

**COLD IN-PLACE RECYCLING OF ASPHALT PAVEMENTS USING SELF-CEMENTING FLY ASH:  
ANALYSIS OF PAVEMENT PERFORMANCE AND STRUCTURE NUMBER**

Final Report

for

Combustion Byproducts Recycling Consortium

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by

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## I. INTRODUCTION

Counties and local governments have large networks of rural and county roads that 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 (Misra et al. 2003). For example, the state of Missouri has over 30,000 miles of primary and supplementary rural roadways as part of the state roadway system and over 70,000 miles of county roads as part of the local system. Performance of these roads is typically inconsistent because of the diversity of materials used in their construction and during maintenance. In addition, many of these roads often go without any maintenance at all because of budgetary constraints. In many cases, problems with these roads go all the way back to subgrade deficiencies.

These conglomerate pavement sections may be converted into durable roads that last many more years by recycling old asphalt pavement and base course into new roadway through the *Cold In-Place Recycling (CPR) with Self Cementing Fly Ash*. 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 longer-lasting pavements, it has been reported that this technology could result in savings of up to 33% over conventional techniques.

In CPR with self cementing fly ash, 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 was conducted in August 2004 by Jackson County, MO, in collaboration with Kansas City Power & Light Company, Lafarge North America Corporation and the PI. The partnership established in this project was recognized through EPA's Coal Combustion Products Partnership (C<sup>2</sup>P<sup>2</sup>) award. The lessons learned from this demonstration project indicate a need for determining (1) the correlation between DCP-based field CBR values and FWD-derived layer elastic modulus for fly ash stabilized recycled asphalt base (RAB), and (2) fly ash stabilized RAB layer coefficient for engineering design of rehabilitated road pavements.

In this project report, we have utilized the data obtained during construction of the demonstration project as well as data to be acquired as part of the post-construction monitoring to establish design methods. Availability of such guidelines will encourage local area governments nationwide to consider similar technologies for their road networks.

## **II. Laboratory CBR and DCP Tests**

### **II.1 Sample Preparation**

The dry density and moisture content of the RAB-fly ash mixes were determined according to AASHTO T99 (Modified Proctor Test). For each sample, RAB and the required amount of

Lacygne Class C fly ash by weight were first mixed dry, and then the proper amount of water was added to the dry mix to obtain the desired moisture content. After the moisture was added, the sample was thoroughly mixed until homogeneous. Then the RAB-fly ash mix was evenly distributed in the modified Proctor mold and compacted in approximately three (3) equal layers, with 50 blows per layer. In consideration of the rapid hydration characteristics of the fly ash, each sample was prepared individually in less than 20 minutes (Misra et al. 2006).

## **II.2 Laboratory Dynamic Cone Penetrometer (DCP) Tests**

CBR was the first test conducted on each of the samples, and was completed on one side of the sample while the DCP test was performed on the flip side of the sample. Both tests were completed during the same testing session. The indentation left in the sample from the CBR test was filled in with soil scrapings. These soil scrapings had to be mixed at the time of the testing since samples mixed prior to the testing had already cured and could not be used to fill the indentation. The indentations were filled to remove their effect on the DCP test results.

The DCP test was completed using standard testing method specified in the new ASTM test D 6951 – 03. This test is a measurement of the penetration rate of the DCP through an undisturbed soil sample. The standard test uses a DCP device with an 8 – kg hammer, which is dropped from a height of 22.6 inches for a given number of blows. The penetration depth of the cone tip into the soil is then recorded for the number of blows. For these experiments, two to three blows per recording were used depending upon the penetration of first blow with a maximum penetration of 10 cm, which is less than the height of the mold. For the DCP test, the cumulative penetration is plotted as a function of cumulative number of blows for each sample. A regression analysis was performed to fit a line of the data.

### II.3 Correlation between CBR and DCP Penetration Rates

Figure 14 shows the variation of CBR as a function of the DCP penetration rate measured in the laboratory on a log – log scale. Based upon the linear regression analysis of this data, the following correlation is obtained for CBR PR (mm/blow):

$$\text{Log}[\text{CBR} (\%)] = 2.224 - 1.01\text{Log}[\text{PR} (\text{mm/blow})] \dots\dots\dots (1)$$

The goodness-of-fit coefficient  $R^2$  for this relationship is 0.9575. The intercept and slope for the correlation in Equation 1 is very close to those given by Webster (1992), who obtained an intercept of 2.465 and a slope of 1.12, for a wide range of granular and cohesive materials.

Literature on CBR – DCP correlations suggests that the field data is over predicted by the laboratory data. Therefore, it is necessary to apply a correction factor to laboratory–developed correlations to match field data. Gabr (2000) has recommended that a reduction factor of 30% be applied to the CBR values predicted from the lab developed correlations. Consequently, by applying the 30% reduction factor the corrected correlations is obtained as:

$$\text{Log} [\text{CBR}(\%)] = 2.067 - 1.01\text{Log}[\text{PR}(\text{mm/blow})] \dots\dots\dots(2)$$

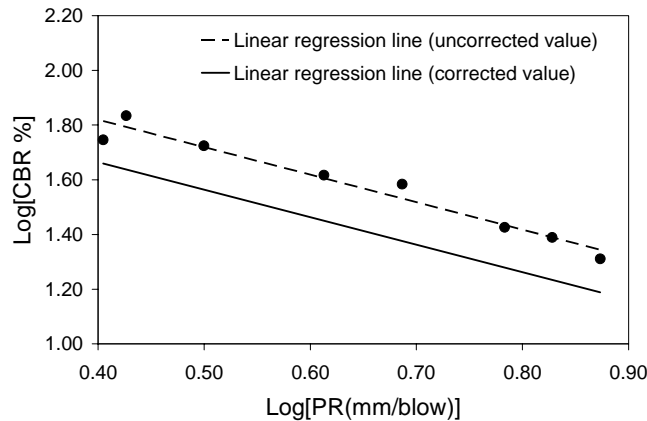
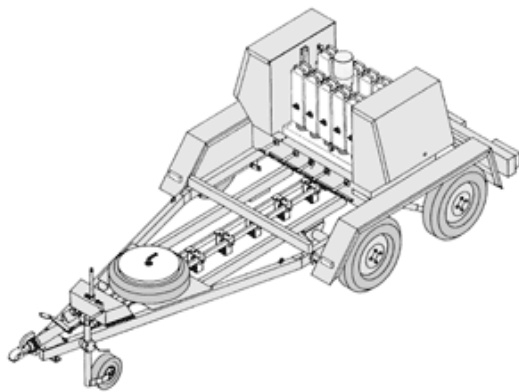


Figure 1. Plot of Log[CBR(%) v/s Log[PR(mm/blow)].

### III. Field FWD tests

The Falling Weight Deflectometer (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 a 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 sensors 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. Figure 1 gives a schematic diagram and a photograph of the device. Figure 2 shows load application through the circular plate as well as the displacement sensors.



(a)



(b)

Figure 1. Dynatest 8082 trailer-mounted HWD used to perform FWD Testing (a) schematic from [http://www.dynatest.com/hardware/fwd\\_hwd.htm](http://www.dynatest.com/hardware/fwd_hwd.htm), and (b) photograph of the device at J.W. Cummins.

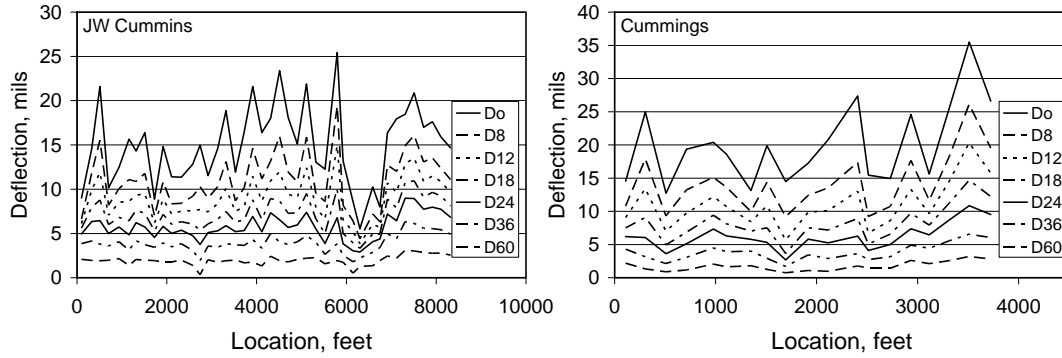


**Figure 2. Picture showing circular plate for FWD load application and tips of dynamic displacement sensors.**

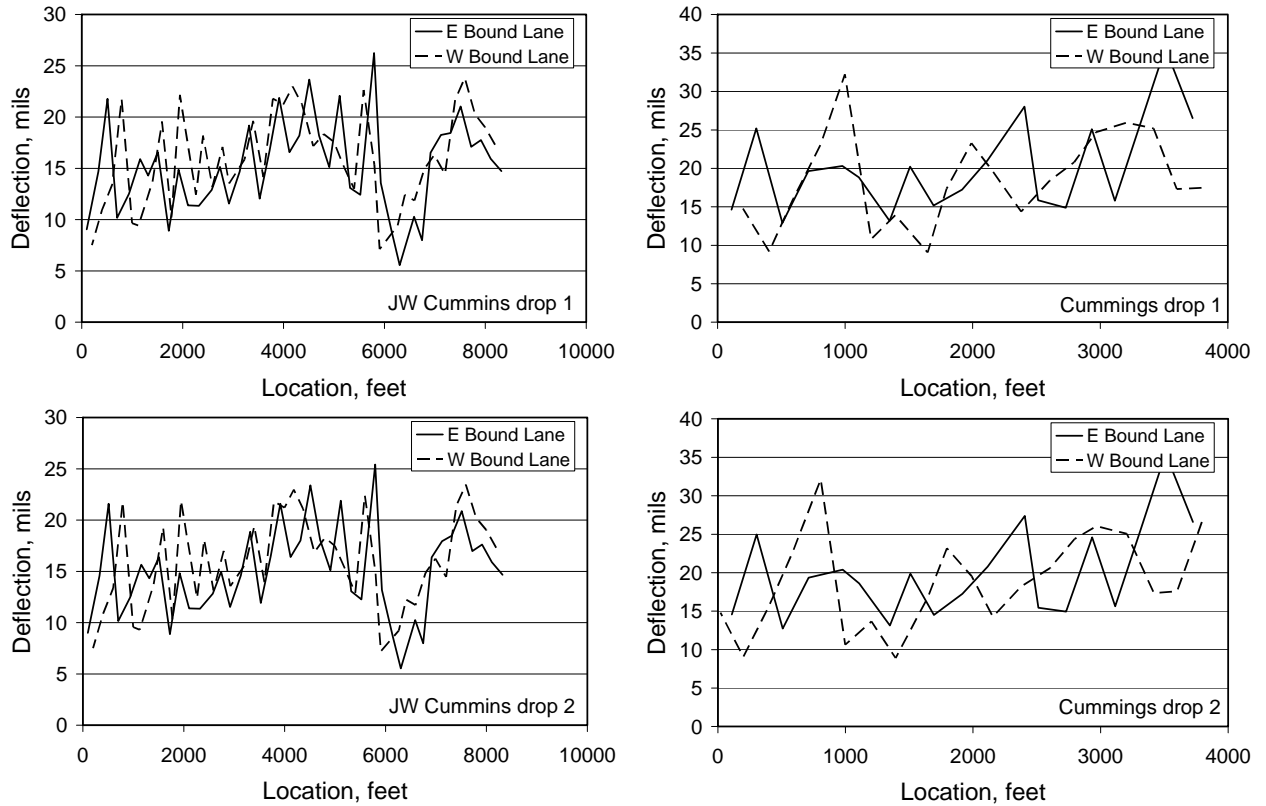
### **III.1 FWD Test Results**

FWD tests were conducted at different location on J.W. Cummins and Cummings road at approximately 200 feet intervals. Dynatest Model 8082 HWD (shown in Figures 1 and 2) 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 3 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<sup>nd</sup> load drop. Figure 4 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.

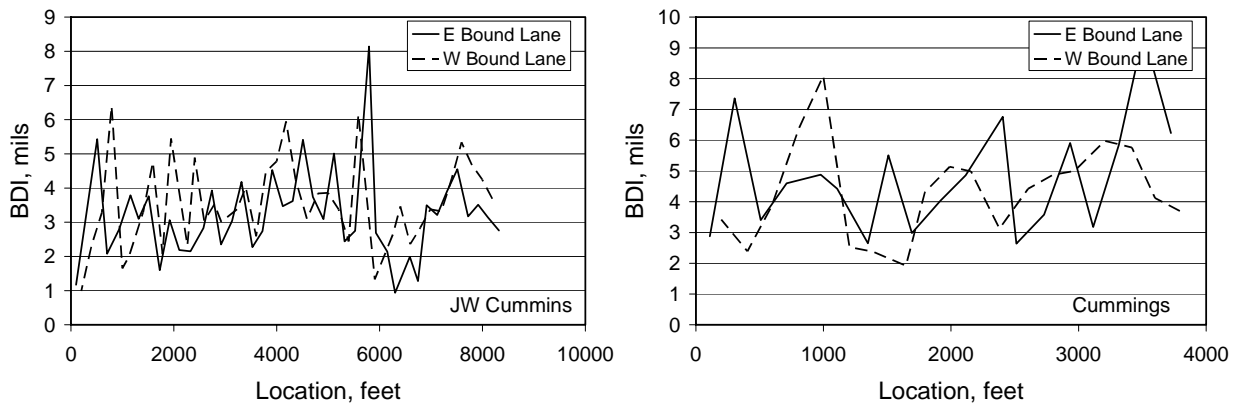


**Figure 3. Pavement displacement from FWD tests on East-bound lanes of J.W. Cummins and Cummings roads, respectively.**



**Figure 4. 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.**

According to NCHRP Project 10-48 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 (NCHRP 2001). 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 5, 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.

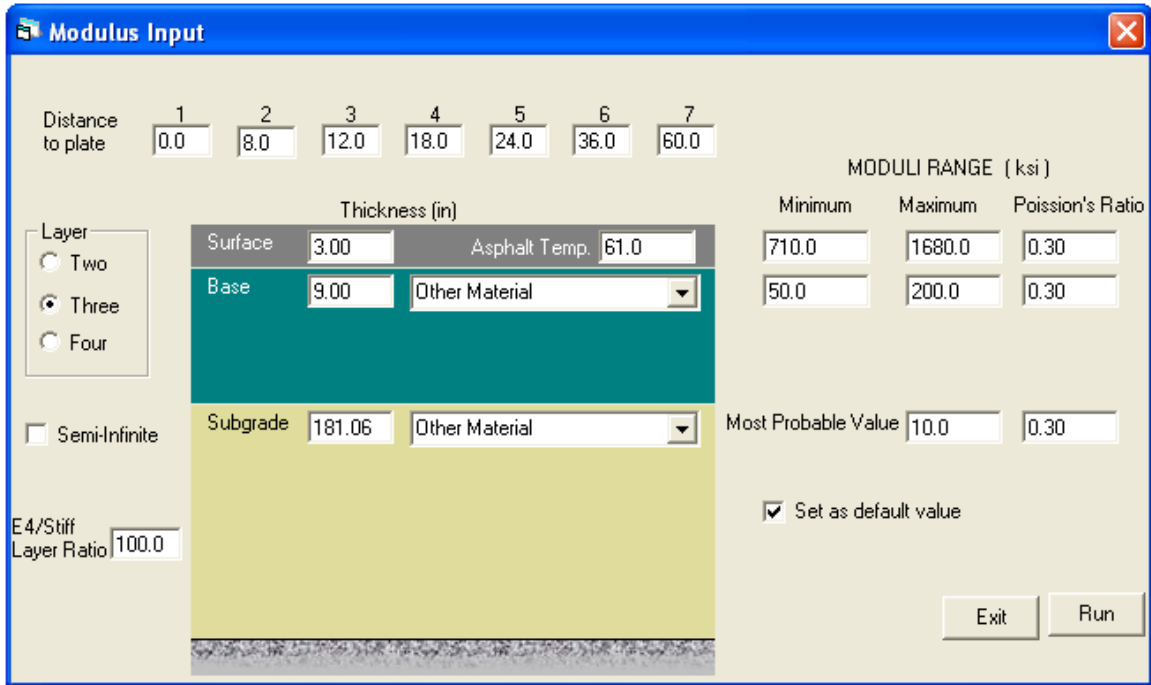


**Figure 5. Base Damage Index (BDI) for J.W. Cummins and Cummings roads, respectively.**

### III.2 Computation of Resilient Modulus using FWD Results

The deflection basins measured with FWD were analyzed using the Modulus 6.0 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

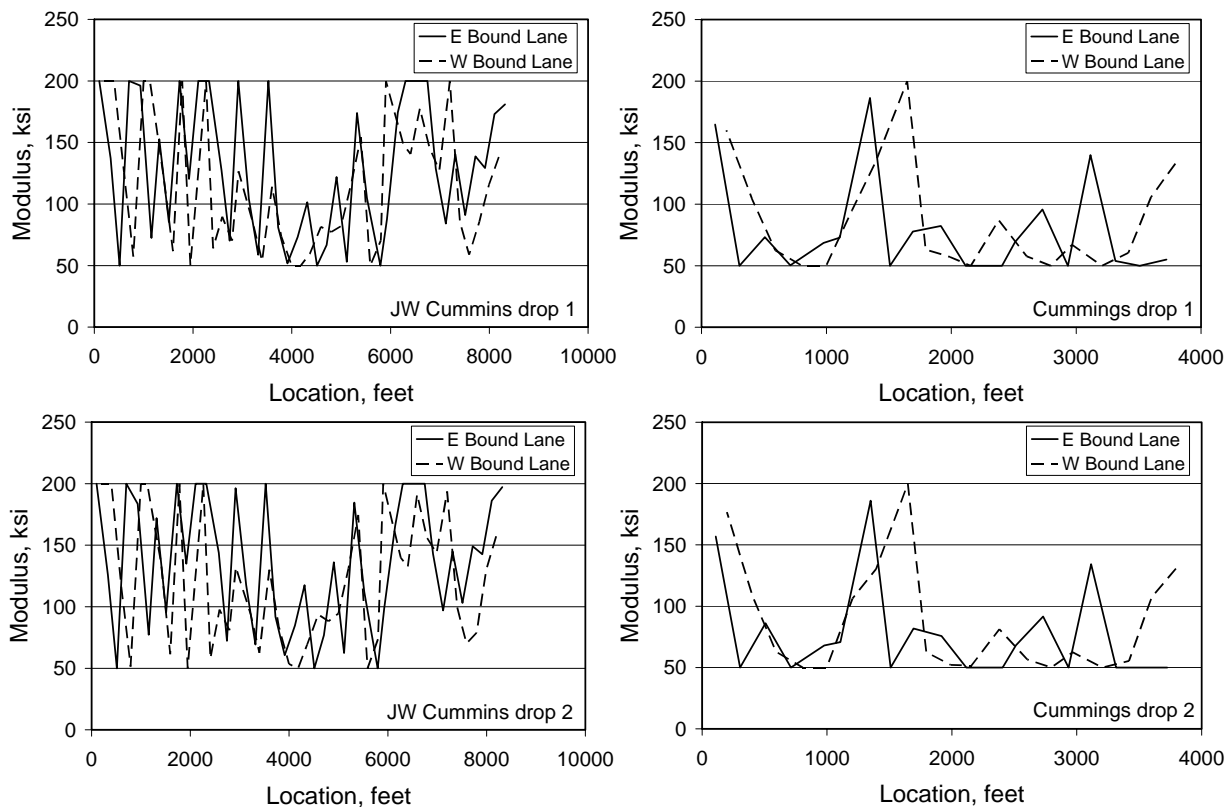
pavements using FWD test data. The input screen for the back calculation module is shown in Figure 6. In this screen, the user can input the sensor distances to the load plate, layer thickness, moduli range and the Poisson's ratio for each layer. If the surface thickness is input in the depth to bedrock calculation, the subgrade thickness is automatically calculated and displayed. This may be bedrock or a stiff clay layer. The user can overwrite this number, if needed.



**Figure 6. MODULUS back calculation input dialog box.**

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 7 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 7 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 7. Plot of back calculated resilient moduli of the fly ash stabilized RAB at different location of J.W. Cummins and Cummings roads, respectively.**

#### IV. Resilient Modulus from Field Tests and Empirical Correlations

Most of the agencies use empirical correlations to determine resilient moduli based upon lab or field tests. 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, empirical correlation for resilient moduli of 100 psi strength base course of granular materials is given by (AASHTO 1993)

$$M_R = 740 CBR \dots\dots\dots (3)$$

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

$$M_R = 2555 CBR^{0.64} \dots\dots\dots (4)$$

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. 3 and 4, respectively. Furthermore, our 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. 3 and 4, 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.

## V. 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 (AASHTO 2002)

$$M_R = 30000(a_i / 0.14)^3 \dots\dots\dots (5)$$

where  $a_i$  = Structural Layer Coefficient. Alternatively, the following equation may be used to estimate layer coefficient for base course from the resilient modulus [Huang 93].

$$a_i = 0.249(\log M_R) - 0.977 \dots\dots\dots (6)$$

Based upon the average laboratory CBR value, the layer coefficients calculated using Eq. 5 comes out to be 0.20 and 0.17 for the correlations given by Eqs. 3 and 4, respectively, while those calculated using Eq. 6 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. 5 and 0.24 to 0.27 based upon Eq. 6 for correlation given by Eq. 3. Similarly, we get layer coefficients ranging from 0.17 to 0.18 based upon Eq. 5 and 0.20 to 0.21 based upon Eq. 6 for correlation given by Eq. 4.

Now, using Eq. 5 and 6, and the resilient modulus ( $M_R$ ) values obtained from the FWD test, the structural layer coefficient is found to be 0.18 and 0.22 for Cummings road, and 0.20 and 0.26 for J.W. Cummins road, respectively. 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.

## **VI. 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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