Center for By-Products Utilization

IN-SITU RECYCLING OF DETERIORATED ASPHALT CONCRETE FOR NEW PAVEMENT BASE COURSES

By Bruce W. Ramme, Haifang Wen, Tarun R. Naik, and Rudolph N. Kraus

Report No. CBU-2005-21
REP-592
October 2005

Presented and Published in the Proceedings of 13th Great Lakes Geotechnical and Geoenvironmental Conference, GEO Institute, ASCE, May 2005.

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ABSTRACT

Class C fly ash is a coal combustion product normally produced from lignite or sub-bituminous coal obtained as a result of the power generation process. In recent years, efforts were taken to incorporate self-cementing fly ash into full-depth reclaimed (FDR) asphalt pavements to improve the structural capacity of asphalt pavement base layers. In this study, an existing asphalt pavement in County Trunk Highway (CTH) "JK" in Waukesha County, Wisconsin was pulverized in place and mixed with fly ash and water to function as a base course. To evaluate the contribution of fly ash to the structural performance of the pavement, nondestructive deflection tests were performed using a KUAB 2M falling weight deflectometer (FWD) on the outer wheel path four days, one year, and two years after construction. The modulus of the fly ash stabilized FDR base course increased by 49% one year after construction, and by 83% two years after construction. The structural capacity of the fly ash stabilized FDR base course in CTH "JK" also increased significantly as it ages, due to the pozzolanic and cementitious reactions. The results of this study indicate that the FDR mixtures with self-cementing fly ash can provide an economical method of recycling flexible pavements and reduce the need for expensive new granular base courses for road reconstruction.

Keywords: fly ash, coal combustion products, pavement recycling, asphalt, full depth reclamation, self cementing, highway, structural number stabilization, and in situ recycling.

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INTRODUCTION

The Full-depth Reclamation (FDR) of asphalt pavements process consists of in situ pulverization of the existing asphalt layer along with the existing aggregate base and sometimes subgrade soils to form a base for a new asphalt overlay. FDR is becoming a widely used rehabilitation technique by highway agencies. There are two reasons: (1) compared to conventional methods, such as milling and patching, FDR is a more effective way to eliminate the reflective cracking of asphaltic overlays and improve the pavement base condition; and (2) FDR utilizes 100% of the existing materials and is a cost-effective technique, considering the shortage of available quality aggregates and more stringent environmental regulations. FDR projects have been performed successfully since the early 1980's in states such as Kansas, Oregon, California and New Mexico.

The pulverized materials without additives could be used as pavement base course after compaction and grading [1, 2]. However, additives are often used to stabilize the pulverized base course. The available additives in common practice include water, cement, asphalt emulsion, hydrated lime, and fly ash [3, 4, 5].

Class C fly ash, a coal combustion product from lignite or sub-bituminous coal obtained as a result of the power generation process, has been used extensively over a wide range of construction applications. Its self-cementitious and pozzolanic properties are valuable in developing strength of concrete or other mixtures containing fly ash. Each year, approximately 68 million tons of fly ash are produced in the U.S.A [6]. About 46 million tons were placed in landfills resulting in significant land purchase costs, landfill costs, and potential environmental issues. It has been reported that some FDR mixes with asphalt emulsion were unstable [4] and cement stabilized base course was prone to cracking [5]. Therefore, it is environmentally friendly and cost-effective to utilize self cementing Class C fly ash to stabilize FDR mixes that have not exhibited these deficiencies. Cross et al. [3] studied the use of Class C fly ash to stabilize Cold In-place Recycled base course and concluded fly ash could be a viable additive. However, Cross et al. [4] also reported that the use of high fly ash application rate could result in the tendency of pavement to crack.

Since Class C fly ash is a pozzolanic material, when compared to Portland cement, mixtures with fly ash have a long-term strength development process. Use of appropriate fly ash contents to stabilize FDR mixes can reduce the brittle behavior of the base course and still provide enough support for the long-term performance of asphalt overlays. Therefore, this study investigated the stabilization of FDR mixes with relatively low fly ash contents in a road with moderate traffic volume. This study is evaluating the strength development and multiple-year field performance of a Class C fly ash stabilized FDR pavement base course in Wisconsin.

Project Description

County Trunk Highway (CTH) JK in Waukesha, Wisconsin was selected as a test section. It was reconstructed in October of 2001, using FDR materials as pavement base course. Fly ash was used to stabilize the FDR pavement base course in place for CTH JK. CTH JK is located in Waukesha County, Wisconsin and the project segment runs between CTH KF and CTH K, with a project length of 1,009 m (3,310 ft.). It is a two-lane road with an average daily traffic (ADT) count of 5,050 vehicles in year 2000 and a projected ADT of 8,080 in design year 2021. The existing pavement structure consisted of approximately a 127 mm (5") asphalt concrete surface layer and a 178 mm (7") granular base course. The new pavement structure consists of a 127 mm (5") asphalt concrete layer and a 305 mm (12") Class C fly ash stabilized FDR base course. The truck percentage on CTH JK in 2000 was 5%.

Laboratory Experiment and Construction

A laboratory mix analysis to evaluate the stabilization potential of pulverized pavement material with Class C fly ash was conducted. A field sample of the existing asphalt pavement and underlying aggregate base was obtained. The results of the grain size analysis on the pulverized materials indicated a sand and gravel mixture with trace fines. The analysis showed that the sample contained 68% gravel size particles (larger than #4 sieve), 26% sand size particle (between #4 and #200 sieves), and 6% silt size (between #200 sieve and a size of 0.005 mm) and clay size (between 0.005 mm and 0.001 mm) particles. Based on published technical literature [3, 4, 5], a laboratory evaluation of fly ash stabilized FDR material was performed at two fly ash contents, 6 and 8% by dry weight of FDR materials. These two relatively low application contents were selected to reduce the potential of cracking of the stabilized mixtures. Laboratory analysis of the fly ash stabilized materials was conducted in accordance with ASTM C593. Moisture-Density (ASTM D1557) and Moisture-Strength (ASTM D1633) relationships of specimens compacted in a 101.6mm (4”) diameter mold were obtained. Results of the moisture density relationship tests on the pulverized asphalt pavement indicated a maximum dry density of 2.27 g/cm^3 (141 p/c.f.) at an optimum moisture level of 5.0%. In addition, the moisture density relationship tests on the recycled asphalt pavement material with 6 and 8% fly ash indicated maximum dry densities of 2.28 and 2.29 g/cm^3 (142 and 143 p/c.f.) at optimum moisture contents of 5.5%, respectively. Maximum unconfined compressive strengths of 1.72 MPa (250 psi) and 2.62 MPa (380 psi) at optimum moisture contents of 5% were obtained after seven day curing for 6% and 8% application rates, respectively.

The pulverized mixes were compacted and graded to form the base for a 127 mm (5”) thick new asphalt overlay. Construction of CTH JK consisted of pulverization, application of 8% fly ash and 5% water, compaction and grading, and placement of the new asphalt overlay. For detailed construction procedures, the readers are referred to reference 7.

Field Performance Evaluation

The nondestructive deflection testing is one of the primary techniques for determining the in situ structural capacities of pavement. The 1993 *AASHTO Guide for Design of Pavement Structures* [8] describes Falling Weight Deflectometer (FWD) testing as a means of evaluating the conditions of existing pavement. The 2002 *Guide for Mechanistic Pavement Design* also proposes the use of FWD for existing pavement evaluation [9]. In this study the FWD was used to evaluate the performance of fly ash stabilized FDR.

KUAB 2M Falling Weight Deflectometer (FWD) tests were conducted to evaluate the field performance of CTH JK. The impact load used in this study was approximately 40KN (9000 lbs). The pavement surface deflections were recorded by seven sensors located at 0, 0.3 m (12”), 0.46 m (18”), 0.61 m (24”), 0.91 m (36”), 1.22 m (48”), and 1.52 m (60”) from the center of loading plate. FWD tests were conducted on CTH JK four days after construction (2001), one year after construction (2002), and two years after construction (2003). The FWD tests were performed at an interval of 30.5 m (100 ft).

Deflection

The pavement deflections under the impact load indicate the structural capacities of existing pavement, including subgrade, base course and surface layer. Deflections may be either correlated directly to pavement performance or used to determine the in situ material characteristics of the pavement layers.

The average deflections measured by the sensors are shown in **Table 1**. The average deflections measured in year 2002 and 2003 were significantly lower than those in year 2001; the deflections in 2003 were slightly higher than those in 2002. The improvement was partly due to the fact that the pavement surface temperature was 16.7°C (62°F) in 2001, 7.2°C (45°F) in 2002, and 8.3°C (47°F) in 2003 at the time of testing. The study of flexible pavements in Texas by Chen et al. [10] shows that only the deflections at a radial distance of 0 and 203 mm (8”) are significantly affected by temperature. Kim et al. [11], however, concluded that the radial distance affected by temperature, is dependent on the thickness of asphalt concrete layer. The effective radial distance for deflection affected by temperature was calculated using the following relationship [11]:

$$Deff = 4.75\ Hac - 413 \tag{1}$$

Where: $Deff$ = effective radial distance affected by temperature, mm, and
Hac = Asphalt Concrete (AC) layer thickness, mm.

In this study, the thickness of the AC layer is 127 mm (5”). Therefore, the effective radial distance affected by temperature is about 190 mm (7.5”). Only the deflections measured at the center of loading plate, D_0, were affected by the temperature in this study. To meaningfully compare the pavement behavior in 2001, 2002 and 2003, the deflections have to be corrected to the reference temperature of 20°C (68°F) in this study. It is well known that the measured pavement surface temperature has to be corrected to that at mid-depth of the asphalt concrete layer to be representative of the effective temperature. The 1993 *AASHTO Guide for Design of Pavement Structures* presents the temperature correction procedure. However, it has been reported that that temperature correction method is inaccurate and impractical [12]. Kim et al. [12] developed the temperature correction method, based on the data collected in North Carolina. Another method was developed by Harichandran et al. [13] for flexible pavement in Michigan. The method which has been developed based on the national, instead of regional data, is BELLS3 by Lukanen et al [14]. Lukanen et al. used the data collected in the Long-term Pavement Performance (LTPP) program. This study examined the above temperature correction methods. The results of temperature correction for CTH JK pavement are shown in **Table 2**. The corrected temperatures at the middle of the asphalt layer using the North Carolina method were higher than the measured pavement surface temperatures; while the other two methods, Michigan method and the BELLS3 method, yielded an opposite trend. It was seen that the effective temperatures from the Michigan method, 15.3°C (59.54°F) in 2001, 5.8°C (42.4°F) in 2002 and 6.9°C (44.4°F) in 2003, were close to those from BELLS3 method, 13.6°C (56.5°F) in 2001, 5.2°C (41.4°F) in 2002 and 6.3°C (43.3°F) in 2003. Considering the climatic similarity between Michigan and Wisconsin, it was decided to use the corrected temperature from the Michigan method.

The next step is to correct the measured deflections at the effective temperature at the middle of the asphalt layer to those at the reference temperature of 20°C (68°F). Several deflection correction methods were developed by Kim et al. [11], Chen et al. [10], and Lukanen et al. [14], using data from pavements in North Carolina, in Texas, and LTTP, respectively. The deflection correction coefficients from the above methods vary significantly, as shown in **Table 3**. Again, considering the climatic difference between Wisconsin and North Carolina or Texas, it was decided to use the method developed by Lukanen et al.

The deflection curves in 2001 and 2002, after the temperature correction, are presented in **Figure 1**. The deflections in 2003 are fairly close to those in 2002, after temperature correction. It was noted that the deflections measured by sensor located at 1.52 m (60”) from the center of loading plate, $D_{1.52}$, in 2002 and 2003 were slightly lower than those in year 2001. Knowing that $D_{1.52}$ is the measurement of the deflection of only the subgrade, it is believed that the slightly reduced deflection of subgrade might be ascribed to further compaction of subgrade by traffic. It appears that the corrected deflections in 2002 and 2003 were significantly lower than those in 2001. Therefore, it is inferred that that the fly ash stabilized cold in-place recycled asphalt base course gained strength significantly between the time of testing in 2001 and in 2002 and 2003, due to pozzolanic and cementitious reactions, and thus reduced the pavement deflection under the loading.

Backcalculation of Layer Modulus

The measured deflection data was used to backcalculate the properties of each pavement layer. Modulus 5.1 and Michback programs were used in the preliminary analysis. It was found that both programs yielded close results. It was decided to continue the backcalculation using only the Michback program. The average moduli of the materials in the asphalt layer, the fly ash stabilized base course and the subgrade were 6.68GPa (968ksi), 1.24GPa (180ksi), and 0.1GPa (14.5ksi), respectively, at the time of testing in 2001; 25.3GPa (3,670ksi), 1.84GPa (267ksi) and 0.1GPa (14.5ksi) in 2002; and 14.3GPa (2,074ksi), 2.27GPa (329ksi), and 0.11GPa (16.0ksi) in 2003, respectively. The modulus of fly ash stabilized FDR base course increased from 1.24GPa (180ksi) in 2001, to 1.84GPa (267ksi) (by 49%) in 2002, and to 2.27GPa (329ksi) (by 83%) in 2003. The results indicated that the structural capacity of the fly ash stabilized CIR recycled asphalt base course developed significantly within two years after construction. This is due to the pozzolanic and cementitious reactions in the mixes containing Class C fly ash, as stated above.

Structural Number

The structural number of the pavement was backcalculated from surface deflection, as follows [15]:

$$SN = \left[1.49 \times (ET)^3\right]^{1/3} \tag{2}$$

$$Log_{10}(ET)^3 = 5.03 - 1.309 Log_{10}(AUPP) \tag{3}$$

$$AUPP = \frac{1}{2}(5D_0 - 2D_{0.3} - 2D_{0.61} - D_{0.91}) \tag{4}$$

where: SN = structural number of pavement, mm,
ET^3 = flexural rigidity of pavement, mm,
AUPP = area under the pavement profile, mm, and
D_i = surface deflection, mm.

Figure 2 shows the structural number SN at individual test locations in year 2001, 2002, and 2003, obtained from the corrected deflections. The increased SN indicates that the structural capacity of CTH JK pavement was improved significantly. The structural coefficient for fly ash stabilized CIR material, α_2, was calculated as follows:

$$\alpha_2 = \frac{SN - \alpha_1 h_{HMA}}{h_{base}} \tag{5}$$

Where: h_{HMA} = thickness of HMA layer, mm, and
h_{base} = thickness of base course, mm.

There are two approaches to backcalculate the structural coefficient of Class C fly ash stabilized FDR base course. The first approach uses the structural number from corrected deflections and the structural coefficients from the corrected asphalt concrete layer modulus at reference temperature, 20°C. The other is based on the structural number from measured deflections and structural coefficients from backcalculated asphalt concrete layer modulus without correction.

In the first approach, backcalculated modulus of the asphalt layer at test temperature was reduced to that at reference temperature. Four correction methods of asphalt layer modulus were examined in this study. They are developed by Kim et al. [16], Chen et al. [10], Harichandran et al. [13], and Lukanen et al. [14]. The results for modulus temperature correction were shown in **Table 4**. It is seen that the corrected moduli vary significantly. The results from Harichandran et al. and Lukanen et al. are significantly larger than those from Kim et al. and Chen et al., especially at low temperature. The corrected moduli in 2002 and 2003 were not comparable to those in 2001. The reason for the significant discrepancy might be: (1) inaccuracy of modulus correction methods; (2) aging of asphalt concrete materials; and (3) further densification of asphalt concrete layer under traffic. Without knowing the exact reason, it was decided to use the second approach to backcalculate the structural coefficient of the base course containing fly ash.

The structural coefficient of asphalt concrete layer was calculated based on the 1993 *AASHTO Guide for Design of Pavement Structures*, as follows:

$$\alpha_1 = 0.40 \times \log\left(\frac{E}{3000Mpa}\right) + 0.44 \qquad (6)$$

where E is the laboratory resilient modulus of asphalt concrete, Pascal.

According to 1993 *AASHTO Guide for Design of Pavement Structures*, the backcalculated layer modulus of asphalt concrete could be up to three times higher than the resilient modulus obtained in the laboratory. Therefore, the backcalculated modulus value of the asphalt layer was converted into resilient modulus and was input in Equation 6. The structural coefficient obtained form Equation 6 was input in Equation 5.

A structural coefficient of 0.16 was obtained for the fly ash stabilized base course in CTH JK at the time of testing in 2001, 0.23 in 2002, and 0.245 in 2003. Since layer coefficient is a measure of the relative ability of the material to function as a structural component of the pavement, the increase of layer coefficient indicates the improvement of structural capacity of the fly ash stabilized FDR base course in CTH JK. From the standpoint of structural number only, the increase of structure number will result in an allowable traffic increase of 130%.

Distress Survey

Condition surveys were conducted to assess the physical condition and distress buildup of the pavement in CTH JK one year and two years after the construction. Of particular interest in these surveys was to identify the possible reflective cracking that was due to the contraction of base course that propagates through the asphalt layer. As seen in **Figures 3 and 4**, no cracking was observed in the pavement of CTH JK.

The rutting of pavement was surveyed in the inner wheel path of pavement in CTH JK. The rut depth was measured using a straightedge and a gage. It was found that the no rutting happened in pavement of CTH JK. To better visualize the rutting, water was placed on the pavement surface of CTH JK. No accumulation of water was observed within the wheel path, indicating that the materials under the asphalt concrete layer provide a strong support to the load of traffic.

CONCLUSIONS

FDR is becoming a widely used rehabilitation technique by highway agencies. It is also environmentally friendly and cost-effective to utilize Class C fly ash to stabilize FDR pavements. CTH JK in Waukesha, Wisconsin was selected as a test section. A laboratory mix analysis to evaluate the stabilization potential of recycled pavement material with Class C fly ash was conducted. Pavement performance of CTH JK was evaluated using the FWD test after construction, one year, and two years after construction. The modulus of fly ash stabilized FDR base course increased from 1.24GPa (180ksi) in 2001, to 1.84GPa (267ksi) (by 49%) in 2002, and to 2.27GPa (329ksi) (by 83%) in 2003. A structural coefficient of 0.16 was obtained for the fly ash stabilized base course in CTH JK at the time of testing in 2001, 0.23 in 2002, and 0.245 in 2003. The increase of layer coefficient indicates the improvement of structural capacity of the fly ash stabilized FDR base course in CTH JK. It is believed this is due to the long-term pozzolanic and cementitious reactions in the mixtures containing Class C fly ash. No distresses occurred in CTH JK two years after construction. The results of this study at this stage indicate that the CIR stabilization with self-cementing fly ash provide a sustainable economical method of recycling flexible pavements and reduce the need for expensive new granular base courses for road reconstruction.

ACKNOWLEDGMENTS

We would like to thank Mr. Tom Kiernan of Lafarge North America for the support we received. Our special thanks go to Mr. Steve Krebs, Mr. Michael Malaney, Ms. Colleen Coolly, and Mr. Craig Villas of Wisconsin Department of Transportation who helped us by providing the FWD testing for this project.

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TABLES AND FIGURES

List of Tables:

Table 1 - Average Deflection Measurements in Year 2001 and 2002
Table 2 - Temperature Correction of CTH JK Pavement
Table 3 - Deflection Correction Coefficient of CTH JK Pavement
Table 4 - Temperature Correction of Asphalt Concrete Modulus

List of Figures:

Figure 1 - Pavement Deflection from FWD Test
Figure 2 - Structural Number of Pavement in CTH JK
Figure 3 - CTH JK Pavement in 2002
Figure 4 - Distress Survey of Pavement in CTH JK in 2003

Table 1–Average Deflection Measurements in Year 2001, 2002 and 2003

		D_0, mm	$D_{0.3}$, mm	$D_{0.46}$, mm	$D_{0.61}$, mm	$D_{0.91}$, mm	$D_{1.22}$, mm	$D_{1.52}$, mm
Year 2001	Mean	0.234	0.177	0.152	0.129	0.099	0.074	0.056
	Std. Dev.	0.045	0.037	0.031	0.027	0.023	0.019	0.016
	C.V. (%)	19.3	20.6	20.7	21.1	23.2	25.5	27.9
Year 2002	Mean	0.138	0.115	0.106	0.094	0.075	0.062	0.049
	Std. Dev.	0.029	0.024	0.022	0.020	0.017	0.015	0.013
	C.V. (%)	19.6	19.9	19.5	19.4	20.6	22.3	24.2
Year 2003	Mean	0.151	0.125	0.110	0.098	0.080	0.065	0.051
	Std. Dev.	0.028	0.025	0.022	0.020	0.017	0.015	0.012
	C.V. (%)	18.5	19.8	20.0	20.2	21.7	23.1	24.3

Table 2–Temperature Correction of CTH JK Pavement

Correction Method by		Kim et al.	Harichandran et al.	Lukanen et al. (BELLS3)
Data Source		North Carolina	Michigan	LTPP
Measured	16.6°C (2001)	19.2°C	15.3°C	13.6°C
	7.2°C (2002)	9.1°C	5.8°C	5.2°C
	8.3°C (2003)	10.3°C	6.9°C	6.3°C

Table 3–Deflection Correction Coefficient of CTH JK Pavement

Correction Method by		Kim et al.	Chen et al.	Lukanen et al.
Data Source		North Carolina	Texas	LTPP
Correction Coefficient	2001	1.08	1.28	1.14
	2002	1.26	2.87	1.42
	2003	1.24	2.48	1.39

Table 4–Temperature Correction of Asphalt Concrete Modulus

Correction Method by		Kim et al.	Harichandran et al.	Chen et al.	Lukanen et al. (BELLS3)
Data Source		North Carolina	Michigan	Texas	LTPP
Measured	6.68GPa (2001)	4.9GPa	5.3GPa	4.8GPa	5.4GPa
	25.3GPa (2002)	10.3GPa	12.3GPa	8.0GPa	13.4GPa
	14.3GPa (2003)	6.23GPa	7.9GPa	3.3GPa	7.4GPa

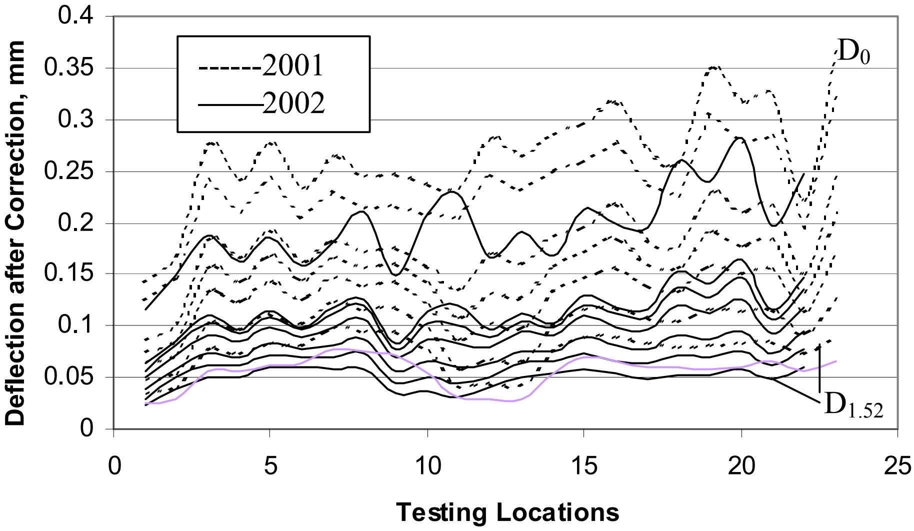

Figure 1–Pavement Deflection from FWD Test after Temperature Correction

Figure 2–Structural Number of Pavement in CTH JK after Temperature Correction

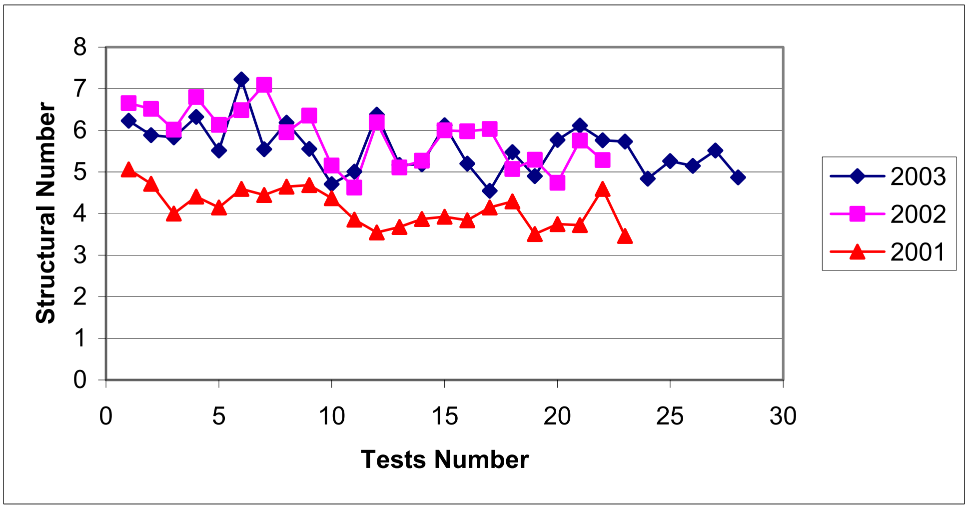


Figure 3–CTH JK Pavement in 2002


Figure 4. CTH JK Pavement in 2003