

Reference to this paper should be made as follows: Kim, S., Gopalakrishnan, K., and Ceylan, H. (2012). “*Structural Characterization of Iowa’s Rubblized PCC Pavements.*” ASCE Journal of Transportation Engineering, Vol. 138, No. 4, pp. 406 – 413.

Structural Characterization of Iowa’s Rubblized PCC Pavements

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Abstract: Rubblization is considered one of the sustainable surface preparation techniques before placing a hot mix asphalt (HMA) overlay that involves breaking the concrete pavement into pieces. The design of the structural overlay thickness for rubblized projects is difficult as the resulting structure is neither a true rigid pavement nor a true flexible pavement. The most important aspect of the design procedure is to characterize the rubblized concrete layer in terms of a layer coefficient and modulus for the empirical and the mechanistic-empirical (M-E) pavement design methods. This paper describes a field study undertaken to investigate the performance of rubblized pavements in Iowa, which includes characterization of the rubblized Portland cement concrete (PCC) layer (layer coefficient and modulus) using a neural networks-based layer moduli backcalculation program. The average rubblized PCC layer coefficient and backcalculated modulus of the rubblized layer in Iowa were found to be 0.19 and 539 MPa (78 ksi), respectively. The M-E design program developed in-house seems to estimate the HMA overlay thickness reasonably well to achieve long-lasting performance of HMA-overlaid rubblized PCC pavements.

CE Database subject headings: Pavement design and rehabilitation, Rubblization, Portland cement concrete, Neural Networks, Hot Mix Asphalt, Mechanistic-Empirical Design

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Introduction

Portland cement concrete (PCC) pavements usually deteriorate during service times because of distresses (for example, cracking, faulting, punchouts) caused by a combination of traffic loads, materials related distresses, and weather conditions. As a result, repair and rehabilitation activities, depending on the type, extent, and severity of distresses, are carried out to extend the service life of an existing pavement.

An asphalt overlay with rubblization of heavily distressed PCC pavements is one of the rehabilitation strategies that many agencies are now using. Rubblization is one of the surface preparation techniques before placing a Hot Mix Asphalt (HMA) overlay in attempts to minimize reflection cracking, which is the reflection of cracks and joints in existing pavement on the HMA overlay surface. Rubblization involves breaking the concrete pavement into pieces. The sizes of the broken pieces usually range from sand size to 75 mm (3 in) at the surface and 305 to 381 mm (12 to 15 in) on the bottom of the rubblized layer (Von Quintus et al. 2007). The rubblized PCC layer behaves like a high-quality granular base layer and responds as an interlocked unbound layer – reducing the existing PCC to a material comparable to a high-quality aggregate base course. This loss of structure must be accounted for in the HMA overlay design thickness (Galal et al. 1999).

Designing the structural HMA overlay thickness for rubblized projects is difficult, as the resulting structure is neither a true rigid pavement nor a true flexible pavement. Classical rigid pavement analysis and design utilizes the Westergaard theory, whereas classical flexible pavement analysis and design utilizes the Burmister multilayer theory. On the basis of the assumption that the rigidity of the PCC slabs has been destroyed, the Burmister approach may be used with HMA-overlaid fractured PCC pavement. The Westergaard approach has been proposed to be used to evaluate the prerubblized PCC slabs, whereas the Burmister theory may be used for postrubblization analysis (Bemanian and Sebaaly 1999).

The most important aspect of the design procedure is to characterize the rubblized concrete layer. The biggest challenge in the structural design of rubblized pavements is determining the appropriate HMA overlay thickness that, satisfies both functional and structural requirements of the pavement. Several approaches have been proposed to establish the required overlay thickness (Ceylan et al. 2005). The most recognized overlay design procedure is the 1993 American Association of State Highway officials (AASHTO) Guide for Design of Pavement Structures (AASHTO 1993). It uses an empirical procedure on the basis of the, results of the AASHTO Road Test. The 1993 AASHTO design method uses the layer coefficient to characterize each layer. Many agencies are also considering the use of mechanistic design procedures, such as the Mechanistic-Empirical Pavement Design Guide (MEPDG) developed under National Cooperative Highway Research Project (NCHRP) 1-37A (NCHRP 2004) for overlay design procedures. The elastic modulus for the rubblized layer is an important design value in MEPDG to determine the thickness of the HMA overlay. Because the use of the layer coefficients with the 1993 AASHTO overlay design procedure and elastic modulus values with MEPDG have not been adequately validated with performance data (Von Quintus et al. 2007), those values vary considerably depending on the state agency and the design procedure used, giving rise to a wide range of HMA overlay thicknesses.

The HMA overlay thickness design methodology currently used in the state of Iowa is purely empirical. In an effort to shift toward mechanistic based design, researchers at Iowa State

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University (ISU) developed a mechanistic-empirical (M-E) thickness design program and a neural networks-based moduli backcalculation program for the HMA-overlaid rubblized PCC pavements under the sponsorship of the Iowa Highway Research Board (Ceylan and Gopalakrishnan 2007; Ceylan et al. 2008). Following the development of these programs, the present study was conducted to evaluate the performance of rubblized pavements across Iowa.

Objective

The primary objective of this study is to characterize the rubblized PCC layer coefficient/modulus of representative rubblized pavements in Iowa. The results would be useful in establishing design modulus and for providing AASHTO layer coefficient recommendations for rubblized PCC layers. The other objective is to examine the accuracy of ISU M-E thickness design program by comparing the predicted model thicknesses with the HMA overlay thicknesses of in-service rubblized pavements showing good performance in Iowa.

HMA overlay thickness design for rubblized PCC pavements

1993 AASHTO overlay design method

To calculate the overlay thickness, both the AASHTO condition survey and the nondestructive testing methods are used. Both of these methods use the concept of structural number (SN). Structural number was developed to provide a single number to represent a “conceptual” pavement thickness. The SN comprises the structural contribution of each layer, using a layer coefficient and thickness, where the layer coefficient is a measure of relative stiffness ($SN = a_1t_1 + a_2t_2 + a_3t_3 + \dots$). Therefore, the AASHTO pavement design procedure requires estimating the layer coefficients.

According to 1993 AASHTO guide, the recommended coefficients for rubblized PCC pavement range from 0.14 to 0.30 (Thompson 1999). A study conducted by the Indiana DOT indicated that a layer coefficient of 0.22 represented a conservative value to ensure structural adequacy with similar conditions (Galal et al. 1999). The Ohio DOT uses a layer coefficient value of 0.14 to represent the rubblized layer, neglecting any existing subbase under the PCC (Von Quintus et al. 2007). Similarly, Arkansas, Michigan, Mississippi, and Pennsylvania use values between 0.14 and 0.20. Minnesota, New York, and Wisconsin have used structural layer coefficients of around 0.25. So far, no agency has published or correlated these structural layer coefficients of the rubblized layer to different rubblization equipment and particle size distribution (Von Quintus et al. 2007).

Mechanistic-Empirical Pavement Design Guide (MEPDG)

Rehabilitation design in the MEPDG requires elastic modulus of the rubblized PCC layer to predict the performance of HMA overlaid rubblized pavement. NAPA (1994) reported and recommended modulus values of 689 and 1034 MPa (100 and 150 ksi) for use in design. The default value recommended for use in the MEPDG Level 3 analysis is 1034 MPa (150 ksi), which is a quite higher value (NCHRP 2004). The study by Indiana DOT reported that the PCC modulus value before rubblization was approximately 26 GPa (3,800 ksi), and the rubblized PCC

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modulus value was about 1,172 MPa (170 ksi) from US-41 in Benton County, Indiana (Galal et al. 1999). A recent study in Wisconsin reported that the average elastic modulus determined for the rubblized PCC layer in Wisconsin rubblization projects is 448 MPa (65 ksi) and in general, the elastic modulus ranged from 241 to 827 MPa (35 to 120 ksi) (Von Quintus et al. 2007). These values are similar to those values determined from deflection basin testing of HMA overlays placed over rubblized PCC pavements—both from the Long-Term Pavement Performance (LTPP) Specific Pavement Studies-6 (SPS-6) experiment and actual construction projects reported by Von Quintus and Tam (2000). In summary, no consistent elastic modulus value has been used to represent rubblized PCC layers, which suggests that the value is site specific and dependent on the rubblization process itself.

Mechanistic-Empirical (M–E) HMA overlay design and layer moduli backcalculation program for rubblized PCC pavements

ISU M–E HMA overlay design program

Researchers at ISU proposed a mechanistic-empirical (M-E) thickness design approach for the HMA overlaid rubblized PCC slabs under the sponsorship of Iowa Highway Research Board (Ceylan et al. 2005). In this approach, the pavement is regarded as a multi-layered elastic system. The design inputs include traffic information, material properties and environmental factors. The materials in each of these layers are characterized by modulus of elasticity (E) and Poisson’s ratio (μ). Material characterization of the existing pavement is typically performed using the falling weight deflectometer (FWD) test data. Poisson’s ratio is customarily assumed for design within reasonable accuracy.

Critical pavement responses, such as the tensile strain at the bottom of HMA layer and the vertical compressive strain on the surface of subgrade, are used as the failure criteria to consider HMA fatigue and subgrade rutting, respectively. Critical pavement responses are calculated using Elastic Layer Programs (ELP) such as JULEA, KENLAYER, etc. Critical pavement responses are then related to various types of distresses through transfer functions. The amount of damage is expressed as a damage ratio (calculated using the Miner’s law (Miner, 1945) between the predicted and the allowable number of load repetitions. Critical damage occurs when the sum of damage ratios reaches a value of 1.0. The final design is selected when the HMA overlay thickness satisfies the design requirements for each type of distress.

The M–E design process developed was coded into a Visual Basic program with a user-friendly graphical interface to facilitate design calculations. Details of the development of M-E HMA overlay design program outlined above are described in Ceylan et al. (2005, 2007).

ISU layer moduli backcalculation program

Backcalculation is the “inverse” problem of determining material properties of pavement layers from their response to surface loading. No direct, closed-form solution is currently available to determine the layer moduli of a multilayered system given the surface and layer thicknesses. Most of the existing backcalculation programs employ iteration or optimization schemes to calculate theoretical deflections by varying the material properties until a “tolerable” match of measured deflection is obtained. However, in these programs, the reliability of the solution is

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dependent upon the seed moduli used as an input. This makes backcalculation a difficult process in which minor deviations between measured and computed deflections usually result in significantly different moduli. In many cases, various combinations of modulus values essentially produce the same deflection basin (Mehta and Roque 2003).

Researchers at ISU developed user-friendly, spreadsheet-based software for layer moduli backcalculation of rubblized PCC pavements. This program employs an Artificial Neural Networks (ANN)-based structural model for predicting not only the moduli of pavement layers based on FWD deflection data, but also the critical structural responses. The critical structural responses that are of interest include the horizontal tensile strain at the bottom of an HMA layer (ϵ_t) and the vertical compressive strain on the surface of the subgrade (ϵ_c). The backcalculated pavement layer moduli include the AC modulus (E_{AC}), the rubblized PCC modulus (E_{PCC}), and the subgrade modulus (E_{SG}).

The ANN-based structural models were developed by relating the structural responses (strains and deflections) to layer thicknesses and moduli values using the synthetic database. A synthetic database was generated using an ELP by computing the critical strains for a wide range of layer thicknesses and moduli values. A total of 2,600 data sets were generated based on different combinations of the layer thicknesses and moduli values.

A multi-layered, feed-forward neural network trained using an error backpropagation algorithm (commonly referred to as backpropagation ANNs) was employed for the prediction of critical responses and the moduli of pavement layers. Backpropagation type ANNs are very powerful and versatile networks that can be taught mapping from one data space to another using examples of the mapping to be learned. The learning process performed by this algorithm is called backpropagation learning, which is mainly an error minimization technique (Haykin 1999).

For the prediction of critical responses (ϵ_t and ϵ_c) and the moduli of pavement layers (E_{AC} , E_{PCC} , and E_{SG}), six inputs—thickness of HMA (H_1), transformed thickness of rubblized PCC layer and subbase layer (H_2), and four FWD surface deflections (D_0 , D_{12} , D_{24} , and D_{36}) at 305 mm (12-in.) offsets starting from center deflection (D_0)—were used. The Odemark’s concept of equivalent thickness was used to transform the thickness of rubblized PCC layer and subbase layer (Ceylan et al. 2005). Based on the parametric analysis, two hidden layers with 60 nodes in each layer were found to be sufficient in this case. Thus, the final ANN architecture was 6-60-60-5 (6 inputs, 60 nodes in the first and second hidden layers, and 5 output nodes, respectively). Details of the development of ANN-based structural models outline above and the validation of ANN-based strain predictions are described in Ceylan and Gopalakrishnan (2007).

Field data collection

Experimental field data were collected from July to November, 2007 to characterize layer properties of existing rubblized concrete pavements across Iowa with ISU layer moduli backcalculation program and validate ISU M-E HMA overlay design program. Seven representative rubblized concrete pavements sections (listed in Table 1) were primarily selected considering state wide location and pavement age. The ages of selective pavement are at least older than 5 years. The experimental test methods include the Falling Weight Deflectometer (FWD), the Dynamic Cone Penetrometer (DCP) and visual distress surveys. Core samples were also conducted to collect in-situ material, identify the layer underneath HMA layer, and provide

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space for conducting the DCP test. FWD and DCP tests and coring were performed on three locations in each test section – start (A), middle (B), and end (C) point. The visual distress survey was conducted on the entire test section.

Table 1. List of rubblized pavement sites for data collection

Test Section No.	Location		Average Layer Thickness (mm)			AADT*	Construction Year
	County	Road	HMA	Granular	Rubblized PCC		
1	Black Hawk	D16	168	0	191	1,280	2001
2	Black Hawk	V43	163	0	201	1,340	2001
3	Delaware	IA 3	246	0	221	740	2003
4	Franklin	C23	191	76	234	120	1998
5	Mills	L55	180	0	155	820	1999
6	Polk	IA 141	193	0	229	18,000	2001
7	Polk	IA 141	234	0	249	18,000	2001

* AADT = Annual Average Daily Traffic in 2005

Falling Weight Deflectometer (FWD) testing

The FWD test is conducted by applying dynamic (impulse) loads to the pavement surface, similar in magnitude and duration to that of a single heavy moving wheel load. The response of the pavement system is measured in terms of vertical deformation or deflection over a given area using geophones (velocity sensor). Deflection data was collected using Iowa DOT’s JILS-20 FWD by applying a step loading sequence of 27, 40, 53, and 67 kN (6,000, 9,000, 12,000 and 15,000 lbs) at three different locations (start, middle, and end point) in each test project. The locations of geophones used in this study are 0 (D_{0mm}), 203 (D_{203mm}), 305 (D_{305mm}), 457 (D_{457mm}), 610 (D_{610mm}), 914 (D_{914mm}), 1219 (D_{1219mm}), and 1524 mm (D_{1524mm}) from an applied FWD load.

Two-frequency FWD tests were conducted on a single location to identify the FWD sensor measurement errors. As seen in Fig. 1, no significant differences were observed, which indicated that the FWD can produce consistent results for the same test materials. The measured deflections on geophones were responding linearly to increasing FWD loads (see Fig. 1). This indicates that the deflections at different FWD load levels can be normalized to the deflections at one FWD load level. The measured deflections at 27, 53, and 67 kN of FWD loads were normalized to the deflections at 40 kN of FWD load. Fig. 2 presents the average normalized deflection at D_{0mm} of the FWD geophone for each test section.

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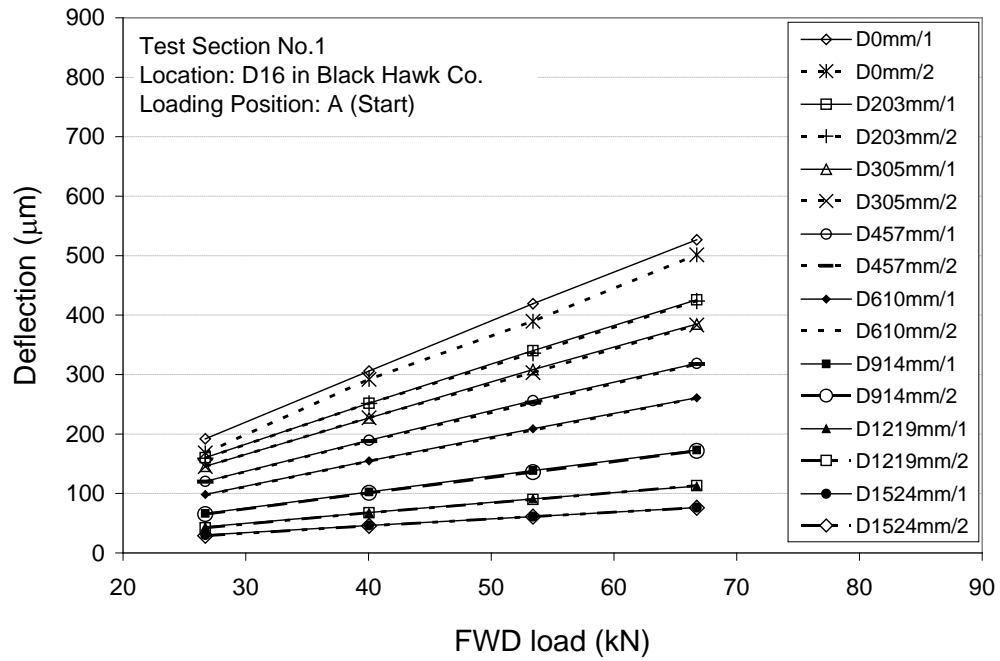


Fig. 1. FWD deflections with loads

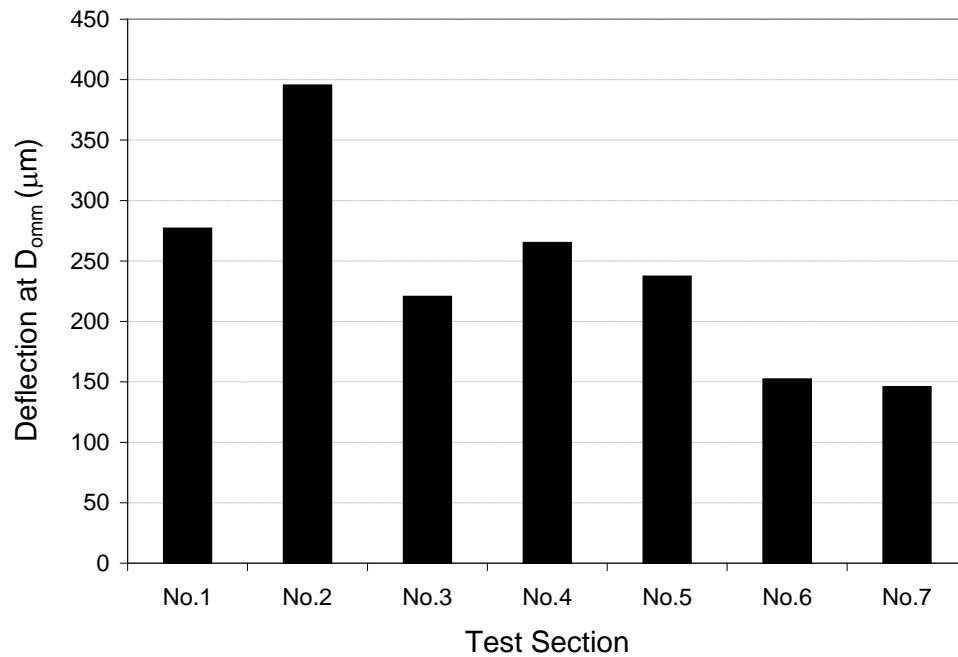


Fig. 2. FWD deflection at D_{0mm} in test sections

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Dynamic Cone Penetrometer (DCP) testing

DCP tests were conducted at the same locations after coring where FWD tests were conducted. The DCP tests were conducted to collect additional information about the in-situ subgrade soil properties. The DCP is an in situ device where measurements of penetration per blow (mm/blow) are obtained. In 2003, the ASTM published a standard for use of the DCP (ASTM 2003), “Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.” The device works by using a standard 8 kg (17.6 pound) hammer, which is lifted to the handle and dropped to the anvil, forcing the rod to penetrate the compacted soil area. The greater the number of blows needed to penetrate the rod into the soil, the stiffer the material.

To represent DCP measures at different depths in each location, the average rate of penetration or penetration index ($DCPI_{wtag}$) is determined by calculating the weighted average using the following Equation (1) (Swangsuriya and Edil 2004).

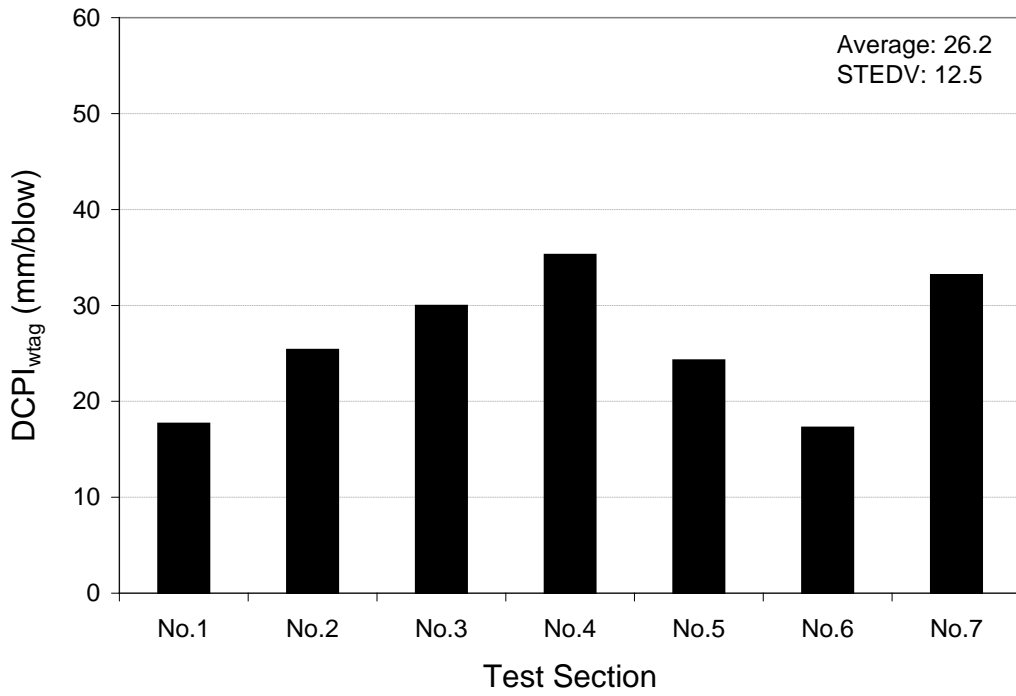
$$DCPI_{wtag} = \frac{1}{H} \sum_i^N [(DCPI)_i \times (z)_i] \quad (1)$$

in which, H = total penetration depth, z = layer thickness, DCPI = penetration index for z, (mm/blow) .

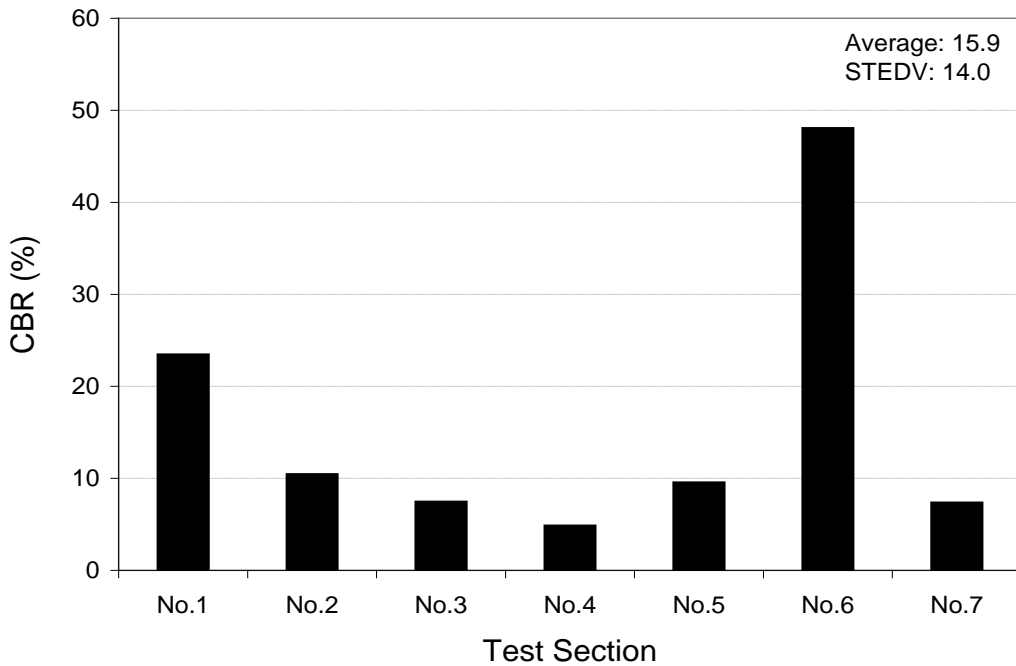
The rate of penetration (DCPI) has been correlated to the California Bearing Ratio (CBR, percent), an in situ strength parameter (ASTM 2003). The DCPI-CBR correlation for soils other than CL below CBR 10% and CH soils is as following Equation (2). DCP test results for test sections are presented in Fig. 3. The average $DCPI_{wtag}$ and CBR values for test sections are 26.2 mm/blow and 15.9%.

$$CBR = \frac{292}{DCPI^{1.12}} \quad (2)$$

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(a)



(b)

Fig. 3. Subgrade strength for test sections: (a) DCPI_{wtag}; (b) CBR

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Visual distress survey

Visual distress surveys over the entire test section were conducted to evaluate the rubblized pavement performance under the actual HMA overlay thickness of in-service rubblized pavement sections selected. The distress survey methodology employed was similar to that described in the Strategic Highway Research Program’s (SHRP) “Distress Identification Manual for the Long-Term Pavement Performance (LTPP) Project.” (Miller and Bellinger 2003).

A distinction was made between reflective cracking and low-temperature (transverse) cracking. Cracking was identified as “reflective cracking” when the transverse cracks were uniformly spaced (corresponding to PCC joint spacing underneath the HMA layer). The direction of crack propagation (top-down or bottom-up) was observed from the cored HMA sample.

Although a large number of pictures were taken as part of the visual distress survey for individual test sections to provide the types and severities of the distress, some representative pictures are included here which are indicative of the overall conclusion. Visual distress survey results are also summarized in Table 2. In general, no load-associated distresses, such as fatigue cracking and rutting, were found in any of the test sections as shown in Fig. 4. The predominant distresses observed in the rubblized PCC sections are longitudinal cracking and low-temperature cracking. No reflective cracking was observed in these rubblized PCC sections. These results indicate that the selected rubblized pavement sections are performing very well under the structural conditions identified in this study.

Table 2. Summary of visual distress survey results

Test Section No.	Location		Visual Distress Survey Results
	County	Road	
1	Black Hawk	D16	11 low temperature cracks
2	Black Hawk	V43	1 block and 8 low temperature cracks 2 longitudinal cracking (about 4.8 km) and 9 low temperature cracks
3	Delaware	IA 3	temperature cracks
4	Franklin	C23	No cracks
5	Mills	L55	14 low temperature cracks
6	Polk	IA 141	14 longitudinal cracks, 3 low temperature cracks
7	Polk	IA 141	2 longitudinal cracks

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Fig. 4. Picture of distress-free HMA surface on rubblized PCC (test section no. 4: C23 in Franklin county)

The observed longitudinal cracking, as shown in Fig. 5, run generally parallel to the pavement centerline and initiated from top surface. This observation indicates that the longitudinal cracking in test sections may be related to materials problem (segregation of the mix) rather than traffic loads. The observed transverse cracking was not uniformly spaced, which indicates that this may be related to low temperature rather than reflection cracking of concrete layer beneath the pavement.



Fig. 5. Picture of Longitudinal crack on HMA surface over rubblized PCC (test section no. 6: IA141 in Polk county)

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Characterization of layer properties for rubblized PCC pavements

Layer modulus of rubblized pavements

The FWD surface deflections obtained for rubblized sections were input into the ISU rubblized pavement layer moduli backcalculation program to compute the moduli of HMA, rubblized PCC and subgrade. The modulus of HMA is more temperature-sensitive than the modulus of rubblized PCC and subgrade. The computed HMA moduli at different temperature conditions were adjusted to the HMA moduli at a reference temperature (25 °C) using the equation (3) reported by Nouredin (1994).

$$E_{AC} = E_{AC,25} \times \frac{2747.5}{(T)^{2.46}} \quad (3)$$

in which, E_{AC} = Asphalt concrete modulus (MPa), $E_{AC,25}$ = Asphalt concrete modulus at 25 °C, and T = Asphalt concrete temperature, °C. Fig. 6 clearly illustrates the effect of temperature on HMA modulus. The backcalculated HMA modulus below 25 °C decreased and the modulus above 25 °C increased after adjustment to a reference temperature of 25 °C.

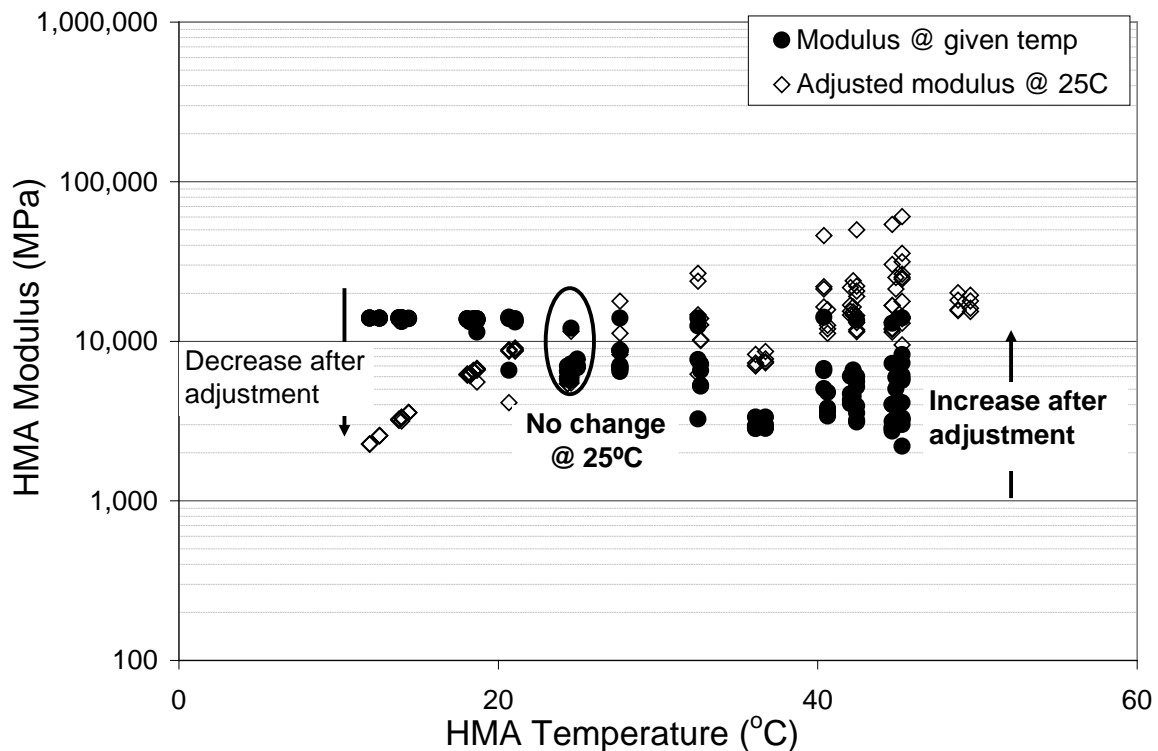


Fig. 6. HMA moduli before and after adjustment to a reference temperature of 25 °C

The CBR obtained from DCP test results has been correlated to the resilient modulus (M_r) of soil, an input parameter representing soil material strength in MEPDG (2004). The M_r -CBR correlation is as following Equation (4):

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$$M_r = 2555(CBR)^{0.64} \quad (4)$$

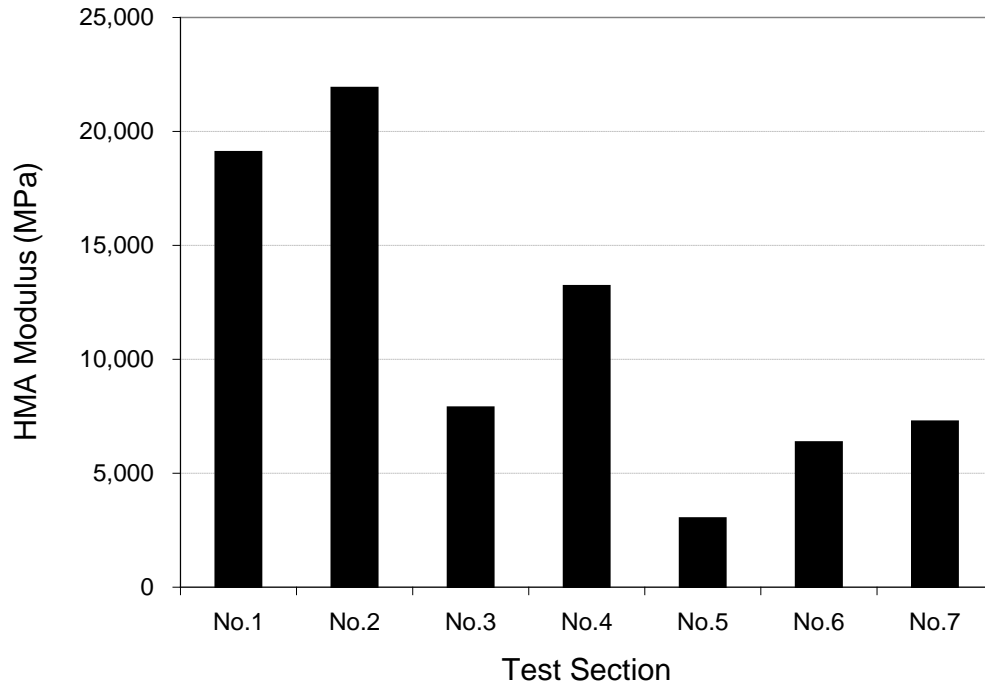
Fig. 7 summarizes the layer moduli results for each of the rubblized sections and Table 3 presents the overall statistical summary for layer moduli results. Note that the reported rubblized PCC modulus of section 4 is the modulus of the combined layer of rubblized PCC and granular layer since the thickness of granular layer (relatively thinner in this case) was added into the thickness of rubblized PCC layer during backcalculation computation. The average rubblized PCC modulus in this study was found to be 539 MPa (78 ksi). The value of 539 MPa (78 ksi) is close to the modulus value of 448 MPa (65 ksi) recommended by the Wisconsin DOT study (Von Quintus et al. 2007). Both of values are lower than the default modulus value of 1034 MPa (150 ksi), which is currently used in MEPDG (NCHRP 2004) and recognized as a quite higher value (Von Quintus et al. 2007). The range from 259 to 1,120 MPa (38 to 162 ksi) of rubblized PCC modulus in this study is also close to the range from 247 to 827 MPa (35 to 120 ksi) reported by Wisconsin DOT study (Von Quintus et al. 2007). These values are similar to those values determined from deflection basin testing of HMA overlays placed over rubblized PCC pavements – both from the Long-Term Pavement Performance (LTPP) Specific Pavement Studies-6 (SPS-6) experiment and actual construction projects reported by Von Quintus et al. (2000).

Table 3. Overall statistical summary for pavement layer moduli

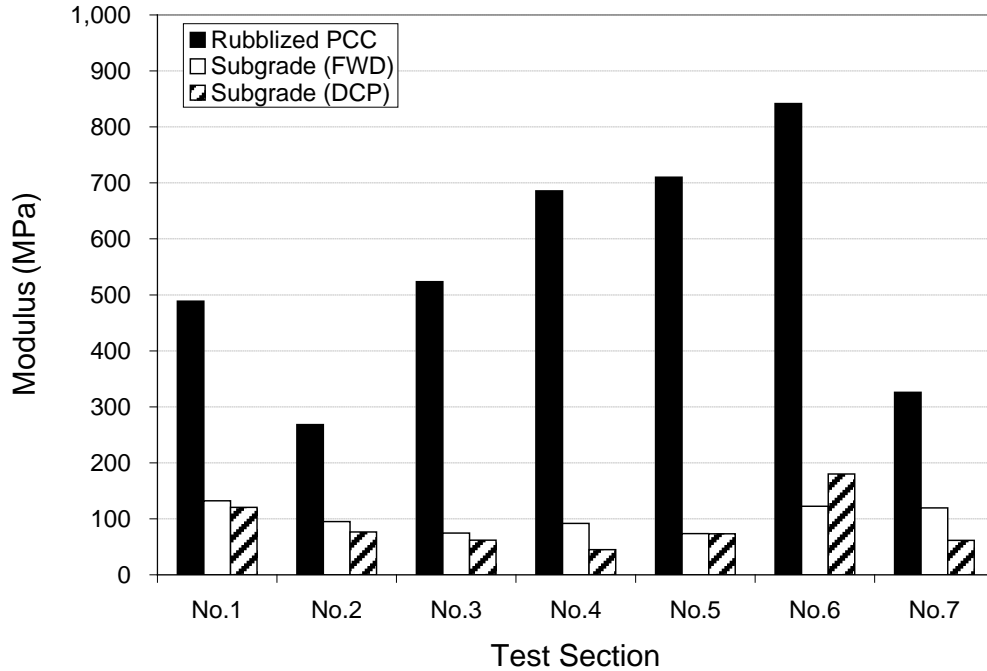
Variable	HMA	Rubblized PCC	Subgrade Modulus (MPa)	
	Modulus* (MPa)		Modulus (MPa)	FWD
Average	12,092	539	100	89
S.D.	9,604	310	28	48
Max	60,416	1,120	140	303
Min	2,268	259	66	34

* At the reference temperature (25°C)

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(a)



(b)

Fig. 7. Pavement layer moduli for test sections: (a) HMA; (b) Rubblized PCC and subgrade

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As shown in Fig. 7 (b) and Table 3, the average subgrade modulus value of 89 MPa (12ksi) obtained from DCP test results is slightly lower than the backcalculated subgrade modulus value of 100 MPa (14 ksi) obtained from FWD data using the ISU layer moduli backcalculation program. This result indicates that the ISU layer moduli backcalculation program provides good predictions for subgrade modulus. The average rubblized pavement subgrade modulus value of 89 and 100 MPa (12 and 14 ksi) meets the minimum strength requirement 69 MPa (10 ksi) of the foundation layers for rubblization project specified by Wisconsin DOT (2007). Considering the fact that the DCP and FWD tests were conducted in summer, the results seem to indicate that the foundation layer of rubblized sections in this study can provide enough strength.

Critical pavement responses

The critical structural responses for the rubblized pavement sections at the reference temperature (25°C) were computed using the ELP. The adjusted HMA modulus (corresponding to a reference temperature of 25°C) was used in the ELP to predict the critical structural responses. Fig. 8 presents the predicted critical structural responses for each of the rubblized PCC pavements. The average tensile strain value of 74 microstrain at the bottom of the HMA layer and the average vertical strain of 235 microstrain at the top of the subgrade are close to the values of 70 microstrain and 200 microstrain for long-lasting HMA pavements. The strain values for individual rubblized pavement sections, except no. 5 (L55 in Mills County), are close to the strain criteria for long-lasting HMA pavements.

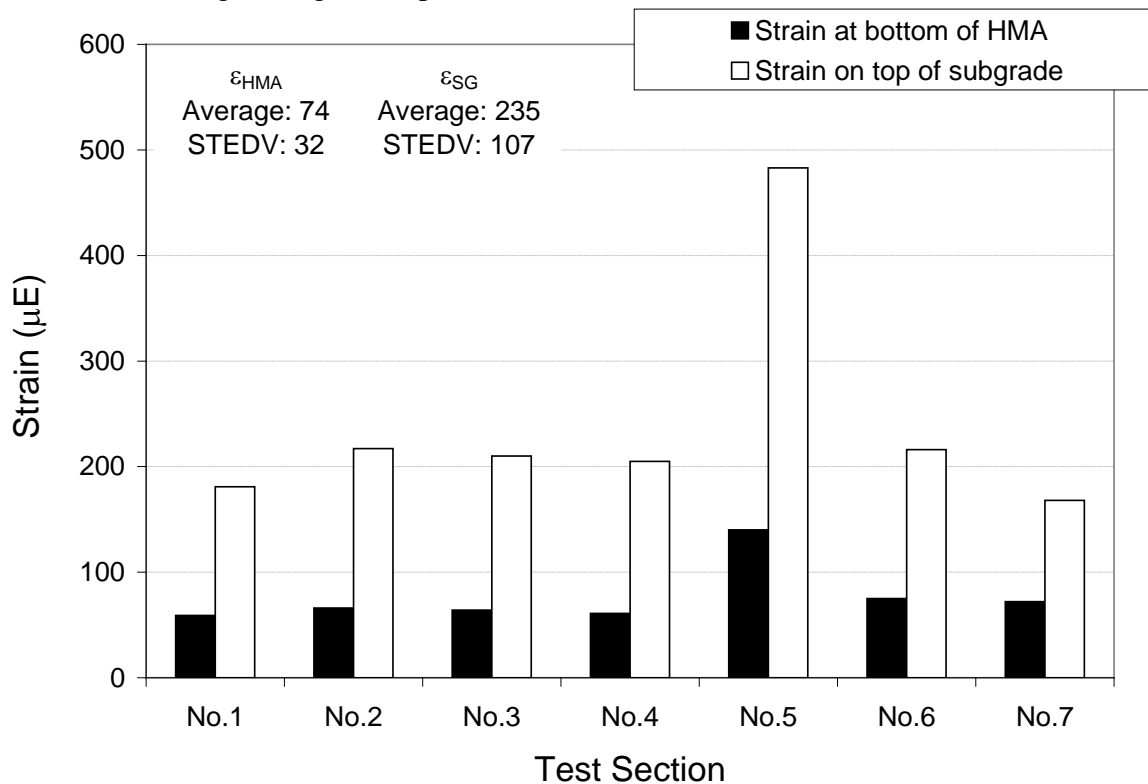


Fig. 8. Predicted critical structural responses

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Layer coefficient of rubblized PCC pavements

Many DOTs and counties use the 1993 AASHTO HMA Overlay Thickness Design method to calculate HMA overlay thickness. The rubblized PCC structural layer coefficient (a_2) is required in this design method. The following is an empirical equation (5), relating a_2 to rubblized PCC layer modulus, suggested by Galal et al. (1999) to provide results equivalent to the layer coefficients derived at the AASHTO Road Test.

$$a_2 = 0.0045\sqrt[3]{E_2} \tag{5}$$

in which, a_2 = rubblized PCC layer coefficient, E_2 = rubblized PCC layer modulus. For this study, the rubblized PCC layer coefficient values obtained for individual rubblized PCC sections are presented on Fig. 9. The average PCC layer coefficient value in this study was found to be 0.19. This is consistent with the range (0.14 to 0.20) used by Arkansas, Michigan, Mississippi, Ohio, and Pennsylvania (Von Quintus et al. 2007). However, those values are lower than a value of around 0.25 used by Minnesota, New York, Indiana and Wisconsin (Von Quintus et al. 2007; Galal et al. 1999).

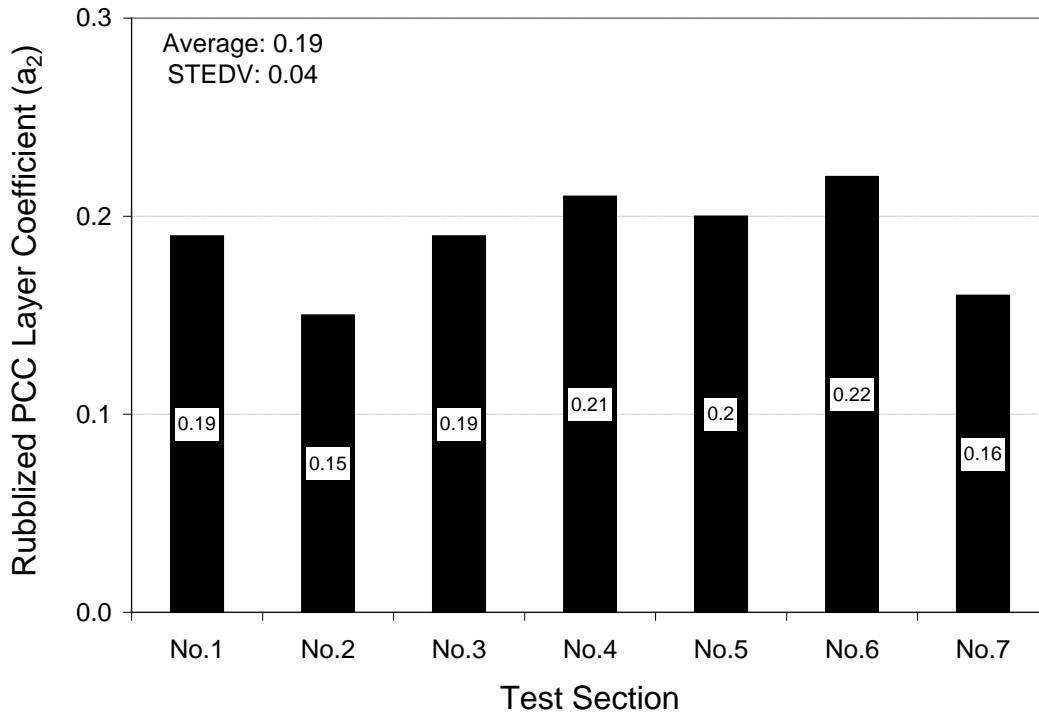


Fig. 9. Rubblized PCC layer coefficient (a_2)

Accuracy of ISU M-E HMA overlay design program

The predictive capabilities of the developed ISU M-E HMA overlay design program were examined by comparing the predicted model thicknesses with the actual HMA overlay

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thicknesses. The predicted HMA overlay thickness errors are presented in Fig. 10. The average predicted and actual thickness ratio value of 1.1 is close to the value of 1.0. The range of ratio values for individual rubblized pavement sections also are not much deviated to the value of 1.0. The actual HMA overlay thickness of in-service rubblized PCC sections are in agreement with the results obtained from the developed HMA overlay thickness design software.

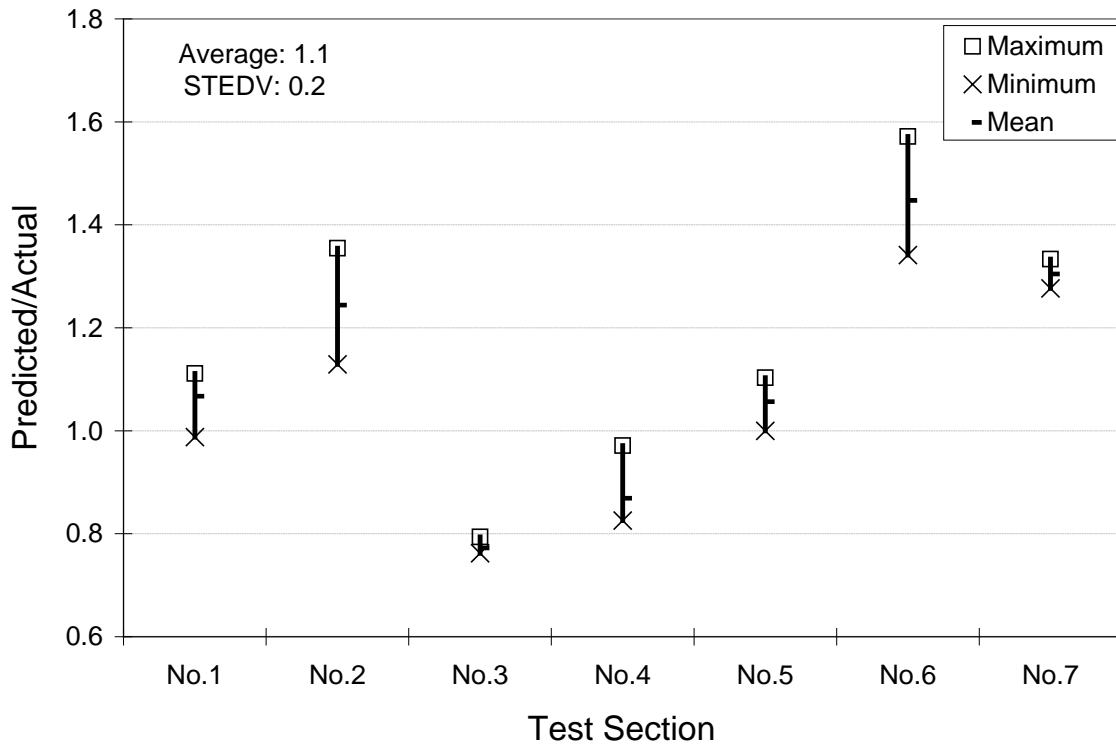


Fig.10. Predicted HMA overlay thickness error

The predicted HMA thickness for test sections 3 and 4 are less than the actual values while the predicted HMA thickness for test sections 6 and 7 are higher than the actual values. However, it should be noticed that the actual layer thickness values for test sections 3 and 4 with lower traffic volume are about the same as those for test sections 6 and 7 with higher traffic volume (see Table 1). These results indicate that the ISU M-E HMA overlay thickness design program can better reflect traffic volume effect in the determination of overlay HMA thickness.

Summary and conclusions

The design properties of rubblized concrete pavements layers have been characterized through field test results and an ISU layer moduli backcalculation program developed. The principal conclusions can be succinctly summarized in the context of the two fundamental objectives raised:

- The average rubblized PCC modulus of the rubblized layer in this study was found to be 539 MPa (78 ksi), which is close to the modulus value of 448 MPa (65 ksi) recommended by the Wisconsin DOT study.

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- The average rubblized PCC layer coefficient value in this study was found to be 0.19, which is consistent with that used by Arkansas, Michigan, Mississippi, Ohio, and Pennsylvania.
- The average tensile strain value of 74 $\mu\epsilon$ (microstrain) at the bottom of HMA layer and the average vertical strain of 235 $\mu\epsilon$ on top of the subgrade are close to the values of 70 $\mu\epsilon$ and 200 $\mu\epsilon$ recommended for long-lasting HMA pavements.
- The M-E HMA overlay thickness design software developed during the first phase of this study seems to reflect the effect of traffic volume reasonably well in the determination of overlay HMA thickness.
- These results indicate the ISU M-E HMA overlay thickness design program can better. The ISU ANN-based backcalculation program provides good predictions for subgrade modulus.

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