Resilient Moduli and Structural Layer Coefficient of Flyash Stabilized Recycled Asphalt Base

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ABSTRACT

A proven method of converting conglomerate pavement sections into durable roads is by recycling old asphalt pavement and base course into new roadbase using a process called Full-Depth Cold In-Place Recycling (CPR). In CPR, existing asphalt pavement sections are pulverized in-place to full pavement depths, mixed with fly ash and appropriate water content and compacted into a roadbase in a single process. A demonstration of this process on approximately 2.5 miles of roadway consisting two different segments was conducted in August 2004. Subsequently, the falling weight deflectometer (FWD) tests were conducted at different locations on these roads after 1.5-year of project completion. In this paper we present an analysis of the FWD data to obtain the resilient moduli and structural layer coefficients of the stabilized roadbases.

INTRODUCTION

County and local roads account for nearly 3 million miles of all roadways in the United States. These roads often consist of what are called low-volume flexible or built-up pavements, best described as conglomerates of base course and wearing course of chip seal or other asphalt overlays that have been built up over many years.[1,2] Performance of these roads is typically inconsistent because of the diversity of materials used in their construction. In addition, many of these roads often go without any maintenance at all because of budgetary constraints.

Full-Depth Cold In-Place Recycling (CPR) with self-cementing fly ash has been shown to be an effective method of converting conglomerate pavement sections into durable roads. Cold in-place recycled pavement and base course (CPR) is similar to subgrade stabilization, in that self-cementing fly ash is used as the cementing agent to stabilize granular materials. CPR demonstration projects have been performed in several states, where it has been shown to produce longer road life at a major saving. This technology provides several economic and environmental benefits. The process recycles deteriorated asphalt pavement and self cementing fly ash, reduces energy consumption, diesel emissions, land disposal requirements and virgin resource utilization, and increases road longevity. In addition to providing environmental benefits and long-lasting pavements, it has been reported that this technology could result in savings of up to 33% over conventional techniques.

In CPR, existing asphalt pavement sections in need of repair/rehabilitation may be pulverized in-place to full pavement depths, mixed with self-cementing (Class C) fly ash and appropriate water content and compacted into a base course in a single process. Up to 12 inches of a conglomerate pavement section (or a combination of base material, chip and seal courses, and asphalt overlays) can be pulverized in one pass and then stabilized in a second pass. To complete the process, a new wearing surface is laid on the recycled section.

A demonstration of this process on approximately 2.5 miles of roadway consisting two different segments was conducted in August 2004[3]. Subsequently, the FWD tests were conducted at different locations on these roads after 1.5-year of project completion. The deflection bowls measured in the FWD tests were analyzed to back calculate the resilient moduli of the roadbase. These moduli were utilized to estimate the structural layer coefficient which is a measure of the relative ability of a unit thickness of a material to function as a structural component of the pavement and may be used to calculate the structure number needed for the design of layer thicknesses. The estimated layer coefficients values, which range from 0.18 to 0.26, compare favorably with the AASHTO design requirement of 0.14 for the layer coefficient of flexible pavement granular base.

BRIEF DESCRIPTION OF DEMO PROJECT

A demonstration of the full-depth cold in-place recycling of low traffic volume road asphalt pavement section process on approximately 2.5 miles of roadway was conducted in August 2004 in Jackson County, MO. The following aspects of the CPR process were studied as part of the demonstration project:
(1) Properties of self-cementing fly ash and recycled asphalt pavement (RAB) material.
(2) Mix designs for mixtures of fly ash and RAB based upon laboratory tests.
(3) Construction procedures.
(4) Performance of stabilized RAB as pavement material through field CBR tests.

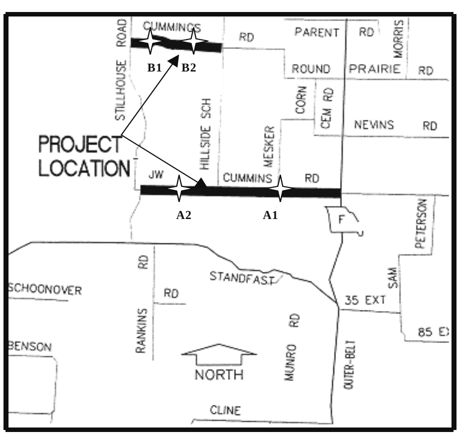

Figure 1. Demonstration project site location map and typical cored sample.

As a result, model construction specifications and pre-construction testing program were developed. In addition, the construction procedures and the monitoring methods were documented and evaluated. Figure 1 gives the location map of the demonstration sites as well as a picture of typical cored samples from the existing deteriorated pavement. Table 1 gives a brief characteristic of the samples.

Table 1. Pavement segment description.

Pavement Segment	Location	Surface Base Thickness, inch	Subgrade Thickness, inch
JW Cummins	A1 A2	1.0” Chip & Seal 1.5” Chip & Seal	5.0” Soil/aggregate Mix 4.5” Soil/aggregate Mix
Cummings	B1 B2	4.0” Asphalt 4.0” Asphalt	3.0” Oily Aggregate 3.0” Oily Soil/Aggregate

The construction procedures were formulated based upon experience from previous recycling and self-cementing fly ash based stabilization projects. The construction procedures emphasized: (1) proper pulverization, (2) uniform distribution and control of fly ash, (3) control of moisture content, (4) proper mixing and blending of fly ash with RAP, (5) final compaction within a prescribed time frame, (6) weather limitations, and (7) finishing and protective cover. Quality control specifications were given in terms of 1) maximum dry density based upon the Standard Proctor compaction test, and 2) moisture range based upon the unconfined compressive strength and the Standard Proctor compaction tests. In order to evaluate the behavior of fly ash stabilized RAB, a DCP with an 8-kg hammer was used during the demonstration project to assess the field CBR values. The DCP tests were performed at the completion of compaction, which was typically 2 hours after the addition of water. DCP tests were also performed at the same locations after 24 to 48 hours of curing. The project was successfully completed and the roadway is being monitored for long term performance. As a part of this monitoring program, FWD tests were conducted at different locations on these roads after 1.5-year of project completion.

FWD FIELD TEST

FWD is a widely used device for non-destructive evaluation of pavement structure mechanical properties. The device is designed to apply a dynamic load on the pavement surface to simulate the force imparted by moving vehicle. The dynamic load is applied by dropping a known weight onto a circular plate to produce a load pulse. The pavement deflection caused by the load pulse is recorded using dynamic deflection sensors. Typically, seven gages are used, which are placed along a straight line in the direction of the traffic flow, at a distance of 0, 8”, 12”, 18”, 24”, 36” and 60” from the location of load application, respectively. The device is placed on a trailer and pulled by a pick-up truck. Thus, FWD test can be performed at several locations along the roadway at predefined intervals.

Falling weight deflectometer (FWD) tests were conducted at different location on J.W. Cummins and Cummings road at approximately 200 feet intervals. Dynatest Model 8082 HWD were used to perform the FWD testing. Two drops of approximately 9000-

lb were applied to the pavement at each FWD test location. Total of 83 locations were tested on the J.W. Cummins road, which were divided into 42 locations on the East-bound lane and 41 locations on the West-bound lane, while 38 locations were tested on the Cummings road, divided into 19 each on the East- and West-bound lanes. Figure 2 shows the deflections measured by the 7 sensors for the East-bound lanes of J.W. Cummins and Cummings roads, respectively, in response to the 2^{nd} load drop. Figure 3 shows the comparison of deflections measured by sensor 1, directly below the applied load, for the East and West-bound lanes of J.W. Cummins and Cummings roads, respectively, as well as the deflections produced in response to drops 1 and 2, respectively. Clearly, consistent results are obtained from two lanes indicating that the constructions process was uniform across the lanes. It may be noted that the RAB stabilization is performed one lane at a time. Furthermore, similar results from 2-drops suggest that the application of load produced recoverable (elastic) deflections, indicating adequate strength of the pavement layers.

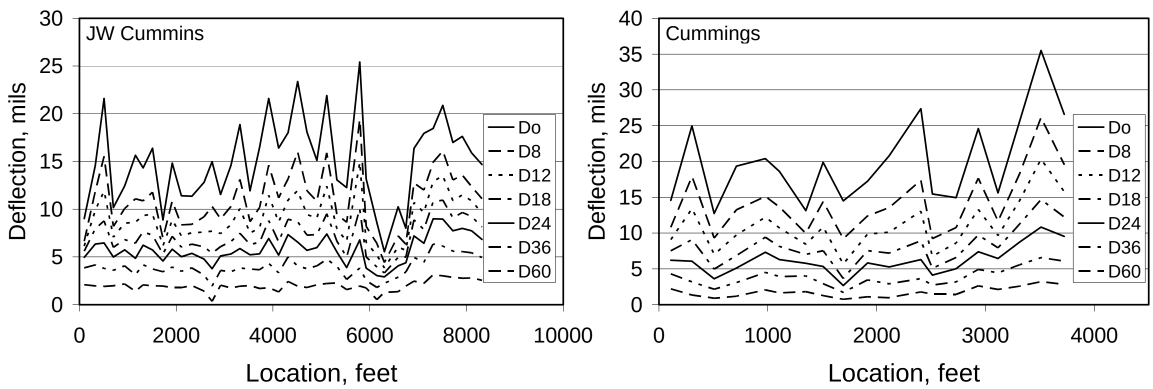

Figure 2. Pavement displacement from FWD tests on East-bound lanes of J.W. Cummins and Cummings roads, respectively.

Base Damage Index (BDI)

According to NCHRP Project 10-48[4] report entitled “Assessing Pavement Layer Condition Using Deflection Data” developed on the basis of FWD results; Base Damage Index (BDI) may be used to assess the condition of base course. BDI is defined as the difference between the deflections of sensors located at 12” and 24” ($BDI=D_{12}-D_{24}$), respectively. For pavement structures composed of asphalt concrete (AC) surface course, granular base course and sub-grade, the report suggests that $BDI \geq 5.8$ mils indicates a poor condition. This criterion was developed based upon the analysis of a database of measured pavement performance of aggregate base pavements with typically >4 inch AC layer. In Figure 4, we show the BDI for the J.W. Cummins and Cummings roads, respectively. The average BDI is found to be 3.4 mils for J.W. Cummins pavement, and 4.6 mils for the Cummings Road pavement. Thus, the BDI criterion for aggregate base pavements is satisfied at most locations on the two road sections.

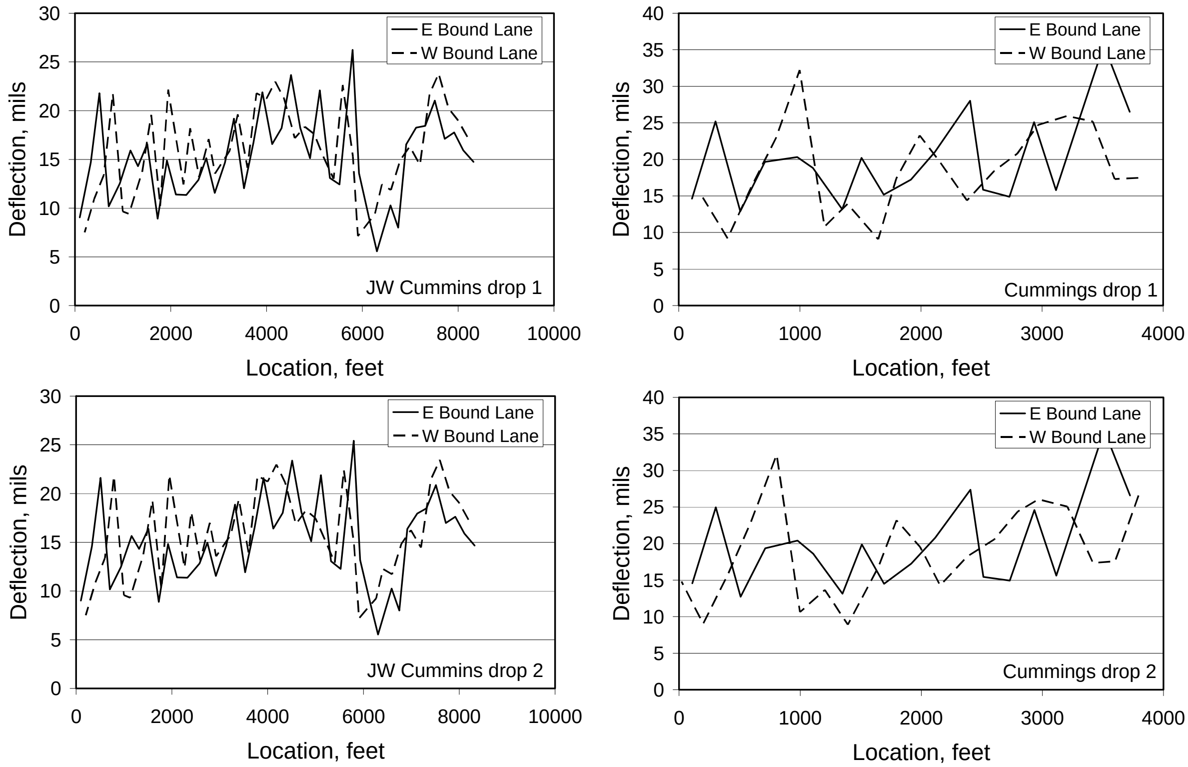

Figure 3. Comparison of deflections measured by sensor 1, directly below the applied load, for the East and West-bound lanes of J.W. Cummins and Cummings roads, respectively, and the deflections produced in response to drops 1 and 2, respectively.

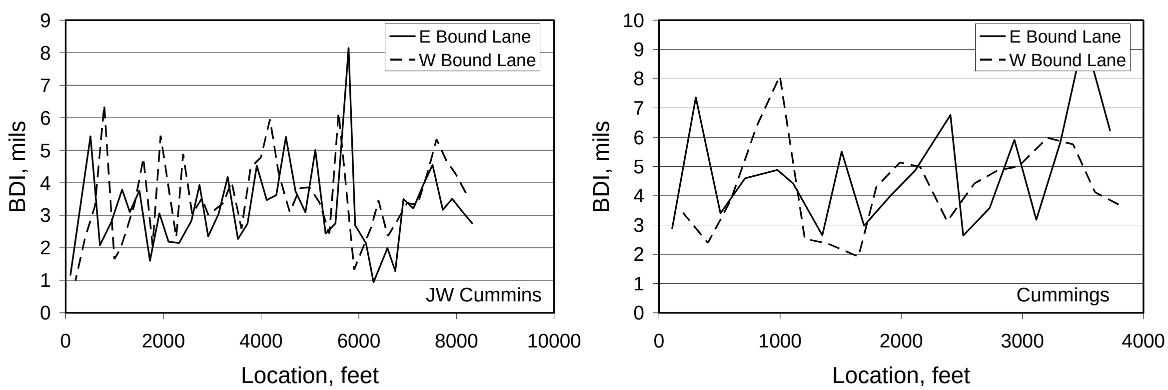

Figure 4. BDI for J.W. Cummins and Cummings roads, respectively.

Computation of Resilient Modulus using FWD Results

The deflection basins measured with FWD were analyzed using the Modulus 6.0[5] software developed by Texas Transportation Institute at The Texas A&M University (TAMU). Modulus 6.0 is designed to back calculate resilient moduli of pavement layers for flexible pavements using FWD test data. The user is required to input the sensor distances to the load plate, layer thickness, moduli range and the Poisson's ratio for each layer. If the surface thickness is input in the depth to bedrock calculation, the subgrade thickness is automatically calculated and displayed. This may be bedrock or a stiff clay layer.

For our analysis, we have used a moduli range of 700-1700 ksi for asphalt concrete based upon typical values reported in the literature, 50-200 ksi for fly ash stabilized RAB based upon our unconfined compression and CBR test data, and 10 ksi for untreated subgrade. Poisson's ratio for all the three layers was taken to 0.3. Figure 5 gives the back calculated resilient moduli of the fly ash stabilized RAB at different locations of both East- and West-bound J.W. Cummins and Cummings roads, respectively. Figure 5 also gives the back calculated resilient moduli based upon load drops 1 and 2. An advanced segmentation routine has been included in Modulus 6.0. This permits the user to analyze the modulus results, and to look for significantly different sections within the test pavement. On J.W. Cummins road, section 4000 to 5000 feet shows a significantly distinct modulus value. The other locations are similar however, they show considerable variability. On Cummings road, the locations in the section between 1000 to 2000 feet have significantly higher modulus value. The other locations show similar results. For further analysis, single segmented mean values of moduli were computed for the two road segments. The mean value and standard deviation for J.W. Cummins road was found to be 96 ksi and 50 ksi, respectively, while that for Cummings road was found to be 63 ksi and 25 ksi, respectively. The mean values of AC layer and subgrade moduli were found to be 1072 ksi and 16 ksi, respectively.

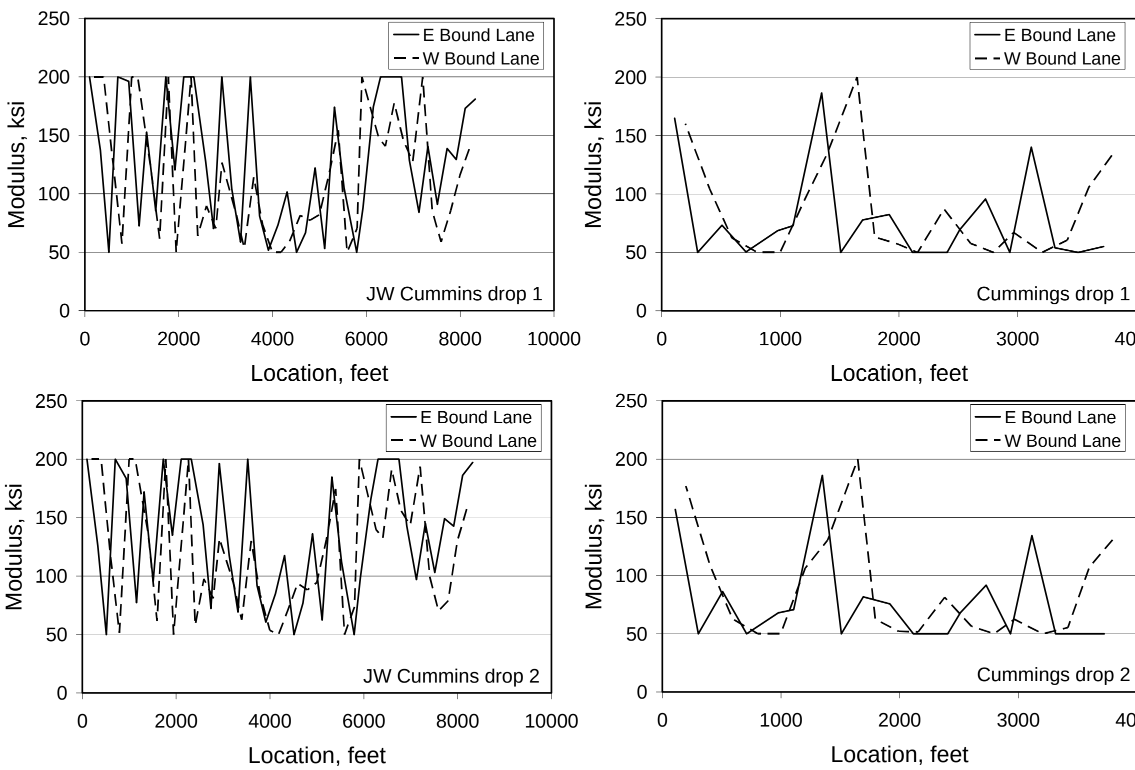

Figure 5. Plot of back calculated resilient moduli of the fly ash stabilized RAB at different location of J.W. Cummins and Cummings roads, respectively.

Comparison of Resilient Modulus from FWD Results and Empirical Correlation

Empirical correlations to determine resilient moduli based upon lab or field tests are widely available. However, empirical correlations for resilient moduli of fly ash stabilized RAB are not available in the literature. Therefore, it is of interest to assess the performance of fly ash stabilized RAB by comparison with other acceptable base course materials. Here we compare the back calculated resilient moduli of the fly ash stabilized RAB obtained from FWD data with those calculated using existing empirical correlations for granular subgrade. According to 1993 AASHTO Guide for Design of Pavement Structures[6], empirical correlation for resilient moduli of 100 psi strength base course of granular materials is given by

$$M_R = 740\,CBR \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where CBR = California Bearing Ratio, M_R = resilient modulus in psi. AASHTO 2002 Pavement Design Guide[7] provides a different empirical correlation for resilient modulus which is shown given as

$$M_R = 2555\,CBR^{0.64} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

Based upon laboratory tests, average CBR value of 120 is obtained for fly ash stabilized RAB after 7 days of compaction and air curing. Using this CBR value, we obtain the resilient moduli of 89 ksi and 55 ksi based upon the correlations given in Eqs. 1 and 2, respectively. Furthermore, our dynamic cone penetrometer (DCP) test results suggest a field CBR value in the range of 108 to 135, assuming that 20% to 25% of the ultimate strength is gained within the first 24 to 48 hrs of curing. Using these CBR values, we obtain a resilient modulus ranging from 80 to 100 ksi and 51 to 59 ksi based upon correlations given in Eqs. 1 and 2, respectively. We observe that the resilient moduli values obtained from the FWD tests lie between the ranges predicted by the above two correlations. Thus, neither of these correlations are directly applicable for fly ash stabilized RAB. Nevertheless, it is encouraging that the resilient moduli values are well within the range of granular base materials.

STRUCTURAL LAYER COEFFICIENT OF FLY ASH STABILIZED RAB

The above resilient moduli values may be utilized to estimate the structural layer coefficient which is a measure of the relative ability of a unit thickness of a material to function as a structural component of the pavement. The structural layer coefficient is used to calculate the structure number needed for the design of layer thicknesses. The layer coefficient can be found from the following equation[7]

$$M_R = 30000(a_i / 0.14)^3 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

where a_i = Structural Layer Coefficient. Alternatively, the following equation may be used to estimate layer coefficient for base course from the resilient modulus[8]

$$a_i = 0.249(\log M_R) - 0.977 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

Based upon the average laboratory CBR value, the layer coefficients calculated using Eq. 3 comes out to be 0.20 and 0.17 for the correlations given by Eqs. 1 and 2, respectively, while those calculated using Eq. 4 come out to be 0.26 and 0.20. Using the CBR values estimated from field DCP tests, we get layer coefficients ranging from 0.19 to 0.21 based upon Eq. 3 and 0.24 to 0.27 based upon Eq. 4 for correlation given by Eq. 1. Similarly, we get layer coefficients ranging from 0.17 to 0.18 based upon Eq. 3 and 0.20 to 0.21 based upon Eq. 4 for correlation given by Eq. 2.

Now, using equation (3) and (4), and the resilient modulus (M_R) values obtained from the FWD test gives structural layer coefficient values as 0.18 and 0.22 for Cummings road. Similarly using equation (3) and (4), and the resilient modulus (M_R) values obtained from the FWD test gives structural layer coefficient values as 0.20 and 0.26 for J.W. Cummins road. Again, it is encouraging that the structural layer coefficients based upon FWD values fall within the ranges predicted by empirical correlation for resilient modulus. Moreover, it is noteworthy that irrespective of the correlation used, the layer coefficients values exceed the AASHTO design requirement of 0.14 for the layer coefficient of flexible pavement granular base. As a conservative estimate, a structural layer coefficient ranging from 0.18 to 0.20 can be used to determine the structural number for calculating the base layer thickness for the fly ash stabilized bases.

SUMMARY AND CONCLUSIONS

Full-Depth Cold In-Place Recycling (CPR) with self-cementing fly ash has been shown to be an effective method of converting conglomerate pavement sections into durable roads. CPR demonstration projects have been performed in several states, where it has been shown to produce longer road life at a major saving. This technology provides several economic and environmental benefits. The process recycles deteriorated asphalt pavement and self cementing fly ash, reduces energy consumption, diesel emissions, land disposal requirements and virgin resource utilization, and increases road longevity. In addition to providing environmental benefits and long-lasting pavements, it has been reported that this technology could result in savings of up to 33% over conventional techniques.

A demonstration of this process on approximately 2.5 miles of roadway consisting two different segments was conducted in August 2004. As a result, model construction specifications and pre-construction testing program were developed. In addition, the construction procedures and the monitoring methods were documented and evaluated. Subsequently, the FWD tests were conducted at different locations on these roads after 1.5-year of project completion. The deflection bowls measured in the FWD tests were analyzed to back calculate the resilient moduli of the roadbase. These moduli were utilized to estimate the structural layer coefficient which is a measure of the relative ability of a unit thickness of a material to function as a structural component of the pavement and may be used to calculate the structure number needed for the design of layer thicknesses. The conservatively recommended layer coefficient values, which range from 0.18 to 0.20, are considerably higher than the AASHTO design requirement of 0.14 for the layer coefficient of flexible pavement granular base. The high layer coefficients for RAB base course may allow for thinner asphalt wear surfaces, thereby reducing construction costs.

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