cycle.

2.2.5 Energy management

Energy-efficiency in buildings is one of the most environmentally benign ways to save energy. From the viewpoint of the EU, it has the highest priority on the three key issues identified as an area of necessary actions. The Directive on Energy performance of buildings aims at improvements in energy-efficiency-related
measures, and action plans in this area are expected to grow in the future. Improvement in energy efficiency brings direct benefits for urban sustainability.

2.3 Roles within COST action C25

Professor Luis Braganca (Portugal) is the Chairman of COST C25 and the two of the Authors (A. Long and R. Venables) have been accepted as the UK representatives. Three Working Groups are proposed:

- **WG1** – Global Methodologies, assessment methods, global models and databases
- **WG2** – Eco-efficient use of natural resources in construction (materials, products and processes)
- **WG3** – Life-cycle performance.

R. Venables is likely to contribute to WG1 on the basis of his long commitment to sustainability and his contribution to the establishment of CEEQUAL (Civil Engineering Environmental Quality Assessment and Awards Scheme). This scheme has now been successfully applied to projects in UK/Ireland, with the total value of projects that have been or are being assessed recently passing £2b (€2.9b). A Long anticipates that his research on structures and materials could be usefully applied to WG3 in relation to life cycle assessment and the utilisation of innovative techniques to enhance the sustainability of structures.

3 IMPROVING THE SUSTAINABILITY OF CONCRETE BRIDGE DECKS BY DESIGN

3.1 Background

Bridges with spans of up to 30 m constitute the vast majority of road infrastructure bridges in service across the world – whether it be for overpasses/underpasses for motorways or for minor river crossings. Within this category of bridges concrete deck slabs are widely used whether in combination with pre-cast pre-stressed concrete beams or steel girders. A similar type of deck can also be utilised for many medium span bridges hence the importance of designing a durable deck system cannot be overemphasised.

Over the past 20 years it has been found that many concrete bridges (concrete was selected in the 1960/70's for its inherent durability) have exhibited problems, such as spalling, associated with reinforcement corrosion. Such problems are particularly prevalent in marine environments or where freezing/thawing conditions require the intensive use of salt to prevent the formation of ice. In the latter case the vulnerability of the reinforcement in the deck slab is exceptionally high and in many instances deck slabs have to be repaired/replaced at great cost within 20–30 years. This causes great disruption to traffic and the associated costs of congestion are high.

A further problem for bridge deck slabs is that they have to be assessed structurally to ensure that they can carry the heavier lorries now on our roads. These deck slabs would in many cases be found to be unsatisfactory were it not for an inherent strength which is not taken into account in normal flexural design approaches. In particular it is accepted that the capacity of the slab elements of beam and slab decks is greatly enhanced due to the restraint provided by the beams and diaphragms. This enhancement has been recognised by a number of bridge authorities worldwide by incorporating it into their national design codes. Whilst BS5400 [1978 & 1990] does not recognise this the current UK assessment codes [BD44/95] for concrete structures do allow arching action to be included in the assessed capacity of deck slabs. The recognition of arching action is most important as it can mean the difference between a bridge deck passing or failing the assessment requirements.

In this paper the greatly enhanced strength associated with arching action, which is clearly of benefit for increased loadings, when taken into account in the design process can be shown to produce concrete bridge decks which are more durable than current designs. Any associated increases in costs will be more than compensated for by the anticipated increase in life expectancy before repairs are needed.

3.2 The concept of arching action in slabs

With the advent of Johansen’s [1962] yield line theory in the 1940s designers and researchers felt that at long last they had a prediction method for slabs which would provide realistic strength estimates. However, the tests carried out by Ockleston [1955] on interior panels of the Old Dental Hospital in Johannesburg revealed collapse loads of 3–4 times those predicted by the yield line method. This enhanced capacity was attributed to the development of an internal arching mechanism arising from the restraining effect of the surrounding panels.

Where a beam is restrained against longitudinal expansion, the concept of arching can best be understood by referring to Fig. 1. With the development of tension cracks at mid-span and at the supports the beam tries to expand longitudinally but as it is restrained, corresponding forces are induced which allow it to sustain a substantial load on the basis of the

![Figure 1. Arching action in a typical bridge deck slab.](image-url)
arching thrusts which develop as the deformation increases.

Similar actions take place in two-way systems where a dome or membrane rather than an arch is generated and this phenomenon is generally referred to as “Compressive Membrane Action” (CMA). The extent of the enhancement provided by compressive membrane action, over and above the flexural strength, depends on the degree of restraint provided by the surrounding structure. A typical load deflection curve with the notional contributions of CMA and flexural action separately identified is given in Fig. 2.

3.3 Relevance to bridge deck slabs

Tests on model bridge deck slabs in the Civil Engineering Department, Queen’s University, Kingston, Canada, in the late 1960s revealed considerable reserves of strength against punching failure [Tong & Batchelor, 1971]. The cause of this enhancement was correctly identified as CMA and its particular relevance to transient concentrated wheel loads was recognised. Here it is important to note that bridge decks represent one of the first areas to be considered appropriate for the utilisation of these design concepts. This is largely because the major localised loading is transient in nature and hence creep, which may reduce the enhancing effects of CMA, is of little importance.

On the basis of small scale model tests a conservative design method was produced. Thus in the Canadian design standards [2005] for beam and slab bridges, nominal transverse reinforcement, only 0.3% was required to resist concentrated wheel loadings as opposed to the 1.7% based on flexural design requirements. Similar design concepts are now accepted in various states in the U.S.A. and to date no adverse effects have been detected from these reductions in levels of reinforcement.

3.4 Research on arching action in the UK

3.4.1 Validation tests in Northern Ireland

In the knowledge of the research carried out in Canada on AASHTO girder based beam and slab bridge decks it was decided that parallel tests should be carried out on spaced M-beam (essentially a range of depths of prestressed I-beams with a narrow top flange and a broad bottom flange 1 m wide) decks to determine whether similar reductions in transverse reinforcement were possible. This would allow a slightly larger M-beam to be used at a spacing of 1.5 m or 2.0 m with consequent savings relative to smaller M-beams at 1.0 m spacing. In order to establish the strength of the slabs spanning between beams a one-third scale model bridge deck was constructed in the laboratory and tested at Queen’s University Belfast, in the late 1970s.

The design of the prototype slab for the two 112.5 kN wheel loads using the equations of Westergaard [1930] indicated that steel reinforcement of the order of 1.7% was required. For test purposes areas of reinforcement equivalent to approximately 1.7%, 1.2%, 0.5% and 0.25% were provided in the model along with three panels equivalent to 2 m spacing and two panels equivalent to 1.5 m beam spacing providing a total of 20 panels for testing.

3.4.2 Model test results and prediction method

The ultimate load capacity of the 20 test panels was determined as the load which caused the loading shoe (simulating the wheel load) to punch through the slab in the characteristic manner. It was found that there was very little variation in the ultimate load capacity of all the panels even though the transverse reinforcement varied from approximately 0.25% to 1.7%. In comparison with USA [1979] and United Kingdom [1978] design code predictions (Fig. 3) the results of the tests on the one-third scale model with the M-beams spaced at up to 2 m apart showed considerable enhancement over the design capacity of the standard slab. This enhancement can be attributed to the considerable in-plane restraint that is inherent in bridge deck slabs.

Figure 3 clearly shows that the codes do not give a satisfactory prediction of the punching shear capacity of typical bridge slabs and a more appropriate method which allows for in-plane restraint was therefore developed. For rigidly restrained bridge slabs, the effect of reinforcement upon the ultimate capacity is small as is evidenced by the results of the model test. Thus a method of prediction has been derived to allow for the compressive membrane forces generated within the slab. This method makes use of an effective steel reinforcement ratio, and full details of the method and its derivation are given in Kirkpatrick et al, 1984. As can be seen from Fig. 3 the proposed method of predicting the punching shear strength of
reinforced concrete bridge slabs gives good correlation with the results from the one-third scale model. This method has now been endorsed by the UK Highways Agency and is included in BD81/02 [2002].

3.5 British cement association (BCA) model tests
Jackson [1990] reported the results of a series of tests on the slabs of half scale bridge decks. These were carried out in the BCA laboratories and the results obtained for the spaced M-beam configuration confirmed the findings of Kirkpatrick et al [1984]. It was concluded that even though a sophisticated finite element model, which allowed CMA effects to be taken into account, had been utilised, the prediction method of Kirkpatrick was considered to give conservative estimates of the ultimate capacity. In this paper Jackson took account of global as well as local effects and even when the diaphragm was not included in one of the models the measured strength was still higher than that predicted by Kirkpatrick, 1984.

3.6 Serviceability of deck slabs
The ultimate load tests referred to above have indicated that strength is not critical in the design of deck slabs – however, designers also have to satisfy the serviceability limit state requirements. The widths of the cracks induced in the slabs were monitored during the model test in the Queen's University of Belfast and it was found that under service load conditions no cracks resulted. However, as scale effects can affect the accuracy of these measurements full-scale tests [Kirkpatrick et al, 1986] were subsequently carried out on a bridge built by the Northern Ireland Roads Service. This bridge incorporated beams at 1.5 m and 2 m spacing, and the reinforcement varied from 0.25% to 1.7% in the standard 160 mm thick deck slab.

The tests showed that current crack control formulae are not applicable because of the enhanced performance which results from the development of compressive membrane action. Initial cracking occurred at loads well in excess of the design service loads and even after cracks had been induced by severe overloading it was found that the slabs still satisfied the serviceability limit state requirements. The findings of this research have led to the adoption of a less conservative design approach for M-beam bridge deck slabs by the then Department of the Environment for Northern Ireland [DOE, 1986]. Provided certain restraint conditions are satisfied, the use of a nominal 0.5% reinforcement in the slab is now acceptable.

3.7 Review of the advantages arising from CMA
From a structural viewpoint the following benefits are evident:
1. Reduction in reinforcement (from 1.7% to 0.5% or less)
2. Same slab depth for greater spacing of beams
3. Lower overall cost of bridge superstructure as one larger beam at 2 m centres is less expensive than two smaller beams at 1 m centres.

Thus substantial reductions in costs can be achieved whilst at the same time retaining comparable strength and durability. However if the long-term durability of the bridge deck could be increased at a modest increase in cost then the whole life cost could be reduced as can be shown schematically in Fig. 4. Thus the challenge to designers is to achieve the type of relative performance achieved by the CMA deck (enhanced durability). Significant progress of this front has been achieved in Canada and the UK in recent years.
3.8 Improved sustainability by design

3.8.1 Canadian developments

The beneficial effects of CMA prompted the development in Canada of a novel reinforcing system [Mufti et al, 1987]. In this system, the in-plane restraint to the slab is provided by external steel reinforcement and the control of cracks due to temperature and shrinkage is provided by relatively inexpensive polypropylene fibres. Since these fibres are not affected by de-icing salts, a deck reinforced by them is not only inexpensive but is likely to be highly durable. The feasibility of a number of possible reinforcement arrangements has been assessed via tests on half-scale models of steel beam/concrete slab composite systems. Arising out of these model tests the Canadian authors have enough confidence in the proposed concept to suggest that conservatively designed fibre reinforced concrete deck slabs could now be used in actual bridges and a number of these bridges have been in service for nearly ten years.

3.8.2 Developments in Northern Ireland and the UK

The system developed in Canada is not applicable to decks with pre-cast prestressed concrete beams which tend to be significantly more popular across the world. Thus a number of alternative approaches have been the subject of ongoing research both in the laboratory at Queen’s University and on site in conjunction with the Department of Regional Development- DRD (NI) Roads Service [Taylor et al, 2001 & 2003; Taylor & Mullin, 2005]. This work focuses on the following subjects:

1. **Concrete** – As well as considering the addition of fibres, advantage is being taken of the fact that for a given degree of restraint the strength of slabs developing CMA is significantly enhanced by increases in concrete strength as illustrated in Fig.5.

2. **Reinforcement** – Apart from considering the lower percentages of top and bottom reinforcement (0.5% vs. 1.7%) site and laboratory tests have been carried out on:
   i) Conventional steel reinforcement located in a single layer at the centre of the slab (greatly increased cover).
   ii) Glass fire reinforced plastic reinforcing bars.

Both approaches have performed well, as anticipated, and it is clear that by using high strength concrete (with or without fibres) in conjunction with corrosion free reinforcement, bridge decks could be produced which should be virtually maintenance free. Because of the lower percentages of reinforcement these need not have a higher initial cost than conventionally decks hence it could be of enormous benefit to bridge owners.

3.8.3 Field tests on Corick bridge (NI)

Following laboratory work, tests were carried out on a reinforced concrete beam and slab bridge in Northern Ireland which incorporated novel reinforcement type and position [Taylor et al, 2007]. The tests were carried out in collaboration with the Department of Regional Development (DRD), Roads Service, and Northern Ireland. Some comparisons have been made with arching theory described in BD81/02 [Highways Agency, 2002] which came about as a result of previous research at Queen’s University.

The Corick bridge tests corroborated the findings of laboratory models and showed that the deck slab behaviour was virtually independent of the percentage of reinforcement. This challenges current flexural analysis which gives a direct correlation between the deck slab strength and the amount of steel reinforcement. Additionally, the slabs strengths were far in excess of those predicted by the standard flexural theory. This will allow for CMA concepts to be incorporated into relevant national design codes as is the case in Canada and in the UK. The enhanced strength and serviceability of laterally restrained slabs can also be taken into account in the assessment of beam and slab bridge decks such by using BD81/02 or the more recent Guide to the Use of Compressive Membrane Action published by the UK Concrete Bridge Development Group [Taylor et al, 2003].

The deck slabs with reinforcement at mid-depth had strengths far in excess of the design ultimate loads and behaved in a similar manner to those with top and bottom reinforcement up to an applied load equivalent to twice the maximum service wheel load for 45units of HB loading. This finding represents a substantial economy in the amount of reinforcement and, due the increase in the concrete cover, could increase the overall durability of the bridge deck without compromising its serviceability.
3.9 Conclusions from CMA research

A sufficient understanding of the structural benefits of compressive membrane action for bridge deck slabs has now been achieved. This will allow:

i. the enhanced strength and serviceability of laterally restrained slabs to be taken into account in the assessment of beam and slab bridge decks.

ii. CMA concepts to be incorporated into relevant national design codes. Already this is accepted in Ontario, Canada and in the UK.

iii. the benefits of the increased design capacity associated with CMA being taken account of in the slab elements of box girder bridge and other forms of cellular structures,

iv. the development of highly durable deck slabs, which will be virtually maintenance free.

The net effect of all the above is more cost effective bridge decks which exhibit greatly enhance sustainability relative to existing designs.

4 A NOVEL FLEXIBLE CONCRETE ARCH SYSTEM FOR SUSTAINABLE BRIDGES

4.1 Background to the novel arch system

Masonry arch bridges are one of the oldest forms of bridge construction and have been around for thousands of years. Brick and stone arch bridges have proven to be highly durable as most of them have remained serviceable after hundreds of years. In contrast, many bridges built of modern materials have required extensive repair and strengthening after being in service for a relatively short part of their design life. This section describes the development of a novel flexible concrete arch system that has the potential to be highly sustainable due to the low or zero amount of steel reinforcement.

It is no longer economically viable to construct a masonry arch in the traditional method due to the cost of skilled labour required to build the accurate centring and to cut the masonry blocks. Progress on this type of work is usually slow and can be weather dependent. In order to provide a viable alternative Queens University of Belfast, in collaboration with Macrete Ireland Ltd, are developing a flexible concrete arch system made of un-reinforced pre-cast concrete voussoirs. The arch is constructed and transported in the form of a flat pack using a polymer grid reinforcement to carry the self weight during lifting but behaves as a masonry arch once in place.

Basically there are two options for the construction of the arch unit. The voussoirs can be pre-cast individually, laid contiguously horizontally with a layer of polymer grid material placed on top. An in-situ layer of concrete, approximately 40 mm thick, is placed on top and allowed to harden to interconnect the voussoirs. The same unit can be made in a single casting operation by using a shutter with wedge formers spaced to simulate the tapered voussoirs. Both forms of construction are shown in Fig 6. The arch unit can be cast in convenient widths to suit the design requirement, site restrictions and available lifting capacity. When lifted, the wedge shaped gaps close, concrete hinges form in the top layer of concrete and the unit is supported by tension in the polymer grid. The arch shaped units are then placed on a precast footings and all self-weight is then transferred from tension in the polymer to compression in the arch.

This is in line with advice given by the UK Highways Agency [1995b] where the arch form, plain structural elements and the elimination of corroding reinforcement are recommended.

4.2 Materials

Control samples of the polymeric reinforcement were tested to ascertain the material properties. The polymer grid is available in a range of mesh sizes and strengths. The manufacturer’s data sheet also gave the longitudinal and transverse tensile strength in kN/m. Table 1 summarises the results of material tests carried out at Queen’s University Belfast. The differences in the measured and specified values for the tensile strength may be accounted for by the difference in the test method. The initial 0.35 m wide arch used a 100/15 Paragrid® but a stronger grid, 150/15,
was used in the 1 m prototype arch for the lifting and placing of the arch. The concrete used for casting the voussoirs was the standard pre-cast mix used by Macrete Ltd with a 7-days compressive strength of \( \frac{40}{100} \) N/mm\(^2\) and a 28-day compressive strength of \( \frac{55}{100} \) N/mm\(^2\).

4.3 Manufacture of the arch unit

A prototype arch unit of 5 m span, 2 m rise and 2.5 m internal radius was constructed and lifted. This arch required twenty-three voussoirs which were 1 m wide and 200 mm deep with a 40 mm slab interconnecting in-situ screed. The arch was lifted at approximately the quarter span points with additional nominal support at the mid span region. During lifting the arch drags at the each end and the mid span point tends to sag; hence the need for the additional support. When the end cantilevers are fully effective they produce a hogging moment in the mid span region which assists in the formation of the arch. During this operation a critical case occurs when the arch is fully formed and suspended at the lifting points. A maximum bending moment occurs at the lifting points for the cantilever ends and to simulate this condition a series of short beam elements were tested to establish the capacity and to investigate the rate of creep in the low modulus polymer reinforcement. A summary of the results of these tests is given in Table 2 and it was found to give an adequate factor of safety during the lifting procedure. The lifting sequence is shown in Fig. 7.

The arch unit complete with tapered seating is shown in Fig 7c. Subsequently, an anchor block detail was designed for the seating of the arch ring. The anchor block caters for the slope of the last voussoir enabling the arch to from correctly. It also provided some lateral restraint to the arch ring both during construction, prior to completion of the arch system with spandrel walls and backfill, and in the long-term under live loading.

4.4 Stability test

4.4.1 Test set-up

To assess the flexibility of this system, a stability test under backfilling operations was conducted and the arch unit was monitored for horizontal deflections, vertical deflections and strain at the voussoirs joints.
It was originally intended to backfill the arch with Type 6S granular backfill under the Specification for Highway Works, Volume 1 [1998]. However, after a preliminary cost estimate for this span, it was decided to trial the use of lean mix concrete as a backfill option. Shuttering for the backfill was set up independently to the arch ring to allow free movement during the placement of the concrete.

4.4.2 Test procedure and results
The backfilling operation was carried out by placing approximately 250 mm deep layers of concrete to each side of the arch up to 1.5 m horizontal distance from the back face of the anchor block. The distribution of load to either side of the arch ring was aimed at minimising the effects of asymmetric load. The depth of the concrete was measured and the transducer and strain gauge readings were recorded at each increment of backfill load. The readings showed a reasonably symmetric response to the backfilling operation.

The maximum movement in the crown was 0.8 mm upwards in the back face and occurred at the full height of the concrete backfill. This was due to the lateral pressure of the wet concrete creating an inwards movement at the sides of the arch which in turn caused the crown to rise. As the concrete hardened, the lateral pressure reduced and the deflection response at the crown reversed in direction. That is the upward movement ceased and the crown deflected very slightly downwards. The results of the stability tests showed very little movement in the arch ring and it was concluded that the arch was stable under the backfill operations.

4.5 Live load testing of the arch system
The 1 m prototype flexible concrete arch system as shown in Fig. 8 was tested in accordance with the requirements of Macrete Ltd. and following the guidelines in BS8110: Part 2 [1985]. A simulated static wheel load was applied at the midspan and the third span of the arch ring. The single wheel load, for the intended category of bridge, is 5.75 t. However, for both loading locations, the arch system carried over 35 t without showing signs of distress (that is, six times the single wheel load).

4.6 Analysis of the arch unit
An analysis of the arch unit was conducted using ARCHIE a numerical analysis package which allowed for interaction with the arch backfill as this software is also used by the DRD Road Service in Northern Ireland for load assessment analysis of their arch bridges [Highways Agency, 1997 and 2001] validation using ARCHIE was important. The arch unit was analysed under different wheel loading conditions and deflection profiles were similar to those for the load test. The predicted ultimate capacity was highly conservative when compared with the actual load which was safely carried by the prototype arch system.

4.7 Conclusions for the novel arch system
Experience has shown that arch bridges are highly durable structures requiring little maintenance in comparison with other bridge forms. Thus, the objective of the new Highway Agency Standard [2004] is welcomed especially if it encourages a renaissance in arch building using unreinforced masonry materials.

The novel arch system has been demonstrated in tests reported in this and other papers, to be a viable alternative to long established methods of construction and the following advantages have been identified:

1. As the Arch system is cast horizontally it can conveniently be transported to site in a “flat pack” form.
2. As centring is not required during installation this greatly simplifies the process and enhances the speed of construction.
3. As there is no corroding reinforcement the long term durability should be assured.
4. Initial estimates would indicate that the system is cost competitive with alternatives such as RC box culverts which do not share the aesthetic benefits or the longevity of an arch.
5. Ease of access to restricted sites. Here the pre-cast voussoirs can be brought to the site separately where they can be transformed into a flexible arch and then lifted into position. in the usual way.

A complete 5 m wide arch bridge with a 5 m span and 2 m rise complete with spandrels, fill and surfacing is currently being constructed. This bridge will be load tested and all necessary data recorded to compare with theoretical predictions. A replacement
bridge is also planned in conjunction with DRD (NI) Roads Service.

Current data indicates that in the UK alone almost 3500 bridges with spans between 3 m and 9 m need to be strengthened or replaced at an estimated cost of £80 million. Thus there is obviously a potential market for this short span bridge system that guarantees high quality pre-cast concrete units, ease of transport, simplicity of erection and exceptional durability. From all view points the system represents a very sustainable alternative for the future.

5 OVERALL CONCLUSIONS

The establishment of COST Action C25, which only commenced its work in late 2006, recognises the need for an integrated approach to advance sustainability within construction. The input from twenty countries will allow best practices from across Europe to be shared and the development of assessment methods will assist in the identification of sustainability issues. Once designers and contractors have a better indication of what they are looking out for they will be better placed to take sustainability into account right form the inception of a project. In this way and by developing innovative technologies to overcome difficulties progress can be accelerated in this important field.

The two examples of innovative approaches to bridges, ie. unreinforced deck slabs and the flexible concrete arch, share the following positive sustainability features relative to conventional designs:

- Little or no maintenance throughout their life
- Greatly extended lives

Since these improvements can be achieved at little or no additional cost the problems associated with traffic disruption/congestion when a bridge is being repaired/replaced can be minimised. This recognition of the significance of full life cycle costing, as opposed to the minimum initial cost approach, is very important from the sustainability viewpoint.

REFERENCES

ARCHIE-M: Masonry Arch Bridges and Viaduct Assessment Software, Version 2.0.8, OBVIS Ltd. UK.


Canadian Standards Association, Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-00 (new version available for comment 2005 CSA-Technical Committee), Canada.


Tong P. Y. and Batchelor B. de V. Compressive membrane enhancement in two way bridge slabs, Cracking, Deflection and the Ultimate Load of Concrete Slab Systems, S.P -30, ACI, Detroit, 1971, pp. 271–286.

Sustain the chapel building – a demonstration of sustainable materials renovation

Elizabeth A. Chien & Thomas S. Tolman  
*US Army Corps of Engineers, Seattle District, Seattle, WA, USA*

Richard G. Lampo, Thomas R. Napier & Nathan D. Mowry  
*US Army Engineer Research & Development Center, Construction Engineering Research Laboratory, Champaign, IL, USA*

Ken J. Smith  
*Environmental and Natural Resources Division, Public Works, Fort Lewis, WA, USA*

**ABSTRACT:** Faced with thousands of square feet of building demolition per year, waste managers at Fort Lewis, WA and the Seattle District Army Corps of Engineers (USACE) seek effective ways to keep demolition debris out of the landfill. Ft. Lewis collects concrete and grinds it for use on training trails. They gather wood pallets and other clean wood waste for use in creating compost and they treat petroleum-contaminated soils. When a World War II-era chapel appeared on the demolition list, Ft. Lewis and the USACE started thinking of waste management in a more holistic view. Why not divert the whole building to a new location and remodel it for a new use rather than disposing it in a landfill? This holistic approach spurred the development and design of a new Environmental Education and Conference Center (E2C2) for Fort Lewis. The E2C2 (the old chapel) will be relocated on a former landfill site, now being reclaimed and transformed into the Sequalitchew Eco-Park (Eco-Park). This paper evaluates the benefits of reusing the existing chapel for the E2C2, rather than building a new facility and disposing the old chapel into a local landfill.

1 PROJECT BACKGROUND

1.1 *Fort Lewis demolition*

Faced with tens of thousands of square feet of building demolition per year for the next five years, waste managers at Fort Lewis, WA and the Army Corps of Engineers (USACE) seek ways to keep enormous amounts of demolition material out of the landfill. They collect concrete, asphalt, and masonry materials and grind it for use on training trails; gather wood pallets, other clean wood and brush for use in creating compost; and treat petroleum-contaminated soils. Construction contractors working at Ft. Lewis, are required to divert (salvage, reuse, recycle) 75% of construction & demolition (C&D) waste, by weight.

When a World War II-era chapel appeared on the demolition list, it sparked an idea of how construction and demolition (C&D) waste management could reach a whole new level. Why not move the whole chapel building straight to the landfill – intact? Fort Lewis needed an environmental education center and more off-post meeting space. A former landfill was being converted into a public outreach site. The old chapel building could be completely salvaged and given a new life on the landfill as an education and community facility rather than “in” the landfill doing little good.

1.2 *The Sequalitchew Eco-Park*

Fort Lewis recently closed up Landfill No. 5. This old landfill was a tremendous liability for the military installation. In 2002, Fort Lewis Public Works began work on a sustainability plan. Military leaders, community partners and dedicated staff members met to set goals for a sustainable future. The landfill slowly emerged as a venue for demonstrating these goals. Sustainability teams converged on the landfill, using an integrated visioning process, and planned projects that will transform Landfill No.5 from a liability into the Sequalitchew Eco-Park (Eco-Park), a community resource.

The old chapel building became a piece of this process. It will be moved to the Eco-Park and renovated into a sustainable structure.

This paper outlines the design process followed for creating a sustainable new education center from...
a dilapidated 60-year old wood structure. Each building system was evaluated for its environmental impact, as well as its ability to give the chapel a new life. Lessons learned in the design stage of the chapel reuse project will be documented, emphasizing systems and methods found to be life-cycle cost effective, benefits of avoiding the landfill, and methods employed to measure the sustainability of the finished project.

2 REUSE VS. BUILDING NEW

2.1 Just how much is in there?

Although it makes sense that salvaging existing resources is more sustainable than using virgin resources, quantifying that “gut feeling” is more challenging. To make a preliminary estimate of the impact of reuse, the amount of material in the existing chapel building was estimated. The building contains roughly 9,300 board feet of lumber, 2,600 square feet of reusable pine flooring, 4,500 square feet of fiberboard finish panels, 560 square feet of windows and doors and 8,000 square feet of finish coatings (paints and stains).

2.2 What's the difference

To build a replacement facility of the same size and function today would take approximately 23 large trees (assumes 400 board feet per tree). The Building for Environmental and Economic Sustainability (BEES) analysis software list the primary environmental impacts of wood as “raw materials acquisition”, “manufacturing” and “transportation”. Most of these impacts are eliminated or greatly reduced by relocating and reusing the existing chapel rather than building new. See BEES summary graphs (Figs 1a & 1b) [NIST 2002].

The National Center for Manufacturing Sciences (NCMS) works to holistically quantify the impact of manufacturing activities across all sectors. Their materials flow chart (see Fig. 2) shows how much of the materials harvested or extracted from our environment are used in the construction industry (designated by a number 23 in the table). Clearly construction activities have profound implications on our environment and future sustainability due to the sheer volume of activities involved. The NCMS graph is also instructive to demonstrate the value of reuse. When a material is reused, the first four steps (extraction, primary processing, refining and fabrication) of the manufacturing process are avoided. These are the steps that use the most energy and have the largest impact on the environment. Reusing whole buildings such as the chapel, also greatly reduces the impact of the assembly stage, providing further benefit [Chalmer 2005].

To better see how reusing the chapel building was a sustainable solution, materials in the building were compared for life cycle impact to similar new or replacement materials using the life cycle analysis software, Athena. For example the 9,300 board feet of lumber, 2,600 square feet of pine flooring and 11,000 SF of diagonal sheathing that will be reused in the building will save roughly 46,275 KW of energy and reduce CO2 equivalent emissions of the building by 7,510 pounds [Athena 2005].

2.3 Looking to the future

The E2C2 project will remake the chapel into a building that will last another 50 years in a much more sustainable manner than building new.

3 AVOIDING THE LANDFILL

3.1 Where does it all go?

Reusing existing materials is one way in which building salvage is sustainable. Another major factor is the reduction in landfill requirements. Fort Lewis no
Major material flows in products, by NAICS Category (to three digit level)

Figure 2. Materials Flow Chart [Chalmer 2005].
longer has a landfill on post for construction debris. Diverting this material from the landfill will help Fort Lewis avoid the enormous liability of caring for more closed landfills in the future. It will also reduce the amount of property used to hold something as waste that still has a usable life.

4 SOCIAL BENEFITS OF REUSE

4.1 Preserving memory
Terry Austin, at Fort Lewis recalls her wedding day standing in front of an old wooden chapel building. Ken Smith relates a story of his father as a young man entering the service at Fort Lewis during World War II, he wonders if his father spent time in the alpha block chapel pondering life’s toughest questions. Walking into the old chapel or any old building connects us to our past. Reusing the chapel gives renewed life to those memories, while allowing new memories to intertwine with the old.

4.2 It’s not historic preservation
The US Army reserves excellent examples of buildings from all eras to preserve as cultural resources and historic properties. The alpha block chapel is not one of the buildings chosen for preservation in this manner, but there is still social and cultural value in respecting its history, as it is reborn into a new facility.

5 MEASURING SUSTAINABILITY

5.1 Rating system
To quantify the sustainability of the building as a whole, the team turned to the United Stated Green Building Council’s (USGBC) LEED Green Building Rating System [USGBC 2005]. An examination of how LEED can be applied to reuse projects follows, outlined by the five major LEED categories: Sustainable Sites, Water Efficiency, Energy and Atmosphere, Materials and Resources and Indoor Environmental Quality.

5.2 Sustainable sites
The E2C2 will be located on a former landfill site and used to educate Fort Lewis and the community about environmental issues. The former landfill cells will be used as troop training areas and nature trails for the community. Grading is such that part of the site will be turned into a stormwater treatment system for a significant portion of North Fort Lewis.

Orientation of a building is a major factor in long-term energy performance. Because the chapel building is being relocated, the design team oriented the building...
on an east-west axis. This provides the maximum amount of south-facing glass which can easily be shaded. The original window pattern of the building was retained to preserve as much of the existing structure as possible. However, the original tall windows were split into an upper portion for daylighting and lower portion for views. Roof overhangs were added to shade daylighting windows. Sunshading devices were added to the vision windows. This effectively restricts direct sun penetration into the building, allowing ample daylight without the overheating problems of the original window design. See Figures 3–6 for shading patterns on the new building façade.

5.2.1 Roof
Metal Roofing was selected for the new roofing because it provides a highly reflective surface to help mitigate the urban heat island and enhance the environment at the Eco-Park. Metal roofing also easily integrates with the self-adhering photovoltaic membranes that will be installed between the standing metal seams.

5.2.2 Site shading
Parking will be constructed in small clusters or “pods” on top of an existing landfill cell. These pods allow existing trees to be preserved and new native species of trees added, providing shading for the parking area.

5.3 Water Efficiency

5.3.1 Landscaping
The community is assisting with landscaping the Eco-Park site. Only native landscaping will be used. No irrigation system will be installed.

5.3.2 Potable water savings
The design team originally planned to use composting toilets in the E2C2, which would have saved 80% more water than a conventional facility of the same type. Composting toilets also eliminate the need for using potable water to convey sewage.

Despite the many advantages of composting toilets, the team decided against using them in the facility. 2 primary reasons for this decision were: 1- The maintenance requirements associated with composting toilets and 2- The cost and complexity of composting toilet systems.

The primary maintenance challenge with composting toilets in a high-use facility is the addition of wood chips or other organic materials to the composter. Due to the transient nature of the E2C2 building occupants and no dedicated maintenance staff, the requirement to add wood chips as often as every day was simply not feasible for the project.

Composting toilets also require a basement area below the toilets for the compost chamber, with an access door to grade. The costs for these additional requirements made composting toilets unaffordable for this facility.

Additionally, Fort Lewis has a solid waste recovery program, taking sewage waste and turning it into compost, so the effluent created in the facility will be dealt with in a responsible manner. Water savings features included in the project are waterless urinals, low-flow faucets and tankless water heaters. Although not included in the project due to cost limitations, a rainwater harvesting system for toilet flushing would add to the sustainability of the project, by eliminating potable water use for sewage conveyance.

5.4 Energy efficiency
The building envelope will be upgraded to include super insulated walls (R25), floor (R30) and roof (R38). Although insulation is obviously a great start for energy efficiency, adding it to the E2C2 will require demolition of much of the interior wall finishes. However the energy savings achieved by adding the insulation offsets the cost and environmental impact of additional finish materials.

Based on an energy model prepared by the design team’s mechanical engineer, the insulated structure will use 75% less energy for heating than the existing building (40% overall energy savings) based on an average weather year.

The building will be entirely heated with electricity via radiant panels integrated into the window light shelves. Methane gases released due to the existing landfill decomposition will be harvested through a future project and used to create electricity for the entire Eco-Park, including the E2C2.

The E2C2 will also feature photovoltaic panels, integrated with a cool metal roof. These panels will supply 5% of the building’s total electricity needs. Approximately 1,000 SF of PV panels are required to provide this amount of energy.

5.5 Materials and resources
Early in the project, guidelines were set for how each material will be handled during construction. Each material in the existing building will be tracked during the project and will be handled according to the prioritized list below:

1) Reuse in the new facility,
2) Salvage to local 2nd-hand construction materials outlet or to Ft. Lewis Army units,
3) Recycling (downcycling).

5.5.1 C&D waste recycling
95% of construction and demolition waste will be recycled in the project. Many materials will be recycled
5.5.2 Local materials
The E2C2 will utilize locally produced materials wherever possible. The reuse of the building is the largest "local" material use. The metal roofing will also be manufactured locally. Composite siding is manufactured locally. Detailed records will be kept by the contractor of the manufacturing and raw materials extraction location for each material used in the project.

5.5.3 Material sustainability
Building materials were evaluated for their sustainable features. A summary of the major building materials is included below:

Siding: Materials considered: Metal siding, vinyl siding, composite panels, and fiber cement siding. The E2C2 will mostly be covered in fiber cement siding. This material provides good durability with low maintenance. Fiber cement siding can be recycled if it is ever removed from the building and can likely be reused as siding on another project. A downside to fiber cement siding is the relatively high levels of CO2 associated with cement production. However the durability of the fiber cement siding helps to offset this due to reduced maintenance requirements. A second siding product is planned for use on the project that is made from Forest Stewardship Council (FSC) certified paper product combined with a resin made from cashew nut pods. The material is a composite that is manufactured locally (within 100 miles of Fort Lewis).

Roofing: Existing asphalt shingle roofing on the chapel has deteriorated and must be replaced. The team seized this opportunity to add insulation to the roof deck and greatly improve the overall energy performance of the building. Several alternative new roofing materials were evaluated. Among these were asphalt shingles, cedar shakes, recycled rubber tiles, lightweight green roof and cool colored metal roofing. Metal roofing was chosen as the material for the project because of its long-term durability. Metal roofing also can be used easily with integrated photovoltaics. Green roofs were planned early in the project for the entire structure but could not be used on the main roof because the existing structure was not able to support the added weight. Green roofs will be used on the new entry roofs and will demonstrate the storm water benefits of green roofing by contrasting the amount of roof runoff for the green roof with that of the metal roof.

Windows: Existing single pane double hung windows will be removed and replaced with energy-efficient insulated window units. Windows also must meet US Department of Defense Antiterrorism/Force Protection Standards, which require reinforced window frame, laminated glazing and structural tube steel around each window opening. The added structural steel is a significant cost to the project and also a source of heat loss in the building envelope, since it fills the void between the exterior sheathing and interior finish material.

Doors: All new interior doors will be manufactured with wheatboard cores that contain no added urea formaldehyde. These doors will utilize renewable resources in the form of wheat straw.

Interior Wall Surface: Salvaged wood siding from recent Fort Lewis demolition projects will become the new interior wall finish in the building. Existing lead-based paint will be removed from this siding through another project that will recover the lead from the paint.

Flooring: The chapel contains existing fir flooring that is covered with underlayment and commercial carpet. The carpet and underlayment will be removed and the existing floor restored. It will be refinished with a polyureseal coating that is made for heavy traffic floors.

5.6 Indoor environmental quality

All materials used in the E2C2 will be low-emitting. A discussion of some materials selected is included below:

Flooring: The existing wood floor will be restored with a waterborne urethane polymer emulsion that is low in volatile organic compounds (VOCs) at 114 g/L. The sealer contains no formaldehyde or hazardous chemicals, yet has been successfully used in high traffic environments.

Another flooring system considered was a terrazzo system over the existing wood floor. This was not used due to the added materials and labor required to install the system. Even with the need for future resurfacing of the wood floor surface, it was a better alternative than adding a whole new flooring system.

The building will feature operable windows and controls appropriate for a large meeting space. Ventilation will be provided in two modes. During winter or heating mode, air will be warmed with a heat exchanger before entering the space. Summer mode ventilation will take advantage of the operable windows and exhaust vents high in the roof to circulate cool western Washington air via the stack effect.
The indoor environment of the facility will be updated to provide excellent occupant comfort with low-maintenance sustainable materials.

6 CONCLUSIONS

Reuse is the ultimate in construction sustainability. The Fort Lewis Environmental Education and Conference Center achieves sustainability through reusing and old chapel building and saving it from demolition. The building saves significant amounts of energy over new construction and contributes fewer pollutants and green house gases to the atmosphere. It preserves social memory and becomes a significant resource for the community. The project also demonstrates how an existing building can benefit from the use of sustainable design principles and can make use of green building rating tools, such as LEED to achieve a greater measure of sustainability.

REFERENCES

Environmental science in building construction: new course development for undergraduate students

C. Diggelman
Architectural Engineering & Building Construction Department, Milwaukee School of Engineering, Milwaukee, WI, USA

ABSTRACT: In response to a growing awareness of the environmental impacts of construction, a new undergraduate course – Environmental Science in Building Construction – was developed and incorporated into architectural engineering and construction management programs at Milwaukee School of Engineering (MSOE). This senior-level required course is offered concurrently with senior design. Course development included surveying the AGC of Greater Milwaukee and completing a NSF sponsored project to assess environmental impacts of a campus construction project. Course elements include introducing students to drivers of the sustainable construction movement (climate change, ozone depletion, environmental degradation and resource depletion); introducing students to the LEED™ suite of standards and scientific evidence that LEED™ strategies reduce environmental impacts; and requiring students to complete a research project on technologies that have the potential to reduce environmental impacts. If environmental impacts of construction are to be reduced, future engineers must understand a wide range of sustainability-related issues.

1 INTRODUCTION

1.1 MSOE’s Architectural Engineering and Building Construction (AE&BC) Department

The AE&BC Department offers BS degrees in architectural engineering and construction management and MS degrees in structural engineering and environmental engineering http://www.msoe.edu/ae/.

In 2002, program changes were made in both undergraduate programs. With the growth of importance and influence of the US Green Building Council’s LEED™ building certification system and student interest in LEED™ accreditation, the department decided to design a new course, Environmental Science in Building Construction (AE4121), to give students a background in sustainable construction. This course was to be taught for the first time in the 2005/2006 winter quarter.

1.2 National Science Foundation project

The National Science Foundation (NSF) issued a solicitation for proposals (NSF 03-510), Technology for a Sustainable Environment- Sustainable Construction Processes, in 2002. The US Green Building Council (USGBC) indicates that there are significant environmental impacts due to buildings which contribute 36% of total energy use, 65% of electricity consumption, 30% of greenhouse gas emissions, 30% of raw materials use, 30% of waste output (136 million tons annually) and 12% of potable water consumption http://www.usgbc.org/DisplayPage.aspx?CMSPageID=291&. The NSF was looking for an assessment of the impact of construction, “the resources used and wastes generated at the construction site over the life of the project should be defined and eventually integrated with life-cycle metrics of the facilities themselves” and the need for “creating new or modifying current construction processes to reduce or eliminate environmental impacts while also considering construction costs and construction competitiveness” http://www.nsf.gov/pubs/2003/nsf03510/nsf03510.pdf.

Milwaukee School of Engineering (MSOE) was in the design phase for a new building, the Kern Center. The Kern Center is a five-story, 210,000 ft² state-of-the-art athletic, health and wellness facility used for organized athletics and intramural sports.

As owner, MSOE was in a unique position of requesting construction documents that would enable us to quantify environmental impacts of the constructing the Kern Center, so we submitted a proposal. Both Wisconsin’s Focus on Energy Program and the National Science Foundation provided funding support for this project. Additional NSF support was received under the Research Experience for Undergraduates (REU) program. It was anticipated
that what we learned in the NSF project would contribute to AE 4121 development.

2 COURSE DEVELOPMENT

2.1 Input from local industry and MSOE faculty and alumni

Members of the AGC of Greater Milwaukee, faculty and alumni were invited help prioritize the list of course topics given below:

- Environmental laws and regulations
- Environmental communications
- Interpret MSDS
- Hazard communications
- Elements of environmental management system
- Life-cycle environmental impacts of energy and materials
- Minimization, management and cost of materials and waste
  - Hazardous
  - Solid
    - Construction
    - Demolition (i.e. asbestos)
- Management of soil
  - Contaminated
  - Erosion and sedimentation control
  - Dust control
- Management of spills
- Management of water
  - Surface
  - Ground
  - Recharge
  - Wastewater
  - Storm water
- Management of air quality
  - Site emissions
  - Indoor construction air quality
- Ozone-depleting gases
- Other considerations
  - Noise management
  - Site light issues at night
  - Knowledge of LEED™ prerequisites
  - Knowledge of elements of green design
  - ISO14001

Industry responses indicated that waste management (hazardous, solid) and construction indoor air quality were the priority topics. Responses from recent graduates indicated that sustainable construction and LEED™ certification and accreditation were the topics of highest priority.

2.2 Kern Center research project contributions

The overall goal was to perform a baseline study of materials and energy used and wastes generated during Kern Center construction. Flows were to be characterized as:

- Inputs of primary building materials, fuel, energy and water
- Stocks of building materials and soil remaining on site and
- Outputs of air emissions, waterborne wastes, solid wastes and soil.

Tasks included determining what construction documents were available to us, developing a data collection methodology, determining where there were gaps in information, comparing quantities from invoices (actual amounts) with quantities calculated from site drawings (estimated amounts) and comparing pollution generated at the site with pollution embodied in materials and fuels generated off-site.

Undergraduate students were involved in the beginning in taking meter readings and collecting daily reports and invoices from the construction manager, Hunzinger Construction Company. They took site photos several times per week and cataloged these photos. We had a copy of the owner’s plans, but did not have access to subsequent modifications to these plans. Our activities could not add costs to the project. Undergraduate students organized construction documents and developed research projects from this data.

Undergraduate projects included an analysis of fuel (diesel from construction equipment and gasoline from worker transport) requirements over the course of the project and the potential for carpooling of workers to reduce fuel emissions; the accuracy of daily reports compared site photos for tracking on-site construction equipment; compiling life-cycle-inventory data on embodied energy and pollution in construction materials used in Construction Specifications Institute’s (CSI) CSI Division 31 (site work) and CSI Division 3 (concrete work); and an analysis of the potential for landfill diversion of construction materials and cost savings had there been construction recycling during Kern Center construction (all material was landfilled).

Two Master of Science in Environmental graduate student projects were also developed using these construction documents. One capstone research project, “Quantifying VOC’S in Products Off-Gassed during the Construction of MSOE’S Kern Center and Proposed Strategies for Reducing Employee Exposure,” evaluated products that off-gassed volatile organic compounds (VOCs) actually used in the Kern Center from the time of building completion to the end of the project. The other project was “Assessing environmental impacts of site activities during construction of MSOE’s Kern Center: site and concrete construction.” The goal of this project was to perform a baseline study of flows of materials and emissions during CSI Divisions 31 and 3.
2.3 Draft course goals

The 3-credit (quarter hours) course needed to satisfy 1.5 credits of science content for ABET (Accreditation Board for Engineering and Technology). It was to meet three hours per week and be taught in the winter quarter of the senior year, concurrently with senior design. Topics included those indicated from the feedback provide from industry and alumni.

Draft course goals:

- Students will gain a foundation in the basics of environmental science, including air, water and soil pollution and hazardous and solid waste management,
- Students will gain an understanding of environmental impacts generated both on-site (air emissions from construction equipment, wind-blown dust and water pollution) and off-site (embodied energy and pollution from specific materials and emissions from transport of workers and materials to the site),
- Students will gain an understanding of current strategies for minimizing environmental impacts
- Students will complete a research project that includes contacting a contractor, identifying an environmental issue of mutual concern, developing a baseline current practice, researching alternatives and recommending one cost-effective alternative with reduced environmental impacts.

3 COURSE ELEMENTS

3.1 Course description

This course introduces students to environmental aspects and impacts of construction materials, fuels and activities; environmental laws and regulations and elements of sustainable construction. Sustainable construction includes strategies for managing and reducing environmental aspects and impacts to land, air, water and living systems.

3.2 Course objectives

Upon successful completion of this course, the student will:

- Be introduced to major drivers, including climate change, ozone depletion, degradation of soil, air, water and biological diversity, and resource depletion, of the sustainable construction movement.
- Be introduced to scientific concepts of sustainable construction.
- Be introduced to the LEED™ suite of standards, including categories, credits and certification process and scientific evidence that LEED™ strategies reduce environmental aspects and impacts.
  - Sustainable sites
  - Energy and atmosphere
  - Building hydrologic system
  - Green building materials
  - Indoor environmental quality (IEQ)
  - Construction operations, including recycling of materials, erosion and sediment control measures, and protection of indoor air quality
  - Building commissioning
- Be introduced to tools for quantifying economic benefits of high performance/green buildings
- Be able to complete an independent research project (a search of the engineering literature, a concise written report and a presentation on that report) on a technology with potential to reduce environmental impacts (and increase LEED™ points) on the student’s capstone project.

3.3 Course topics

- Environmental drivers
  - Population
  - Energy
  - Water – quantity and quality
  - Minerals
  - Air pollution
  - Water and soil pollution
  - Solid and hazardous wastes
  - Biological diversity
  - Global atmospheric changes
  - Green building definitions and foundations
  - Green building assessment and process
  - Sustainable sites and landscaping
  - Energy and atmosphere
  - Water
  - Materials
  - Indoor environmental quality
  - Construction operations
  - Building commissioning
  - Economic analysis of green buildings

3.4 Textbook and other required materials


3.5 Grading matrix

- Assignments (9%)
- Weekly quizzes and final exam (41%)
- Technical report (41%)
3.6 Invited speakers

The course was designed with one common meeting time each week which enabled us to bring in speakers from the community to talk on a wide range of topics. Presenters included the executive director of Wisconsin Green Building Alliance, an executive at Johnson Controls who was active in the development of LEED™, architects with LEED™ project experience, an environmental attorney, engineers to discuss storm water management projects, indoor air quality and building commission, a C&D recycler from a local environmental organization and a representative from a construction management firm discussing construction operations.

3.7 Technical report

The goal of the technical report is to identify one line item from the senior design (or other) project to evaluate and possibly upgrade for LEED™ points. There is a maximum length of five pages. Students can choose from a wide range of topics. Potential topics include engineered wood products, high performance steel, stainless steel, aluminum, insulation, concrete admixtures or replacements (flyash, blast-furnace slag), sealants and adhesives, painting systems, new applications for plastics, CFC replacements, fire retardant materials, lighting fixtures, roofing materials, low-flow toilets, etc.

The report requires a search of the engineering literature. The student may also search the general literature and the Internet and are encouraged to interview practitioners and vendors. The comparison of a state-of-the-art to alternative construction material/application/product must include a discussion of performance, costs and environmental impacts. Students address capital and operating/maintenance costs over the life cycle, energy impacts, hazardous and solid waste impacts, and construction issues. The student must also evaluate the material/application/product for potential LEED™ points.

4 KERN CENTER PROJECT CONTRIBUTIONS

Following are some of the useful findings of the NSF project that generated class discussion and student research projects:

- Over 23,000 gallons of diesel and 20,000 gallons of gasoline were consumed on this project.

- On-site fuel peaked during the first three months of construction during site excavation; worker fuel peaked during January and May of 2004.

- Daily reports were not a reliable source of information for equipment and electronic tracking of equipment was recommended for the future.

- Life Cycle Analysis, which includes Life Cycle Inventory, is a tool to evaluate the environmental impact of a material or process. Publicly available data was obtained for concrete, softwood lumber, electricity, gasoline, diesel fuel, BOF (basic oxygen furnace) and EAF (electric arc furnace) steel. The impacts from materials were added to impacts from fuel consumed by construction equipment and from vehicles used for material, fuel and labor transport. Impacts were quantified as generated on site or generated off site. It was found that LCA data, if publicly available, can be useful for determining environmental impacts. If not publicly available, purchased software with life cycle inventory information is available, but costly. The quality of the data provided often seems questionable.

- Fuel required to bring labor to the construction site was quantified for the period April through December 2003 (about 30% of the construction project). Assumptions included a 53 mile round trip and 18 miles per gallon per worker. It was also assumed that no one in the City of Milwaukee would carpool, no more than three persons would carpool per car (most workers drove trucks), and five miles was added to the round trip. Over 345,000 miles was driven by workers during this time; over 19,000 gallons of fuel were consumed. Two carpooling alternatives were assessed- subcontractor organized or general contractor organized. If carpooling was organized by the subcontractor, a 13% reduction in fuel was possible; if organized by the general contractor a 17% reduction was possible. It was concluded that carpooling is a feasible way to reduce worker fuel emissions.

- The potential for landfill diversion of construction materials and cost savings if there had been construction recycling during Kern Center construction was evaluated. The construction site was small and MSOE was unable to get permits allowing recycling dumpsters on sidewalks, so all material (about 790 tons of waste) was landfilled. No information was available on the composition of that material so total weight was calculated from invoices and waste composition was based on WasteCap Wisconsin's data. It was found that about 590 tons (5800 cubic yards) could have been diverted from local landfills. About $19,400 of the $41,400 spent for landfilling could have been avoided. It was found that construction waste recycling is beneficial and cost effective.

- One MS in Environmental Engineering capstone project, “Quantifying VOC’S in Products Off Gassed...”
during the Construction of MSOE’s Kern Center and Proposed Strategies for Reducing Employee Exposure,” evaluated products that off-gassed volatile organic compounds (VOCs) actually used in the Kern Center from the time of building completion to the end of the project. A paper based on this project is being presented at the conference.

- A second MS in Environmental Engineering project, “Assessing environmental impacts of site activities during construction of MSOE’s Kern Center: site and concrete construction,” was completed. The goal of this project was to perform a baseline study of flows of materials and emissions during CSI Divisions 31 and 3. Activities for CSI Division 31 included demolition of existing site improvements, site excavation, hauling of excavated material, installation of the soil retention system, installation of building foundation, and fuels to transport construction materials, fuels and site personnel to the site. Activities for CSI Division 3 included cast-in-place and pre-cast concrete, concrete sealants, fuels to transport construction materials, fuels and site personnel to the site.

Actual quantities of materials came from invoices and were compared to quantities estimated from plans and drawings. There was good agreement (within about 7%) between actual and estimated values.

Flows to the environment are given below:

- Air emissions (73% of total outputs); 75% of the air emission outputs occurred off site and 25% of occurred on site.
- Solid wastes (21% of total outputs); 77% of the solid waste outputs occurred on site and 23% occurred off site.
- Waterborne wastes (6% of total outputs); 93% of the waterborne waste outputs occurred on site and 7% occurred off site.

Overall, almost 2/3 of the total flows to the environment from this project were attributable to pollution embodied in fuels and materials; 1/3 of the flows were attributable to on-site construction. Most of the emissions were air and almost all of these were carbon dioxide. Water flows were primarily due to washing out concrete trucks. Solid wastes were attributable to landfilled contaminated soil excavated from the site.

5 COURSE OBSERVATIONS AND FEEDBACK

There were three sections of AE 4121 and a total of 52 students. We solicited feedback on both course and teacher evaluations.

Feedback on course elements included:

- Goals and objectives
  - “too much a LEED class”
  - “being an environmental major, this course was very helpful in the understanding of LEED”
  - “this class was a waste of my time”
  - “this course seemed to extremely academic and theoretical”
  - “after working in the construction industry for 6 years I have never run across any Green Building issues; now I am planning on getting LEED accredited this summer”
  - “information in the course is very important”
  - “overall a good class”
  - “talk to the USGBC and try to get them to let us take the actual LEED test here”
  - “there was too much emphasis on research”
  - “all subjects were covered well, but rushed unnecessarily”

- Topics
  - “need more examples”
  - “focus more on green building than LEED”
  - “learned about many of the topics in other classes”
  - “economic analysis most important for me”
  - “topics covered well, but in many cases there was too much information”
  - “much of the information seemed to be just research”
  - “very helpful in explaining LEED”
  - “not useful at all”
  - “never opened or read it”
  - “excellent, a good resource”
  - “very detailed and gives useful examples”
  - “the text is fair; the industry is changing so it will be hard to keep the text up to date”
  - “I don’t think there should be a text in this class”

- Assignments
  - “great integration to senior project”
  - “waste of time”
  - “I learned a lot about the topic of the paper I wrote”
  - “good, allowed us to explore different means of solving environmental issues”
  - “representative of material covered”

- Invited speakers
  - “too much repetition of material”
  - “speakers were good and had a lot of information to convey”
  - “showed real world use of LEED in construction”
  - “good points of view”
  - “some were good and some were boring”
  - “maybe have fewer that explain what LEED is and more on how to design some of the different systems and choose materials to gain LEED points”

- Technical report
  - “great integration to senior project”
  - “liked the report because of insights gained into what I can implement”
“reasonable as a means of individual investigation, but generally just more work in an already busy year”
“a tough paper, but good”
“informative”
“very effective”
“research paper lets us dig into a LEED topic and explore the inner workings”
“the presentations on reports were a good idea; they let us hear about all sorts of different products”
“exposed everyone to the alternatives that exist in the building environment”
“honestly, I had so little time to spend on it that I am not really happy with the result, but had little choice about that”
“biggest waste of time of the whole class”

As indicated, there was a wide range of student comments. On balance most were positive, but a number were clearly negative. Students indicted that the total work load (AE 4121 and senior design) was high, “sustainable construction” is important and likely to become increasingly so, it was valuable to get the perspective of invited speakers and the research report was useful. For a first time offering, the course was considered successful. We will continue to integrate elements of this course with the senior design or other projects. Completing a LEED checklist will continue to be required for senior design. Incorporating environmental science content remains a major objective. The research report and presentation will continue to be included.

In general, teacher evaluations were favorable on the topics surveyed – organization and planning, communications, faculty/student interaction and assignments/grading.

6 CONCLUSIONS

The course was perhaps too ambitious for a three-credit course offered concurrently with senior design, considering the large time commitments required for senior design. Student receptivity to environmental science content is less than optimal; they expect to have courses with science content earlier in their academic programs. There is a wide variation in student receptiveness to “sustainable” construction that ranges from “environmentalist propaganda” to “this course was a true eye opener for me”. If environmental impacts of construction are to be reduced, future engineers must understand a wide range of sustainability-related issues. Teaching “sustainable construction” to construction students effectively promises to be an ongoing, but interesting, challenge.

REFERENCES

Milwaukee School of Engineering’s Architectural Engineering & Building Construction Dept. http://www.msoe.edu/ae/
Cardboard: An innovative construction material

J. Schönwälder & J.G. Rots
Building Technology, Faculty of Architecture, Delft University of Technology, The Netherlands

ABSTRACT: Cardboard is indispensable for our daily use, but considering it as a construction material is not a common practice yet. Cardboard is sustainable, it is based on renewable or even recycled resources, environmentally friendly and recyclable. It is also an inexpensive material; light but very strong. Hence, its application for temporary constructions is reasonable. In this paper this innovative and sustainable construction material is presented. In particular it is focused on the mechanical and structural behaviour of cardboard and cardboard elements. Therefore different types of cardboard structures are analysed. With these different types of cardboard, beam components were designed and tested. The aim was to use the qualities of each basic material for the adequate part of the beam and to find a preferable design with a favourable strength to weight relation. The prototypes showed, that cardboard is strong enough to be used as a construction material and through smart design it can be the preferable construction material.

1 INTRODUCTION

The waste disposal of building materials at expiration of the service life of a construction is still a problem for the building industry. Most materials cannot be reused or recycled economically with the current technologies. This is a particular problem for constructions which are designed for a short period of time, such as exhibition halls or emergency shelters. For temporary buildings the capacity of common materials is not fully used concerning the short service life and the proportion of demolition and recycling in the life cycle costs is much higher compared to permanent buildings. Here, the application of cardboard can be a solution.

Cardboard is not as durable as concrete or steel, but it is recyclable, biodegradable and made from renewable resources. It is an inexpensive material which is remarkably strong considering its light weight. Hence, application for temporary constructions is reasonable. However, to make reliability cardboard constructions further research into several aspects is required.

2 CARDBOARD AND ARCHITECTURE

From architectonic point of view not only the ‘green’ aspect of cardboard is interesting, it is also a very appealing material as it is variable in form and structure, easier than any other building material. By bending, folding and gluing many types of structural components can be produced.

2.1 Application fields

The Japanese Architect Shigeru Ban has made cardboard presentable as structural material. He already designed numerous cardboard buildings mainly using paper tubes [Siegfried 2000]. Among them are the Japan Pavilion for the 2000 World Expo in Hanover [McQuaid 2003] and the Paperdome in Amsterdam [Eekhout 2004]. Some of his paper buildings are still in use, even though they were only designed and constructed for a short period of time.

Other examples of realized projects where cardboard was used as building material are the Cardboard School in Westcliff on Sea, UK [Buro Happold 2001] or the emergency shelters from Durakit, Canada [Durakit 2006].

All these projects show that cardboard has potential for application in many structural fields.
However, due to the non-durable nature of cardboard they will be restricted to temporary buildings. One of the main application fields for the use of cardboard is probably emergency houses. Here the cheap price of the material, the light weight considering shipping and assembling on site and the recyclability are the essential aspects to choose for cardboard. Also the event and tradeshow sector is supposed to be a large market for cardboard constructions. Here the structures are used for a short period and must then be either disposed or transported to another location for reuse. Furthermore, there is also the possibility to use cardboard for temporary interior walls. In this case acoustic and thermal properties are essential as well. Structures like honeycomb combine good thermal isolation with excellent mechanical properties for walls. Other possible applications for cardboard are auxiliary means during construction. Examples would be cable supports or formwork for casting concrete, like it is already common practice for round concrete columns.

2.2 Water and fire resistance

The main objection considering cardboard as a building material will concern the impact of water and fire. It is not possible to make cardboard absolutely fireproof but special flame-retardants in the pulp or surface treatments can make it sufficiently fire-resistant to meet most fire code requirements as tests at the TU Delft and TNO have proven [EET 2003].

Water is a significant problem for cardboard constructions, due to the hygroscopic nature of paper. The cellulose fibers are connected by hydrogen bonds which are very sensitive to water. Any absorption of water loosens the fiber bonds and leads to the decrease of consistency and strength of the material. Thus cardboard constructions always have to be adequately protected against direct water contact and humidity in the air.

Therefore already the design should avoid that the cardboard elements come in direct contact with water by constructional means such as roofs, covers, skins or other facades. Additionally the elements have to be treated to avoid the penetration of the humidity from the air. This can be done by water-rejecting additives mixed in the pulp during production and by applying special water-resistant layers or coatings on the surface. Thereby it is of utmost importance not to omit the cut edges and treat them carefully with a water-repellent coating.

All these water and fire resistant additives and coatings have primarily the function to protect the cardboard construction, but they should also be recyclable, not impact the mechanical properties of the elements and not increase the material costs significantly. This can be a problem when the treatments are water-based, which again loosens the bond of the fibers in the paper and decreases the maximum strength of the cardboard elements. If the additives or coatings are not recyclable themselves at least they should not impact the recyclability of the cardboard and it should be possible to remove them easily either before or during the re-pulping.

There are many different products to provide fire and water resistance. Typical products for fire-resistance are antimony compounds added as pigment in the pulp or surface treatments of e.g. ammonium sulfate salts or sodium silicates. Waterproof layers can be PE-foils or very thin aluminum foils which are preferably applied as a sandwich layer during the converting process of the paper. A water-resistant treatment creates a hydrophobic surface which can be received by silicates (special waterglasses), polymerized silicon compounds, oils, waxes or thermosets (such as epoxy or phenolic resins) or thermoplastics (PE or PVC). Whereas oils and waxes are not very lasting and should be only considered in not rainy areas. Thermosets are water repellent and self-extinguishing, and give extra strength to the elements. However, they are not recyclable. Thermoplastics are recyclable, but also flammable.

The choice of the water and fire resistant product is depending on the environmental circumstances (temperature and humidity), on the quality of paper or board that is used (virgin or recycled fibers), on the desired service life, on the function of the respective element and the effect the coating can have on its mechanical properties.

The topic of fire and water resistance requires definitely further research to provide a reasonable and sustainable construction material.

3 CARDBOARD AND SUSTAINABILITY

Sustainability seeks to provide the best outcomes of developments for human and natural environments. It relates economic aspects, the social responsibility
and the protection of the environment, so they together lead to an improved quality of life for present and future generations.

A sustainable building material qualifies by giving the least impact on the environment and protecting the natural resources. It is renewable, reusable and recyclable. The material should be used in an economic and ecologic way during the entire life cycle. The lifecycle of a building material comprehends material exploration, production, transportation, construction, service life, dismantling and material disposal.

In order to evaluate the proper environmental and economical sustainability index and the embodied energy of a construction material a life-cycle assessment of the respective building is necessary.

### 3.1 Resources and raw material

Paper and board are based on cellulose fibers. The primary raw material of paper is wood. Wood is a renewable resource. 1/10 of the world wide clearing is used for the paper production. Mainly the wood supply for the paper production is the byproduct from timber production or low quality wood from thinning the forests. However it is important that this wood comes from sustainable forestry management which actively protects the ecosystem forest. The introduction of a world wide certification system is necessary.

Furthermore paper can also be based on recovered fibers. Then paper products are recycled and used as secondary raw material. The use of waste paper is the most sustainable solution concerning the protection of raw materials and energy consumption. However the strength of recycled paper products is lower than from virgin (fresh) fibers. In structural application this has to be considered.

### 3.2 Production process

During the production process the sustainability of paper products was significantly improved since the past 15 years concerning energy efficiency, water consumption and CO₂ emission.

Paper is a very energy intensive material. The energy can represent up to 25% of the production costs. In Europe the specific energy consumption could be reduced by 31% in the past decade [ICFPA 2005]. To produce one ton of solid board approximately 7.5 GJ of energy is needed [EET 2003]. The more waste paper is used in the end product the lower is the energy consumption. To determine the embodied energy of a building material also the transportation gives impact to the energy, which comes to the embodied energy of 9.4 GJ/ton for solid board and 8.8 GJ/ton for corrugated board [EET 2003]. This is 2 times higher than the embodied energy of timber which is 4.7 GJ/ton but much lower than the one of steel, which is 25 GJ/ton.

Water is the essential means for the papermaking process. Meanwhile through modern technology the amount of fresh water to produce one kilogram paper could be reduced by one third in the last decade [ICFPA 2005] and is now 6–12 litres. This is also because modern paper mills have an internal water circulation where the water is reused up to 10 times before it leaves the mill mechanical and biological cleaned. However, it is important to what extent the water is chemically soiled. According to a study of FEFCO [FEFCO 2000] the amount of polluting substances like metals, chlorine and phosphor is less than 0.001 kg/ton.

In Europe the emission of CO₂ during the paper production process could be reduced by 30% in the last decade.

### 3.3 Recycling

Paper is the most recycled material in our society. Globally, the consumption of recovered fibers amounted to nearly 46% of all fibers used for production in 2000 [ICFPA 2005]. However, the dream of an endless cycle of waste paper is an illusion. During each recycling process the fibres shorten and lose their strength. In general a fibre can be reused 7 times.

The recycling of paper is simply done by re-pulping but also depending on the quality of the collected waste paper. Concerning the recyclability of cardboard elements of a construction, the cardboard is not the issue of matter. The additives, fillers, coatings and adhesives which are necessary for the strength and impregnation of the construction material, bring the recycling into question. Some coatings have to be removed mechanically before the cardboard can go into the re-pulping. The most economic and ecologic solution of this issue is still essential for the use of cardboard in architecture, and a balance between sustainability and material improvement is necessary.

Furthermore, cardboard constructions can be designed in a way they can easily be dismantled which facilitates the reuse or recycling of the material.

### 3.4 Economical

Cardboard is economical because it is inexpensive and light. The light weight reduces the energy consumption and the costs for transportation and assembling. A lower dead load of a construction also reduces the dimensions of the structural components.

For temporary constructions the use of cardboard will definitely be favorable, considering the material costs. At the moment cardboard is still an unknown building material and the investigation and the development of technologies and prototypes make cardboard buildings rather expensive. A collected knowledge and standard technologies will facilitate and cheapen cardboard constructions.
4 MECHANICAL PROPERTIES OF PAPER AND CARDBOARD

In order to establish cardboard as a ‘real’ building material further research in several areas is necessary. Hereby the structural safety of the building is important and the investigation of the mechanical behavior of the material is essential. In the following the mechanical properties of single sheets as well as of components are characterized.

4.1 General

Paper is the generic term of all cellulose fibre products made in form of a web or sheet by felting virgin or secondary fibres from a specially prepared suspension. Furthermore there is the subdivision of two main grades: paper and cardboard. The classification of these grades is not very uniform within the paper industry. Cardboard is usually thicker and stiffer than paper. It also comprehends paper products composed of several layers such as solid, corrugated or honeycomb boards. In general, all products heavier than 225 g/m² are referred to cardboard.

The research on the mechanical and structural behavior of cardboard is essential for the further development of ‘Cardboard in Architecture’. It gives insight in the material response and hence delivers indications for design and technology development. A considerable database of knowledge of paper properties is available in the paper industry, but rather concerning production processing, visual quality, packaging and personal hygiene. Most of the mechanical properties required for structural application are generally not determined for paper or board.

4.2 Single sheet

As all cardboard products are composed of several layers of single sheets it is elementary to study the mechanical behavior of a single sheet first.

Paper is a disordered network of cellulose fibres. The properties of the fibres and the bonding between the fibres determine the mechanical behaviour of a sheet. These characteristics are influenced by the choice of the raw material and the papermaking operations [Niskanen 1998]. The mechanical behaviour of paper thus depends on various factors and is influenced by both network and fibre scale. That means that all papers and cardboards differ from each other unless they consist of the same raw material and were produced in the same way. This makes paper and board difficult to standardize in mechanical point of view. However, in general paper and cardboard is an inhomogeneous, anisotropic, non-linear, viscoelastic and hygroscopic material. In the following general tendencies and relations of the in-plane properties of paper and board are presented without taking into account fiber or bond parameters.

4.2.1 Anisotropy

The anisotropy is due to the manufacturing process. During the forming and drying process the fibres align more in the production direction (the machine direction [MD]), than in the perpendicular direction (the cross-machine direction [CD]). This has the consequence that in MD the paper or board is stronger than in CD. The MD/CD-ratio, or the anisotropy, depends on the fibre properties and production processes and vary between 1,5 and 4.

4.2.2 Strength and stiffness

Figure 3 shows typical stress-strain curves of paper in tension and compression for MD and CD. The four different curves indicate the anisotropy of paper. In MD the material is stronger than in CD. Papers can be produced with a maximum tensile strength of 80 MPa in MD. In CD the board is less stiff, the strength is lower and the deformation higher. Also a sheet of paper has higher tensile strength than compression strength. Generally it is assumed that the E-modulus is the same for compression and tension. The curves show, except for tension in CD, a relatively brittle failure, that means there is no significant plastic deformation before breaking.

For the mechanical properties such as tensile and compressive strength, elastic modulus and maximum strain, general relationships exist. These are collected in Table 1. It can be seen that most of the properties could be derived from one single tensile test, if no more experimental data is available. However, as the relations are still rather vague and depend on the single paper it is always advisable to do a complete test series for more specific information.

4.2.3 Creep

Time and rate-dependent properties characterize cardboard as a viscoelastic material. Creep is an increase
of strain at a constant stress level in time. The creep rate depends on the type of cardboard, stress level, relative humidity and other factors. Different papers exhibit different creep curves. Stress-relaxation is the decrease of stress at a constant strain level. Most of the stress decay is log linear with time. For both creep and relaxation no reference values are provided.

At the TU Delft tensile creep fracture on solid board was investigated at various constant stress levels [Schönwälder 2006a]. The outcome was that the specimen charged at 40% of the ultimate tensile strength still failed within a few months.

4.2.4 Hygroscopy
The cellulose fibers make cardboard hygroscopic. That means the moisture content of cardboard is related to the ambient relative humidity, RH, and temperature. The moisture content is highest in humid and cold conditions. When the moisture level in paper increases the fibers soften and the fiber-bonds loosen. As a consequence the stress-strain behavior of paper changes with moisture content. Increasing moisture contents reduce the elastic modulus and the failure stress. At 23°C and a RH of 50% the moisture content in cardboard is approximately 5%. At a relative humidity of 90% the moisture content is around 14% and the stiffness and strength properties decrease by 50%.

4.2.5 Comparison to other materials
In order to compare the mechanical properties of cardboard with common building materials, their properties are listed in Table 2. Steel, concrete and glass are very strong and stiff construction materials, however they are rather heavy. Concerning the weight density cardboard is comparable to wood. Wood is also an anisotropic material whereas wood is stronger
in the grain direction but has almost negligible properties in perpendicular direction. One can say during the processing from wood to paper, the product looses strength in the x-direction and gains strength in the perpendicular direction. The table shows that cardboard fits in the range of building materials, however the stiffness is very low.

The table also includes the outcome of tests performed on a solid board with the grammage of 1050 g/m² [Schönwälder 2006b], to give an idea of the strength of common solid board. The results confirm the relationships of the properties as given in Table 1.

4.3 Components

In component level the responses can be different than in sheet level. The behavior of the component is depending on the paper properties, the adhesive and the geometry. In the paper industry several cardboard components are available. The most common ones are solid board, corrugated board, honeycomb board and paper tubes. Each of them has its advantages for structural application. With a composition of these components structural elements such as panels, columns and beams can be produced.

4.3.1 Solid board

Solid board is a generic term for all massive boards which are produced as single-, multi-ply (couched), multilayered (laminated) or coated boards. They are available in many different grades. The main characteristics of solid board are the high density and homogenous structure of this material. Figure 4 illustrates the composition of a solid board.

Solid board always consists of a filler ply, a top liner and mostly a back liner. The plies can vary in thickness and strength. The filler is always made of recycled paper. The thickness of the filler ply influences the stiffness in the z-direction of the board. The strength of the liners determines the strength of the solid board in MD and CD. The top and back liner can have special properties. Mostly it is a Kraftliner, which is a very strong paper produced of at least 85% of virgin fibers. The top and back liner can also have special treatments like an extra PE-coating to make the board water-prove, or special fire-retardants can be added to make the board fire proof.

The basis weight of solid board is between 650 g/m² and 1650 g/m², and the density about 700 kg/m³. The recycling factor keeps the board environmentally friendly and cheap. The price for solid board is approximately 0.8 €/m².

The advantages of solid board for structural application are its high strength and stiffness, the possibility to add easily special refinements on the surface (for water or fire protection) and the possibility of lamination with honeycomb or corrugated board.

4.3.2 Corrugated board

Corrugated board is the most traditional packaging material. It is produced by guiding a paper web, the corrugating medium or fluting, through a slit between two corrugated rolls and pressing it into a waveform through a combination of pressure and heat. In the same machine, an even paper web (facing or liner) is then glued onto this corrugated paper on one or both sides (Fig. 5).

Kraftliner is usually used as facing, but also stronger solid board can be applied. The corrugated medium is mostly 100% recycled paper, and can have different thicknesses. A number of standard flute dimensions (classification in A, B, C, E and F) are available, which differ in flute height and flute length. Also boards with double or triple flutings can be produced.

The characteristics of corrugated board are the light weight and the high tensile strength in y-direction. In compression it is not very strong as the thin layers buckle easily. The air between the layers makes it a good isolation material. For the traditional corrugated board with one corrugated medium and Kraftliner as facing the density of corrugated board is about 370 kg/m³ and the costs about 0.5 €/m².

In construction corrugated board finds rather application as isolation material or for non-structural elements such as interior walls.

4.3.3 Honeycomb board

Honeycomb board is a packaging material, but it also already finds application in the building sector, as a filler material of doors.

Honeycomb board consists of two layers of paper or board and one layer of honeycomb core in between
as illustrated in Figure 6. The honeycomb core is manufactured by a special technique of cutting and gluing pattern and can be made of variable paper grades from 140 g/m² to 250 g/m². The core can be produced in different heights and dimensions.

Due to the hexagonal structure of the core honeycomb board is characterized by a high compressive strength in z-direction. This is received with relatively little material. The load can be up to 100 kN/m². The bending strength can also be very high but is dependent on the grade of the top and bottom liner and of the height of the honeycomb ply.

The density of honeycomb board is dependent on the core height, but is approximately 500 kg/m³. With 5–10 €/m² the honeycomb board is rather expensive.

4.3.4 Paper tubes
For most of the cardboard buildings realized so far cardboard tubes were used as structural element. Through the geometry this component is mainly used as bar or column.

The manufacturing of paper tubes is done by a winding technique. Several paper stripes are pulled out from a paper roll cassette, put into a jar filled with glue and by a winder they get rolled in a spiral around a steel core. The amount of paper stripes determines the wall thickness of the tube. Cardboard tubes can be processed into any length and thickness.

Due to the winding the anisotropy of the basic material is for the tubes not essential anymore. Tubes have a very good strength in x-direction. In tension the tubes are as strong as in compression, however so far it was not possible to develop appropriate joints that can transfer the tensile forces from the cardboard tube to the connecting element. Therefore in the design the tensile strength of cardboard tubes was often assumed as zero [Eekhout 2004].

The strength of the tubes is very dependent on the paper that was used, on the winding technique and on the glue. Cardboard tubes which were produced of recycled paper had generally a compressive strength between 6 and 12 MPa [McQuaid 2003, Buro Happold 2001], whereas tubes produced of virgin fibers had a strength of about 16 MPa [Eekhout 2004]. For structural application the wall thickness of the tubes should be more than 10 mm. Tubes cost about 10 €/m and have a density of approximately 800 kg/m³.

5 CARDBOARD BEAMS
The strength of cardboard in structural components was evaluated by constructing different designs of cardboard beams. These were tested in a four-point bending test to their maximum capable barring load. The requirement for the design was to construct a beam only made of cardboard (solid board, honeycomb, etc) and glue. The aim was to use the qualities of the basic materials and to find a preferable design for a beam with a good strength to weight relation. The outcome of three designs is presented. Their cross-sections are given in Figure 8. All beams had a span of 2.75 m and were tested in a four-point bending test (Fig. 9).

5.1 The solid board beam
This construction was built of only solid board. The main structure was a double I-beam where the flanges and the webs consisted of several sheets glued together with wood glue. As solid board is much stronger in tension than in compression, the top flange counted 12 layers and the bottom one 8 layers. The web was designed with 5 layers. The two webs were stiffened by a triangle construction to prevent early buckling. The flanges and the web were connected by toothing without any additional glue. The overall dimension of this
beam was 30 × 20 cm and the own weight approximately 27 kg.

The maximum load of this beam was 10 kN. The load-deformation response was linear up to the maximum strength. The failure occurred in the top flange and was due to compression, as the top flange buckled in the middle of the beam length, as shown in Figure 10, left. With further loading also the web started to buckle in this zone (Fig. 10, right). The load-deformation curve (Fig. 11) also shows clearly these two failures. The first peak belongs to the buckling of the top flange and shows a significant decrease of load capacity which increased again until the failure of the web (second peak).

This beam showed very good results, and the design was clever and easy for manufacturing. Only the overall dimensions could be reduced to make the load-slenderness relation more attractive.

5.2 The honeycomb beam

As the solid board beam was rather heavy a design with lighter material such as honeycomb was chosen. In the compression zone of the beam honeycomb boards were placed vertical in order to benefit from the high compression strength of honeycomb structures. Around the honeycomb structure solid board was glued in a box shape. The bottom, the tensile zone, of the beam was strengthened by extra layers of solid board. This beam had an overall dimension of 25 × 15 cm and a weight of 17 kg.

The maximum load of the beam was nearly 4 kN. The beam showed local buckling underneath the load transmission (Fig. 12, left). Later examination of the inside of the beam showed that at these areas the solid board delaminated from the honeycomb structure, but also a crashing of the top horizontal placed honeycomb layer in the compression zone (Fig. 12, right).
The load-deformation response (Fig. 13) was linear until reaching the maximum strength and showed a non-brittle and almost ideal plastic behavior.

5.3 The reinforced honeycomb beam
As the previous beams failed in the compression zone, a beam was designed with a tube in the upper part of the beam as a compressive reinforcement. The rest of the construction resembled the honeycomb beam to have a clear comparison for the effect of compressive reinforcement. The cardboard tube increased the own weight of the beam to 22 kg.

The maximum load of this beam was 6 kN. As expected the beam could bear more compressive stress and started to crack in the tension zone. After reaching the maximum strength the crack mouth opened very fast and the beam immediately lost strength (Fig. 14). The load deformation response of this beam was hence very brittle (Fig. 15). After the crack opening the tube in the top flange was exposed to bending and contributed to the residual strength of the beam (Fig. 14, right).

5.4 Evaluation of cardboard beams
The results of the preliminary beams showed that the glued connections were the weak parts of the structures. Often delamination occurred. All beams showed local buckling due to elementary slenderness.

The reinforced honeycomb beam was 50% stronger compared to pure honeycomb beam. Hence the compressive reinforcement improved the load bearing capacity of the beam enormously. However, the reinforced beam showed brittle failure, which should be avoided in construction as it gives no warning in terms of cracks before failure. A good balance between strengthening the compression zone and non-brittle failure has to be found.

Table 3. Results of the cardboard beams.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Max F [kN]</th>
<th>Weight [kg]</th>
<th>Price [€]</th>
<th>Weight [%]</th>
<th>Price [€/kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10</td>
<td>27</td>
<td>21</td>
<td>2.7</td>
<td>2.10</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>17</td>
<td>24</td>
<td>4.3</td>
<td>6.00</td>
</tr>
<tr>
<td>3</td>
<td>6</td>
<td>22</td>
<td>28</td>
<td>3.7</td>
<td>4.67</td>
</tr>
</tbody>
</table>

Regarding the deflection of the beams, all beams were not very stiff and showed high deformations. These were due to local buckling and deformation, but also because the basic material did not have a significant stiffness. A stiffer beam can be obtained with a higher modulus of elasticity of the basic material and higher moment of inertia of the cross-section.

In order to evaluate the efficiency of each material the maximum resisting load, the own weight and the price of all three beams are listed in Table 3.

The list shows that the honeycomb beams are lighter but not very strong compared to the solid board beam.
and rather expensive. For a better comparison the ratios of weight to maximum load and price to maximum load are calculated. The solid board beam has the best ratio of own-weight to maximum load and the best price to maximum force ratio. A comparative analysis of beams of the same dimensions produced with common building materials showed that the solid board beam is comparable with concrete and wood in the price/max F ratio and better than concrete in the weight/max F ratio.

Nevertheless, these beams were just prototypes and improvements of the maximum strength to own weight ratio are expected with the further development. The prototypes showed that cardboard is strong enough to be used as a construction material and through smart design and maybe in combination with other materials it can be a preferable material to use.

5.5 Advice for the design of cardboard elements

In general, for the design of a structural component in cardboard it is important to consider the anisotropy while placing the sheet, so that the MD veers towards the principle stress direction of the component.

The structural design should take the possibility of local buckling of paper sheets into account and minimize the free element length in the compression zones.

Also load concentrations should be avoided in the structural component, as the fiber network of paper and board is very sensitive to point loads. This is particular important for the design of connections.

The viscous nature of paper and board has an influence on the long-term behavior of the structure and should be considered in the construction and in the safety factor of the material.

As the current available cardboard is not very stiff and huge deformations arise, it is advisable to apply pure cardboard elements only where deformations are acceptable, for instance in roofs or trusses, or to stiffen the cardboard with other materials such as fiberboard.

Also it is very important that the cardboard components are sufficiently impregnated and have no unsealed areas, to avoid that humidity can penetrate in the cardboard structures and decrease the mechanical behavior.

6 CONCLUSION

Cardboard has been introduced as an innovative sustainable building material with structural application for temporary constructions. It was shown that cardboard is ecological sustainable, as it is from renewable or even recycled resources, bio-degradable and recyclable and the impact of the production process on the environment was ameliorated in the past decades. Cardboard is inexpensive and light which makes it also economic sustainable.

Furthermore, the mechanical behavior of cardboard and cardboard structures was closer examined. The strength and the density of cardboard is comparable to wood, however cardboard is less stiff. Cardboard is also viscous and hygroscopic material. By designing and testing the cardboard beams it was demonstrated that cardboard is variable in many forms and structures and that structural components can be easily composed by cutting folding and gluing. The cardboard beams also showed that with smart design strong, light and cheap elements can be produced.

As cardboard is still a new and undeveloped construction material, the research area is still very wide. Especially the fields of water and fire-resistance, improvement of the material stiffness and development of connections have to be further investigated. However, even though there is still a long way to go, cardboard is attractive and has big potential for an innovative, sustainable construction material.

REFERENCES

Improvement mechanism of bondability in uf-bonded reed board by pf

J. Torkaman
Faculty of Natural Resources, University of Guilan, Sowmehsara, Guilan, Iran

ABSTRACT: In this study, reed (phragmites australis) particles and urea formaldehyde (uf) resin were used for particleboard manufacturing. The properties of uf bonded reed particle boards were relatively lower than those of commercial particleboard. The purpose of this study was to improve bondability by pf resin. In this experiment, phenol formaldehyde (pf) was used to replace 5 and 10% of uf adhesive. The properties of the resulting mixed adhesive products were then evaluated and compared with particleboard bond with only uf. Properties evaluated were modulus of rupture (MOR), internal bond (IB), thickness swelling (TS) and water absorption. The results showed that replacement of uf with pf improved the mechanical and physical properties of particleboard. The best condition according to Iranian standard obtained in 5% replacement and 190°C press temperature and 6 min press time.

1 INTRODUCTION

In view of the global decline in the forest resources and population growth, agro-based resources have been exploited in recent years to supplement the supply of wood materials from the existing forest resources. One of the main potentials utilization of these agro-based materials is conversion to composite products. Research studies have been conducted on these aspects for years. And the manufacturing technology of processing bamboo, bagasse, sorghum, and oil palm stems into composite products has been developed (ANDY et al., 1996; ZHANG et al., 1996; KIMURA et al., 1996; YAHYA, 1994). Some problems still exist with seasonality, storage, scattering sources and bondability. Among these factors bondability remains a major unsolved technical problem, especially when urea-based resins are applied (ZUCARO et al., 1995; HAN et al., 1995; SAUTER, 1996). Reed has traditionally been used to make paper but in recent years other utilization are being considered such as particleboard.

2 MATERIALS AND METHODS

Reeds (phragmites australis) with 25% moisture content were obtained from sowmehsara city of Guilan province, Iran. Reed air-dried density was 0.5 gr/cm³. The stems were first cut into 70 mm lengths by a drum chipper and further refined using a Pullman flaker. The particles were then screen according Bison Quality Control 44011 standard.

Table 1. The composition of particles based on mesh analysis.

<table>
<thead>
<tr>
<th>Mesh size (mm)</th>
<th>Reed particles (%)</th>
<th>S = l/t</th>
<th>J = w/t</th>
<th>A = l/w</th>
</tr>
</thead>
<tbody>
<tr>
<td>≥4</td>
<td>25</td>
<td>115.5</td>
<td>4.7</td>
<td>15</td>
</tr>
<tr>
<td>4–2</td>
<td>35</td>
<td>105.5</td>
<td>4.8</td>
<td>14.5</td>
</tr>
<tr>
<td>2–1</td>
<td>30</td>
<td>120.5</td>
<td>4.8</td>
<td>13.8</td>
</tr>
<tr>
<td>1–0.4</td>
<td>7.5</td>
<td>12.5</td>
<td>4.1</td>
<td>12.7</td>
</tr>
<tr>
<td>≤0.4</td>
<td>2.5</td>
<td>90.5</td>
<td>3.5</td>
<td>11.5</td>
</tr>
<tr>
<td>Min</td>
<td>–</td>
<td>110.5</td>
<td>4.36</td>
<td>13.5</td>
</tr>
</tbody>
</table>

Table 2. Synthetic adhesives properties.

<table>
<thead>
<tr>
<th>Adhesive</th>
<th>Density (g/cm³)</th>
<th>Solid content(%)</th>
<th>pH</th>
<th>Viscosity(cp)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pf</td>
<td>1.15</td>
<td>63.5</td>
<td>8.5</td>
<td>150</td>
</tr>
<tr>
<td>Uf</td>
<td>1.20</td>
<td>52</td>
<td>6.8</td>
<td>180</td>
</tr>
</tbody>
</table>

Table 1. Summarizes the composition of particles based on mesh analysis. All particles used were oven-dried at 100°C to about 2% moisture content. The specific surface area of the particles is increased with a reduction in the surface containing silica and wax; and most of the silicon on the surface could be removed by the milling process (WILLIAM, 1996). The uf and pf resins used in this study were supplied respectively by Nafis and rositan campanies, Iran. Table 2. Show their properties the pf resin used for improving the properties of particleboard produced from reed in this study.
2.1 Board manufacturing

The uf resin was sprayed on the particles in a blender at 10% resin content on the oven-dried weight of particles. Two percent of pf resin (5 and 10%) was mixed with the uf resin prior to blending based on the weight of the uf resin solid. One percent of NH$_4$Cl based on the weight of the resin solid, was added as the curing catalyst. The hand-formed mats were pressed into 15 mm thick boards using 30 KP/cm$^2$ bars at two press temperature (180 and 190°C) and two press time (4 and 6 min) (table 3). The board size was 500 $\times$ 500 $\times$ 15 mm with targeted densities 0.71 g/cm$^3$. Three boards were made in the same condition: all together 36 boards were manufactured.

2.2 Board testing

Specimens were cut from the boards after conditioning and tested according DIN-68761 standard. The specimen size for the bending test was 250 $\times$ 50 $\times$ 15 mm. The sample sizes for internal bond(IB) and thickness swelling(TS) were 50 $\times$ 50 mm and 25 $\times$ 25 mm respectively. Thickness swelling was determined by measuring the changes of board thickness after immersing in 20°C water for 2 h and 24 h.

3 RESULT AND DISCUSSION

The average Static bending, Internal bond, Thickness swelling and Water absorption values are shown in table 4.

Data for each test were statistically analysed. Multifactor analysis of variance was used ($\alpha = 0.05$) to test for significance between factors and levels. When the variance analysis indicated a significant difference among factors and levels. A multiple comparison of the means was performed employing a Dancan test to identify which groups were significantly different.

Based on Iranian standards Isiri(1985). The Static bending, Internal bond, the maximum Swelling (24 h) and the maximum water absorption (24 h) requirement for general purpose boards are 18 Mpa, 0.4 Mpa, 15 and 65% respectively. According to table 4 results static bending data ranged 13.8 to 30.02 Mpa. The increasing of all factors effect on Static bending. The best Static bending (30.02 Mpa) was obtained at 10% replacement and 190°C press temperature and 6 min press time condition.

Internal bond data ranged from 0.3 Mpa to 0.9 Mpa. Except for board types 1 and 4 all of the other boards had higher Internal bond than the general purpose requirements. The best Internal bond like to static bending was obtained in board types 12 and 8. According to table 4 the best thickness swelling (2 and 24 h) and Water absorption (2 and 24 h) were obtained in board types 7-8 and 11-12.

The results above indicate that the board properties are not satisfactory using uf resin and must be improved. Figure 1–4 and table 4 show the effects of pf on the properties of reed board. Except MOR all properties of board were improved at 5% replacement. According Dan can test there is not significantly different in all properties of board type 8 and 12 except MOR at 5 and 10% replacement.

<table>
<thead>
<tr>
<th>Board type</th>
<th>Replacement (%)</th>
<th>Press time (min)</th>
<th>Press temperature (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>4</td>
<td>180</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>4</td>
<td>190</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>6</td>
<td>180</td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>6</td>
<td>190</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>4</td>
<td>180</td>
</tr>
<tr>
<td>6</td>
<td>5</td>
<td>4</td>
<td>190</td>
</tr>
<tr>
<td>7</td>
<td>5</td>
<td>6</td>
<td>180</td>
</tr>
<tr>
<td>8</td>
<td>5</td>
<td>6</td>
<td>190</td>
</tr>
<tr>
<td>9</td>
<td>10</td>
<td>4</td>
<td>180</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>4</td>
<td>190</td>
</tr>
<tr>
<td>11</td>
<td>10</td>
<td>6</td>
<td>180</td>
</tr>
<tr>
<td>12</td>
<td>10</td>
<td>6</td>
<td>190</td>
</tr>
</tbody>
</table>

Figure 1. Effect of using pf as replacement of uf resin on Modulus of rupture of particleboard.

Figure 2. Effect of using pf as replacement of uf resin on Internal bond of particleboard.
The bondability of reed and wheat straw particle-boards which bonded with a uf resin using silane coupling agents were improved (HAN, 1998). Like other nonwood lignocelluloses both reed and wheat straws have higher hemicellulose and ash contents than wood. the outer surface of these two straws are covered with much silica and wax (HAN, 1995). However, the natural characteristics of these materials make them difficult to bond with urea– formaldehyde (uf) resin. an adhesive popularly used in particleboard manufacture (SAUTER, 1996; HAN et al 1998).

4 CONCLUSIONS

The following conclusion can dawn from the analysis of the results obtained in this study:

- Increasing of Phenol formaldehyde as replacement of Urea formaldehyde resin increase Modulus of rupture of reed boards.
- Maximum internal bond of particleboards was obtained in 5% replacement.
- Replacement of Urea formaldehyde with Phenol formaldehyde in 5 and 10 percentage increase both properties of thickness swelling and water absorption in 2 and 24 hr immersion.
- The best condition according to Iranian standard was obtained in 5% replacement and 190°C press temperature and 6 min press time.

REFERENCES


1 INTRODUCTION

This qualitative study explores current design, construction, and reporting practices for sustainable architecture in the United States. The primary purpose is to reconstruct and analyze the design and construction processes for the Xanterra houses and provide an example of how the architects, clients, and builders established a design philosophy and made tradeoffs vis-a-vis the Leadership in Energy and Environmental Design (LEED) scorecard. Such decisions were made in the context of a high-profile, exemplary project with significant external pressure to succeed. These houses were recently designated as the first building projects in the National Park system to receive LEED certification from the United States Green Building Council (USGBC). Moreover, the homes are promoted as a model for future building projects at Yellowstone and other national parks. A secondary purpose is to explore how the project achieved LEED certification and to assess the process of reporting and scoring. The paper addresses persons interested in sustainable architecture, passive solar heating, housing, and LEED.

This paper builds upon the existing literature on sustainable housing by demonstrating the efficacy of concrete construction and passive solar design. Some of this literature is oriented to practical recommendations [Steven Winter Associates 1983, Chiras 2002]. In fact, the paper shows that the designers of the Xanterra houses referenced this literature, and thus it analyzes how sustainable theories are applied and modified in practice. The academic and scientific literature includes several technical studies of the performance of specific individual houses or abstract building systems [Barley et al. 2004, Gadja & Van Geem 2000, Petrie et al. 2004]. These studies, however, followed a prescriptive design process rather than a collaborative one, so the authors also intended this paper to contribute significantly to the small body of literature found in popular publications that focuses on the collaborative design process as it relates to sustainability [ScottHanson & ScottHanson 2005].

This paper also builds upon the existing literature that examines the LEED system and its shortcomings. A few important papers conclude that a major problem of the LEED rating method is its failure to account for a building’s performance over its projected life [Trusty & Horst 2002, Bunz et al. 2006]. This paper verifies those conclusions by showing that the lifespan of concrete construction was not considered during the Xanterra homes’ LEED rating process. More importantly, very little academic attention has been paid to the process of LEED reporting and scoring within a professional setting, despite the fact that common business practices and human limitations can greatly affect a project’s LEED score [Pise 2006]. This paper shows that the LEED scorecard does not accurately reflect the architectural and engineering design of the Xanterra houses.

The larger questions that stimulated this research are consistent with problems that have motivated the current widespread interest in sustainable construction: How can architects design buildings that consume less energy? How can materials and construction practices...
be used more responsibly in terms of reducing pollution and waste? How should these practices be evaluated to understand their true efficacy? Such questions are particularly urgent for the American homebuilding industry, which in general produces structures that consume a large amount of energy since they are reliant on mechanical systems for heating and cooling. Since the Xanterra houses were consciously developed as a positive alternative to typical homebuilding practices, an analysis of their performance, from design through assessment, will have significant implications for future projects of a similar type.

2 METHODOLOGY

The area of exploration is quite broad with regard to the architectural design, construction, and assessment processes for sustainable housing in the United States, in the context of complex public-private relationships. To examine this subject, the authors conducted a qualitative inquiry in the case study tradition with a constructivist approach [Hatch 2002]. The case study methodology is used to reconstruct the design and construction activities of the Xanterra houses. The intent is to provide a descriptive narrative with a theory-building structure [Groat & Wang 2002]. The qualitative approach provides a unique insight into issues of design and professional practice.

3 FINDINGS

3.1 Project data and design intent

The Xanterra houses are two single-family homes for seasonal workers (see Figure 1). They are situated in Gardiner, Montana, population 851 (2000 census). They were completed in 2003, and certified under LEED-NC v2.0, project #1353, December 10, 2004. (The LEED Rating System for Homes was introduced in September 2005.)

The data includes interviews, artifacts, and observations. The interviews were unstructured and included the client’s representative, architect of record, and sustainability consultant. The unobtrusive artifacts included elements of the project record such as drawings, construction photographs, communications, energy simulations, etc. The observations were derived in the form of raw field notes and photographs taken directly at a visit to the research site. The data is analyzed to discover relationships, make interpretations, develop explanations, and generate theories. The process involves a combined inductive and interpretive analysis.
commissioning agency was Xanterra Parks & Resorts, a private company that holds a renewable agreement to operate commercially inside Yellowstone. Since green architecture is a priority of the federal government, Xanterra essentially “wanted to prove they were green” when building new housing for its workers in order to impress the National Park Service [Williams 2006]. The lead architect was Mark Headley of Overland Partners, Bozeman, Montana. The project’s sustainability consultant was Kath Williams, Ph.D., whose considerable credentials include seven years as vice-chair of the US Green Building Council. She was instrumental in developing the LEED system itself.

Xanterra set the performance parameters as “at least 40% more energy-efficient than a standard home,” defined by ASHRAE 90.2. As Deru and Torcellini noted, “Building performance typically follows the design goals established early in the design process; in other words, you get what you ask for” [Deru & Torcellini 2004]. Due to his engineering background, Hendrickson was thoroughly familiar with the basic principles of sustainable design, and thus he took a more active role than many clients’ representatives. He determined some of the major materials and systems selections and enforced the 40% standard set by Xanterra.

Hendrickson’s basic approach to the project was to follow the innate wisdom of buildings in ancient Native American civilizations. “Inhabitants of the Rocky Mountain west 1,000 years ago knew how to capture the sun’s energy and store it in thermal mass” [Hendrickson 2004]. He wonders why modern homebuilders rely on mechanical heating and cooling rather than following this timeless and fundamental lesson. He is also strongly influenced by Daniel Chiras’ 2002 book The Solar House, which he considers the authoritative text on this subject. Thus a guiding design philosophy for the Xanterra houses was provided by a combination of ancient wisdom and modern techniques based on scientific method and advanced technology. This philosophy was a driver of design decisions and tradeoffs during the collaborative process.

3.2 Site dynamics

The Xanterra houses are situated against the backdrop of one of America’s most spectacular natural landscapes, a few hundred yards from the north entrance to Yellowstone National Park. Gardiner, Montana, is a cold, mountain climate with 7560 annual heating degree days [NOAA 2002], located at 45° latitude. These characteristics suggested the need for a superinsulated building envelope. The elevation of 5300 ft (1615 m), high annual sun hours, and high atmospheric clearness numbers provided excellent conditions for passive solar heating, active solar power generation, and daylighting.

The shape of the particular building site, however, would not be considered ideal for sustainable architecture, since it included a considerable north-facing slope. The site measured 18,259 ft² (0.51 acre; 1,696.32 m²). Original plans called for three identical houses of 1,995 ft² (185.34 m²) each, which would have been a relatively dense building development by standards of this area, and thus the project would be relatively sustainable in this respect. In the end, only two of the three houses were built. Regardless, a LEED credit for density was not obtainable simply because of the non-urban location. The project did receive two LEED credits for a reduced development footprint and preserving green space.

The building site was considered contaminated because its former use was a fuel transfer station, and it contained abandoned industrial tanks and other hazardous material. Of course, site repair is one of the most fundamental and consequential aspects of sustainable development, as described by Christopher Alexander in A Pattern Language, “On no account place buildings in the places which are most beautiful. In fact, do the opposite.… Leave those areas that are the most precious, beautiful, conformable, and healthy as they are, and build new structures in those parts of the site which are least pleasant now” [Alexander 1977]. In the larger context of Yellowstone and its environs, the choice of a contaminated former industrial site in the town of Gardiner was clearly motivated by this notion of building on the worst part of the “place.” Furthermore, if the site was selected with the LEED scorecard in mind, the design team might have anticipated that remediation of site contamination would be rewarded.

3.3 The design process

The most significant aspect of the Xanterra houses’ design in terms of energy efficiency is their superinsulated envelopes. From the outset, the design team narrowed their options to structural insulated panels (SIPs) and insulated concrete forms (ICFs). A strong internal dispute emerged between these two systems, with the architects favoring SIPs and Hendrickson insisting on insulated concrete for its mass. According to Hendrickson, “I regarded SIP panels as simply good insulation. Insulation is indeed a good thing, but our nation had been using good insulation since the 70’s, and I felt strongly that insulation was not significant enough for a ‘leadership’ house” [Hendrickson 2004]. He felt that LEED offered an opportunity to incorporate thermal mass, and was convinced that ICF was a superior system and more appropriate for demonstrating leadership in sustainable design. He won the debate, and the homes were constructed with a wall system of 6 in (0.15 m) thick concrete walls with an outside layer of 4.25 in (10.80 cm) expanded polystyrene and an inside layer 2.25 in (5.72 cm)
thick. The overall insulation value may be as high as R53, if calculated by ‘equivalent R-value’ [Kosny et al. 2001]. By comparison, the International Energy Conservation Code requires R18.4 for mass walls in residential buildings in this climate zone [IECC 2003].

Although SIPs were rejected for the wall system, they were a logical choice for roof construction, and this decision contributed greatly to the high performance of the building envelope. These particular panels were constructed of oriented strand board (OSB) and 12 in (0.305 m) foam insulation, resulting in a roof that offered an insulation value of R45 versus the standard R30 [IECC 2003]. In addition, the panels omitted the need for an attic, allowing additional ceiling height and increased daylighting on the upper level.

The design philosophy sought to incorporate passive heating, but interestingly, without optimization. According to Chiras, “Overglazing and undermassing a home, common problems of the past, can cause severe overheating. Pay attention to the glass-to-mass ratio to achieve optimum comfort” [Chiras 2002]. Both the client Hendrickson and the architect Headley had experienced the negative effects of overglazing and undermassing. Headley had built his own passive solar house that was 100% glazed on the south elevation but lacked adequate thermal mass. According to Hendrickson; it routinely overheated even in Montana’s cold winter climate [Hendrickson 2006b]. Hendrickson had a similar experience during childhood when he lived in an all-glass “House of Tomorrow” that had been designed by architect George Keck for the 1933 Chicago World’s Fair. The same overheating problem existed when Hendrickson lived in the house in the 1970s. These personal experiences lead to the appropriate use of ICF and SIP construction.

Paradoxically, though, one of the major features that would come to characterize the Xanterra houses was the sunspace, a technique specifically not recommended by Chiras. “In many cases, sunspaces provide their own heat, but not enough to be of great benefit to the rest of the house. Another significant problem is that it is often difficult to transfer heat generated inside a sunspace to neighboring rooms. Heat doesn’t flow laterally very well” [Chiras 2002]. The architect’s use of the sunspace, and the client’s acceptance of it, suggests that the design process was not prescriptive but marked by negotiation and compromise.

Moreover, the south-facing wall of the sunspace was under-designed according to typical rules of thumb. It includes 53.4 ft² (4.96 m²) of glazing for a floor area of 85 ft² (7.90 m²). The recommended ratio for a cold climate is between 0.90–1.50, where the Xanterra houses utilize a value of 0.82. Perhaps in an effort to store more heat, the concrete slab floor of the sunspace was built 12 in (0.305 m) thick, although the Sustainable Buildings Industry Council advises that any slab thickness above 6 in (0.15 m) “has little effect” on storage [Chiras 2002].

The design of the upper level conformed more closely to recommended practices for passive solar design. It includes 86.7 ft² (8.05 m²) of glazing for a floor area of 87.75 ft² (8.15 m²), for a ratio of 0.99. The upper level was also finished with a concrete floor for thermal mass. A 2 in (5.08 cm) slab was placed above a sub-structure of engineered wood i-joists.

The glazing was designed to maximize solar gain for the south-facing spaces. On all orientations, the houses employed a low emissive (low E) glass, which is common in the United States and standard in Europe. The insulated glass is argon filled with the low E coating on the interior surface of the exterior pane. The low E coating is designed to “allow maximum daylight and short wavelength infrared radiation to enter the building and warm it, but at night to reduce heat loss by reflecting long wavelength heat back into the room” [Button & Pye 1993].

For passive solar architecture, it is important to treat the south elevation differently, in order to maximize solar gain in the winter. Thereby the architect’s initial design called for clear glass without the low E coating on the south windows. However, the use of clear glass would have allowed significant heat loss as well as gain. Instead, the design team located a new product that possesses both low E and a high solar heat gain coefficient (SHGC) of 0.65, which they placed on the south side alone. Then, in order to mitigate the problem of excessive heat gain in the summer, the architects designed south-facing shades and overhangs according to typical passive-solar methodology with dimensions appropriate to solar path.

The last major aspect of the design in terms of sustainability (aside from interior equipment such as dual-flush toilets, which are also very progressive in relation to typical American homes) was the installation of building integrated photovoltaics (BIPV) on the south-facing roofs of the sunspaces. BIPV is an energy collection system composed of photovoltaics partnered with construction assemblies. The intent is to avoid a stand alone solar array by offsetting the BIPV initial costs with the savings earned from substituting BIPV for the other envelope materials such as roof elements. The Xanterra houses employed BIPV in conjunction with the sunspace on the southern orientation although the photovoltaics were placed above conventional roofing rather than fully integrated with the building envelope.

3.4 The construction process and project costs
The houses were built by a local contractor specializing in commercial construction with little experience in sustainability. Furthermore, the contractor had no experience working with ICF; according to Hendrickson,
the contractor “never wanted to build one of these things and never will again” [Hendrickson 2006b].

One problem encountered during construction serves to illustrate the importance of effective communication between the designers and the builder, particularly when the builder may be unfamiliar with principles of sustainable construction. In this case, the interior wall between the sunspace and the main body of the house was constructed of ICF, which would have tended to insulate the sunspace, keeping it hot and depriving the rest of the house of the benefits of the solar gain. Since a sunspace is only effective if its heat can be transferred to the rest of the structure, Hendrickson argued correctly that it would be beneficial to remove the rigid foam insulation from this interior mass wall. The wall would become a thermal storage unit, collecting heat from the sunspace during the day and radiating it to the rest of the house at night. However, he had such difficulty convincing the builders of the importance of this detail that Hendrickson visited the jobsite in the dark of night and removed the insulation, leaving a bare concrete wall 6 in (0.15 m) thick.

The project was also notable for its aggressive recycling of construction waste, which is a goal of the LEED system. Kath Williams developed a ‘Construction Waste Management Plan’ for the project and worked with the builder on implementation. To obtain the maximum number of credits under the LEED system, at least 75% of construction, demolition, and land-clearing waste must be recycled. Moreover, it is a prerequisite requirement that there must be an area of the construction site dedicated to separation and collection of recyclables.

A commonly-asked question with regard to green buildings and LEED certification is: what are the added construction costs? Betsy del Monte, a LEED-certified architect, has argued, “There is a common misconception that . . . green buildings . . . are more expensive to construct than traditional buildings, [but] constructing a high-performance facility doesn’t necessarily mean using more costly materials or methods” [del Monte 2005]. The most thorough scientific research on this subject concluded that the average premium for the green buildings that they studied was “slightly less than 2%,” or about $3/ft² (£25.38/m²) to $5/ft² (£42.33/m²). These costs, the report concluded, were “substantially lower than is commonly perceived” [Kats et al. 2003].

For the Xanterra houses, the parties involved were hesitant to share the actual project costs, although everyone admitted that the houses were considerably more expensive than typical houses of that size and far more expensive than the typical housing for Park Service employees. The project most certainly exceeded the 2% premium that the Kats study suggested.

There were a few extraordinary contingencies in this project that triggered additional costs and justified the expense. As described above, the decontamination of the site was unusual and expensive, and the site’s steep slope required that the garages be built above-grade, a significant expenditure relative to slab-on-grade garage construction. But the most important justification for a larger construction budget was the nature of Xanterra’s relationship to the National Park Service. One member of the design team remarked that “Xanterra wanted it to be such a stellar project that they threw too much money at it” [Williams 2006]. Still, because the project was built under the auspices of the park service, it was required to be “reasonable,” and several features, such as fireplaces, were eliminated during the design process in order to reduce cost.

3.5 LEED certification process

LEED is now essentially a self-reporting and self-policing certification process. It operates, in the words of Kath Williams, “on the honor system” [Williams 2006], but at the time this project was certified there was a fairly strong formal review process involving reviewers from the USGBC. Under this system, most of the review materials took the form of a written narrative; the reviewer did not visit the project and did not necessarily review plans or photographs. Thus the certification depends largely on writing ability, which has frequently been acknowledged as a skill that many architects and engineers lack [Cuff & Robertson 1982]. Moreover, the USGBC reviewer’s judgment on any given point is able to be appealed, which places a further premium on the design team’s ability to communicate persuasively. LEED-accredited professionals are able to exploit this need by marketing their ability to communicate and shepherd a project through this reporting and appeals process.

For the Xanterra houses, the LEED reporting paperwork was not completed by Williams but instead was assigned to an intern architect, and some of the senior members of the design team never saw the final USGBC submittal. In fact, there were several mistakes made during the reporting process.

Despite the copious attention to the performance of the building envelope, the houses failed to get the proper number of LEED credits for Optimizing Energy Performance. This category requires that a mathematical energy model be developed in order to analyze the building’s theoretical performance compared to a ‘base case’ of the same type and size (defined by ASHRAE 90.2). When a third-party energy model was commissioned, the modeler mistakenly employed a high-performance house with an insulated concrete envelope and southern orientation for the ‘base case’ house. Even with the higher standard of comparison, the project received seven out of ten possible points for Optimizing
Energy Performance because the model showed reduced energy costs of 42%. Hendrickson later showed that if the ‘base case’ had been modeled correctly, the Xanterra houses would have performed significantly better, likely earning all ten LEED credits for Optimizing Energy Performance.

The final performance of the houses is not measurable, since they are not instrumented, although Williams said that the houses perform better than expected [Williams 2006]. Residents were not interviewed, since they are employees of Xanterra, which has a vested interest in good publicity for the houses. This is a limitation to the study.

Moreover, the Xanterra houses did not get full LEED credits for Construction Waste Management because the recycling process was documented incorrectly. In essence, the intern architect did not properly distinguish between excavated material and land-clearing debris. For example, the project was improperly punished for not having recycled two large concrete cradles which (decades prior) had held fuel tanks on the site. In fact, Hendrickson recalled, “we really did recycle pretty much everything,” and the project should have easily qualified for those recycling credits [Hendrickson 2006c].

Similarly, the project was denied a credit for Brownfield Redevelopment because of improper reporting, or the failure to make a persuasive argument, despite the fact that it clearly satisfied the sustainability goal of site repair (a brownfield is defined as a site that has actual or perceived contamination). In addition, the project received only one of three possible points for Renewable Energy, indicating that 5–10% of the buildings’ energy use was supplied through the use of on-site renewable energy systems when in fact the BIPV system produces 40–50% of the homes’ electricity.

Some of the other LEED credits that the project received were relatively easy to achieve and to document, and frankly had little to do with architectural design or the sustainable characteristics of the building. For example, the Xanterra houses received a point in Alternative Transportation for putting what Hendrickson described as “some $1.68 hooks” in the garage for bicycle storage [Hendrickson 2006a].

The Xanterra houses received 30 out of a possible 69 points, qualifying it as ‘LEED certified,’ but the authors have concluded that if the LEED reporting had been accurately and persuasively completed, the project should have received at least 38 points, enough for a silver medal and one point away from gold.

4 DISCUSSION

This study has several implications for future research and practice. The tradeoff between SIP and ICF construction for the wall system was the correct choice. Clearly the Xanterra houses confirm a principle that is already reasonably well-established: concrete houses perform better in terms of energy efficiency than wood-frame houses while insulated concrete provides even better results. In general, energy modeling shows that a house with ICF walls of R15 will use 5.5 to 8.5% less heating and cooling energy than a house with conventionally wood-framed walls of R10.6 [Petrie et al. 2002]. A typical ICF wall is rated much higher than R15 and will perform even higher than its rating [Kosny et al. 2001].

The use of ICF for residential construction in the United States is relatively small but growing rapidly. As of 2005, ICF technology accounted for 5.7% of residential construction nationwide, up from only 0.2% in 1995, according to the Insulating Concrete Form Association. A major barrier to the broad acceptance of ICF is economic. Using ICF rather than wood framing for above-grade walls will add 7.2 to 8.4% to the initial construction cost of a home, according to the National Association of Homebuilders’ Research Center.

This case study also suggests that the practices of the American window industry can create a significant barrier to homebuilders interested in pursuing passive solar heating. The low-e glass with a higher SHGC was labeled “northern glass” by one major manufacturer at the time these houses were designed, since they recommended it for northern climates. But Xanterra’s design team found this label to be confusing: ideally, for passive solar heating, it would be best to use low-e glass on all four elevations, but a higher SHGC (“northern glass”) on the southern elevation, as they did in this instance. However, the major glass companies in America recommend using the same glazing on all elevations of a house, according to Hendrickson, because they assume that homebuilders and consumers will be confused if a different type of glazing is specified for only the south-facing windows [Hendrickson 2006a].

Perhaps most significantly, the LEED certification system requires a difficult learning curve for clients and architects and difficult choices about trade-offs during the design process. For example, all LEED projects require the significant added expense of a LEED-accredited professional and an energy model. Architect Mark Headley notes that a typical computer energy model costs approximately $16,000 (€12,596), which is “a stumbling block for a lot of clients” [Headley 2006]. In general, architects typically suggest that the LEED process adds about $40,000–$50,000 (€31,489–39,362) to a project’s bottom line, regardless of its size or total budget.

Different members of the Xanterra project team ultimately formed various opinions about the process of working within the LEED system. Jim Hanna, Xanterra’s director of environmental affairs, remarked, “The good thing about LEED is that it gives you a
comprehensive checklist of options. You get to choose. It is not a rigid standard that tells what you must do. You pick the ones you want to do. If you earn enough points, they will certify your project” [McMichael 2005]. While some clients regard this as an attraction, the authors identify it as a weakness of the LEED system.

Hendrickson, the Park Service engineer, became rather disillusioned with the LEED system as a result of this project. He remains incredulous that the project received no points for ICF construction (because of the faulty energy modeling). In Hendrickson’s view, “LEED rewards ‘gizmo’ ideas and building practices” (a reference to the bicycle hooks in the garage) but does not emphasize strongly enough the fundamental principles of sustainable residential construction: solar orientation, super-insulated envelope design, and thermal mass. Even the new LEED Rating System for Homes, which is supposed to be simplified for the mainstream home building industry, is “absurdly complex and complicated. I don’t think any sensible person would ever follow it” [Hendrickson 2006a].

Virtually all of the popular literature related to LEED is positive in nature and meant to endorse the system; only a few academic studies address ‘green architecture’ from a critical cultural perspective [Bosch 2000]. Further research is needed to examine LEED’s true effectiveness as a tool for measuring sustainability versus its increasing importance as a public relations tool.

Some additional criticisms of LEED were discovered when attempting to situate the Xanterra houses in the context of international standards for sustainable design. First, LEED follows other international sustainable programs (Germany, United Kingdom, and Hong Kong, for example) which do not acknowledge the usable life of a building. “Despite emphasizing the life cycle, current guides do not address extension of the building life span” [Bunz et al. 2006]. LEED evaluates the performance of the (projected) building rather than the (actual) life cycle of the site, and the Xanterra houses were given no credit for the durability of their construction materials.

Another criticism is that the LEED system only recognizes positive sustainable elements and does not penalize for inappropriate use of non-sustainable design. The LEED scorecard gives a single point for positive achievement in each category and zero points for its absence. Since there is no method for subtracting points, ‘neutral’ and ‘negative’ performance are equivalent. Thus a building containing products with high volatile organic compound (VOC) emissions, no recycled materials, and ozone depleting refrigerants, may qualify for LEED certification and be perceived as ‘green.’ By contrast, the Japanese system, Comprehensive Assessment System for Building Environmental Efficiency (CASBEE), uses a five-point assessment scale for each category, and a score of level 3 indicates “average” performance. Japanese buildings can therefore ‘lose’ points due to negative performance in any given category. Altogether, the LEED rating system with its narrow perspective of sustainability based on positive scoring plus the failure to account for useful building life undoubtedly constitutes an inadequate measure of true sustainability.

A significant limitation of this study was that the Xanterra houses were not instrumented, and the authors could not make an independent determination about the buildings’ performance. In fact, this raises another limitation of the LEED rating system: buildings are evaluated according to their hypothetical performance (which may be calculated in error, as shown above) rather than their actual performance. If LEED wishes to be taken seriously as an instrument of assessment, rather than a public-relations tool, why not require an actual performance measurement as a prerequisite to certification? In light of the fact that the LEED process already adds significantly to a project’s bottom line (see above), it would seem difficult to argue that cost would be a prohibitive factor. A possible solution is for the projects’ electricity, gas, and water usage to be tabulated and correlated after one year of use.

This research suggests that the process of scoring and reporting within the setting of a collaborative professional environment may have a significant effect on a project’s assessment. Any of the problems encountered here – a faulty third-party energy model, a contractor inexperienced in sustainable construction and its documentation, an intern architect given new responsibilities and perhaps lacking writing ability, and no internal procedure for appealing the decisions of the LEED reviewer – might reasonably be assumed to exist in many other examples of green architecture. Given these contingencies, how accurate is a typical LEED rating? This single example shows a margin of error of at least 26% between the characteristics of the building and its LEED score. Additional research is needed to further explore contingencies of professional practice and their effect on assessments of sustainability. It is likely that some canonical examples of green architecture are assessed and perceived incorrectly.

REFERENCES


Hendrickson, Erik 2006a. Correspondence with the authors. 14 June.

Hendrickson, Erik 2006b. Interview by authors. Gardiner, MT, 13 July.

Hendrickson, Erik 2006c. Correspondence with the authors. 16 October.


McMichael, Carole 2005. LEEDing Yellowstone to a ‘Greener’ future, *Concrete Homes* January.


Williams, Kath 2006. Interview by authors. Bozeman, MT, 13 July.
1 INTRODUCTION

Ultrasound is defined as the sound range to which a human ear cannot respond. The sound range for human hearing is from about 16 Hz to 20 kHz. This limit applies for humans and sound beyond this upper range limit is impossible to hear. Ultrasonic frequency has an upper limit of about 5 MHz for gases and 500 MHz for liquids and solids. Frequency (Hz) is the number of waves that pass a fixed point in unit time, whereas power is related to the intensity of the sound source (W cm\(^{-2}\)). This large frequency range leads to dividing the ultrasonic usage into two areas. The first area is concerned with the effect of a medium on the wave. It involves low amplitude and high frequency propagation. This type of ultrasound is known as low power or high frequency ultrasound. It is also known as diagnostic ultrasound (1–10 MHz). This type is very commonly used in medicine (scanning), physiotherapy and chemical analysis.

The second area is called power ultrasound and involves high energy, low frequency ultrasound. Power ultrasound ranges from 20 kHz–100 kHz. This type of ultrasound is used in cleaning, plastic welding and chemical reactivity.

The power and the production of the chemical effects of ultrasound are provided by a phenomenon known as cavitation. Cavitation is the production of microbubbles in a liquid which are formed when a large negative pressure is applied to the liquid [Mason, and Peters, 2002].

In this project a scientific and rational model for the production of gypsum that was validated on a pilot plant scale. Therefore it was necessary to investigate how the variation in chemical composition will affect the synthetically produced gypsum quality, i.e. analyzing the gypsum produced for some trace metals, such as Pb, Fe, Cu, As, Sb, Zn, Ni, Cd and Na. Most of these metals can cause a serious hazard to humans through inhalation, skin contact, eye contact or ingestion [Rai and Amit, Jan 2002].

Na is not toxic however it can cause efflorescence and Fe causes discolouration of the plasterboard. Synthetic gypsum can be produced by a simple chemical reaction between sulphuric acid (H\(_2\)SO\(_4\)) and lime (Ca(OH)\(_2\)) or limestone (CaCO\(_3\)). The non-ferrous metallurgical industry produces large volumes of acidic effluents, or wastewaters, which are treated using lime neutralization in order to remove the acidity and heavy metals. The success of this research lead to investigating the possibility of making synthetic gypsum during the neutralization of acidic waste waters with lime or limestone. Gypsum samples were produced at different operational conditions (pH, temperature, mixer speed, residence time, etc.). The samples were analysed for trace metal content using flame atomic absorption spectroscopy and inductively coupled plasma spectrometry.

Ultrasound assisted crystallisation during the synthesis of gypsum was performed to observe the possibility of obtaining a cleaner gypsum product. A 40 kHz ultrasonic bath and a 20 kHz ultrasonic probe were used in this investigation. All the experiments performed in this investigation were lasted for 4 hours and were carried out at room temperature. Different ultrasonic powers and different methods of sonication methods were carried out at pH 2 and pH 4 in order to obtain the cleanest gypsum product. Various sonication methods were also carried out using the 20 kHz ultrasonic probe at 25%, 50% and 75% sonication powers.
2002]. It is only at powers above the cavitation threshold that sonochemistry can occur. That is because at only these powers, the great energies associated with cavitational collapse will be released into the liquid.

1.1 Ultrasonic equipment

1.1.1 Ultrasonic bath
Sonication with an ultrasonic cleaning bath is one of the easiest and cheapest methods of sonication. A typical ultrasonic cleaning bath consists of a stainless steel tank with a transducer at the base (see figure 1.1.1). Sonicating a chemical reaction is performed by immersing a glass beaker or vessel containing the reactants in an ultrasonic bath filled with a liquid medium and switching on the power.

1.1.2 Ultrasonic Probe
In order to increase the power of sonication, a probe system was designed. A typical probe system consists of a horn with a detachable tip, a transducer and a generator (see figure 1.1.2). The length of the probe is an important point to take into consideration.

2 GENERAL EXPERIMENTAL PROCEDURE

Before commencing the experiment, the pH meter was calibrated with pH 4 and pH 7 buffer solutions.

Table 2a. Experiments performed in the absence of ultrasound.

<table>
<thead>
<tr>
<th>Experiment number</th>
<th>pH</th>
<th>Lime</th>
<th>Acid</th>
<th>Period/minutes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>analytical</td>
<td>analytical</td>
<td>240</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>commercial</td>
<td>commercial</td>
<td>240</td>
</tr>
<tr>
<td>3</td>
<td>4</td>
<td>commercial</td>
<td>commercial</td>
<td>240</td>
</tr>
</tbody>
</table>

Table 2b. Experiments performed using a 20kHz ultrasonic bath.

<table>
<thead>
<tr>
<th>Experiment number</th>
<th>pH</th>
<th>Power</th>
<th>Sonication method</th>
<th>Period/minutes</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>2</td>
<td>100%</td>
<td>continuous</td>
<td>120</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>100%</td>
<td>continuous</td>
<td>20</td>
</tr>
<tr>
<td>6</td>
<td>4</td>
<td>100%</td>
<td>continuous</td>
<td>15</td>
</tr>
</tbody>
</table>

Table 2c. Experiments performed using a 40kHz ultrasonic probe.

<table>
<thead>
<tr>
<th>Experiment number</th>
<th>pH</th>
<th>Power</th>
<th>Sonication method</th>
<th>period/minutes</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>2</td>
<td>25%</td>
<td>pulsed</td>
<td>240</td>
</tr>
<tr>
<td>8</td>
<td>2</td>
<td>50%</td>
<td>pulsed</td>
<td>240</td>
</tr>
<tr>
<td>9</td>
<td>2</td>
<td>75%</td>
<td>pulsed</td>
<td>240</td>
</tr>
<tr>
<td>10</td>
<td>2</td>
<td>25%</td>
<td>continuous</td>
<td>240</td>
</tr>
<tr>
<td>11</td>
<td>2</td>
<td>25%</td>
<td>pulsed on lime addition</td>
<td>5</td>
</tr>
<tr>
<td>12</td>
<td>2</td>
<td>50%</td>
<td>pulsed on lime addition</td>
<td>5</td>
</tr>
<tr>
<td>13</td>
<td>2</td>
<td>75%</td>
<td>pulsed on lime addition</td>
<td>5</td>
</tr>
<tr>
<td>14</td>
<td>2</td>
<td>50%</td>
<td>continuous to lime</td>
<td>5</td>
</tr>
<tr>
<td>15</td>
<td>4</td>
<td>25%</td>
<td>pulsed</td>
<td>240</td>
</tr>
<tr>
<td>16</td>
<td>4</td>
<td>50%</td>
<td>pulsed</td>
<td>240</td>
</tr>
<tr>
<td>17</td>
<td>4</td>
<td>75%</td>
<td>pulsed</td>
<td>240</td>
</tr>
<tr>
<td>18</td>
<td>4</td>
<td>25%</td>
<td>continuous</td>
<td>240</td>
</tr>
<tr>
<td>19</td>
<td>4</td>
<td>25%</td>
<td>pulsed on lime addition</td>
<td>5</td>
</tr>
<tr>
<td>20</td>
<td>4</td>
<td>50%</td>
<td>pulsed on lime addition</td>
<td>5</td>
</tr>
<tr>
<td>21</td>
<td>4</td>
<td>75%</td>
<td>pulsed on lime addition</td>
<td>5</td>
</tr>
<tr>
<td>22</td>
<td>4</td>
<td>50%</td>
<td>continuous to lime</td>
<td>5</td>
</tr>
</tbody>
</table>

Tables 2a, 2b and 2c illustrate the experiments performed in this research.

2.1 Using the 40 kHz ultrasonic bath
A 5-litre reaction beaker was placed in the ultrasonic bath through a hole in the aluminium cover. The aluminium cover was designed in order to act as a template so that the 5-litre reaction beaker could be located in exactly the same position each time an experiment was performed.

The beaker was filled with 2-litres of sulphuric acid (analytical grade or battery acid). The agitator was fitted
on a metal stand and positioned centrally above the reaction beaker. The mixer blade was immersed in the acid (the distance between the blade and the bottom of the beaker was 1.5–2 cm) and the agitator was turned on at a variable speed (see each experiment). About 5 cm of the thermometer probe was immersed in the acid to obtain an initial temperature reading. The pH probe was also immersed in the acid to obtain the initial pH reading.

2.2 Using the 20 kHz ultrasonic probe

When the probe was used, the author decided to use a smaller reaction beaker (3 L). The 3 L reaction beaker was filled with 1.5 L of the battery acid and placed in a fume cupboard. The ultrasonic probe was securely fitted on a metal stand and positioned over on the top of the reaction beaker (nearer the left edge of the beaker). 25 mm of the tip of the probe was immersed into the acid. The pH and the temperature probes were also immersed in the acid in order to obtain a pH and temperature readings respectively. The sonication time, method, power and pH were varied.

The remainder of the experimental procedure is the same for both sonication methods.

While the pH probe was stabilising, a 500 g L⁻¹ of analytical grade CaCO₃ solution was prepared by the addition of 1 kg of CaCO₃ to 2 L of distilled water into a 5 L glass beaker (CaCO₃ stock solution). A magnetic follower was used and the beaker was placed on a magnetic stirrer. The magnetic stirrer was turned on to provide a homogenous mix of calcium carbonate milk.

The time required to stabilise the pH reading varied from 30–60 minutes. When the reading was stabilised, the initial pH and temperature readings were recorded.

The stopwatch was started when the first 50 ml of the CaCO₃ milk was pipetted from the CaCO₃ stock solution into the (3 or 5) litre reaction beaker. CaCO₃ milk was added to the reaction beaker in different amounts but at regular intervals in order to reach and maintain the pH at approximately 2. All the experiments lasted for 4 hours. In some experiments, samples were taken from the reaction beaker during the experiment at different pH levels in order to investigate the impurity level change at different pH points (vary for each experiment). Each reaction ended when the time reached 4 hours and the agitator plus ultrasonic source was turned off. The final pH and temperature reading were recorded and a final gypsum product was obtained.

Up to six (75 ml) samples were taken from the final gypsum product and placed into 100 ml glass beakers in order to investigate the effect of extra sonication at different powers and times on the final product. The ultrasound was applied using a 20 kHz ultrasonic probe. The temperature of the solution was measured before and after sonication. The samples were “blasted” with continuous ultrasound as follows; a “blast” of 25% continuous ultrasound for 1 minute was applied to the first sample, and a “blast” of 25% continuous ultrasound for five minutes was applied to the second sample. The third sample was “blasted” with 50% ultrasound for 1 minute, and the fourth sample was “blasted” with 50% ultrasound for five minutes. The fifth sample was “blasted” with 75% ultrasound for 1 minute, and the sixth sample was “blasted” with 75% ultrasound for five minutes. The number of samples taken for this investigation varied for each experiment.

The final gypsum pulp was filtered, washed with two separate portions of 100 ml of distilled water. The other samples taken (from the impurity level change & extra sonication investigations) were also filtered and washed with two separate portions of 20 ml distilled water. The final gypsum product and all the samples were dried in a fan-assisted oven at 40°C for 24 hours.

2.3 Experiments performed in the absence of ultrasound

2.3.1 Analytical grade reagents at pH 2

The experimental technique for this experiment follows the general experimental details. 2L analytical grade sulphuric acid and 500 g L⁻¹ analytical grade limestone were used in this experiment. The 40 kHz ultrasonic bath was used but not switched on. The experiment was run at pH 2, and the agitator was set at 900 rpm. No samples were obtained for the impurity change investigation. At the end of the experiment, the gypsum product was washed, filtered, and dried according to the general experimental details section.

2.3.2 Battery acid and commercial grade CaCO₃ at pH 2

The experimental technique for this experiment follows the general experimental details. The 40 kHz ultrasonic bath was used (power switched off), 2 L battery acid and 500 g L⁻¹ commercial grade limestone were used. The experiment was run at pH 2 and the agitator was set at 900 rpm. As in the previous experiment, no further samples were collected for extra investigation. Only one gypsum product was obtained and analysed. At the end of the experiment, the gypsum product was washed, filtered and dried according to the general experimental details section.

2.3.3 Battery acid and commercial grade CaCO₃ at pH 4

The experimental technique for this experiment follows the general experimental details. 1.5 L of battery acid and 500 g L⁻¹ commercial grade limestone were used. The experiment was run at pH 2 and the agitator was set at 756.8 rpm. In this experiment, a 3 L glass reaction beaker was used and the
reaction was carried out without any sonication. In this case the reaction beaker was placed directly on the base of the fume cupboard. No further samples were taken for extra investigation. Only one gypsum product was obtained and analysed.

At the end of this experiment, six (75 ml) samples were taken from the final gypsum product for the “extra sonication” investigation (see general experimental procedure). The final gypsum product and all the samples obtained in this experiment were filtered, washed and dried according to the general experimental details section.

2.4 Experiments performed using a 40 kHz ultrasonic bath

- Battery acid and commercial limestone at pH 2 for 4 hours
- Battery acid and commercial limestone at pH 4 with 15 minutes sonication
- Battery acid and commercial limestone at pH 2 with 20 minutes sonication

The experimental technique for this experiment follows the general experimental details. The experiment was run at pH 4 and the sonication lasted for 20 minutes.

Two 20 ml samples were taken from the reaction beaker during the experiment at the following pH levels; pH 0.98 (at 10 minutes), pH 1.60 (at 118 minutes). At the end of the experiment, the final gypsum product and all the samples obtained in this experiment were filtered, washed and dried according to the general experimental details section.

2.5 Experiments using a 20 kHz ultrasonic probe

In all the following experiments, 1.5 L battery acid and 500 g L⁻¹ of commercial grade limestone were used.

- Pulsed 75% ultrasound at pH 2
  The experimental technique for this experiment follows the general experimental details. This experiment was identical to experiment 4, except that in this experiment 75% pulsed sonication was applied. At the end of the experiment, six (75 ml) samples were taken from the final gypsum pulp and “extra sonication” was applied to them at the following ultrasonic powers 25%, 50%, 75% (see general experimental details).
- Pulsed 75% ultrasound at pH 4
  The experimental technique for this experiment follows the general experimental details. Four 50 ml samples were taken from the reaction beaker during the experiment at the following pH levels; pH 0.93 (at 20 minutes), pH 1.39 (at 45 minutes), pH 2.46 (at 75 minutes), and pH 3.37 (at 150 minutes). At the end of the experiment, six samples from the final gypsum pulp were taken and sonicated at different powers (25%, 50%, and 75%).
- Continuous 25% ultrasound at pH 2
  The experimental technique for this experiment follows the general experimental details (see the ultrasonic probe section). Four 50 ml samples were taken from the reaction beaker during the experiment at the following pH levels; pH 0.93 (at 20 minutes), pH 1.39 (at 45 minutes), pH 2.46 (at 75 minutes), and pH 3.37 (at 150 minutes). At the end of the experiment, six samples from the final gypsum pulp were taken and sonicated at different powers (25%, 50%, and 75%).
- Continuous 25% ultrasound at pH 4
  The experimental technique for this experiment follows the general experimental details (see the ultrasonic probe section). Four 50 ml samples were taken from the reaction beaker during the experiment at the following pH levels; pH 0.93 (at 20 minutes), pH 1.39 (at 45 minutes), pH 2.46 (at 75 minutes), and pH 3.37 (at 150 minutes). At the end of the experiment, six samples from the final gypsum pulp were taken and sonicated at different powers (25%, 50%, and 75%).
- Pulsed 50% ultrasound at pH 2
- Pulsed 50% ultrasound at pH 4
- Pulsed 50% ultrasound for 5 minutes on CaCO₃ addition at pH 2
- Pulsed 50% ultrasound for 5 minutes on CaCO₃ addition at pH 4
- Continuous 50% ultrasound at pH 2
- Pulsed 50% ultrasound for 5 minutes on CaCO₃ addition at pH 4
- Sonicating limestone milk at 50% for five minutes prior to addition at pH 2
- Sonicating limestone milk at 50% for five minutes prior to addition at pH 4.

3 RESULTS AND DISCUSSION

3.1 Experiments in the absence of ultrasound

3.1.1 Analytical grade reagents at pH 2
Discussion:
Since this experiment was carried out using analytical grade sulphuric acid and limestone and performed at pH 2, low impurity levels were expected. However the Pb level seem to be higher than expected which could have risen from cross contamination. As the reagents used in this experiment were all analytical grade and
the acids used in the digestion procedure were also analytical grade, therefore the contamination could have either originated from the reaction beaker or the filtration funnel.

3.1.2 Battery acid and commercial grade CaCO₃ at pH 2

Discussion:
This experiment shows a significant increase in impurity levels in comparison with the previous experiment. This is due to the fact that this experiment was carried out using contaminated battery acid and commercial limestone.

3.1.3 Battery acid and commercial grade CaCO₃ at pH 4

Graph 3.1.3:

Discussion:
The solubility of most metal ions and salts generally decreases as the pH of the solution increases [Qingyum, Louis, McDonald, Jeffrey and Skousen]. In other words they become less soluble at higher pH levels. Since experiment 3.1.3 was performed at pH 4, higher impurity levels were expected in the product. Graph 3.1.3 show that the lead levels in experiment 3.1.3, which was performed at pH 4, are lower than the lead levels in experiment 3.1.2 which was performed at pH 2. Since that the most dominant form of Pb is likely to be PbSO₄, the solubility of PbSO₄ increases as the concentration of the sulphuric acid decreases [Blaskett and Boxall, 1990]. In other words as the pH of the solution rises, the solubility of PbSO₄ increases leaving less to be co-precipitated in the gypsum product.

3.2 Experiments using a 40 kHz ultrasonic bath

3.2.1 Battery acid and commercial limestone at pH 2 with 20 minutes sonication

Graph 3.2.1:

Discussion:
This experiment was carried out using a 40 kHz ultrasonic bath. The sonication commenced when the first 50 ml of CaCO₃ was added to the reaction and stopped after 20 minutes. The remainder time of the reaction was carried out without sonication and the experiment lasted for four hours. The final pH of the reaction was approximately pH 2. When the final product of this experiment was compared with the final product from experiment 3.1.2 (no US), it was found that there was a decrease in impurity levels when ultrasound was applied for 20 minutes (see table and graph 3.2.1). The product from this experiment provided the cleanest gypsum produced in this investigation.

3.3 Experiments using a 20 kHz ultrasonic probe

3.3.1 Pulsed 75% ultrasound at pH 2

Graph 3.3.1:

Discussion:
In this experiment, a 20 kHz probe was used for the sonication. The reaction was sonicated with 75%
pulsed power throughout the duration of the experiment (4 hours). The final pH of the experiment was approximately pH 2. When the final product of this reaction was compared with the product from experiment 3.1.2, which was carried out without any sonication, it was found that there was a significant reduction in impurity levels with the exception of Pb, Sb and Fe. As previously discussed, sonication can cause particle size to decrease by fragmenting the crystals [Perez-Rodriguez, Carrera, Poyato, Perez-Maguedu, 2002] [Franco, Perez-Mqueda, Perez-Rodriguez, 2004]. The fragmentation and the breakage of the crystals could cause the impurities to go into solution rather than precipitating on the surface of the crystals. Since the sonication was pulsed, the crystals could be continuously breaking which would allow minimum coprecipitation of impurities on the surface or even as part of the crystal matrix.

On the other hand, Pb and Sb levels increased when ultrasound was used in the reaction.

As previously mentioned the free radicals formed in the course of sonication could have caused the oxidation of Pb$^{+2}$ to Pb$^{+4}$, consequently forming PbO$_4$, which is insoluble in water.

Sb$_2$(SO$_4$)$_3$ is insoluble in water but soluble in acid [West, Robert 1972–1973]. Therefore as the experiment continues and the acid concentration decreases, the level of Sb increases due to the precipitation of Sb$_2$(SO$_4$)$_3$.

As mentioned earlier, ultrasound causes the crystals to break, which leads to larger surface area [Chatakondu, Green, Thompson, Suslick, 1987] [Perez-Rodriguez, Carrera, Poyato, Perez-Maguedu, 2002]. Therefore as the surface area is increasing it is possible that Sb$_2$(SO$_4$)$_3$ is precipitating on the surface of the crystals. All the other metal sulphates included in the analysis are soluble in water [West, Robert. 1972–1973]. Another factor worth mentioning is that the temperature of the solution also increases when ultrasound is used [Suslick, Kenneth, 1989] [Mason and Lorrimer, 1988] consequently leading to better dissolution. As for Fe, it can be oxidised to Fe$_2$O$_3$, which is insoluble in water [West, Robert. 1972–1973].

### Discussion:

In this experiment, a 20 kHz probe was used for the sonication. The reaction was sonicated with a 75% pulsed power throughout the duration of the experiment (4 hours). The final pH of the experiment was approximately pH 4. When the final product of this reaction was compared with the product of experiment 3.1.3, which was carried out without any sonication, it was found that there was a significant reduction in impurity levels with the exception of Pb, Sb and Fe which mirrored results found at pH 2. The discussion given in experiment 3.3.1 is also valid for this experiment as the metal ions in both experiments are behaving in the same manner.

### 3.3.2 Pulsed 75% ultrasound at pH 4

**Graph 3.3.2:**

A comparison between normalized impurity contents in experiment 3.3.2 (with US) and experiment 3.1.3 (without US) both carried out at pH 4

### 3.3.3 Continuous 25% ultrasound at pH 2

**Graph 3.3.3:**

A comparison between normalized impurity contents in experiment 3.3.3 (with US) and experiment 3.1.2 (without US) both carried out at pH 4

**Discussion:**

In this experiment a 20 kHz ultrasonic probe was used for the sonication. Continuous 25% sonication was applied to the reaction for the entire duration of the experiment (4 hours). The pH of the solution
was approximately pH 2. When the product of this experiment was compared to the product of experiment 3.1.2, which was carried out without any sonication, it was found that there was a significant decrease in impurity levels with the exception of Pb, Sb and Fe. Since the sonication was continuous, there was continuous “damage” being applied to the surface of the crystals, which caused any clusters to deagglomerate and fragment releasing impurities trapped within or on the surface of the crystals.

As previously mentioned the free radicals formed in the course of sonication could have caused the oxidation of Pb$^{2+}$ to Pb$^{4+}$, consequently forming Pb$_2$O$_4$, which is insoluble in water.

Sb$_2$(SO$_4$)$_3$ is insoluble in water but soluble in acid, therefore as the experiment continues and the acid strength decreases, the level of Sb increases due to the precipitation of Sb$_2$(SO$_4$)$_3$.

In addition the temperature increase of the solution due to sonication also helps dissolution in general. As for Fe, it could have oxidised to Fe$_2$O$_3$, which is insoluble in water.

3.3.4 Continuous 25% ultrasound at pH 4

Graph 3.3.4:

Discussion:

In this experiment a 20 kHz ultrasonic probe was used for the sonication. Continuous 25% sonication was applied to the reaction for the entire duration of the experiment (4 hours). The pH of the solution was approximately pH 4. When the product of this experiment was compared to the product of experiment 3.1.3, which was carried out without any sonication, it was found that there was a significant decrease in impurity levels with the exception of Pb, Sb and Fe mirroring the findings at pH 2. Since the sonication was continuous, there was continuous damage being applied to the surface of the crystals, which caused any clusters to disintegrate and fragment releasing impurities trapped within or on the surface of the crystal.

The impurities in this experiment are behaving in the same manner as the previous experiment.

4 CONCLUSION

Refer to tables 2a, 2b and 2c for the following experiment numbers. Some of the results of the experiments discussed are not included in the paper due to the length of the paper, however they can be found in reference [Abdul-Jabber, 2005].

All the gypsum samples produced in this investigation are not suitable for manufacturing plasterboard since that the levels of the trace metals analysed remain elevated in all the gypsum samples and exceed the maximum limits provided by Lafarge plasterboard. The only suitable gypsum product obtained from this investigation was the gypsum produced in experiment 1 (pH 2), which used analytical grade reagents.

This investigation showed that sonication could either increase or decrease impurities in the gypsum product. This depends on the source of sonication, the method of sonication, power, frequency, length of sonication. However this effect varies from one metal ion to another. For example the levels of Pb is always increased when ultrasound was introduced, this is due to the solubility of PbSO$_4$ in sulphuric acid as discussed before.

Experiments 2 and 3 which were carried out without any sonication and used commercial grade limestone (provided by Lafarge plasterboard) and battery acid (provided by BRM), carried out at pH 2 and pH 4 respectively. The gypsum produced in both of these experiments were not suitable for plasterboard manufacturing since most of the elements analysed exceeded the maximum limits with the exception of Sb and Fe in experiment 2 (pH 2) and Pb in experiment 3 (pH 4).

The gypsum produced in experiment 2 (pH 2) has Pb content that is higher than experiment 3 (pH 4). Since the most dominant form of Pb in gypsum is likely to be PbSO$_4$, the solubility of PbSO$_4$ increases as the concentration of the sulphuric acid decreases [Blaskett and Boxall, 1990]. In other words as the pH of the solution rises, the solubility of PbSO$_4$ increases leaving less to be coprecipitated in the gypsum product.

When (pH 2, 4 hours US, using the bath) experiments were compared with experiment 2 (no US, pH 2), it was observed that ultrasound had a positive effect on the gypsum produced, since the levels of Cd, Cu, Na, Fe and Zn were decreased after the sonication with the exception of Pb and Sb. However the gypsum obtained from the sonicated experiment had impurity levels which exceeded the maximum levels for FGD.

(pH 4, 15 minutes US, using the bath) experiment had lower Pb, Cd, Na, Fe and Zn levels when compared with experiment 3 (pH 4, no US) with the exception of Cu and Sb. However the final product obtained from the sonicated experiment did not meet the requirements for plasterboard manufacturing with the exception of Zn (13.9 ppm) since the maximum limit is 50 ppm.
When (20 minutes US) experiment, (4 hours US) experiment and experiment 2 (no US) were compared, it was observed that the lowest impurity levels reported were for 20 minutes sonication at the beginning of the experiment. Some of these levels such as Cu, Sb, Fe and Zn were within specification whereas Pb, Cd and Na were higher than the specified levels.

Using the probe, the following experiments were compared; (pulsed 25% US) experiment and experiment 2 (no US), both experiments were carried out at pH 2. It was observed that ultrasound has increased the impurity content in the gypsum produced. This is due to the power used in this experiment was only 3.4705 \times 10^{-3} \text{ kW}. Perhaps the power was not sufficient to break the crystals but instead “pitted” the crystals, consequently the impurities could have penetrated into the crystal and become part of the crystal structure which has led to higher levels of impurities in the product. Therefore this sonication method is not recommended and the metal ions analysed in the final product exceed the maximum limits specified for plasterboard.

When experiment 15 was compared to experiment 3 which was carried out without any sonication, it was found that there was a significant increase in impurities when ultrasound was used perhaps due to the low power used as mentioned earlier.

When experiment 8 (pulsed 50% US) and experiment 2 (no US) were compared, it was observed that there was a significant increase in impurities when ultrasound was introduced with the exception of Cd and Zn.

Experiment 16 was identical to experiment 8, except that it was carried out at pH 4.

When experiment 16 was compared to experiment 3 which was carried out without any sonication, it was found that there was an increase in the levels of Pb, Sb, Na and Fe with a slight reduction in the levels of Cd and a significant reduction in the levels of Cu and Zn.

When experiment 9 (pulsed 75% US) with experiment 2 (no US), it was found that there was a general significant decrease in impurity levels with the exception of Pb and a slight increase in the levels of Sb when sonication was used. This is due to the solubility of Pb and Sb as previously mentioned in this chapter. However the final product is not suitable for plasterboard manufacturing since only Sb, Fe and Zn levels are within specification.

Experiment 17 was identical to experiment 9, except that it was carried out at pH 4.

When experiment 17 was compared to experiment 3 which was carried out without any sonication, it was found that there was a general significant decrease in impurity levels with the exception of Pb, Sb and Fe. The final product is not suitable for plasterboard.

When experiments 11 (Pulsed 25% US for 5 minutes on lime addition) and experiment 2 (no US) were compared, it was observed that there was a slight decrease in the levels of some metal ions such as Cd, Cu, Zn with a significant reduction in the levels of Na. Whereas there was an increase in the levels of some metal ions such as Pb, Sb and Fe.

The final product is not suitable for plasterboard manufacturing.

Experiment 19 was identical to experiment 11, except that it was carried out at pH 4.

When experiment 19 was compared to experiment 3 which was carried out without any sonication, it was found that there was a slight decrease in the levels of Cd and a significant decrease in the levels of Cu, Na and Zn when sonication was used. The levels of Pb, Sb and Fe were increased significantly after the sonication.

When experiment 12 (pulsed 50% US for 5 minutes on lime addition) was compared to experiment 2 (no US), it was found that there was an overall increase in impurity levels with the exception of Na when ultrasound was used. The final product is not suitable for plasterboard manufacturing.

Experiment 20 was identical to experiment 12, except that it was carried out at pH 4.

When experiment 20 was compared to experiment 3 which was carried out without any sonication, it was found that there was an overall increase in impurity levels with the exception of Na. The final product is not suitable for plasterboard manufacturing.

When experiment 13 (pulsed 75% US for 5 minutes on lime addition) and experiment 2 (no US), were compared, it was found that there was a significant reduction in the levels of Cd, Cu, Na and Zn when ultrasound was used, whereas there was an increase in the levels of Pb, Sb and Fe after the sonication. The levels of Cu, Sb, Fe and Zn are within specification and the levels of Pb, Cd and Na are outside the specified limits for manufacturing plasterboard.

Experiment 21 was identical to experiment 13, except that it was carried out at pH 4.

When experiment 21 was compared to experiment 3 which was carried out without any sonication, it was found that there was a significant decrease in the levels of Cd, Cu, Na and Zn and an increase in the levels of Pb, Sb and Fe when ultrasound was used. However the final sonicated products not suitable for making plasterboard because all the metal ions analysed exceed the maximum limits with the exception of Zn.

When experiment 14 (sonicating CaCO3 for 5 minutes before addition) and experiment 2 (no US) was compared, it was found that there was a significant reduction in the levels of Cu, Sb, Na, Zn and a small reduction in the levels of Cd when ultrasound was introduced. The levels of Pb and Fe were increased after the sonication. Despite the low levels of Cu, Sb, Fe and Zn obtained after the sonication, the final product remains unsuitable for manufacturing plasterboard due to the high levels of Pb, Cd and Na.
Experiment 22 was identical to experiment 14, except that it was carried out at pH 4.

When experiment 22 was compared to experiment 3 which was carried out without any sonication, it was found that there was a small decrease in the levels of Cu, Sb, Na and Fe, however this reduction in the impurity levels is not very significant if experimental errors were taken into consideration. The levels of Pb and Zn were significantly increased and Cd was slightly increased after the sonication and the final product is not suitable for plasterboard manufacturing.

When experiment 10 (continuous 25% US) was compared with experiment 2 (no US), it was found that there was a significant decrease in the levels of Cd, Cu, Na and Zn whereas the levels of Pb and Fe were slightly increased and the levels of Sb was increased significantly after the sonication. Despite the low levels of Cu, Fe and Zn, the final product remains unsuitable for making plasterboard due to the elevated levels of Pb, Cd, Sb and Na.

Experiment 18 was identical to experiment 10, except that it was carried out at pH 4.

When experiment 18 was compared to experiment 3 which was carried out without any sonication, it was found that there was a decrease in the levels of Cd, Cu, Na and Zn after the sonication, whereas the levels of Pb, Sb and Fe were slightly increased when ultrasound was used. The final product is not suitable for manufacturing plasterboard.

The author recognises experiment 5 (20 minutes US/bath) as the best sonication method that gives the lowest impurity content. Experiments 9 (pulsed 75% US), 13 (75% US on lime addition) and experiment 10 (continuous 25% US) as the next best experiments that gives low impurity content results at pH 2. However non of the products produced in these experiments are suitable for manufacturing plasterboard as they have elevated levels of unwanted metal ions even when sonication was proved to be effective.

REFERENCES


Qingyum, S., Louis, M., McDonald, Jr. and Jeffrey, G., Skousen. Effects of armouring on limestone neutralisation of AMD. Division of plant and soil sciences, West Virginia university, Morgantown.


MANUFACTURED AGGREGATE FROM WASTE MATERIALS

R. Lupo, M. Tyrer, C.R. Cheeseman & S. Donatello
Centre for Environmental Control and Waste Management, Imperial College, London

ABSTRACT: The move from natural to synthetic aggregates for use in construction is driven both by mounting legislation and taxation intended to reduce the growing demand for natural aggregates. Despite increasing re-use of crushed concrete from demolition waste, this source alone cannot satisfy the demand for aggregate materials. This paper reports work using cement-bound mineral processing fines, which have been processed to form a synthetic aggregate material. High-shear processing has been used (in combination with polymer additives) to optimise the highest strength with the lowest practical water content, allowing aggregates to be produced by extrusion and kibbling. Using polyvinyl alcohol at loadings up to 3% by mass, produces aggregates with a compressive strength around 25 MPa comparable to many commercial concrete products which use limestone aggregates.

1 INTRODUCTION

The aggregate levy introduced by the UK government has had a beneficial impact on the aggregates market. It is now financially increasingly more attractive for some companies to wash and screen excavated soils from both brown- and greenfield sites, to produce coarse and fine aggregates. This has had the dual favourable effect of first reducing the need for quarries for virgin mineral extraction, and secondly reducing the dependence on landfill for this construction and demolition waste stream. Approximately 800,000 tonnes of aggregate a year are produced this way in the UK.

The design of the process ensures that particles with nominal size of greater than 73 µm are separated and sold for aggregate. The wash water is cleaned and reused but the cleaning process (flocculation, settlement and filtering) generates waste filter cake which is sent to landfill. As the filter cake contains particles less than 73 µm it is collectively classed as silt by the minerals industries, but also includes the clay-sized fractions. The silt also presents an environmental hazard as accidental spills into river courses result in large fish kills [Lester 1999]. In addition, the cost of transporting and disposing of this waste is a threat to the financial viability of the aggregate recycling plants and remains a barrier to the introduction of others.

At a time when landfill costs are increasing and land availability is decreasing, it is increasingly important to view waste streams as a resource, rather than liability which be disposed of with little regard to the social, environmental and economic consequences.

The current work concentrates on characterising the filtrates from aggregate washing and reports preliminary trials on conversion of these filter cakes to a commercially viable aggregate.

Methods of binding other “silty” materials commonly involve sintering at high temperature and produce useful materials e.g. bricks from reservoir sediment [Huang et al. 2001] amongst others. The main disadvantage of sintering technology is the energy costs associated with the process along with the greenhouse gasses released in its generation.

Ordinary Portland Cement (OPC) has been used as a binder for clay to form cold-cured bricks [Temimi et al. 1995], with soils, again to make bricks in developing countries [Khedari 2005] and is common practice for producing sub-base for road building. OPC has however been used specifically with waste silts for producing aggregate.

This work seeks to identify a route by which the fine filtrates recovered during aggregate washing may be cold-bonded, principally using cements with polymer additions to modify the properties of the materials where appropriate. Water-dispersed or soluble polymers are expected to allow minimal quantities of mix water to be used in these formulations; ideally little more than is stoichiometrically necessary for cement hydration. In this way, high strengths are expected after hydration, whilst maintaining sufficient workability of the green pastes to allow high shear mixing, kibbling and pelletising. The objective is to develop low-cost and low energy materials, which will allow these mineral fines to be economically re-used. This is likely to
be of greatest commercial impact to the silt producing companies, as it will divert their solid wastes from landfill back into a buoyant market.

2 MATERIALS AND METHODS

2.1 Characterisation

Samples from five aggregate recycling plants around the UK were visited to assess quantities of silt produced and to collect samples.

Water content was measured by drying at 105°C for 24 hours and calculating weight difference. The dried samples were then subject to TGA, XRD, particle size analysis, nitrogen absorption (for surface area measurement), pH measurement (on dispersal in water) and optical microscopy.

In addition samples were reacted with a solution of barium chloride to test for the presence of sulphate, which can lead to sulphate attack in concrete. The samples were also tested with silver nitrate solution to test for the presence of chlorides, which may present a corrosion problem if present in structural concrete with steel reinforcement.

2.2 Binder systems

Mixing was carried out in a Hobart N50 mixer which was sufficiently powerful to blend the binders and filter cake (the latter has a similar consistency to potters clay). Water was added on an empirical basis, until a consistency suitable for extrusion was reached. Each batch weighed 300 g including water content already in the silt, prior to any further water additions.

For trials with OPC and PVA, table 1 shows the compositions used. The PVA used was supplied by Nippon Gohsei, grade GH17S. Dry sand was added in the mass proportion of 23:57 sand to wet silt because it decreases the water to cement ratio (owing to its own water demand) and to reduce sticking of the mixture to processing equipment. Additional trials were conducted using fast-setting high alumina cement and calcium chloride used both as a setting accelerator and a flocculant for clay dispersions. The compositions for these trials are shown in table 2.

Once mixed, each batch was separated into two parts. From one part compression test samples (cylinders) of 20 mm diameter and approximately 20 mm high were pressed into moulds. The other part of the batch was pushed through a 6.25 mm sieve and the kibbled particles processed through a laboratory pan pelletiser in order to shape into rounded pellets.

Curing took place at room temperature and humidity and compression test samples were taken at 14 day and 28 day cures.

<table>
<thead>
<tr>
<th>Wet silt</th>
<th>Dry sand</th>
<th>OPC</th>
<th>PVA</th>
<th>Total water content</th>
</tr>
</thead>
<tbody>
<tr>
<td>68</td>
<td>27</td>
<td>5</td>
<td>0</td>
<td>23.9</td>
</tr>
<tr>
<td>61</td>
<td>24</td>
<td>15</td>
<td>0</td>
<td>24.8</td>
</tr>
<tr>
<td>57</td>
<td>23</td>
<td>20</td>
<td>0</td>
<td>23.9</td>
</tr>
<tr>
<td>50</td>
<td>20</td>
<td>30</td>
<td>0</td>
<td>24.8</td>
</tr>
<tr>
<td>36</td>
<td>14</td>
<td>50</td>
<td>0</td>
<td>22.2</td>
</tr>
<tr>
<td>57</td>
<td>23</td>
<td>19.5</td>
<td>0.5</td>
<td>24.8</td>
</tr>
<tr>
<td>57</td>
<td>23</td>
<td>19</td>
<td>1</td>
<td>20.3</td>
</tr>
<tr>
<td>57</td>
<td>23</td>
<td>18</td>
<td>2</td>
<td>22.0</td>
</tr>
<tr>
<td>57</td>
<td>23</td>
<td>17</td>
<td>3</td>
<td>19.8</td>
</tr>
</tbody>
</table>

Table 1. Compositions used for OPC and PVA mixing trials: percent by mass. (Note water additions made on an empirical basis to achieve consistency suitable for kibbling).

<table>
<thead>
<tr>
<th>Wet silt</th>
<th>Dry sand</th>
<th>Cement</th>
<th>PVA</th>
<th>CaCl₂</th>
<th>Total water content</th>
</tr>
</thead>
<tbody>
<tr>
<td>67</td>
<td>28</td>
<td>5 HAC</td>
<td>0</td>
<td>0</td>
<td>23.5</td>
</tr>
<tr>
<td>57</td>
<td>23</td>
<td>20 HAC</td>
<td>0</td>
<td>0</td>
<td>28.8</td>
</tr>
<tr>
<td>57</td>
<td>23</td>
<td>15 OPC</td>
<td>2</td>
<td>3</td>
<td>21.0</td>
</tr>
</tbody>
</table>

Table 2. Compositions used for mixing trials with HAC and calcium chloride: Percent by mass. (Note water additions made on an empirical basis to achieve consistency suitable for kibbling).

2.3 Aggregate testing

Standard aggregate testing requires “resistance to fragmentation” type tests, which, by various standard methods, crush an aggregate sample. These tests give a value for the percentage of fines generated from a standard compaction. The Los Angeles Coefficient and Aggregate Crushing Value (ACV) are two such examples [British Standards 1998 and 1990a]. A further approach is the 10% Fines test [British Standards 1990b] in which the load required to generate 10% fines is reported. In all cases, the results are not given in MPa and hence could not be compared directly to the results obtained from this work. In addition the testing does not give a value which can be readily used for engineering design purposes. For example, a “20% fines” value would not enable a concrete mix designer to specify material suitable for a particular application. Rather, it provides an indication of the aggregates’ “crushability” in the un-bound state and their tendency to break into smaller particles.

Preliminary experimental work was carried out by applying loads specified in the ACV test in British Standard BS 812-110:1990 [British Standards, 1990a] (100 kN applied to aggregate contained in a 75 mm diameter vessel). It was found that aggregate with low...
OPC contents broke down and re-consolidated into one mass with the fines sticking together. This meant that once the crushed sample was sieved, fewer fines passed the fine sieve giving the deceptive impression that the aggregate had a low ACV and hence high strength.

For these reasons it was felt that it would be necessary to test the aggregate in a way which would be more akin to the loads it would encounter in service. Typically, compaction of backfill would generate loads of up to 0.7 MPa [Chen, 2000] and subsequently a building would generate approximately 0.2 MPa. Loads experienced by road materials are often far smaller. Therefore, the standard ACV test apparatus was chosen, but the load applied was 2 MPa. Moreover, it was felt appropriate that both the degree of compression under load, as well as the quantity of fines generated should be measured. If the quantity of fines exceeded the space between aggregate particles, effects such as pore pressure would markedly change the character of the aggregate.

3. RESULTS

3.1 Quantity

Four plants, located throughout the UK, wash excavated soils brought in from a variety of construction and demolition (C&D) sites in their respective area. A fifth plant, washed soils excavated from one site only and is co-located at that site. The excavated soils may or may not contain varying levels of crushed concrete and/or natural stone, depending on the site from which they were excavated. Together, the plants generate a combined total of approximately 1000 tonnes of silt per day, which is predominantly sent to landfill sites as waste.

3.2 Characteristics

Details of the characteristics of the silts gathered from the sites are shown in table 3, below.

The factors which influence the water content of the filtrate were both the degree of fines input into the process and the cycle time of the filter press.

The samples were subjected to instrumental analysis, chemical spot tests and optical microscopy as a matter of course. Thermal analysis showed a slight weight loss at 450°C which gave an indication of some organic matter in the silt.

X-ray diffraction showed high levels of quartz and clay particles. Particle size analysis confirmed the presence of high levels of fine particles including clay. A typical distribution of particle sizes is shown in figure 1.

No soluble chlorides were detected (soluble chlorides could have a deleterious effect on reinforcing steel in concrete structures). Similarly, no soluble sulphates were detected and the equilibrium pH of the samples in water varied between 7 and 8.8. These results indicate that there are no readily soluble salts present that could be deleterious to concrete containing aggregate manufactured from this material.

The surface analysis gave results lower than typical clay results, most likely due to the fact that there were larger particles present. Optical microscopy confirmed the results shown above, particularly the presence of quartz and clay particles and identified occasional opaque magnetic particles.

3.3 OPC

The results of unconfined compressive stress (UCS) testing are shown in figure 2. It is clear that at higher levels of binder addition the materials are stronger, as would be expected. Also, at higher cement levels there are correspondingly lower wet silt contents. Therefore the water to cement ratio is also higher than for lower cement ratios which may contribute to the increased strength as well as the increased level of binder.

3.4 OPC PVA

Modest polymer additions (strictly replacement of OPC by PVA) induce a marked change in properties of the fresh mortar samples.

The variations in unconfined compressive strength are shown in fig 3. Again it is clear that at higher levels of PVA addition the materials are stronger. Even the replacement of 0.5% OPC with PVA gave a marked increase in strength. This could be attributed to three factors. First, PVA is an adhesive and would therefore act to bind individual particles together. Secondly, the PVA absorbs some of the water from the mixture which reduces the water to cement ratio and hence increases the strength of the material. Thirdly, PVA is a rheological aid, which would allow particles to compact closer together during sample processing, resulting in higher mechanical binding.

The low result at 2% PVA is most likely due to excess water addition during mixing.
3.5 HAC and calcium chloride addition

The UCS results for HAC are slightly higher than those for OPC but the higher cost of HAC would probably not justify its use in this application.

There was no appreciable strength increase as a function of curing time and this is to be expected of HAC as it achieves its maximum strength more rapidly than does OPC.

The addition replacement of 3% OPC with calcium chloride gave a comparable result to 18% OPC and 2% PVA. Whilst this appears to be a promising result, the higher cost of calcium chloride in comparison with OPC limits its application, even on this modest scale. For massive concrete applications, calcium chloride accelerators are acceptable, but are avoided where they may induce corrosion in structural steel.

3.6 Aggregate testing

The results of aggregate compression testing are shown in Figure 4. It can be seen that a typical lightweight aggregate compresses markedly under typical service loads and that the fines generated are also at a relatively high level.

Figure 1. A typical distribution of particle sizes in filter cake silt from aggregate washing plants. Particle sizes <2 µm are normally considered as clay particles for soil classification purposes.

Figure 2. Variation of Unconfined Compressive Strength with cement content in silt, sand and cement mixtures.

Figure 3. Variation of Unconfined Compressive Strength with cement content in silt, sand, PVA and cement mixtures.
In this case, the control aggregate was a sample obtained from one of the aggregate recycling plants. It had a high natural stone content (quartz gravel) but particles of crushed brick, concrete, glass and roadstone were also present. The stress-strain relationship of the control is very similar to the synthetic aggregate containing 2% PVA, 3%CaCl₂, 15% OPC and mineral washing fines. Whilst this is economically marginal, it demonstrates that suitable aggregates can be manufactured at costs approaching those required by the market.

4. DISCUSSION AND CONCLUSIONS

We must, of course, consider the major use of industrial aggregates; concrete production. British Standard BS EN 12620:2002 (Annex E) states that aggregate normally has a strength greater than the concrete itself [British Standards 2002]. Indeed, in high strength applications, the compressive load is carried largely by the aggregate, through particle-particle contact and the role of the cementitious binder phase is to limit rotation or slipping of the aggregate particles. In non-structural concretes, this may not necessarily be the case; often both the aggregate and the concrete made from it are far stronger than is necessary, (or in some cases, even desirable!). Nonetheless, it would seem reasonable to assume that concrete made from these manufactured aggregates will have a reduced strength compared to structural concrete. Consequently, it is expected that aggregates of this type should be used principally in non-structural applications, such as blinding concrete, foundation engineering and mass pours.

It can be concluded that it is feasible to manufacture aggregates suitable for selected backfill and low strength concrete applications. Increasing OPC and PVA additions increase the strength of the aggregates but also the cost.

Further work is necessary to test the aggregates and the concretes made from them under different service environments. This lengthy process will be necessary to demonstrate the physical and chemical stability of the materials in the long term.

REFERENCES


Evaluation and minimization of life cycle environmental risk of concrete structures

T. Noguchi & S. Fujimoto
The University of Tokyo, Tokyo, Japan

ABSTRACT: Concrete is the second most widely consumed substance on Earth, after water. Total amount of concrete produced in 2000 in Japan was approximately 500 million ton, which amounted to almost 50% of resource consumption in construction industry and to 25% of total input resources in Japan. Various environmental burdens are produced in cement and concrete production, execution and demolition of concrete structures, recycling and disposal of concrete waste, etc. and these pose potential hazards for the environment and may result in monetary damages to industries. To establish sustainable concrete related industries, it is absolutely imperative to minimize the total environmental risk. This paper proposes a simulation system for resource flow based on multi-agent-system which contributes to investigate an optimum material flow of concrete materials, an optimum corporate strategy and an optimum political measure to minimize the total environmental risk.

1 PRODUCTION OF CONCRETE RELATED MATERIALS AND CONCRETE IN JAPAN

1.1 Total input and output of concrete

According to the Ministry of the Environment, Japan’s total material input in 1999 and 2000 amounted to 2 billion to 2.2 billion tons [Ministry of the Environment 2003]. In view of the fact that Japan’s concrete production in 2000 approximately totaled 500 million tons, concrete accounts for 25% of Japan’s total material input. On the other hand, Japan’s waste totaled 458 million tons (general waste: 52 million tons; industrial waste: 406 million tons) in 2000, of which concrete lumps discharged from demolition sites accounted for 35 million tons [Ministry of Land, Infrastructure and Transport 2002].

Concrete thus accounts for large parts of both the material input and discharged waste, making the recycling of demolished concrete lumps a major issue in Japan, where the final disposal sites for industrial waste have been nearly filled up in recent years.

1.2 Production of cement and additives

The materials of portland cement include limestone, clay, silica stone, iron oxide materials, and gypsum, most being domestically available. Approximately 85% of the fuel for producing cement is coal imported from countries including China, Australia, Russia, and Indonesia, while 10% and 3% are petroleum coke and industrial waste fuel, respectively. Heavy oil accounts for only 1% of the fuel. Japan’s production technology achieved the highest level of energy efficiency thanks to the energy conservation measures taken during the period between 1970 and 1990. Figure 1 shows the changes in the cement production by type over time [Japan Cement Association 2002].

The cement production increased as the construction projects for buildings and infrastructures swelled during the high economic growth period, hitting an all-time peak of 99 million tons in 1996. Amid the decreasing tendency of cement production thereafter
associated with the reductions in construction projects, the ratio of blended cement production has been gradually increasing. Its increasing trend is expected to continue in view of the fact that the expected increase in the number of thermal power plants will increase the amount of resultant coal ash; that the crude steel production tends to increase affected by the construction rush in economically buoyant China; and that blast-furnace slag cement and fly ash cement were designated as green procurement products by the Law on Promoting Green Purchasing in 2001.

Industrial waste and byproducts including slag and coal ash have long been utilized as cement materials, as these contain significant amounts of major ingredients of cement. Combustible waste including waste tires and waste plastics has also been used as fuels for cement production. Recently, sewage sludge and incineration ash from municipal solid waste have been actively utilized as raw materials and fuel for cement, because (1) the high temperature safely decomposes hazardous materials including dioxins; (2) no new waste is generated; (3) this requires no additional incineration facilities; and (4) it extends the service lives of final disposal sites. Table 1 shows the waste and byproducts utilized by the cement industry in 2005 [Japan Cement Association 2006].

### Table 1. Waste and byproducts used in cement industry in 2005.

<table>
<thead>
<tr>
<th>Type</th>
<th>Uses</th>
<th>Amount (1,000 t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>blast furnace</td>
<td>raw material, addition</td>
<td>9,200</td>
</tr>
<tr>
<td>slag</td>
<td></td>
<td></td>
</tr>
<tr>
<td>coal ashes</td>
<td>raw material, addition</td>
<td>7,200</td>
</tr>
<tr>
<td>by-product</td>
<td></td>
<td></td>
</tr>
<tr>
<td>gypsum</td>
<td>addition</td>
<td>2,700</td>
</tr>
<tr>
<td>dirt, sludge</td>
<td>raw material</td>
<td>2,500</td>
</tr>
<tr>
<td>nonferrous slag</td>
<td>raw material</td>
<td>1,300</td>
</tr>
<tr>
<td>steelmaking slag</td>
<td>raw material</td>
<td>500</td>
</tr>
<tr>
<td>coal tailing</td>
<td>raw material, fuel</td>
<td>300</td>
</tr>
<tr>
<td>others</td>
<td>–</td>
<td>5,900</td>
</tr>
<tr>
<td>total</td>
<td>–</td>
<td>29,600</td>
</tr>
</tbody>
</table>

1.3 Production of aggregate

Similarly to cement, the production of aggregate soared during the period of high economic growth when the construction projects for buildings and infrastructures increased, recording an all-time high of 949 million tons in 1990 as shown in Figure 2 [Japan Crushed Stone Association 2004]. In recent years, crushed stone and crushed/pit sand have become major coarse and fine aggregates, respectively, as good quality river sand and river gravel have become depleted. The use of sea gravel and sea sand for concrete began in the mid-1960s particularly in western Japan and the extensive use of these aggregates caused exposure of bedrock on the bottom of the Seto Inland Sea, while causing the erosion of embankments and endangering sand spits and fishing grounds. The deterioration of the coastal environment was so serious that an absolute ban on the collection of sea gravel and sea sand from the Seto Inland Sea is being implemented. Regulations on crushed stone collection have also been tightened from the aspect of natural landscape protection. With this as a

![Figure 2. Change in aggregate production.](image-url)
Table 2. Various types of aggregates made from industrial waste.

<table>
<thead>
<tr>
<th>Type</th>
<th>Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td>blast furnace slag aggregate</td>
<td>JIS A 5011-1</td>
</tr>
<tr>
<td>ferronickel slag aggregate</td>
<td>JIS A 5011-2</td>
</tr>
<tr>
<td>copper slag aggregate</td>
<td>JIS A 5011-3</td>
</tr>
<tr>
<td>electric arc furnace oxidizing slag aggregate</td>
<td>JIS A 5011-4</td>
</tr>
<tr>
<td>melt-solidified slag aggregate for concrete derived from municipal solid waste and sewage sludge</td>
<td>JIS A 5031</td>
</tr>
<tr>
<td>fly ash calcined lightweight aggregate</td>
<td></td>
</tr>
</tbody>
</table>

background, ready-mixed concrete (RMC) plants in the metropolitan coastal area have been receiving not only crushed limestone from western Japan but also river sand from China including Taiwan in large amounts, giving rise to wide-area transfer of aggregate over the regions and borders.

Aggregate, which occupies 70% by volume of concrete, is anticipated as a recipient of waste and byproducts from other industries and various types of aggregates have been developed as given in Table 2. However, such recycled aggregate still accounts for only a small fraction of the total aggregate production for concrete.

1.4 Production of concrete

The distribution of ready-mixed concrete plants and development of concrete pumping technology at construction sites enhanced the productivity and placeability of concrete, contributing to the rapid increase in the construction projects for buildings and infrastructures during the rapid economic growth period. As shown in Figure 3, the production of concrete, 60% of which is RMC, peaked at nearly 600 million tons in fiscal 1990, in which RMC shipment was 198 million m³. The RMC shipment then decreased to 123.74 million tons in fiscal 2003, which is less than two-thirds of the peak value, and is expected to continue to decrease due to reductions in investment in new construction of buildings and civil structures.

Waste and byproducts generated from RMC plants include returned concrete brought back from job sites as excess concrete and wash water resulting from the cleaning of concrete mixers and agitating trucks. Aggregate in wash water is recovered and reused for concrete. Recovered water is divided into supernatant and sludge in sedimentation tanks, and the supernatant is reused as mixing water for concrete as it is. Sludge water is also used as mixing water for concrete if the solid percentage is 3% or less of cement. The sludge solid is dewatered and dumped as sludge cake or used as road bottoming. Returned concrete and sludge cake resulting from 1 m³ of ready-mixed concrete amount to 0.009 m³ and 5.88 kg, respectively, on national average.

1.5 Flow of concrete related materials

Figure 4 shows the current resource flow of concrete related materials, which is produced by mixing cement, cementitious additions, aggregate, and water, together with those of asphalt concrete, gravel, and crushed stone for uses other than concrete. The discharge of resources related to concrete is as low as 10% of the input, leaving enormous amounts of concrete structures in stock.

2 RECYCLING SYSTEM OF CONCRETE

2.1 State-of-the-art of concrete recycling

Thanks to the active and continued effort by the Ministry of Construction (the present Ministry of Land, Infrastructure and Transport) including “Recycling Plan 21” and “Construction Recycling Promotion Plan ‘97,” the recycling ratio of concrete lumps increased to 98% in fiscal 2002 as shown in Figure 8. However, nearly all the recycled concrete lumps have been bound for road bottoming and mechanical stabilization materials for roads.

Concrete buildings and civil structures built during the period of rapid economic growth, which have caused durability concerns such as the premature deterioration problems referred to as the “concrete crisis” in the 1980s, are expected to reach the ends of their service lives and be demolished in the period between 2010 and 2130, generating an enormous amount of concrete lumps as shown in Figure 3. On the other hand, the demand for concrete lumps for roads, which is currently their largest recipient, is destined to decrease, as new road construction is expected to gradually decrease while the method of repairing existing roads is expected to shift from repavement to “mill and overlay”. The reuse of concrete lumps as aggregate for
concrete will therefore become crucial [Nagataki & Iida 2001].

2.2 Concrete recycling into aggregate for concrete
The uses for concrete lumps to be recycled are determined by the qualities of the recycled material, such as density and water absorption, which vary depending on the percentage of cement paste contained within or adhering to the surfaces of original aggregate, and the quality of recycled aggregate depends on the production method. Figure 5 shows general methods for producing recycled road bottoming, recycled aggregate for leveling concrete (low quality recycled aggregate), and recycled aggregate having qualities comparable to those of natural aggregate and used for structural concrete (high quality recycled aggregate). For producing high quality recycled aggregate, it is necessary to repeat crushing or carry out high level treatment such as heating and scrubbing as shown in Figure 6 in order to minimize the adhering cement paste. When obtaining high quality recycled aggregate only by crushing, the recovery percentage decreases, with increased fines generation, as the number of treatment increases, since the aggregate itself is crushed and reduced in return for the improved quality.

A large amount of by-product powders result from the production of high quality recycled aggregate, posing problems. Possible uses for such powders considered so far include the following: addition to road bottoming, a cement material, concrete addition, asphalt filler, ground improving material, and inorganic board material. However, these are in competition with inexpensive natural resources, demanding quality stabilization and quality control cost reduction of such fines.

2.3 Future system for concrete recycling
In order to make concrete recyclable any number of times in the future like steel and aluminum, the production process of concrete should be changed from...
the conventional downstream approach focusing on the cost reduction and efficiency in production without consideration to the product recyclability to a production system incorporating upstream (inverse) processes in consideration of recyclability. While completely recyclable concrete (CRC) based on this concept is capable of forming a material flow in a nearly closed loop as shown in Figure 7 [Tamura et al. 2002], the material flow should be further optimized in cooperation with other industries as part of the resource recycling of society as a whole, reflecting back on the abundant resource consumption of concrete and the long service lives of buildings and civil structures.

3 SIMULATION SYSTEM FOR RESOURCE FLOW (ECOMA)

3.1 Purposes of development of ecoMA

There have been several technical and political proposals to reduce the emission of concrete waste. High quality recycled aggregate is one of the most major technical proposals that are assumed as promising. However, the impacts on the whole supply chain system, the social and economical applicability of the materials and the location decision of new material plants etc. have not yet been well investigated.

To evaluate technical and social aspects of the resource flow of concrete related materials and then to propose the optimized flow, we have been developing a resource flow simulation system named “ecoMA”. The system adopts the concept of multi-agent-system and is designed to focus not only on the production process of each plant and company including cement factory, quarry, ready-mixed-concrete plant, waste-disposal plant, recycling plant etc. but also on the decision-making dynamics between each company and government within the city-scale so that social constraint of resource flow can be simulated properly.

3.2 Overview of ecoMA

The main characteristic of ecoMA is in the modeling of social, time and geographic factors of the resource flow related to concrete structures.

Production policy of each company, governmental policy and economic circumstances significantly affect the resource flow as well as the environmental impacts from the whole resource flow system. Therefore, the decision-making process at the branch of the flow should be taken into consideration. In ecoMA, a set of decision-makers called agent as shown in Figure 8 is categorized into 3 types, PL agent as a model for material plants, TR agent as a model for transportation companies, and GV agent as a model for governmental organizations. In addition, agents in ecoMA have some characteristics in common as shown in Table 3 [Bradshaw 1997].

Concept of time is introduced by the event model. Event, in ecoMA, is defined as a social activity that
Concrete crushed beforehand into 50 mm lumps is charged in a vertical heating furnace and subjected to hot air with approximately 300°C from below to make the cement paste brittle and weak. It is then scrubbed in a tube mill to separate cement paste from the aggregate.

(b) Mechanical Scrubbing-1 (eccentric tubular type)

Concrete crushed beforehand into 50 mm lumps is crushed using an eccentric tubular vertical mill to produce recycled coarse aggregate by removing adhering cement paste.

(d) Wet Scrubbing and Gravity Classification

Concrete crushed beforehand into 50 mm lumps is crushed using rods in a drum. It is then moved up and down in water to remove lightweight mortar and woodchip.

Figure 6. Equipments for high quality recycled aggregate.

Figure 7. Closed loop of CRC.
evokes the reaction of agents. For example, promotion of a new building, decision of old building demolition, demand of road repair and tax imposition is defined as event. This modeling is on the assumption that production of any material and demolition of any structure never occurs without their needs. Events are queued based on the survey and the estimation and evoked in order as shown in Figure 9. Supply chain is defined as a tree structure, as in graph theory, having the event as a base node. Social constraints, such as recycling without the usage demand, are well introduced by this modeling.

Concrete and concrete related materials are relatively heavy and used in high volume in comparison with other building materials. Consequently, fluctuation of energy consumption due to transportation induced from the difference in geographic distribution of construction site, ready-mixed-concrete plant and materials plant is not negligible even within the city-scale of the target system. In ecoMA, geographical information such as plant location and statistical date of building construction is introduced to reflect the space constraint of resource flow in the real world.

3.3 Case studies

Scenarios of the first case study are designed to examine the effect of dissemination of high quality recycled aggregate in Tokyo area as an alternative to the decrease of the demand of road bottoming and the increase of demolished reinforced concrete buildings. A scenario in the second is set to examine the effect of building life-time on the amount of concrete waste disposal. The target area of simulation is 100 km*100 km of Tokyo area as shown in Figure 10 and the target period is from the year 2000 to 2050. Basic input/output unit of plants is based on the interview survey. To simplify the situation, amount of construction of buildings and road are set constant decrease by 1% a year after 2003. Amount of demolition of building is based on the estimation that remaining lifetime of all the buildings follows the Weibull distribution [Oikawa et al. 2002].

The agents’ strategies at each scenario in the first case study are shown in Table 4. The scenario A represents...
the realistic and no-developing-change situation where agents act only for their profit, with no recycling plants or governmental support for the recycle promotion. Recycled aggregate made using heated scrubbing method is introduced in the scenario B and C. Unrealistically, each agent has strategies to corporate to expand use of the recycled aggregate at its best. The scenario B is designed to simulate “the best spread of recycled aggregate” world to examine the problem that is to happen even in the best case. In the scenario C, cement plants use recycled-aggregate-origin by-product powders as raw materials of cement clinker, so as to examine the problem that heated scrubbing method generates approximately 34% in weight of powders. Strategy change, e.g. price-up etc., is only evoked per year so that excessive sensitivity of agents is suppressed.

Figure 11 shows the estimated total amount of land-filled waste concrete including backfilled and illegal dumping concrete in the scenario B. The result shows that amount of final disposal of demolished
concrete will drastically increase in the next 30 years even the full promoted usage of recycled aggregate. Figure 12 shows the estimated total of the amount of land-filled waste concrete with a series of building lifetime. The result shows that extension of the lifetime can reduce the total amount of land-filled waste concrete. This scenario is not realistic since the quality of buildings constructed during the period of rapid economic growth is not good, but at least indicates the possible solution to the run-out of waste disposal area.

4 CONCLUSIONS

A new life-cycle-assessment simulation system named “ecoMA” based on multi-agent-system is proposed. The system is shown to be effective in the timeline, geographical and socio-economical evaluation of environmental impacts in resource flow of concrete related materials, which will become a key issue in minimizing an environmental risk in Japan.

ACKNOWLEDGEMENT

This research was partially supported by the Ministry of the Environment, Grant-in-Aid for Scientific Research K1701, 2005–2007, “Development of resource-flow simulation system and optimization supporting system for the evaluation of environmental impact of concrete related industries”, Representative Researcher Takafumi Noguchi, the University of Tokyo.

REFERENCES

Comparison between mechanical properties of SCC containing Rice-Husk Ash and normal concrete

A. Sadr Momtazi, M.A. Mehrdad, K. Afshin Nia & A. Azhari
University of Guilan, Faculty of Technology, Civil Engineering, Iran

ABSTRACT: In this article we investigated different mix designs of concrete which has the various amount of Rice-Husk Ash as a pozzolan, water-cement ratio and super-plasticizer. Then we tested properties with the Slump-Flow test, L-Box test, V-Funnel test in order to evaluate bleeding, resistance to segregation and workability. To survey physical and mechanical properties of Self-Compacting Concrete (SCC) and compare with a reference normal concrete (NC), we used 48 cubic moulds to determine their compressive strength at the age1, 3, 7, 14, 28, 60, 90 and 120 days. 12 prismatic 5 × 5 × 30 cm moulds, 6 prismatic 7 × 7 × 30 cm and 3 cylinders 15 × 30 cm moulds are prepared to approach their tensile strength, shrinkage and Modulus of Elasticity subsequently in required ages. These samples cured in three different environmental conditions such as water, humid and air. The results show higher compressive strength in SCC, while shrinkage does not differ with that of in NC. We can also find out that the Modulus of Elasticity and Poisson’s ratio are almost higher in SCC.

1 INTRODUCTION

During recent years many researches have been made on SCC to be used in construction field [Khayat, K.H., Hu, C. & Monty, H. 1999, Khayat, K.H. & Assadi, 2002, Khayat, K.H. 2003]. SCC is a concrete that can be placed under its weight and without vibration in places where vibrating is difficult, such as the concrete pouring place which has compact network of reinforcement [Persson, B. 2003, Nasvik, J. 2003, Zhu, W & Bartos, P.J.M. 2003 & PCA-Jl. 2003]. Improving quality of concrete, in terms of durability and mechanical properties are the most important parameters that justify using SCC, particularly in recent years [Kuennen, T.2003]. As it has been mentioned, omitting vibration from concrete stages in this type of concrete and its Elasticity and Self-Compactness case decrease in the holes in concrete and subsequently, increase in its resistance against destructive environmental factors such as chlorides [Tang, L. et al. 1999], and controlling reinforcement corrosions particularly in places which seems necessary due to the holes of concrete coat on reinforcement.

Essentially, SCC must have enough plasticity to fill all parts of cast parts; therefore, its W/C must be increased and this is not desirable. Thus, by adding the super-plasticizer, the W/C is lowered and in turn, workability increases [Sri Ravindraaiah, R. et al. 2003]. Using alternative materials for cement, as named in this paper, will increase compressive strength of samples due to the existence of silica [Persson, B. 1999].

2 MATERIALS

The materials used in this test is divided into two status:

- Coarse aggregate which is gravel 5–16 mm in the first six cases of SCC; however in the last 4 states the gravel is 5–12.5 mm.
- Sand used for 6 primary cases is 3–5 mm but for 4 others is divided into two parts 0–3 mm and 3–5 mm. The cement was Portland cement (Type II). The pozzolan used in this study is the Rice-Husk Ash. The Rice Husk is burnt in the 500 centigrade degree for two hours. Then, it is placed in a room with mild temperature until reach to environmental temperature. The special mass of ash was around 300 Kg per cubic meters. Table 1 shows the chemical compounds of ash.

In this research, the super-plasticizers type Sikavisoccrete-1 is used. This type of super-plasticizers is the third generation of the super-plasticizers used in concrete. This product is compatible with both Switzerland standard SIA162 (1989) and the European standard PrEN934-2. This product has an effective role in producing concrete by absorbing
from the surface of fine stones and keeping them away from hydration reactions which are being done. Among the properties of this material, one may name lower of concrete carbonation, improvement of concrete properties in creep and compactness, increase in resistance against penetrability and positive effect on the self compacting methods of concretes.

3 PROCESS OF TESTS ON FRESH SCC

There are 6 different mixture proportions of materials which are shown in table 2. In the first to three samples, the amount of aggregates is kept constant. In addition, amount of powder and cement materials are kept constant while in the first three types, the amount of water has increased and subsequently, the ratio of water to cement has been increased as well. No bleeding is observed in the first three samples; however Slump-Flow test shows considerable segregation.

As it can be seen in table 2, the main problem occurred in the first three samples, is segregation between water and fine particles from the coarse aggregates we kept the water-cement ratio fixed and less than the amount of super-plasticizers to certain amount. In three samples (No.4, No. 5 and No.6) the bleeding was observed in the V-Funnel (5 min).

In samples 7, 8, 9 and 10, the gravel and fine sand is sieved and the gravel is limited to 5–12.5 mm range and separated sand in 0–3 mm and 3–5 mm sections. By this arranging, we made granules finer and with changes in the ratio of water to cement, while cement was constant, the results in table 3 were obtained comprehensively. Design number 10 is the most successful design and the necessary tests were performed for the hardened concrete on this plan.

Then, the plan 10 was made in real size, as shown in table 4. The goal of this paper is to study the compressive strength of SCC with high resistance in

<table>
<thead>
<tr>
<th>Constituents</th>
<th>Weight percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>92.5</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>0.41</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>0.21</td>
</tr>
<tr>
<td>CaO</td>
<td>0.41</td>
</tr>
<tr>
<td>MgO</td>
<td>0.45</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.08</td>
</tr>
<tr>
<td>k₂O</td>
<td>2.31</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Constituents</th>
<th>Weight percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>92.5</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>0.41</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>0.21</td>
</tr>
<tr>
<td>CaO</td>
<td>0.41</td>
</tr>
<tr>
<td>MgO</td>
<td>0.45</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.08</td>
</tr>
<tr>
<td>k₂O</td>
<td>2.31</td>
</tr>
</tbody>
</table>

Table 2. Mixture proportions for SCC.

<table>
<thead>
<tr>
<th>Mixture proportions</th>
<th>Particles' size (mm)</th>
<th>No.1</th>
<th>No.2</th>
<th>No.3</th>
<th>No.4</th>
<th>No.5</th>
<th>No.6</th>
<th>No.7</th>
<th>No.8</th>
<th>No.9</th>
<th>No.10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel(Kg)</td>
<td>5–12.5</td>
<td>10.8</td>
<td>10.8</td>
<td>10.8</td>
<td>10.8</td>
<td>10.8</td>
<td>10.8</td>
<td>10.8</td>
<td>11</td>
<td>10.8</td>
<td>10.8</td>
</tr>
<tr>
<td>Sand(Kg)</td>
<td>0–5</td>
<td>16.2</td>
<td>16.2</td>
<td>16.2</td>
<td>16.2</td>
<td>16.2</td>
<td>16.2</td>
<td>16.2</td>
<td>11.8</td>
<td>12</td>
<td>11.83</td>
</tr>
<tr>
<td>Sand(Kg)</td>
<td>0–3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4.37</td>
<td>4.4</td>
<td>4.37</td>
</tr>
<tr>
<td>Sand(Kg)</td>
<td>3–5</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>1.75</td>
<td>1.75</td>
<td>1.75</td>
<td>1.75</td>
<td>1.75</td>
<td>1.75</td>
<td>1.75</td>
</tr>
<tr>
<td>Rice husk ash (kg)</td>
<td></td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>1.75</td>
<td>1.75</td>
<td>1.75</td>
<td>1.75</td>
<td>1.75</td>
<td>1.75</td>
<td>1.75</td>
</tr>
<tr>
<td>Water (Kg)</td>
<td></td>
<td>2.83</td>
<td>3.06</td>
<td>3.42</td>
<td>3.06</td>
<td>3.06</td>
<td>3.06</td>
<td>3.06</td>
<td>3.8</td>
<td>4.56</td>
<td>3.06</td>
</tr>
<tr>
<td>Ratio of water to powder materials</td>
<td></td>
<td>0.32</td>
<td>0.35</td>
<td>0.38</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.4</td>
<td>0.52</td>
<td>0.35</td>
</tr>
<tr>
<td>Super plasticizer (CC)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>175</td>
<td>131</td>
<td>131</td>
<td>131</td>
<td>181</td>
<td>175</td>
<td></td>
</tr>
<tr>
<td>W/C</td>
<td></td>
<td>0.35</td>
<td>0.38</td>
<td>0.43</td>
<td>0.44</td>
<td>0.44</td>
<td>0.44</td>
<td>0.44</td>
<td>0.65</td>
<td>0.44</td>
<td></td>
</tr>
</tbody>
</table>

Table 3. Results of tests on four final mixtures.

<table>
<thead>
<tr>
<th>Test</th>
<th>Slump flux</th>
<th>V-shape funnel type</th>
<th>L-shape box</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Least dimension cm</td>
<td>Highest dimension cm</td>
<td>Time to achieve diameter</td>
</tr>
<tr>
<td>No.7</td>
<td>70</td>
<td>72</td>
<td>5</td>
</tr>
<tr>
<td>No.8</td>
<td>71</td>
<td>74</td>
<td>5</td>
</tr>
<tr>
<td>No.9</td>
<td>72</td>
<td>74</td>
<td>4</td>
</tr>
<tr>
<td>No.10</td>
<td>69</td>
<td>72</td>
<td>4</td>
</tr>
</tbody>
</table>
different times and in various curing conditions and comparing it with control concrete. The mixture of NC is given in table 5. The cubic casts were lubricated and prepared. They were 48 casts. The resistance was read by computers with ability of calculating compressive strength, even in high resistance in the days 1, 3, 7, 14, 28, 60, 90 and 120. The samples, except control concrete were not vibrated. Three different environment conditions for curing samples of SCC and NC are air, water and 95 percent humidity. In each three state, three cubic samples were tested and the average was declared as the compressive strength of samples.

All samples were weighted and their densities were calculated before breakage. The necessary density and forces for breaking of the samples were calculated per kilogram and the compressive strength f’c was calculated per Kg/cm2. Same stages were taken on control concrete. As it could be seen in figure 1, the compressive strength of concrete along with the pozzolan of Rice-Husk Ash before 14 days is 40 percent higher than compressive strength of control concrete while in the dates after 14 days, the compressive strength of concrete along with the pozzolan of Rice Husk Ash could even show more than 50 percent increase than control concrete.

In addition, it is observed that consumption of the Rice Husk Ash increases compressive strength and density to an acceptable limit. The fine materials in SCC (High Performance) increase density which increase the resistance and impenetrability of concrete. We know that if concrete penetrability is low, concrete will show resistance against destructive attacks like chloral ion, corrosion of reinforcement, and the durability of concrete is increased as well. It is also observed that curing method of concrete could affect compressive strength and concrete density. The samples cured in water have less porosity due to loss of internal water and thus, show higher compressive strength while in dry samples, due to loss of water inside the sample, it has porosity and this causes decrease in density and lowering in compressive strength.

Figure 1 shows the results of compressive strength of both concretes, including Rice Husk Ash and control concrete. The results showed that along with the Rice Husk Ash, one might expect a 60-day resistance of around 80 MPa from the concrete. While in the same conditions, control concrete could only obtain around 40 MPa compressive strength. This happens while the concrete of first type is a SCC that could bring about high resistance for us and at the same time, it could be profitable for the employer and workshop in environmental and economic terms.

Figure 1 shows difference between compressive strength of samples made of Rice Husk Ash and control concrete samples in three areas; dry, humid and into water in all times of test. In all times, whether in the beginning or later times, resistance of concrete into water is slightly more than resistance of concrete kept outdoors.

### 4 TENSILE RESISTANCE

To study tensile strength, prismatic samples of $5 \times 5 \times 30$ cm were used. Results showed that the relationship between tensile strength and compressive strength in this type of concrete is similar to Normal Concrete. The results of this tensile resistance are given

---

**Table 4. Final mixture proportions of SCC.**

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Size (mm)</th>
<th>Final design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel (Kg)</td>
<td>5–12.5</td>
<td>66.528</td>
</tr>
<tr>
<td>Sand (Kg)</td>
<td>3–5</td>
<td>26.92</td>
</tr>
<tr>
<td>Sand (Kg)</td>
<td>0–3</td>
<td>72.892</td>
</tr>
<tr>
<td>Rice-Husk Ash (Kg)</td>
<td></td>
<td>10.768</td>
</tr>
<tr>
<td>Cement (Kg)</td>
<td></td>
<td>43.072</td>
</tr>
<tr>
<td>Water (Kg)</td>
<td></td>
<td>18.852</td>
</tr>
<tr>
<td>Water-cement ratio</td>
<td></td>
<td>0.44</td>
</tr>
<tr>
<td>Super-Plasticizer (CC)</td>
<td></td>
<td>1078</td>
</tr>
<tr>
<td>Water-powder ratio</td>
<td></td>
<td>0.35</td>
</tr>
</tbody>
</table>

**Table 5. Final mixture proportions of NC.**

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Size (mm)</th>
<th>Final design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel(KG)</td>
<td>5–12.5</td>
<td>57.66</td>
</tr>
<tr>
<td>Sand (Kg)</td>
<td>3–5</td>
<td>21.6</td>
</tr>
<tr>
<td>Sand (Kg)</td>
<td>0–3</td>
<td>57.66</td>
</tr>
<tr>
<td>Cement (KG)</td>
<td></td>
<td>44.44</td>
</tr>
<tr>
<td>Water (Kg)</td>
<td></td>
<td>17.776</td>
</tr>
<tr>
<td>W/C</td>
<td></td>
<td>0.4</td>
</tr>
<tr>
<td>Slump (mm)</td>
<td></td>
<td>130</td>
</tr>
</tbody>
</table>

---
5 ELASTIC MODULUS AND POISSON’S RATIO

The Elastic Modulus and Poisson’s ratio were made on cylindrical samples with 15 cm diameters and 30 cm height in water and environmental conditions. Tests were performed on the 28-day age of the samples. The results are given in Table 6.

6 SHRINKAGE

To study shrinkage, prismatic moulds with 7 x 7 x 30 cm dimensions were studied. After 24 hours, the casts were opened and their compactness was calculated up to 150-day age for the three states, inside water and humidity of 95% and outdoor. As it was expected, compactness in concrete was higher in dry environment than humid samples [Vierira,M. & Bettencourt, A. 2003]. In addition, samples containing pozzolan had higher compactness due to the cement function of substitute materials. Results of compactness and inflation tests are given in Figures 3 and 4. In figure 3, three states of samples were compared. Figure 4 shows compactness of control concrete. Figure 5, 6 and 7 compare shrinkage between SCC and NC in different conditions.

7 CONCLUSIONS

Following results were obtained with respect to the tests:
- Using fine-particle materials leads to an increase in workability and decrease in segregation in SCC.
- Increase in cement amount increases resistance against segregation.
- As the age increases, the compressive strength increases for both ordinary samples and the samples that contain Rice Husk Ash.
- Using Rice Husk Ash causes better compactness and less porosity of concrete.
- By using super-plasticizers, the amount of water is decreased.
- Rice Husk Ash decrease amount of water even in very low quantities and shows bleeding.
- Keeping the samples in humid rooms has the same properties against the dried sample. Samples into the water would show more density and higher compressive strength compared.
- Compressive strength of self compacting concrete made with approximately 20 percent Rice Husk Ash to replace cement in the 60-day cement could be increased up to 80 Mega Pascal.
- The super-plasticizers mixture with water and adding it to the mixture would help in homogeneity of concrete.
- Substitution of cement with Rice Husk Ash provides economic gain in Iran because of insufficient production of Silica Fume and no production of Fly Ash.

REFERENCES
Nasvik, J. 2003. The ABC’ s of SCC:As a Cast-in-Place Concrete Contractor, Here ‘s What you Should Know – many Believe Self-Consolidating Concrete is the Way of the Future", Concrete Construction, March.
Comparative study between properties of SCC containing lime-stone powder and NC

A. Sadr Momtazi & A. Hajati Modaraei
University of Guilan, Faculty of Technology, Iran

A. Azhari & K. Afshin Nia
University of Guilan, Faculty of Technology, Iran

ABSTRACT: This article outlines an experimental study on different properties such as compressive strength, tensile strength, the modulus of elasticity and shrinkage of SCC, containing milled limestone powder. There is a comparative study on NC, NC (reference mixes). At first, we found an optimum mix design for SCC, then, with molding in different sizes for the required numbers in three various curing conditions (water, humid, air cured), we started to investigate. The parallel study also was carried out in order to compare these two types of concrete in different curing conditions. The results show that shrinkage in SCC is approximately twice more than that of in NC, while, elastic modulus, compressive and tensile strength do not differ significantly.

1 INTRODUCTION

During recent years, many researches and progresses have been made in the field of manufacturing SCCs [Collepardi,M. 2003,EFNARC,2002, Saak,A.N. et al 2001, & Masahiro,Q. et al 2003]. The SCC usually contains large amount of adhesive pastes which may cause a great amount of shrinkage. The Wollongong University in Australia has conducted extensive research on the shrinkage of SCCs that contain lime-stone powder, as well as on normal vibrated concrete [Bui,V.K & Montgomery,D.1999].

In Sweden in 1997, a research was made on the amount of creep, shrinkage and elastic modulus of SCC containing lime-stone powder [Persson,B.1999]. Similar researches have been carried out in Switzerland [Bosiljkov,VB.2003] and Japan [Oh,S.G. et al 1999]. The highest tendency for using SCC has been for the improvement in concrete quality as well as its high capacity to be used in different functional environments. The tests introduced for SCC at the time of mixing differ significantly with those of for NCs. The differences are mostly due to the characteristics of consumed materials in these concretes. One may say that SCC is a new generation of High-Performance Concrete that could be spread and fill the casts without vibration and with minimum risk of segregation constituents materials [Collepardi,M.2003]. Of course, special cares are needed in the mixture proportions of consumed materials in SCCs for homogeneity of concrete, deformation and high durability.

In this research, in order to evaluate the flowing behavior of SCC s in quantities method, laboratory methods such as Slump-Flow test, V-Funnel test, L-box test are used for studying segregation resistance, filing ability and passing ability, respectively. After producing a mixture of SCC and its control by using the mentioned tests and casting in different rigid metal molds, the long-term characteristics of these concretes were studied. Then this SCC was compared with a NC by using conventional tests.

2 MIXTURE PROPORTIONS FOR SCC

Many researchers have already tried to propose mixture proportions for SCC [Gettu,R.2003 & Nasvik,K.J. 2003]. However, no standard method has been presented for SCC and most often; this is done by trial and error.

The mixture proportions for optimized mixture of SCC were developed by testing on 10 mixtures. Since the properties and type of using limestone powder is almost unknown, in two mixtures, it was tried to find a suitable amount for this material and in others, it has been tried to vary the superplasticizers amount and also changing the Water-Cement ratio.
The mixture constituents is gravel, sand, Portland cement (type II), Lime-Stone powder, water and superplasticizer (Viscocrete-1), which were added to the mixer in certain time; respectively. Dry materials are mixed minimum 30 seconds before adding water and admixtures. The mixing time after adding water was minimum 5 minutes. Before casting the molds, the specifications of the concrete were studied on Slump-Flow test, L-Box test and V-Funnel test. Table 1 shows the amount of materials consumed in different mixes. Table 2 shows the laboratory results of samples 7, 8, 9 and 10.

3 COMPARISON BETWEEN SCC AND NORMAL CONTROL CONCRETES

3.1 Compressive strength

Generally, the compressive strength of SCC is relatively higher than NCs [Kuennen, T. 2003].

To study the compressive strength, the cubic rigid metal molds 15 × 15 × 15 cm size was used. These molds have been prepared with sufficient number to cure in three types of different environmental conditions and after casting, relevant tests were made on them in different ages Figures 1 and 2 show the increase in compressive strength in different ages.

Tests were continued for 120 days. There is no significant difference between the two types of concretes.

---

Table 1. To achieve of a suitable mixture proportions for Self-Compacting Concretes with different amount of materials.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>NO.1</th>
<th>NO.2</th>
<th>NO.3</th>
<th>NO.4</th>
<th>NO.5</th>
<th>NO.6</th>
<th>NO.7</th>
<th>NO.8</th>
<th>SCC-1</th>
<th>SCC-2</th>
<th>NC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel (5–12.5).Kg</td>
<td>10.8</td>
<td>10.8</td>
<td>10.8</td>
<td>13.04</td>
<td>13.04</td>
<td>13.04</td>
<td>13.04</td>
<td>13.04</td>
<td>13.04</td>
<td>50</td>
<td>57.66</td>
</tr>
<tr>
<td>Sand (0–5).Kg</td>
<td>16.2</td>
<td>16.2</td>
<td>16.2</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Sand (3–5).Kg</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>3.68</td>
<td>3.68</td>
<td>3.68</td>
<td>3.68</td>
<td>3.68</td>
<td>3.68</td>
<td>15</td>
<td>21.6</td>
</tr>
<tr>
<td>Sand (0–3).Kg</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>45</td>
<td>57.66</td>
</tr>
<tr>
<td>Lime-stone powder .Kg</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>1.75</td>
<td>2.55</td>
<td>4.24</td>
<td>4.24</td>
<td>4.24</td>
<td>4.24</td>
<td>10</td>
<td>0</td>
</tr>
<tr>
<td>Cement .Kg</td>
<td>8.1</td>
<td>8.1</td>
<td>8.1</td>
<td>7.5</td>
<td>7.5</td>
<td>7.5</td>
<td>7.5</td>
<td>7.5</td>
<td>7.5</td>
<td>25</td>
<td>44.44</td>
</tr>
<tr>
<td>Water .Kg</td>
<td>2.83</td>
<td>3.06</td>
<td>3.24</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3.2</td>
<td>9.75</td>
<td>17.776</td>
</tr>
<tr>
<td>Water/Cement ratio</td>
<td>0.35</td>
<td>0.38</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.425</td>
<td>0.39</td>
<td>0.4</td>
</tr>
<tr>
<td>Superplasticizer .cc</td>
<td>160</td>
<td>160</td>
<td>160</td>
<td>138</td>
<td>138</td>
<td>138</td>
<td>138</td>
<td>138</td>
<td>207</td>
<td>150</td>
<td>400</td>
</tr>
</tbody>
</table>

Table 2. Results of tests on the fresh mixtures.

<table>
<thead>
<tr>
<th>Test</th>
<th>Slump-Flow</th>
<th></th>
<th>V-Funnel</th>
<th></th>
<th>L-Box</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D-min(mm)</td>
<td>D-max(mm)</td>
<td>T-50(sec)</td>
<td>5-min(sec)</td>
<td>10- Sec (sec)</td>
</tr>
<tr>
<td>No. 7</td>
<td>73</td>
<td>79</td>
<td>4</td>
<td>5</td>
<td>4</td>
</tr>
<tr>
<td>No. 8</td>
<td>73</td>
<td>76</td>
<td>4</td>
<td>7</td>
<td>5</td>
</tr>
<tr>
<td>SCC-1 (final)</td>
<td>68</td>
<td>72</td>
<td>5</td>
<td>9</td>
<td>7</td>
</tr>
<tr>
<td>SCC-2 (final)</td>
<td>65</td>
<td>71</td>
<td>5</td>
<td>13</td>
<td>9</td>
</tr>
</tbody>
</table>

---

Figure 1. Compressive strength of SCC-1 and SCC-2.

Figure 2. Compressive strength of SCC-1 and NC.
Perhaps, higher resistance of SCC in the 7-day age is due to the presence of limestone powder which has an accelerating effect on the hydration of C_3S and initial setting. Other researchers have also noted this phenomenon [Sonebi, M. et al 2004 & Edamatsu, Y. & Nishida, N. 1999].

3.2 Tensile strength
To study the tensile strength, prismatic specimens with 5 x 5 x 30 cm dimensions were used. The results showed in Figure 3, indicate the fact that the relationship between tensile strength and compressive strength is very much similar to NC.

Figure 3 also shows the process of increase in tensile strength in different ages for the concretes cured in water. It seems that one may use the researchers’ experiences on NC for the SCC as well.

4 SHRINKAGE AND SWELLING IN SCCS
The amount of shrinkage for different SCC and NC is presented in Figure 4. The shrinkage in SCC-2 is greater than SCC-1 and NC. Amount of swelling depends on reactivity of cement past and water absorption which is very important in SCC (Figures 5 and 6). These results are conform to researches (7 and 18). In the mixture of SCC, the amount of cementitious material is usually more than NC. These cementitious materials could be only cement, cement and fillers, or cement and Pozzolans. Since the shrinkage and its changes often depend on cement reactions [Bosiljkov, V.B. 2003], the amount and changes in the SCC are usually more than NC. Of course hardness of coarse-aggregates affect the amount of shrinkage. The lower the amount of coarse aggregate is, the higher the shrinkage is expected to be [Collepardi, M. 2003]. Of course, researchers have presented contradictory information on the shrinkage of SCCs. In a research in Australia, it has been reported that using limestone powder with suitable size and volume would lead to the lowering amount of drying shrinkage [Chopin, D. et al 2003].

Results of another research have shown that amount of shrinkage in SCC does not differ with vibrated NCs [Poppe, A.M. & Schuttr, G.D. 2003].
5 ELASTIC MODULUS AND POISSON’S RATIO IN SCC

The measurement of elastic modulus and poisson’s ratio were achieved by testing on the cylindrical standard samples with 30 cm length and 15 cm diameter. The results show that many of experimental standard formulas in different codes of practice for NC can be used for SCC. The results of the tests are shown in Table 3.

6 CONCLUSIONS

By comparing the information obtained, one can say that there is no precise relation between mechanical properties of the self-compacting and NC. The results obtained from data could be summarized as follows;

1. The strength of Self-Compacting and NCs under comparative conditions are similar to some extent; while, strength of SCCs with the similar water-cement ratio is relatively higher. There are no significant equations to be presented for the time being.
2. Increase in the compressive strength concrete with time is similar in both two types of concrete.
3. The bending strength and elastic modulus in Self-Compacting and NCs are similar; however, there are more scattered than for NCs.
4. The shrinkage of SCC is clearly more than NCs. This could be due to large amount of cement materials in self-compacting concretes. Shrinkage is particularly more in the concretes with less than 28-days age.

REFERENCES

Collepardi, M. 2003. Recent Development in SCCs in Europe, Civil Engineering Faculty Leonardo da Vinci, Polytechnic of Milan, Italy.
Wombacher, F. J. shrinkage and shrinkage reduction of the self compacting concrete, sika, R. & D. Zurich, Switzerland.

<table>
<thead>
<tr>
<th>SC(W)</th>
<th>SC(D)</th>
<th>NC(W)</th>
<th>NC(D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E(GPa)</td>
<td>35.95</td>
<td>36.42</td>
<td>29.41</td>
</tr>
<tr>
<td>ν</td>
<td>0.23</td>
<td>0.24</td>
<td>0.17</td>
</tr>
</tbody>
</table>

Table 3. Elastic modulus and Poisson’s ratio in Self-Compacting and NCs.
1 INTRODUCTION

1.1 Background

With an increasing awareness of the negative environmental impacts associated with many commonly used construction materials, there exists a desire to find modern alternatives which are both economically and environmentally viable. Given that straw is an annually renewable agricultural bi-product, which is relatively cheap to purchase, and which provides excellent insulation as a building material, it seems that straw bale construction has the potential to become a practical alternative to traditional building methods [Magwood & Mack 2000].

There are two main types of straw bale construction; load-bearing and post-and-beam. With post-and-beam construction, the straw acts as an insulating infill material, while additional timber framing provides the structural support for the building. For load-bearing straw bale construction, the plastered straw bale wall system provides the structural capacity to withstand any and all external live loads applied to the building, in addition to the dead loads associated with the roof and other components of the structure. In order to resist these loads, a plastered straw bale wall behaves as a composite sandwich panel, where the plaster skins act as the primary structural members, and the straw acts as a tie between the skins, providing lateral support for the skins, and tying the skins together, allowing the wall to act in a composite manner.

Because of its anecdotally noted structural adequacy, and the numerous environmental benefits associated...
with straw bale construction, there has been a recent resurgence in the popularity of plastered straw bale construction around the world, including in North America [Lerner et al. 2000].

1.2 State of straw bale construction in Canada

In many locations around the globe the amount of ‘red-tape’ that must be cut in order to build a straw bale structure (or any ‘alternative’ structure) has diminished to the point of being nearly non-existent. Unfortunately, this is not true for Canada. The Canadian building codes are typically very strict, which can make it difficult to obtain building permits for construction [DSA Architects 2006]. This is especially true for load-bearing straw bale construction. Furthermore, the administrative difficulties extend beyond those associated with obtaining building permits. Owners also have noted difficulties with obtaining mortgages and insurance for their homes [DSA Architects 2006]. In order to meet regulations associated with a straw bale building project, there is often a need for experimental proof of the acceptable performance of plastered straw bale walls.

1.3 Currently available data

Of the large range of questions that are often asked regarding the viability of load-bearing straw bale construction, some of the most common ones are in regards to the structural adequacy of the walls. Other issues such as fire resistance, moisture penetration and resulting rotting, and insects and other pests are pertinent, but are beyond the scope of this paper. Such issues, specifically those related to moisture within plastered straw bale walls have been researched, and continue to be researched. Work by Fugler [2000] and Goodhew et al. [2004] are just a few examples of such research suggesting that although moisture can become a problem, it can be avoided by proper detailing and construction practices, and appropriate site selection. Further research in this area is ongoing.

The focus of this paper is on the compressive performance of plastered straw bale wall assemblies. Although this is a topic that has been researched to some extent already, the data that is currently available is still limited in its overall usefulness. Data describing the compressive performance of plastered straw bale walls is available from experiments such as those conducted by Platts [1996], Carrick & Glassford [1998], Grandaert [1999], Zhang [2002], Faine & Zhang [2002], Dreger [2002], and Fibrehouse Ltd & Scanada Consultants Ltd. [1996]. Unfortunately, because there are currently no standards for testing related to straw bale construction, there is a wide range of reported test results. Vardy & MacDougall [2006] noted that in the literature the ‘results obtained for ultimate compressive strength on plastered bale walls varied significantly from 28 kN/m to 90 kN/m for various tests conducted using a wide range of wall dimensions, plaster proportions and thicknesses, reinforcement schemes and bale types and sizes’. There is a difficulty with comparing the various results because of a lack of consistency among these experiments with regards to a number of important parameters such as wall dimensions and plaster mix. Furthermore, it appears that although the wall dimensions, plaster proportions, bale type and bale size are usually reported, the plaster thickness and plaster strength are often not detailed.

Because of the wide range of reported test results and the lack of standards related to the structural testing of plastered straw bale wall assemblies there exists a need for research which can explain some of these variations noted in the literature. An understanding of the parameters which may vary from wall-to-wall, and the effect that these variations have on the structural performance of the wall would be beneficial to providing a more detailed understanding of the structural behaviour of plastered straw bale wall assemblies.

1.4 Need for further research

Previous experiments conducted by Vardy & MacDougall [2006] described the performance of individual plastered straw bales. The strength and stiffness was reported for bales plastered on-edge and flat with a variety of plaster strengths and thicknesses. Although it appeared that all of the individual plastered bales provided adequate compressive resistance, it was noted that they may have failed prematurely as a result of stress concentrations developing at the top of the plaster skins due to un-evenness in the plaster layer as a result of warping in the formwork. In addition, it was noted that for taller wall sections, there may exist failure mechanisms which were not present for the individual bale experiments. Thus it was decided to conduct similar experiments on larger wall specimens to determine their performance in relation to the individual plastered bales. Specifically it was intended that observations would be made on the effect of increasing wall height on the performance of the walls. It was also intended that these specimens would be constructed and tested in such a way as to minimize the possibility of stress concentrations in the plaster in order to determine if the individual bale experiments may have been compromised as noted by Vardy & MacDougall [2006].

Furthermore, there is a need to predict the performance (strength, stiffness, failure mechanism etc.) of a plastered bale wall assembly without having to conduct a full-scale laboratory test. Such a prediction will require a greater understanding of the stress-strain performance of a plastered straw bale wall assembly, and an understanding of the failure mechanism (or mechanisms) for a plastered straw bale wall.
loaded in compression. Considering this, an attempt was made to fit a non-linear stress-strain model to the data obtained from the experiments discussed within this paper. In addition, lateral displacement measurements were conducted using a number of methods throughout the experimentation process. These measurements were intended to give indication of the failure mechanisms which exist for the plastered straw bale walls, and specifically to determine if buckling was occurring within the wall prior to ultimate failure. Careful attention was paid to the failure mechanisms observed for each of the experiments.

2 METHODOLOGY

2.1 Design of fabrication and testing jig

One difficulty that has been noted with conducting research on plastered straw bale wall assemblies is the difficulty in constructing a wall with a consistent plaster thickness. Coupled with this is the difficulty in constructing a wall which is square, with plaster skins which are vertical, and parallel to one another. Considering these issues, it was determined that it was necessary to design a construction and testing jig that could be used to create consistent, reproducible specimens. It should be noted at this point, that the ability to create dimensionally consistent specimens is not reflective of common building practice. Among other imperfections, a typical plastered straw bale structure may have significant variations in the plaster thickness and strength, and in the verticality of the walls themselves. The effect that these imperfections may have on the real-life performance of a plastered straw bale wall is outside of the scope of this paper, but is something that must be considered, and is discussed in the recommendations for future work section of this article.

Vardy & MacDougall [2006] described the design of a fabrication jig, and separate testing apparatus, for individual plastered straw bales. It was noted however, that the plaster formwork in the fabricating jig may have become warped and thus, the top and bottom surfaces of the plaster may not have been flat. It was hypothesized that this may have led to stress concentrations at the top of the plaster (where most failures were observed to occur), leading to premature failure of the individual plastered straw bales. The new jig consists of a steel base, constructed with 50 mm × 100 mm hollow structural steel (HSS) tubes as shown in Figure 1.

The base consists of one steel section that will support each of the plaster skins, with a number of sections connecting these two pieces to support the straw. The base is thus shaped like a ladder as can be seen in Figure 1. There is also a top piece of the jig, which is the same shape as the bottom base. The top and bottom pieces are connected by four vertical 50 mm × 50 mm HSS sections. The vertical pieces can be bolted easily to the top and bottom pieces to create a steel frame. Vertical pieces of varying height can be constructed in order to create plastered straw bale walls of varying height. The top and bottom pieces of the jig provide the top and bottom formwork for the plaster, and also provide a top bearing surface for loading the walls, and a bottom base for supporting the wall during testing.

The vertical formwork is provided by wood strips placed between the top and bottom pieces and laid against the bales as seen in Figure 2. The thickness of
these pieces determines the thickness of the plaster skins. The entire wall can be moved in the jig, even after plastering, as long as the vertical steel pieces remain connected. In order to test, the vertical pieces (steel pieces and wooden formwork) were removed so that the plastered straw bale wall was supporting the top piece, and any load that was applied to the top piece. Figure 2 is a photograph of a 2.31 m wall being plastered using the jig.

2.2 Fabrication procedure

The fabrication procedure is as follows:

1. The bales were trimmed and compressed individually using the jigs described by Vardy & MacDougall [2006]. They were trimmed to a width of approximately 405 mm.
2. The bales were placed in the steel frame shown in Figure 2. The vertical bars determined the height of the wall and were designed to be 0.99 m high for the three-bale wall section and 2.31 m high for the seven-bale wall section. The vertical bars were bolted to the steel ladders at the top and bottom of the wall, which acted as the top and bottom of the formwork.
3. Vertical wood strips were placed against the straw, 600 mm apart, on each side of the wall. These acted as the vertical plaster formwork and were cut to a thickness of 25 mm to provide a guide for the plaster thickness.
4. The plaster was hand applied in three layers, all applied in the span of approximately 4 hours, allowing very little time for curing between layers. This is not necessarily representative of common construction practice. Often the plaster is applied in two separate coats, with enough time between coats to allow the first layer to cure significantly before the second layer is applied. Further testing is required to understand how this variation in practice will affect the structural performance of a plastered straw bale wall.
5. Burlap was placed over the wall once plastering was completed and provided a moist curing environment for approximately three days. The wall then cured for another four days without the burlap cover.
6. The wall sections were tested after seven days, at which time the wall and jig were craned into the loading apparatus, and the vertical steel and wood pieces were removed.

2.3 Testing procedure

The testing program consisted of compression experiments on six plastered straw bale wall sections, as well as a number of cube and cylinder experiments on the plaster that was used for the walls.

The three 0.99 m wall sections were tested in a stroke-controlled loading machine. The load was applied at a rate of 1 mm/min until the ultimate load was reached. Beyond this point the loading rate was increased to 3 mm/min. The load was applied concentrically, directly to the top steel ladder, and the wall was supported at the base by the bottom steel ladder. This configuration allowed the load to be applied directly to the straw and the plaster at the same time, and ensured that strains were consistent in both materials.

The three 2.31 m wall sections were tested under a large reaction frame. The frame was fitted with a 900 kN hydraulic jack, which was controlled with a hand pump. The rate of loading was variable due to the nature of the hydraulic jack – hand pump system used. It was attempted to load at a slow rate until the ultimate load was reached, and then to increase the rate of loading. Again, the load was applied concentrically, directly to the top of the steel ladder, and the wall was supported at the base by the bottom steel ladder.

For each experiment conducted, three 45 mm × 45 mm plaster cubes were tested in compression to determine the approximate plaster strength for the plaster applied to the wall. The cubes were tested at a rate of 1 mm/min.

2.4 Instrumentation

For all six experiments the load, vertical displacements, and lateral displacements were recorded. The load was recorded for the three 0.99 m experiments using the load output from the testing machine. For the three 2.31 m experiments, the load was recorded using a 100 kN load cell located between the hydraulic jack and the steel ladder at the top of the wall.

For all six experiments, the vertical displacement was recorded at each corner of the top of the wall using displacement transducers. The techniques used for measuring the lateral displacements varied from wall to wall. For the first 0.99 m wall, lateral displacements were measured with displacement transducers at the top, bottom, and middle of one side of the wall. For the second and third 0.99 m walls, and the first 2.31 m wall, the lateral deflections were measured with displacement transducers at mid-height on both sides of the wall. For the second and third 2.31 m walls, the lateral deflections were measured using displacement transducers at mid-height on both sides of the wall and at the top of the wall on one side of the wall.

In addition, for the third 2.31 m wall, the deflections were recorded using Particle Image Velocimetry (PIV) as described by White et al. [2003]. This involved colour texturing the edges of the plaster skins using splatters of black paint, and having a camera take photographs of the edge of the wall at 20 second intervals throughout the duration of the experiment. The photographs were used to track the lateral and
vertical deformations of the plaster skins (and the bales in-between) throughout the entire experiment.

3 MATERIALS

3.1 Wheat straw bales

Two-string wheat bales were used for all of the experiments. The bales were purchased from a local farmer in the summer of 2005 and were used for the experiments in the summer of 2006. At the time of purchase the bales were dry and were stored in a barn. After purchase, the bales were stored indoors at room temperature and at constant humidity. At the time of testing the bales were dry and in good condition. Similar to the bales described by Vardy & MacDougall [2006], the bales were all approximately 12 kg with approximate dimensions of 350 mm in height, 500 mm in width and 800 mm in length. They were found to have a density of approximately 85 kg/m³, which is similar to values available in the literature [Watts et al. 1995]. The stress-strain behaviour of the bales was also found to be very similar to the bales described by Vardy & MacDougall [2006]. A typical stress-strain curve for an un-plastered wheat bale laid flat is given in Figure 3.

3.2 Lime-cement plaster

The plaster used for all six wall experiments was a lime-cement plaster with proportions of 4.5:1.25:0.25 of sand, lime and cement respectively. These proportions, with a water-cementitious materials ratio (W/Cm) of 1.08 have been shown to produce a plaster with 28 day compressive strength of 1.72 MPa as described by Vardy et al. [2005]. Vardy et al. [2005] and Taylor et al. [2005] determined the Modulus of Elasticity for lime-cement plasters to vary significantly from less than 100 MPa to more than 800 MPa. This variation appeared to be random, and was not a function of plaster strength or curing time. The proportions used represent a plaster with a very high percentage of lime, and thus produced a relatively weak plaster. Furthermore, with the experiments conducted after only 7 days of curing, the strength values obtained for the experiments will likely be significantly lower than the 1.72 MPa given in the literature. The modulus values, though variable, should not be affected significantly by the difference in curing time.

Vardy et al. [2005] described the applicability of a concrete stress-strain model for modelling the behaviour of a lime-cement plaster in compression. A model described by Todeschini [1964] has been shown to provide a good prediction of the stress-strain behaviour of the plaster used for the experiments described herein. Figure 4 compares the theoretical behaviour determined using the Todeschini model with a typical stress-strain curve found for the lime-cement plaster used for the plastered straw bale wall experiments described in this paper. The equation used for the stress-strain relationship of the plaster is given as Equation 1 below:

$$f_c = \frac{\varepsilon \times E_c}{1 + \left(\frac{\varepsilon \times E_c}{2 \times f'_c}\right)}$$

where $f_c = \text{theoretical stress}$; $\varepsilon = \text{strain}$; $E_c = \text{concrete modulus of elasticity}$; $f'_c = \text{concrete compressive strength}$.

As indicated in Equation 1, the model can be used to predict the stress-strain curve when given the modulus and ultimate strength of the plaster. The ultimate strength used can be determined from cube tests, but determining the appropriate modulus has proved to be more of a challenge. Vardy et al. [2005] and Taylor et al. [2005] showed that the experimentally determined modulus for lime-cement plaster is highly variable, even when the mix proportions remain constant.
4 RESULTS AND DISCUSSION

4.1 Plaster tests

4.1.1 Plaster strength

For each of the six experiments, three 45 mm × 45 mm cubes were tested in compression to determine the strength of the plaster applied to the wall. Each wall was constructed using the same plaster mix, and there were multiple batches of plaster mixed for each wall. Each of the batches was mixed with the same proportions of materials and thus, should have the same structural properties. Realistically, however there will always be variability in the mixes and thus, variability in the plaster structural properties. The results obtained from all 18 cubes tested give an indication of the expected wall plaster strength, and the variability which may be expected in the strengths. Table 1 gives a summary of the plaster strength information determined from the cube tests.

As can be seen from Table 1, the cube strength of the plaster was found to be extremely low. The average cube strength was found to be only 0.764 MPa. Furthermore, it was found that there is significant variability in the plaster strength, with the lowest measured cube strength being 0.606 MPa and the highest being 0.963 MPa. This variability will translate directly to variability in the strengths found for the plastered straw bale wall specimens.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of tests</td>
<td>18</td>
</tr>
<tr>
<td>Average strength</td>
<td>0.764 MPa</td>
</tr>
<tr>
<td>Maximum strength</td>
<td>0.963 MPa</td>
</tr>
<tr>
<td>Minimum strength</td>
<td>0.606 MPa</td>
</tr>
<tr>
<td>Standard deviation</td>
<td>0.103 MPa</td>
</tr>
</tbody>
</table>

4.2 Plastered straw bale wall experiments

4.2.1 Load-deflection response

Figure 5 gives a typical load-deflection response for the entire duration of a 2.31 m plastered straw bale wall compression test. The wall appears very stiff for the initial portion of the test, until the ultimate load of approximately 22 kN is reached. At this point a crack forms on one side of the wall and the load drops significantly to about 5 kN. The load then rises again to approximately 10 kN, and holds steady for a short period of time before dropping again to less than 5 kN when a crack forms on the second side of the wall. The wall then regains some strength to approximately 7 kN and maintains this residual strength for the remainder of the test. This behaviour was noted for all six wall experiments and is similar to the response observed for individual plastered straw bales [Vardy & MacDougall 2006].

The load-deflection diagrams for the six experiments were used to determine the ultimate stress and the wall modulus for each of the plastered straw bale walls. The ultimate load was determined as the maximum load obtained prior to the first crack appearing in the plaster. This corresponds to the point on the figure just before the first significant drop in load was observed. The ultimate stress (in MPa) was calculated by dividing the ultimate load (in N) by the area of plaster upon which the load was applied (= 600 mm × 25 mm × 2). For this calculation, the contribution of the straw to the compressive strength of the composite wall is ignored. Vardy & MacDougall [2006] showed that prior to ultimate failure, the straw contributes very little to the compressive strength of a plastered straw bale wall. This can be further seen by noting that the plaster strain at maximum stress shown in Figure 4 is only approximately 0.006. Looking at Figure 3, it can be seen that the stress in the straw at this strain is essentially zero.

The wall modulus was determined using the slope of the line connecting the first point on the figure (zero load, zero displacement) with the point on the curve corresponding to a load equal to 40% of the ultimate load (as used to define the Modulus of Elasticity for concrete). The slope (in N/mm) was then multiplied by the appropriate wall height (in mm) and divided by the area of plaster upon which the load was applied (again, equal to 600 mm × 25 mm × 2). This gave the wall modulus in MPa. A summary of the ultimate stress and wall modulus values for the six experiments is given in Table 2.

4.2.2 Ultimate stress

Comparisons of the ultimate stresses for the three 0.99 m walls and the three 2.31 m walls are given in Figure 6 and Table 2.
Figure 6 also shows the maximum, minimum, and average compressive cube strengths for the plaster as described previously. It can be seen that both the 0.99 m wall, and the 2.31 m wall achieved ultimate stresses very similar to what would be expected based on the compressive cube strengths observed for the plaster. All wall specimens tested reached an ultimate stress only slightly below the average cube strength found for the plaster cubes, and only one wall failed at an ultimate stress lower than the minimum cube strength.

All six wall sections reached ultimate stresses of between 73.6% and 97.6% of the average plaster cube strength. This indicates that when assuming pure compression failure of the plastered straw bale wall, the cube strengths over-estimate the strength of the plastered straw bale wall. This may be an indication that other failure mechanisms exist for the wall other than pure compression failure.

Figure 7 gives the wall ultimate stresses as a percentage of the average plaster strength of the plaster applied to the wall. Data is presented for the 0.99 m walls and the 2.31 m wall in addition to data for 0.33 m walls available in Vardy & MacDougall [2006]. It can be seen that the 0.99 m and 2.31 m walls performed even better than the 0.33 m walls. For a wide range of experiments the 0.33 m walls typically reached about 60–80% of the average plaster strength.

There were two 0.33 m wall experiments for which the wall ultimate stresses actually exceeded the average plaster cube strength. Given the previously noted variability in the plaster strengths, it is not entirely surprising that a number of walls performed better than expected. However, the great extent to which these walls exceed expectations may indicate error in the mixing of the plaster. Possible sources of error could include adding too much cement or lime, or not enough water. These changes to the mix have been shown to lead to increased plaster strength.

This data reinforces previous comments that the 0.33 m walls may have succumbed to premature failure as a result of stress concentrations at the top and bottom of the plaster skins due to the procedure used for testing.

It is difficult to compare these results with any other values from the literature as in most cases either the plaster thickness or the plaster strength was omitted.

### 4.2.3 Plastered straw bale wall modulus

Figure 8 and Table 2 give the range of plastered straw bale wall modulus values obtained from the experiments discussed in this paper. In addition, Figure 8 gives the bale wall modulus values determined for the individual plastered straw bale experiments described by Vardy & MacDougall [2006]. The average value for all experiments was determined to be just over 218 MPa, but it should be noted that there is significant variation in the values found throughout all of the experiments. There does not appear to be a correlation between the measured wall modulus and the height of the plastered straw bale wall, or any other parameters studied by Vardy & MacDougall [2006].

The bale wall modulus values are in the same range as the plaster modulus of elasticity values determined by Vardy et al [2005] and Taylor et al [2005], who found values to range from less than 100 MPa to over
800 MPa. It is clear that there exists a large variability in the modulus of elasticity for the plaster, which then leads to a large variability in the bale wall modulus values measured for the plastered straw bale walls.

4.2.4 Stress-strain relationship
The next step was to attempt to model the pre-failure stress-strain behaviour of the plastered straw bale walls. The load deflection data for each of the six walls was converted to stress-strain data and plotted. Figure 9 gives the pre-failure stress-strain behaviour of the six tested walls. Also included in Figure 9 are three lines representing theoretical stress-strain curves developed using the Todeschini model discussed above. These three lines represent the minimum, maximum, and average stress-strain relationships expected based on observations from the experiments.

In order to use the Todeschini model a number of important assumptions were made:
1 It was assumed that the walls failed by compressive crushing of the plaster skins, hence assuming that the full compressive strength of the plaster is developed, and thus the full stress-strain curve.
2 It was assumed, as mentioned previously, that the contribution of the straw to the compressive strength of the plastered straw bale wall was negligible.
3 Three separate curves were drawn, one using the minimum compressive cube strength found for the plaster, another using the maximum compressive cube strength found for the plaster, and a third using the average compressive cube strength found for the plaster, as given in Table 1.
4 The curve using the minimum compressive cube strength used the lowest modulus found for the plastered straw bale walls, while the curve using the maximum compressive cube strength used the highest modulus found for the plastered straw bale walls. The curve using the average compressive cube strength used the average modulus found for the plastered straw bale walls.

It can be seen from Figure 9 that the experimental curves all fall within the boundaries described by the maximum and minimum theoretical curves. The average theoretical curve falls directly between the experimental curves found for the 0.99 m walls and the 2.31 m walls. In addition, it can be seen that the experimental curves follow a similar shape as the theoretical curves. It is important to note that it appears the walls all failed slightly prematurely, and were unable to develop the full plaster compressive strength. This is shown by the fact that the curves appear to end rather abruptly, rather than reaching a maximum value with a smooth curve before having the stress drop (as is seen with the theoretical curves). This is an immediate indication that the failure of the walls may not be as simple as pure compression failure of the plaster skins.

One major difficulty that was noted with modelling the stress-strain behaviour of the walls was in regards to the appropriate modulus of elasticity to use for the model. With such a wide range in experimental modulii measured for both the plaster itself, and the plastered straw bale walls, it is very difficult to determine what modulus should be used for the modelling. It appears that if one were to use the average plaster cube strength, and the average plastered straw bale wall modulus (as determined to be 218 MPa), a curve can be found which is a reasonable fit for both the 0.99 m and 2.31 m walls.

4.2.5 Lateral displacements
Measurements of the lateral displacements can give light to a number of important issues with regards to the structural performance of the plastered straw bale walls, and the mechanism (or mechanisms) which lead to the ultimate failure of the walls.

The configurations for measuring the lateral displacements varied from wall-to-wall. A trial and error process was used from test-to-test to determine which
measurements were the most important. It was determined early-on that the bottom of the wall did not move laterally throughout the experiments as it was fixed quite securely to the base support. Lateral displacements were found to be most significant at the mid-height of the wall, thus it was decided to measure the displacements of each side of the wall at mid height. The top of the wall essentially did not move laterally at all, as indicated by displacement transducer measurements. Figure 10 gives a typical plot of lateral displacement with load for these points on the wall. The measurements are provided from the start of the test until ultimate failure.

As expected, the lateral displacements at the top of the wall remain at about 0.0 mm throughout the duration of the experiment. This indicates that there is no ‘sway’ in the wall prior to failure. This is an important observation as any sway in the wall will lead to bending in the wall as a result of the P-Δ effect (the application of load eccentric to the base of the wall).

Also of importance are the mid-height lateral displacement measurements. Both the left and right readings indicate that as the load is applied there is some lateral movement at the mid-height of the wall. This indicates that the wall may be experiencing global buckling as the load is applied. This is reinforced by the observation that both the left side and the right side of the wall moved laterally in the same direction. The magnitudes of the movements do not appear to be equal though, and this suggests that either, one of the plaster skins is buckling more than the other or that there is some error in the measurements. Given that the measured displacements are, for the most part, less than 1.0 mm, it is possible that there is significant error in the readings, as the surface roughness of the plaster skins may be upwards of 1.0 mm.

Given the uncertainty in the measurements, it was decided for the final experiment (the third 2.31 m wall) that Particle Image Velocimetry (PIV) would be used. This allowed the lateral displacements of both plaster skins to be measured simultaneously for the entire height of the wall, and for the full duration of the experiment. PIV has been shown to be an excellent technique for monitoring deformations in soil [White et al. 2003], and it is beginning to gain recognition as a means of measuring deformations for structural engineering research. The results from the PIV monitoring are compared with measurements using displacement transducers to validate the PIV results. The lateral deformation plot obtained from PIV is given in Figure 11, and describes the magnitude of lateral displacement of both plaster skins for the entire wall height from the start of the experiment until ultimate failure.

Figure 11 gives an indication of the laterally deflected shape of the plastered straw bale wall at failure. It shows that the deflections at both the base and the top of the wall are negligible. This reinforces the previous observation that there is no sway in the wall prior to ultimate failure. Furthermore, Figure 11 also supports the initial observation of the possibility of global buckling occurring prior to failure. Both the left and right sides of the wall appear to buckle laterally a distance of approximately 1.6 mm. This corresponds very well with the displacement measured using the displacement transducer on the left side of the wall (approximately 1.5 mm). Since Figure 11 indicates that the lateral displacements are almost identical for both the left side and right side of the wall, it appears as if the mid-height displacement transducer measurements for the left side of the wall (given in Figure 10) are erroneous as discussed previously.

In order to understand the full effect that the buckling has on the performance of the plastered straw bale wall one may consider the increase in stress that will result at mid-height of the wall due to the lateral movement in the plaster skins. If one assumes that the load is applied concentrically to the top of the plaster skin, then at mid-height, with the lateral displacement,
the line of action of the load will be offset by a distance of 1.6 mm. Thus, assuming the plaster skins are considered to be individual stand-alone compression members, the total stress at the extreme fibres of the plaster skin can be calculated using Equation 2 below:

\[
\sigma = \frac{P}{A} + \frac{Pe y}{I}
\]

where \(\sigma\) = extreme fibre stresses; \(P\) = applied load; \(A\) = area of plaster upon which load is applied; \(e\) = eccentricity at which load is applied; \(y\) = distance to extreme fibre; \(I\) = moment of inertia of the plaster skin. At the ultimate load for this wall, the applied load on one plaster skin was 22,380/2 = 11,190 N, the area of plaster upon which the load was applied was 600 \(\times\) 25 = 15,000 mm\(^2\), the eccentricity at which the load was applied was 1.6 mm, the distance to the extreme fibre was 25/2 = 12.5 mm, and the moment of inertia was 600 \(\times\) 25\(\frac{3}{2}\) = 781,250 mm\(^4\). Thus, the extreme fibre stresses at ultimate failure were 1.032 MPa, and 0.460 MPa.

Given the newly calculated ultimate maximum compressive stress of 1.032 MPa and the previously calculated ultimate stress of 0.746 MPa assuming no buckling, it appears that the initially assumed stress is only about 70–75% of the actual maximum stress reached when buckling is considered. If we assume that this is the case for all six of the completed experiments, Figure 6 can be adjusted to account for buckling. Figure 12 gives a plot of the maximum stress for each of the six experiments assuming that the stresses calculated ignoring buckling are only 75% of the actual maximum stress once buckling is considered. Again, the maximum, minimum, and average compressive plaster strengths are shown on Figure 12. The adjusted values for the maximum stresses for the six experiments are also given in Table 3.

Table 3. Summary of adjusted ultimate stresses.

<table>
<thead>
<tr>
<th>Height (mm)</th>
<th>Ultimate stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>990</td>
<td>0.777</td>
</tr>
<tr>
<td>990</td>
<td>0.880</td>
</tr>
<tr>
<td>990</td>
<td>1.004</td>
</tr>
<tr>
<td>2310</td>
<td>0.989</td>
</tr>
<tr>
<td>2310</td>
<td>1.017</td>
</tr>
<tr>
<td>2310</td>
<td>1.032</td>
</tr>
</tbody>
</table>

Figure 12 indicates that if a maximum eccentricity of 1.6 mm is assumed at failure, the ultimate stresses measured in the plaster skins all lie above the average compressive cube strength for the plaster. Furthermore, four of the six columns appear to have reached stresses greater than the maximum compressive strength found for any of the plaster cubes tested. This indicates that the plastered straw bale walls are performing better than anticipated given the assumption for eccentricity. This could be explained by the fact that during plastering, it was always assured that there was at least 25 mm of plaster, but in most areas there was actually more than 25 mm of plaster as the plaster was worked into the straw.

4.2.6 Failure mode

It is evident from the preceding discussion, that failure of the plastered straw bale walls is most likely as a result of compression of the plaster at one of the extreme fibres of the plaster skin. Exactly how this initial compression failure propagates into a full failure of the plaster skin is not as clear. Figure 13 provides a close up photograph of a typical initial failure cracking pattern. This pattern was seen throughout all six tests for the initial crack at the ultimate failure point.

The angular cracking gives an indication that there may have been a mechanism of shear involved in the failure of the plastered straw bale wall. It is likely that this shear failure propagated from an initial compression failure of the plaster at the extreme compression fibre. For the case of Figure 13, this fibre is located closest to the straw. Figure 14 describes a number of likely steps which lead to the failure observed in Figure 13.

Prior to failure, the plaster skin is experiencing eccentric loading. This is shown as “pre-failure” in Figure 14. This loading causes an increase in compressive stress at the right hand side of the plaster skin (closest to the straw in Figure 13). Eventually this stress exceeds the capacity of the plaster, and a portion of the plaster skin fails in compression. This essentially removes a portion of the plaster from the skin “column” as shown in “step 1” of Figure 14. With this section of the plaster skin removed, the load on the wall continues to attempt to drive the plaster
above and below the gap together. Eventually shear failures develop as represented by the dashed lines in “step 2” of Figure 14. The plaster fails along the shear planes and a section of plaster is broken off of the skin as shown in “post-failure” of Figure 14. The initial space created by the compression failure is then closed as the plaster above the failure is pressed down toward the plaster below the failure. This description of the failure is supported by the locations of the cracks observed in Figure 13, and was noted for all six plastered wall experiments.

5 CONCLUSIONS

The experiments conducted on the six plastered straw bale walls and the plaster used for those walls highlighted a number of important findings related to the compressive performance of plastered straw bale wall assemblies:

1 For the lime-cement plaster used in the experiments, the ultimate compressive cube strength varied significantly from 0.606 MPa to 0.963 MPa with an average value of 0.764 MPa.

2 Calculated assuming pure compression failure, the stresses which existed in the plaster at ultimate failure of the bale walls varied from 0.562 MPa to 0.746 MPa. These values were about 80% of the average compressive cube strength for the plaster.

3 Calculated considering buckling of the plaster skins, the stresses which existed in the plaster at ultimate failure of the bale walls varied from 0.777 MPa to 1.032 MPa. These values are all slightly greater than the average compressive cube strength for the plaster.

4 The plastered wall modulus was found to vary significantly for all of the experiments. The average modulus determined from previous individual bale experiments, and the six experiments discussed in this paper, was found to be 218 MPa.

5 A non-linear stress-strain model for concrete was used to model the stress-strain behaviour of the plastered straw bale walls. The model provides a good representation of the behaviour of the walls up to ultimate failure using plaster cube strengths in addition to typical values for the plastered wall modulus.

6 The failure of the walls appeared as a compression failure of the plaster as a result of increased stresses from global buckling. This initial compression failure then lead to the appearance of shear cracks in the plaster, and eventually the loss of a portion of the plaster skin, corresponding to a significant drop in the load resistance.

7 Although PIV provided evidence of buckling of the plastered straw bale walls. It was found that the wall height (an important factor for buckling) did not appear to impact the ultimate load found for the walls. Further experimentation using PIV appears warranted.

6 RECOMMENDATIONS FOR FUTURE WORK

The observations of the lateral displacements and their effect on the behaviour of the plastered straw bale walls has provided indication of the need for further
work in studying the global buckling phenomenon in plastered straw bale walls. PIV could be used for further experiments on plastered walls of varying height, which are loaded in compression. Such experiments would provide a greater understanding of the buckling that exists in plastered straw bale walls, and how this buckling affects the performance of the walls.

In addition, similar experiments may be conducted on plastered straw bale walls which are constructed using techniques common on construction sites. These walls would typically have less uniform plaster thickness and a greater degree of wall out-of-straightness. It is important to have an understanding of the impact that these common imperfections may have on the performance of plastered straw bale walls.

REFERENCES


Fibrehouse Ltd and Scanada Consultants Ltd. 1996. Developing and Proof-Testing the ‘Prestressed Nebraska’ Method for Improved Production of Baled Fibre Housing. Canada Mortgage and Housing Corporation (CMHC), Ottawa, ON.


Author index

Abali, Y. 589
Abaza, O.A. 675
Abdel-Razek, M.M. 427
Abdul-Jabbar, H. 753
Abe, O. 333
Al-Daffaee, H. 753
Al-Gadhib, A.H. 457
Al-Tabbaa, A. 397
Ampadu, K.O. 319
Anderson, D.B. 137
Augonis, A. 503
Azhari, A. 779, 785

Al-Baluch, M.H. 457
Belie, N.D. 411
Bhayani, B.V. 375
Bleziffer, J. 487
Bolivar, J.C. 339
Bondar, D. 313
Botic, S. 527
Brough, A.R. 367
Burdette, E.G. 237
Butou, Y. 147
Byars, E.A. 121
Bydžovský, J. 265

Cann, G.M. 189
Canpolat, F. 325, 589
Cardenas, H.E. 603
Carley, S.W. 645
Carsana, M. 367
Chan, B.K.C. 623
Chapman, D.N. 215
Cheeseeman, C.R. 287, 763
Chien, E.A. 717
Chou, C.Y. 107
Christiansen, M.U. 515
Cizer, O. 611
Claisse, P.A. 11, 189, 203, 209, 753
Cope, J. 575
Corinaldesi, V. 59, 69
Coventry, K.A. 657
Cox, K. 411

Daukšys, M. 85
Davoodi, M.G. 271
de Jong, T.P.R. 27
de Wilde, P. 697

Deathage, J.H. 237
Denzer, A.S. 745
Diggelman, C. 527, 725
Donatello, S. 763
Dubasi, P. 603
Dudeney, A.W.L. 623
El Sayad, H.I. 427, 437
Elsen, J. 611
Emery, S.B. 575
Erdem, T.K. 279
Erdoğan, T.Y. 279
Farahi, E. 359
Fava, G. 295
Feenstra, L. 27
Firth, J.P. 367
Fowler, D.W. 101
Freer-Hewish, R. 685
Fujimoto, S. 769
Fujita, K. 303
Fujiwara, H. 303
Fukuzawa, K. 333

Ganjian, E. 11, 203, 209
Ghataora, G. 215, 685
Gomà, F. 493
Gong, X. 183
Gong, Y. 183
Goodhew, S. 697
Goss, D.C. 521
Greaves, R. 203
Griffiths, J. 657
Griffiths, R. 697
Grinys, A. 85
Guangcheng, L. 115
Gutiérrez, V. 339

Haddad, H.S. 427
Heathcote, K.A. 541
Hedges, K.E. 745
Helene, P. 45, 223
Holsen, T.M. 375
Hsu, Y.S. 645
Huang, J.S. 107

Ideris, Z. 41
Ishii, M. 147
Jamieson, E. 271
Jansen, T. 137
Jiebin, Z. 417
Johansson, S. 575
Jones, T.R. 367
Kandie, B.K.T. 121
Karadelis, J.N. 417
Karamis, S. 203, 209
Katzer, J. 91
Khan, A. 417
Kharazi, M.M.A. 479
Khestl, F. 265
Kirkpatrick, J. 705
Kitamoto, Y. 665
Koibuchi, K. 303
Koutselas, K. 417
Kramer, K.W. 549
Kumanlioglu, A.A. 589
Kuo, W.Y. 107
Kurian, V.J. 41
Kwak, D. 333

Lampo, R.G. 717
Levy, S. 45
Liska, M. 397
Long, A.E. 705
Lorimer, J.P. 189, 753
Loveridge, F.A. 645
Lupo, R. 763
Lynsdale, C.J. 313

Ma, X. 183
MacDougall, C. 789
Maghsoudi, A.A. 447, 471
Maher, J.E. 549
Maldonado, N.G. 223
Mannan, M.A. 41
Marroccoli, M. 389
Maruoka, M. 303
Matulova, P. 231
McKay, D.T. 563
McLean, S.N. 593
Mehrdad, M.A. 779
Meral, C. 279
Meyer, C. 509
Michalcova, G. 231
Milestone, N.B. 313
Millard, S.G. 77
Modaraei, A.H. 785
Mokal, M. 131

801
Momtazi, A.S. 779, 785
Monosi, S. 59
Moore, G. 541
Moore, D.R. 593
Moriconi, G. 35, 59, 69
Morohashi, N. 63
Moscicka, K. 417
Motavaselian, M. 471
Mowry, N.D. 563, 717
Mulder, E. 27
Muynck, W.D. 411
Naik, T.R. 19, 631
Nakagawa, T. 165
Napier, T.R. 563, 717
Narayanan, S.P. 41
Neithalath, N. 375
Ng, C.H. 41
Nia, K.A. 779, 785
Nobili, M. 389
Noguchi, T. 769
Oh, B. 255
Ohno, Y. 165
Ojima, H. 333
O’Brien, A.S. 645
Page, C.L. 367
Page, M.M. 367
Park, S. 255
Paturi, P. 603
Persson, B. 461
Petranek, V. 231
Plachta, A. 417
Plunk, G.C. 521
Pour, S.M. 447
Pouya, H.S. 203, 209
Purnell, P. 359
Quiroga, P.N. 101
Radic, J. 487
Rahgozar, R. 471
Rahman, M.K. 457
Rahman, W. 215, 685
Ramezanianpour, A.A. 313
Rame, B.W. 137
Razza, A. 457
Reddy, D.V. 339
Rots, J.G. 731
Ruello, M.L. 295
Sadeghi-Pouya, H. 11
Sakai, K. 1
Sakurada, T. 63
Saleh, Z.A. 339
Salem, R.M. 237
Sani, D. 295
Sarhat, S.R. 53
Sasaki, K. 147
Savor, Z. 487
Schnöwälder, J. 731
Sear, L.K.A. 155
Seki, H. 353
Seo, T. 165
Shao, Y. 381
Shaw, S.J. 367
Short, N.R. 359
Silva, M.R.Q. 631
Simmons, T. 697
Skrpiččius, G. 85
Smith, D.N. 575
Smith, K.J. 717
Soutsos, M.N. 77
Sugino, T. 353
Tan, T.E. 107
Tang, K. 77
Targan, S. 325
Tawil, A. 137
Taylor, S.E. 705
Telesca, A. 389
Tittarelli, F. 35
Tokay, M. 279
Tolman, T.S. 717
Torri, K. 319
Torkaman, J. 741
Tyrer, M. 763
Valenti, G.L. 389
Van Balen, K. 611
Van Gemert, D. 611
Vandeperre, L.J. 397
Vandivort, T.F. 171
Vardy, S. 789
Venables, R.K. 705
Verstraeet, W. 411
Vlasopoulos, N. 287
Voandoagu, J.J. 319
Vruno, D.M. 243
Williams, A. 593
Yilmaz, K. 325
Yoo, J. 255
Yoshida, H. 333
Yoshida, T. 665
Youjun, X. 115
Yunhua, L. 115
Yuno, K. 147
Yurdusev, M.A. 325, 589
Zeybek, M.S. 589
Zhou, X. 381
Ziemkiewicz, P.F. 171
Ziliukas, A. 503
The construction materials industry is a major user of the world’s resources. While enormous progress has been made towards sustainability, the scope and opportunities for improvements are significant.

To further the effort for sustainable development, a conference on Sustainable Construction Materials and Technologies was held at Coventry University, Coventry, U.K., from June 11th - 13th, 2007, to highlight case studies and research on new and innovative ways of achieving sustainability of construction materials and technologies. This book presents selected, important contributions made at the conference. Over 190 papers from over 45 countries were accepted for presentation at the conference, of which approximately 100 selected papers are published in this book. The rest of the papers are published in two supplementary books.

Topics covered in this book include: sustainable alternatives to natural sand, stone, and portland cement in concrete; sustainable use of recyclable resources such as fly ash, ground municipal waste slag, pozzolan, rice-husk ash, silica fume, gypsum plasterboard (drywall), and lime in construction; sustainable mortar, concrete, bricks, blocks, and backfill; the economics and environmental impact of sustainable materials and structures; use of construction and demolition wastes, and organic materials (straw bale, hemp, etc.) in construction; sustainable use of soil, timber, and wood products; and related sustainable construction and rehabilitation technologies.