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Rehabilitation of concrete pavements utilizing rubblization: a mechanistic based approach to HMA overlay thickness design

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Abstract

In Iowa, there are many Portland Cement Concrete (PCC) type highway pavements. These pavements deteriorate over time due to materials, traffic and environmental related distresses and they are commonly rehabilitated by providing a Hot-Mix Asphalt (HMA) overlay. To mitigate reflection cracking, a frequently observed distress on HMA overlaid PCC pavements, various fractured slab techniques are used, of which rubblization is considered to be the most utilized and effective technique. This paper describes the development of a Mechanistic-Empirical (M-E) thickness design system for HMA overlaid rubblized PCC pavements. In this computerized design procedure, HMA fatigue failure and subgrade rutting failure are considered using Asphalt Institute transfer functions. The design system strain predictions were validated using field results from an instrumented trial section in Polk County, Iowa.

Keywords: PCC rehabilitation; Rubblization; Fractured slab technique; Reflection cracking; Mechanistic-empirical
1. Introduction

In Iowa, there are many Portland Cement Concrete (PCC) type highway pavements which include Jointed Reinforced Concrete Pavements (JRCP), Jointed Plain Concrete Pavements (JPCP), and Continuously Reinforced Concrete Pavements (CRCP). These pavements usually deteriorate due to distresses (cracking, faulting, punchouts, etc.) 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.

Among the various alternatives, construction of Hot-Mix Asphalt (HMA) overlays over existing concrete pavement is considered to be the most common type of rigid pavement rehabilitation (PCS/Law 1991, Freeman 2002, Timm and Warren 2004). Although the use of HMA overlays is regarded as relatively quick and inexpensive measure to repair the deteriorated PCC pavement, their performance has proven less than satisfactory because of the ‘reflection cracking’, which significantly reduces the pavement serviceability (Sherman 1982). The movement of the PCC pavement (caused by traffic loading or thermally induced expansions and/or contractions or a combination of both) causes excessively high strains to develop at the bottom of the HMA overlay, above the joints and cracks, which leads to upward crack propagation, resulting in reflection (or reflective) cracking (Freeman 2002).

In climates such as Wisconsin, the initial reflection cracks often appear within a year or two (Makowski et al. 2005). Over the years, agencies have tried various techniques to eliminate or reduce reflection cracking in HMA overlays. During the past 15 years, the fractured slab techniques, which seek to destroy the slab action by reducing the slab into smaller component sizes, have gained increased acceptance as a means of retarding the formation of reflection cracks (Makowski et al. 2005). Among the fractured slab techniques, the break/seat technique is generally applicable to JRCP, crack/seat for rehabilitation of JPCP while rubblization can be recommended for any type of deteriorated PCC pavement (PCS/Law 1991).

The results from a comprehensive investigation conducted by PCS/Law (PCS/Law 1991), the National Asphalt Pavement Association (NAPA) study (NAPA 1994), and a nationwide survey conducted by the Florida Department of Transportation (DOT) (Ksaibati et al. 1999) all indicate that rubblization is the most utilized procedure for addressing reflection cracking (Heckel 2002). Rubblization involves breaking the existing PCC slab into pieces and overlaying it with HMA. Typical rubblization specifications require a majority of the PCC segment sizes to be less than about 50-75 mm (2-3 in.) at the surface and 225-300 mm (9-12 in.) in the lower part of the slab (Thompson 1999). It has been concluded that the rubblized PCC behaves like ‘a high-strength granular base’ with strength between 1.5 to 3 times greater than a high quality dense graded crushed stone base in load distributing characteristics (PCS/Law 1991).
2. HMA overlay thickness design

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. Classical rigid pavement analysis and design is based upon the Westergaard theory (Westergaard 1939), while classical flexible pavement analysis and design is based upon the Burmister multi-layer theory (Burmister 1945). Based on the assumption that the rigidity of the PCC slabs has been destroyed, the Burmister approach may be used with HMA overlaid fractured PCC pavement. It has been proposed that the Westergaard approach may be used to evaluate the pre-rubblized PCC slabs, whereas Burmister theory may be used for post-rubblization analysis (Bemanian and Sebaaly 1999).

HMA overlay thickness design procedures for rubblized PCC pavements have been proposed by the NAPA (NAPA 1994) and the Asphalt Institute (1989) based on the structural number-layer coefficient principles used in the American Association of State Highway and Transportation Officials (AASHTO) guide (AASHTO 1993). The existing versions of the AASHTO design guide are empirically based on performance equations developed using the 1950s AASHO Road Test data (AASHTO 1962). A nationwide survey conducted in 1999 on the use of rubblization indicated that the AASHTO (1993) is the most widely used design procedure by state DOTs in the USA (Ksaibati et al. 1999).

Thompson (1999) summarized the deficiencies associated with the AASHTO-based procedures and proposed Mechanistic-Empirical (M-E) based design concepts and procedures for the analysis and design of HMA overlay thickness for rubblized PCC pavements.

In the latest Mechanistic-Empirical Pavement Design Guide (MEPDG) released in the USA (NCHRP 2004), the design of an HMA overlay for fractured PCC slabs is similar to the design of a new flexible pavement structure. Typical values for the elastic modulus of the fractured slab layer are recommended in the MEPDG. The design analysis for HMA overlays on fractured slabs consider thermal and alligator cracking, and rutting. Reflection cracking is not considered in the fractured slab analysis (Rodezno et al. 2005).

The HMA overlay thickness design methodology currently used in the state of Iowa is purely empirical. In an effort to shift towards mechanistic-based design, a study was undertaken to develop a mechanistic-empirical design methodology for HMA overlaid rubblized PCC slabs at the Iowa State University under the sponsorship of Iowa Highway Research Board (Kota 2004, Mathews 2004). In this paper, the details of this study are presented.

3. Objective

The primary objective of this study is to develop a Mechanistic-Empirical (M-E) procedure for the design of HMA overlaid rubblized PCC pavements in Iowa. Other objectives include validating the design system structural response predictions using field measurements and to computerize the developed M-E HMA overlay thickness design procedure.

4. M-E HMA overlay design method for rubblized PCC

In figure 1, a generic flow chart of the M-E HMA overlay thickness design method is presented (Kota 2004). 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.

Based on engineering judgment, the thickness of HMA overlay is assumed after material characterization. Critical pavement structural responses such as the tensile strain at the bottom of the HMA overlay and the vertical compressive strain on the top of the subgrade are calculated using elastic layer programs such as JULEA, KENLAYER, etc. Surface deflections can be easily measured using the FWD.

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 assumed pavement thickness satisfies the design requirements for each type of distress.

Figure 1. Flow chart for mechanistic-empirical HMA overlay thickness design.
5. Design inputs

Mechanistic design requires fundamental material properties and material failure criteria as a function of load and environmental effects. Design inputs are divided into three categories: traffic information, material properties and, environmental factors.

5.1 Traffic and Loading

Traffic and loading are considered as important factors in pavement design, which include axle loads, number of load repetitions, tire contact areas, and vehicle speeds. To design a pavement, it is necessary to predict the number of repetitions of each axle load group during a design period, typically 20 years. The design procedures are based on cumulative expected 80 kN (18-kip) Equivalent Single Axle Load (ESAL). The traffic loading used in the design is based on the average traffic during the design period. The minimum traffic information required for the pavement design is the Average Daily Truck Traffic (ADTT) at the start of the design period. In Iowa Counties, traffic is generally estimated in the form of Average Daily Traffic (ADT) and percent truck traffic (Iowa 2004).

A simplified procedure used by the Iowa DOT to calculate the design period ESALs (Iowa 2004) was incorporated into the M-E overlay design approach developed in this study. In this simplified procedure, the base year design lane ESALs (obtained from software generated tables depending on the facility information) is multiplied by the growth factor to obtain the total ESALs for the analysis period.

5.2 Material Properties

The materials considered are HMA, rubblized PCC, unbound/stabilized subbase (if present), and the subgrade. In the M-E design, the elastic modulus (E) and Poisson’s ratio (μ) are the two response properties, which are required to predict the state of stress, strain and displacement within the pavement structure.

Subgrade. The performance of a pavement system primarily depends on the performance of the subgrade. The resilient modulus of the subgrade can be determined through backcalculation of nondestructive test (FWD) data, by performing laboratory testing, or by soil classification. In the developed methodology, the user can either input the FWD test data which is used for backcalculating the subgrade modulus or the user can provide the subgrade soil classification information. A typical Poisson’s ratio of 0.40 is assumed for the subgrade.

In this study, an improved and simplified procedure reported by Hall and Mohseni (1991) for backcalculation of AC/PCC pavement layer moduli using FWD data based on available closed form solutions was used. For a bare PCC pavement, the PCC slab’s elastic modulus ($E_{pcc}$) and the subgrade k value or elastic modulus ($E_s$) may be backcalculated from the maximum deflection $d_0$ and the AREA of the deflection basin. AREA is defined by the following equation:


\[
\text{AREA} = 6 \times \left[ 1 + 2 \left( \frac{d_{12}}{d_0} \right) + 2 \left( \frac{d_{24}}{d_0} \right) + \left( \frac{d_{36}}{d_0} \right) \right]
\]  

(1)

where \( d_0 \) is the maximum deflection at the center of the FWD load plate in mm (in.), and \( d_{12}, d_{24}, \) and \( d_{36} \) are deflections at 305 mm (12 in.), 610 mm (24 in.), and 914 mm (36 in.) from the plate center, respectively.

Ioannides (1990) demonstrated that the AREA parameter is uniquely related to the ‘elastic solid’ radius of relative stiffness (\( l_e \)) of the pavement system. The radius of relative stiffness values are expressed as follows:

\[
l_e = \left[ \frac{E_{\text{pcc}} D_{\text{pcc}}^3 (1 - \mu_s^2)}{6(1 - \mu_{\text{pcc}}^2) E_s} \right]^{\frac{1}{3}}
\]

(2)

where \( E_{\text{pcc}} = \) PCC layer elastic modulus (kPa or psi), \( D_{\text{pcc}} = \) PCC layer thickness (mm or in.), \( \mu_{\text{pcc}} = \) PCC Poisson’s ratio, \( l_e = \) elastic solid radius of relative stiffness (mm or in.), \( \mu_s = \) subgrade Poisson’s ratio, and \( E_s = \) subgrade elastic modulus (kPa or psi).

Hall and Mohseni (1991) determined \( l_e \) as a function of AREA using nonlinear regression:

\[
l_e = \left[ \ln \left( \frac{36 - \text{AREA}}{4521.676} \right) \right]^{0.187}
\]

(3)

Using this estimate of \( l_e \) and AREA, the elastic modulus of the subgrade (\( E_s \)) may be obtained from Losberg’s deflection equation (Losberg 1960):

\[
E_s = \left[ \frac{2P(1 - \mu_s^2)}{d_0 l_e} \right]^{0.19245 + 0.0272 \left( \frac{a}{l_e} \right)^2 + 0.0199 \left( \frac{a}{l_e} \right)^2 \ln \left( \frac{a}{l_e} \right)}
\]

(4)

where \( P \) is the FWD load magnitude and \( a \) is the radius of the load plate.

The developed design methodology uses this approach for estimating the subgrade modulus if the user chooses to input the FWD test data. It is important to note that this backcalculation procedure was originally developed and therefore valid for intact PCC slabs and many PCC pavements that are candidates for rubblization are not intact. Therefore, caution must be exercised in using this procedure.

In the absence of non-destructive test data, soil classification systems can be used to obtain material properties. This is similar to the ‘Level 3’ input in the MEPDG (NCHRP 2004). Note that a new feature in the MEPDG, which is not present in the existing versions of the AASHTO Design Guide, is the hierarchical approach to design inputs. Depending on the desired level of accuracy of input parameter, three levels of input are provided from Level 1 (highest level of accuracy) to level 3 (lowest level of accuracy). Depending on the criticality of the project and the available resources, the
designer has the flexibility to choose any one of the input levels for the design as well as use a mix of levels.

The two commonly used soil classification systems in the USA include the AASHTO Classification System for soils, and the Unified Soil Classification System (USCS). Typical resilient modulus values at optimum moisture content for unbound granular and subgrade materials using both of these classification systems are provided in the MEPDG and these values were used in this study. Typical CBR ranges for the USCS materials and AASHTO classification materials extracted from literature (Yoder and Witczak 1975, NAPA 1994) were used to estimate these resilient modulus values reported in the MEPDG for level 3 input. It has been reported that these values are very approximate as they are based on Long-Term Pavement Performance (LTPP) averages and therefore must be used with significant caution (NCHRP 2004). This module of the developed design approach is currently under revision. Since the subgrade soil classification inputs are not very accurate and the MEPDG recommended values are under review, a provision will be made in the developed design system for the user to provide his/her own subgrade modulus input.

**Subbase.** In Iowa counties, typically an unbound/stabilized subbase layer is used on top of the subgrade (Mathews 2004). Since this will introduce an additional factor of uncertainty in the already complicated analysis of HMA overlaid rubblized PCC pavements, it was decided to combine the subbase and subgrade layers to analyze the overall pavement structure as a three layer interface in this study. For this, the concept of equivalent thickness developed by Odemark (1949) was used.

Odemark’s (1949) concept of equivalent thickness is based on the assumption that the stresses and strains below a layer depend on the stiffness of that layer only. If the thickness, modulus and Poisson’s ratio of layer are changed, but the stiffness remains unchanged, the stresses and strains below the layer should also remain unchanged. In other words, the deflection produced by a particular load in the equivalent subgrade should be equal to the deflection produced by the same load in the original condition. Using this concept, the subbase and subgrade was combined into a single layer of equivalent thickness, $H_e$:

$$H_e = h_2 \sqrt[3]{\frac{E_2}{E_3}}$$

where $h_2$ is the thickness of the subbase, $E_2$ is the subbase modulus and $E_3$ is the subgrade modulus.

**Rubblized PCC Slab.** It has been concluded that the rubblized PCC layer is superior to a high-quality granular layer in terms of load distributing characteristics (Thompson 1999). Typical modulus values recommended in the MEPDG (for Level 3 input) for rubblized PCC layers fall in the range of 345 MPa (50,000 psi) to 1,034 MPa (150,000 psi) (NCHRP 2004). In the developed design methodology, the user has the choice to select a modulus value for the fractured slab layer within this range. A Poisson’s ratio of 0.35 is assumed.

It is acknowledged that these moduli values have not been validated against experience in Iowa. There is an ongoing research effort related to the performance evaluation of rubblized pavements in Iowa which will address this issue. Future research efforts will focus on collecting performance related data from rubblized pavements in Iowa for validating the developed design procedure as well as for recommending AASHTO layer coefficient values and rubblized PCC layer moduli values for structural design of rubblized concrete pavements. Data collected during 2003 and 2004 from projects rubblized between 1997 and 2003 indicate a total of 21 rubblization projects in Iowa.

**Hot-Mix Asphalt (HMA).** The primary stiffness property of interest for HMA is its dynamic modulus, \( E^* \), as a function of loading frequency and temperature. Of all the dynamic modulus predictive equations, the one developed by Witzcak and Fonseca (1996) is considered as one of the most comprehensive models as it allows for the evaluation of dynamic modulus for a wide variety of asphalt mixtures/properties (NCHRP 2004). This equation has been included in the MEPDG for prediction of \( E^* \).

The HMA moduli values were obtained for the coldest (Estherville) and hottest weather stations (Burlington) in Iowa using the MEPDG software for each month. The moduli were calculated at various HMA thicknesses: 12.7, 25.4, 50.8, 76.2, 101.6, 203.2, and 406.4 mm (0.5, 1, 2, 3, 4, 8, and 16 inches). A general observation showed that the HMA moduli values of Estherville and Burlington were very similar since the temperatures do not vary significantly across the state in general. By interpolating the moduli values between these two places, Iowa statewide HMA moduli values were determined for each month and the results were tabulated. The monthly average HMA modulus for a given HMA overlay thickness can be determined using this table (Kota 2004).

6. Design process

The design process and the steps followed in the development of the HMA overlay thickness design system for rubblized PCC pavements are discussed in this section.

6.1 Inputs

Traffic input consists of the anticipated design period traffic, in terms of equivalent single axle loads (ESAL). The design period is generally assumed to be 20 years.

All existing and overlay pavement components must be defined in the design process, both by geometry (thickness) and by material characteristics (resilient modulus and Poisson’s ratio).

The thickness of the HMA overlay is an unknown parameter in the design. Therefore, the thickness must be initially assumed and the design process is repeated several times iteratively to achieve the design thickness. Once the design inputs are known, the next step is to calculate the critical pavement responses using a pavement structural model.
6.2 Structural Analysis

The critical structural responses considered in this design process include the horizontal tensile strain at the bottom of HMA layer (\(\varepsilon_t\)) and the vertical compressive strain on the surface of subgrade (\(\varepsilon_c\)). The popular multi-layer linear elastic analysis program, JULEA, was selected for computing the structural responses. A rapid structural analysis approach was developed in this study to facilitate design computations in batch mode. Given that both the critical strains are functions of geometry and elastic modulus of each pavement layer, it should be feasible to estimate the strains given the values of layer thickness and the moduli, using a regression model or a trained Artificial Neural Network (ANN) model.

A synthetic database was generated using JULEA by computing the critical strains (\(\varepsilon_t\) and \(\varepsilon_c\)) for a wide range of layer thicknesses and moduli values. The HMA layer thicknesses varied between 51 to 305 mm (2 to 12 in.) and the rubblized PCC layer thicknesses between 152 to 356 mm (6 to 14 in.) in 50-mm (2-in.) increments. The moduli values ranged from 1,724 to 13,790 MPa (250,000 to 2,000,000 psi) for HMA; 345 to 862 MPa (50,000 to 125,000 psi) for the rubblized PCC, and 34.5 to 345 MPa (5,000 to 50,000 psi) for the subgrade. A total of 2,600 data sets were generated based on different combinations of the layer thicknesses and moduli values.

Prediction models were derived by relating the critical strains to layer thicknesses and moduli values using the synthetic database. As part of the future work, a pavement structural model which can accommodate stress-dependent resilient modulus models for the unbound granular base and subgrade materials will be utilized to enhance the predictive capabilities of the design system.

Prediction of subgrade compressive strain (\(\varepsilon_c\)). A second-order polynomial regression analysis was performed between the computed values of \(\varepsilon_c\) and equivalent thicknesses, \(H_{eq}\). A linear relation was obtained between \(\varepsilon_c\) and \(H_{eq}\) for different levels of subgrade modulus when the results were plotted on a log-log plot. The \(R^2\) values were above 0.98 indicating a strong correlation between the two variables for any given subgrade modulus. A second-order polynomial equation was obtained to predict the vertical compressive strain on top of the subgrade (\(\varepsilon_c\)) as a function of \(H_{eq}\). A shift factor \(\alpha\), dependent on the subgrade modulus, was observed and it was determined using the Solver function in Microsoft Excel. The final prediction model for \(\varepsilon_c\), as incorporated in the overlay thickness design Visual Basic program, is as follows:

\[
\log(\varepsilon_c) = -0.19445 - 1.50787 \left( \log(H_{eq}) + \alpha \right) - 0.07794 \left( \log(H_{eq}) + \alpha \right)^2
\]

\[
R^2 = 0.9911, \text{ SEE } = 0.000016
\]

where

\[
H_{eq} = h_1 \left( \frac{E_3}{E_1} \right)^{\frac{1}{3}} + h_2 \left( \frac{E_3}{E_2} \right)^{\frac{1}{3}}
\]

\[
\alpha = 0.568474 \ln(E_3) - 1.93734
\]
where $E_1$, $E_2$, and $E_3$ refer to moduli of HMA, rubblized PCC, and subgrade, respectively; and $h_1$ and $h_2$ are thicknesses of HMA and rubblized PCC layers, respectively. The predicted $\varepsilon_c$ values are compared with the computed $\varepsilon_c$ values in figure 2 (a).

![Graph](image1)

(a)

![Graph](image2)

(b)

Figure 2. Predicted strains versus JULEA calculated strains: (a) subgrade compressive strain ($\varepsilon_c$) and (b) HMA tensile strain ($\varepsilon_t$).

**Prediction of HMA tensile strain ($\varepsilon_t$).** Conventional linear and non-linear regression techniques did not lead to a satisfactory relationship between the horizontal tensile strain at the bottom of the HMA layer, $\varepsilon_t$ and the pavement layer parameters. Therefore an Artificial Neural Networks (ANN) based approach was used for predicting $\varepsilon_t$. Over the past few years, several studies have successfully demonstrated the use of trained ANN for...

Artificial neural networks (ANN) are statistical models of real world systems which are built by tuning a set of parameters. The mathematical model that a neural network builds is actually made up of a set of simple functions linked together by the weights. The weights describe the effect each simple function, known as unit, will have on the overall model. The network has a set of input units whose function is to take input values from outside, a set of output units which report the final answer and a set of processing hidden units which link the inputs to the outputs. The function of the hidden units is to extract useful features from the input data which are, in turn, used to predict the values on the output units. The network is arranged in layers of units: an input layer, one or more hidden layers, and an output layer (Haykin 1999).

In this study, 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 \( \varepsilon_t \). Backpropagation type ANNs are very powerful and versatile networks that can be taught a 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 \( \varepsilon_t \), five inputs, \( E_1, h_1, E_2, h_2, \) and \( E_3 \) were used. Based on the parametric analysis, two hidden layers with 50 nodes in each layer were found to be sufficient in this case. Thus, the final ANN architecture could be represented as 5-50-50-1 (5 inputs, 50 nodes in the 1st and 2nd hidden layers, and 1 output node, respectively). The same synthetic database used for developing the \( \varepsilon_c \) regression prediction model was utilized for ANN training and testing. Out of the 2,600 data sets, 2,200 data sets were used for training the ANN and the remaining 400 data sets were used for testing. The Root Mean Squared Error (RMSE) was used to track the performance of the network during the training process. Once the network was successfully trained, it was tested using the 400 data vectors. Excellent agreement was found between the ANN predicted \( \varepsilon_t \) values and the target \( \varepsilon_t \) values, as shown in figure 2 (b). This ANN prediction model was included in the design system for estimating \( \varepsilon_t \). It is noted that an ANN-based prediction model was also developed for estimating \( \varepsilon_c \) which showed higher \( R^2 \) values compared to the regression approach. It is anticipated that the regression-based \( \varepsilon_c \) prediction model will be replaced by the ANN-based model in the updated version of the developed design program.

7. Transfer functions

In the design process, the critical strains, \( \varepsilon_t \) and \( \varepsilon_c \) are estimated for each month of the design year using the prediction models described in the previous section. The monthly HMA modulus values (Kota 2004) are used to estimate the monthly strains. The estimated monthly strains are then mapped to capacity using the appropriate transfer functions. The fatigue and rutting equations developed by the Asphalt Institute (1981) were used in this study to estimate the capacities. It is important to note that these transfer functions are not necessarily representative of the many HMA that are currently used.

In the Asphalt Institute (1981) design method, the allowable number of load repetitions \( N_f \) to cause fatigue cracking is related to the tensile strain \( \varepsilon_t \) at the bottom of the HMA and to the HMA modulus \( E_1 \). For the standard mix used in design, the Asphalt Institute (1981) equation for 20% of area cracked is as follows:

\[
N_f = 0.0796(\varepsilon_t)^{-3.291}(E_1)^{0.854}
\]  
(7)

Typically, transfer functions such as these are developed through laboratory testing and the results are then related to the field through the application of shift factors. Frequently, very large shift factors are needed to reconcile ‘laboratory fatigue’ and ‘performance-derived fatigue’ relations. Over the years, many researchers have proposed HMA fatigue algorithms (Newcomb et al. 1983, NCHRP 1990) and many highway agencies have developed their own based on laboratory testing and field experience. In other words, there is not a ‘unique’ HMA fatigue algorithm. Therefore, future research efforts will include modification of the design software so that the user can input an HMA fatigue algorithm of their choosing. The Asphalt Institute (1981) equation for estimating the allowable number of load repetitions to cause subgrade rutting, \( N_d \), is as follows:

\[
N_d = 1.365 \times 10^{-3}(\varepsilon_t)^{-4.477}
\]  
(8)

8. Capacity ratio estimates

Using the Miner’s law (Miner 1945), the total capacity ratios are computed for each distress using the following equation:

\[
\sum_{i=1}^{12} \left( \frac{n_i}{12} \right) N_{f_i}
\]

where \( (n/12)_i \) represents the total traffic anticipated in month \( i \), over the design period. This ratio should be numerically less than or equal to unity. Proper care must be taken so that the capacity ratios for both the failure types (HMA fatigue and subgrade rutting) will simultaneously meet the failure criteria.

According to Thompson (1999), HMA fatigue is the controlling overlay thickness design criterion for practically all rubblized PCC pavements. However, adequate in-situ subgrade strength is essential for successfully supporting rubblization and high-quality HMA overlay construction operations. The combined thickness of rubblized PCC and subbase protects the subgrade during HMA overlay construction and field experience has shown that rubblized PCCs with larger PCC segment sizes provide more effective cover than do smaller PCC segments (Thompson 1999). According to Illinios DOT’s Subgrade Stability Manual which is used to consider constructability issues in Illinois, if the subgrade strength is not adequate during construction, it is recommended to increase the PCC segment size (to a maximum of about 300 mm), or it may be necessary to employ innovative HMA delivery procedures to supply the paver. In

projects with very low subgrade strengths, rubblization may not be the best option (Thompson 1999).

The M-E design process developed in this study for determining the thickness of HMA overlay for rubblized PCC pavements was coded into a Visual Basic program with a user-friendly graphical interface to facilitate design calculations. Screenshots of the thickness design program are shown in figure 3.

![Figure 3. Screenshots of HMA overlay thickness design program for rubblized PCC pavements: (a) traffic screen; (b) slab treatment screen; and (c) subgrade support screen.](image-url)

9. Validation of structural response predictions

The successful use of transfer functions is predicated upon the use of appropriate values of input strains. The mechanistic predictive capabilities of the design program developed in this research were validated by comparing the model predictions with the results obtained from instrumented trial sections on highway IA-141 in Polk County, Iowa. The test site is located approximately one mile north-west of the I-80/I-35 junction near Des Moines, Iowa. A brief description of the test sections follow.

9.1 Instrumented trial sections

As shown in figure 4, there were four instrumented rubblized PCC test sections labeled T9 thru T12 on highway IA-141. The instrumented test sections were located in the southbound lanes. The strain gauges were placed in the outside lane.

Sections T9 and T10 comprise a nominal 190-mm (7.5-in.) HMA overlay over a 254-mm (10-in.) rubblized PCC slab while sections T11 and T12 comprise a nominal 230-mm (9-in.) HMA over the same 254-mm (10-in.) rubblized PCC slab. The PCC slab was rubblized using an Antigo® Multi-Head Breaker, covering the full width of the lane (see figure 5). The rubblized slab exhibited smaller pieces in the top half (approximately 25.4-mm [1-in.] to 76.2-mm [3-in.] size), while the bottom half comprised of particles up to about 8-in (see figure 6).

High quality AC strain gauges (manufactured by Dynatest PAST 2AC™) were used for measuring the tensile strains at the bottom of the HMA overlay (see figure 7). These were located in-line with the anticipated outer wheel path, and placed on the surface of the rubblized PCC slab by embedding in sand/bituminous emulsion slurry.

Sections T9 and T12 each had two strain gauges spaced 0.61 m (2 ft) apart (labeled 2 and 3; 10 and 11, respectively), while Sections T10 and T11 had three gauges spaced 0.61 m (2 ft) apart (labeled 4, 5, and 6; 7, 8, and 9, respectively) (see figure 4). At the location of each set of gauges, a thermocouple was installed to allow measurement of the temperature at the bottom of the overlay/top of the rubblized PCC interface.

The HMA overlay mixtures were conventionally placed and compacted using paver and rollers. Care was taken during the placement of the first base layer to avoid damage to the gauges by direct contact with the paver tracks or by displacement. The construction operations on these test sections were undertaken on September 15, 2001. All instrumentation was installed between construction operations.

Figure 4. Rubblized instrumented trial sections on highway IA-141, Polk County, Iowa.

Figure 5. Rubblization of highway IA-141 section using a multi-head breaker.

Figure 6. Close-up of rubblized PCC slab.

Figure 7. HMA strain gauge instrumentation on highway IA-141.
9.2 Subsequent testing

After construction and on three subsequent occasions, the trial sections were revisited and a series of tests were performed. The Iowa DOT FWD equipment was located as closely as possible over each strain gauge. Three drops were made (nominally 40-kN [9,000-lb] load) and the surface deflections were recorded. The peak strains measured in the embedded gauges were also recorded. An Iowa DOT truck, loaded to closely simulate a ‘standard’ 80-kN (18,000-lb) axle loading condition, was driven over the gauges at creep speed. The strain history (strain vs. time) in the gauges was measured. The temperature measured at the bottom of the HMA overlay was recorded.

Table 1 summarizes the tensile strain values at the bottom of the HMA overlay obtained from the testing done on different trial sections on IA-141. The pavement layer moduli values were backcalculated from the FWD data using the MODULUS backcalculation program (Uzan 1988) and the results are reported in table 1 (Ceylan et al. 2005b). Also, the HMA tensile strain values obtained under the FWD load and the standard truck axle load are compared with the ANN model predictions in table 1. The ANN model predicted strain values were obtained using the moduli values backcalculated from the FWD data. It is interesting to note that the HMA strains obtained under creep speed truck loading are lower than those obtained from FWD loading in general, while the opposite is intuitive (i.e., increased frequency for the FWD \(\rightarrow\) higher HMA modulus \(\rightarrow\) lower HMA strains). The reason for this behavior is not known although part of it could be attributed to the difficulty of aligning the truck directly over the strain gauges and the dual tire configuration of the truck.

From table 1, it can be observed that the strains recorded on gauges 10 and 11 have consistently provided low values when compared to gauges 7, 8, and 9. It is suspected that this may be due to a misalignment of these gauges during the construction process.

Table 1. Comparison of field measured and ANN predicted HMA tensile strains.

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<th>Month</th>
<th>AC Strain Gauge</th>
<th>HMA Thickness, mm</th>
<th>Backcalculated Modulus, MPa</th>
<th>HMA Tensile Strain (microstrain)</th>
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<th>Truck</th>
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9.3 Key findings

Tensile strains at the bottom of the HMA layer recorded under the FWD are generally in agreement with the ANN model predicted results, thus providing a measure of validity to the use of the model in the HMA overlay thickness design program.

Strains recorded under the ‘standard’ 80-kN (18,000-lb) truck axle loading are more variable and of generally lesser magnitude than those under the FWD. This can partly be attributed to the difficulty of aligning the truck directly over the gauges and the dual tire configuration.

The asphalt strain gage measurements from the trial sections and the design model strain predictions were less than or close to 70-microstrain both for 230 mm (9.0 in.) and 190 mm (7.5 in.) thick pavement sections even in peak summer period where high pavement temperatures exist. This suggests that, in this case, an HMA overlay thickness of 190 mm (7.5 in) or even lower would have been sufficient in place of a 230-mm (9-in.) HMA overlay to resist against HMA fatigue failure. Note that the 70-microstrain limit is considered as limiting strain value for long-lasting HMA pavements (TRB 2001).

More structural condition assessment data from existing rubblized pavements in Iowa is required to validate the developed design procedure and provide rubblized PCC layer elastic modulus recommendations for future design of rubblized concrete pavements. Future research efforts will also address questions such as: are the HMA overlay thickness values obtained from the developed design approach reasonable? how do they compare with other design procedures? how are the required thicknesses sensitive to the assumed PCC modulus and other input parameters? etc.

10. Summary and observations

Various research studies have reported that amongst the several available fractured slab techniques for deteriorated PCC pavements, rubblization is considered to be the most utilized approach and that it is a viable, rapid and cost-effective rehabilitation method.

In this paper, the development of a HMA overlay design system for rubblized PCC slabs in Iowa based on a mechanistic-empirical design approach was discussed. A multi-layer elastic analysis program was used to generate a database of results for developing regression and ANN (artificial neural networks)-based models for predicting the critical structural responses. In this design procedure, failure criteria such as the tensile strain at the bottom of HMA layer and the vertical compressive strain on top of the subgrade layer were used to consider HMA fatigue and subgrade rutting, respectively. Design parameters, including material properties, traffic, and environmental factors were established.

The developed mechanistic-empirical design system was also implemented in a Visual Basic computer program with a user-friendly interface. The strain predictions using the developed design system were validated using results from an instrumented trial project at highway IA-141 located in Polk County, Iowa.

Acknowledgments/Disclaimer

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