Hot-Mix Asphalt Overlay Design Concepts for Rubblized Portland Cement Concrete Pavements

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The Illinois Department of Transportation's (IDOT's) first interstate rubblization project was constructed in 1990 on I-57 near Pesotum as part of a rehabilitation study. Hot-mix asphalt overlays (HMA OL) were constructed over a rubblized jointed reinforced concrete pavement (JRCP) with a granular subbase. Excellent performance was achieved on the project, which had accommodated 7.5 million equivalent single-axle loads through 1998. Periodically collected falling weight deflectometer (FWD) data indicate the sections have retained their structural capacity and integrity. HMA fatigue distress has not developed. The success of the project prompted IDOT to consider portland cement concrete pavement (PCCP) rubblization and HMA OL a viable and cost-effective rehabilitation option. Rubblization is particularly appropriate for eliminating reflective cracking and for use when PCCP patching quantities are high or concrete deterioration is in an advanced stage. HMA OL fatigue considerations control the HMA OL thickness requirement for rubblized PCCPs. HMA OLs for rubblized PCCPs, generally in the 150-mm to 250-mm (6-in. to 10-in.) range, are thicker than those used in the traditional PCCP rehabilitation and HMA OL projects. IDOT uses mechanistic-empirical (M-E) flexible pavement design procedures for full-depth asphalt and conventional flexible pavements. M-E–based HMA OL design procedures for rubblized PCCPs are being developed. In the M-E design procedures, the tensile strain at the bottom of the HMA layer is used to consider HMA fatigue. The structural behavior and HMA OL fatigue performance of the Pesotum sections are used in the initial development of the M-E–based HMA OL design procedures for rubblized PCCPs. Results from other rubblized PCCP projects are being used to refine and validate the M-E design concepts.

Various portland cement concrete pavement (PCCP) types were originally constructed on the Illinois Interstate system. The PCCPs included jointed reinforced concrete pavements (JRCP), jointed concrete pavements (JCP), and continuously reinforced concrete pavements (CRCP). Major PCCP distresses are slab cracking, joint deterioration and faulting, CRCP "punchouts," PCC deterioration (extensive D-cracking) and hot-mix asphalt overlay (HMA OL) reflective cracking and subsequent crack deterioration. PCC deterioration extent and severity increase considerably as the pavements age.

Many of the original PCCPs on the Illinois Interstate system have been rehabilitated. The Illinois Department of Transportation (IDOT) used repair and rehabilitation techniques such as slab jacking, slab patching, joint repair and replacement, diamond grinding, milling, and HMA OL in initial Interstate PCCP rehabilitation projects. Some PCCPs have been rehabilitated more than once. Many complicating factors arise in selecting PCCP rehabilitation options, particularly when the section has been rehabilitated previously.

Fractured slab techniques have been used for many years as a rehabilitation option. Before 1990, IDOT had not used a fractured slab technique on an Interstate PCCP rehabilitation project.

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FRACTURED SLAB TECHNIQUES

Thompson's 1989 NCHRP Synthesis of Highway Practice (*1*) summarized breaking/cracking/seating (B/C/S) practices and technology. The stated primary goal of B/C/S was to reduce (preferably eliminate) HMA OL reflective cracking. IDOT constructed and monitored several B/C/S with HMA OL projects in the 1980s, but not on Interstate routes. Follow-up monitoring studies (*2*) indicated that the B/C/S techniques delayed, but did not eliminate, HMA OL reflective cracking. The delay period typically varied from 3 to 5 years, and longer delays were achieved with the thicker HMA OLs. Similar trends were noted in the NCHRP Synthesis (*1*) and a comprehensive nationwide National Asphalt Paving Association (NAPA) study published in 1991 (*3*).

At the time of the NCHRP Synthesis, rubblization applications were not as widespread as B/C/S, but several states had used the procedure. The NAPA study (*3*) indicated rubblization was the most effective procedure for addressing reflective cracking. Rubblization destroys PCCP slab continuity and eliminates transverse joints and the associated joint opening and closing that cause reflective cracking. PCCP rubblizing breaks the concrete into pieces that are substantially debonded from any steel reinforcement.

Typical rubblization specifications require a majority of the PCCP 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. In some instances, near surface segment size requirements are relaxed, and larger sizes are permitted. In recent years, rubblization with HMA OL also has been used in PCCP rehabilitation construction when concrete durability problems (D-cracking, alkali-silica-reactivity), excessive PCCP patching requirements, or unacceptable slab faulting are encountered. In projects in which construction and lane closure time are limited, rubblization is an attractive option and can be built "under traffic" in a shorter period (compared with conventional PCCP rehabilitation techniques).

RUBBLIZATION EXPERIENCE IN ILLINOIS

IDOT's first Interstate rubblization project was constructed in 1990 on I-57 near Pesotum, as part of an SHRP rehabilitation study. HMA OLs 150 and 200 mm (6 and 8 in.) thick [sections 150 m (500 ft) long] were constructed over a rubblized 250-mm (10-in.) JRCP with a 150-mm granular subbase.

Excellent performance was achieved on the Pesotum project, which had accommodated approximately 7.5 million equivalent single-axle loads (ESALs) through the summer of 1998. Periodically collected falling weight deflectometer (FWD) data indicate the sections have retained their structural capacity and integrity. A 1997 IDOT report (unpublished) indicates the sections are performing very well. Rutting is minimal [3.5 mm (0.14 in.) for the 200-mm section and 4.8 mm (0.19 in.) for the 150-mm section], and HMA fatigue has not developed. The sections show a limited amount of transverse cracking (probably thermal cracking) and block cracking. The current international roughness index (IRI) values are 120 cm/km (79 in./mi) for the 200-mm section and 143 cm/km (94 in./mi.) for the 150-mm section. The 200-mm HMA OL rubblized section has the lowest IRI of all the experimental rehabilitation treatments in the IL SPS-6 project. IDOT also constructed two rubblized JRCP projects on low traffic volume state routes in the early 1990s. These projects also have performed very well. IDOT's favorable rubblization experience (particularly the 1990 I-57 project) prompted the planning and construction of a larger-scale rubblization project on an Interstate route carrying a moderate traffic volume.

The new project, 5.3 km (3.2 mi) long, was constructed on I-57 (northbound lane) near Edgewood in 1996. The original pavement was a 200-mm CRCP over a 100-mm (4-in.) bituminous aggregate mixture (BAM) subbase. The primary HMA thickness was 200 mm, but a short 150-mm HMA OL section also was constructed.

The 10-year design traffic was 7.1 million ESALs. A multiple head breaker capable of rubblizing PCCP lane width in one pass was successfully used on the project. A comprehensive paper (*4*) describing the Edgewood project has been published. The success of the Pesotum and Edgewood projects prompted IDOT to consider PCCP rubblization and HMA OL a viable and cost-effective PCCP rehabilitation option that is particularly appropriate for eliminating reflective cracking and for use when PCCP patching quantities are high or concrete deterioration is in an advanced stage.

In 1997, IDOT constructed two additional rubblized PCCPs with HMA OLs on Interstate projects:

- I-57 (southbound lane) in Union County. The original pavement was a 250-mm CRCP over a 150-mm granular subbase. The project was 4.2 km (2.5 mi) long. The 20-year design traffic was 13.3 million ESALs, and the HMA OL was 225 mm. The rubblization specification was that a majority of the PCCP segments be less than 75 mm (3 in.) above the steel and the majority be less than 225 mm below the steel. A "coarser" rubblization specification (75 percent of the PCC segments less than 225 mm and maximum PCC segment size of 300 mm) was used on a 150-m (500-ft) section.
- I-70 (westbound lane) at Greenup. The original pavement was a 200-mm CRCP over a 100-mm BAM subbase. Extensive rehabilitation (concrete patching, full-depth HMA patching, and HMA OLs) previously had been constructed on this project. The project was 4.3 km (2.6 mi) long. The primary HMA OL thickness was 250 mm, but two short [300 m (1000 ft) long] 225-mm and 275-mm (11-in.) HMA OL sections also were constructed. The 15-year design traffic was 28.3 million ESALs.

Periodic pavement condition surveys and FWD testing have been conducted on all IDOT Interstate rubblized PCCP projects. The sections generally are in excellent condition, except for some greater-than-expected HMA OL rutting (a mixture problem) that has occurred on the Union County project.

Rubblized IDOT Interstate projects constructed to date include a range of HMA OL thicknesses from 150 to 275 mm, JRCP and CRCP pavement types of varying thicknesses, and granular and BAM subbases. Experience gained from constructing this range of sections has contributed to the development of improved specifications and construction procedures. The pavement response and performance information and data from the sections facilitate the continuing evaluation of the PCCP rubblization procedure.

A current concern is the determination of appropriate HMA OL thicknesses for a range of project variables. Factors such as PCCP pavement type and thickness, subbase type and thickness, rubblization specifications, and subgrade properties are under consideration.

RUBBLIZED PAVEMENT BEHAVIOR

A rubblized and compacted PCCP is an assemblage of PCC segments that form a tightly keyed, interlocked, high-density material layer. A rubblized PCCP layer is fractured, lacks continuity, and cannot sustain a flexural stress. However, it possess high shear strength and rutting resistance. It is not a typical granular material.

The NAPA study (*3*) indicated that a rubblized PCCP layer behaves like an "unbound material": The product of the rubblization process leaves an in situ layer of material similar to the appearance of unbound base layers. However, it has been concluded that the inherent strength of the rubblized layer is between 1.5 and 3 times as effective in load distributing characteristics as a high-quality dense-graded crushed stone base.

University of Illinois analyses of several sets of IDOT FWD data for the rubblized PCCP section on I-57 at Pesotum indicate the rubblized PCCP layer is superior to a high-quality granular layer. The data in Figure 1 show the expected 40-kN (9-kip) FWD deflections for a conventional flexible pavement (with a high-quality dense-graded granular base) compared to the measured FWD deflection for the rubblized pavement. The deflection estimates are based on an ILLI-PAVE–based design algorithm developed by Thompson and Elliott (*5*).

$$\text{Log } \Delta = 1.9692 + 0.0465\,T_{AC} - 0.5637\,(\text{Log } T_{GR})/T_{AC} - 0.0464\,(\text{Log } E_{AC})\,T_{AC} - 0.2079\,(\text{Log } E_{Ri}) \quad (R^2 = 0.95\ \text{SEE} = 0.046) \qquad (1)$$

where

Δ = surface deflection, mils;
T_{AC} = AC surface thickness, inches;
T_{GR} = granular base thickness, inches;
E_{AC} = AC modulus, ksi;
E_{Ri} = breakpoint resilient modulus of subgrade, ksi (the resilient modulus at a repeated deviator stress of approximately 6 lbf/in.^{2});
R^2 = coefficient of determination; and
SEE = standard error of estimate.

E_{AC} and subgrade E_{Ri} were backcalculated from the FWD data. Thus, a direct comparison can be made between the rubblized PCCP layer and the granular base. It is apparent that the rubblized PCCP layer is more effective in load distribution.

The NAPA study (*3*), a recent Indiana Department of Transportation (INDOT) report (*6*), and other available data indicate that backcalculated moduli for rubblized PCCP layers are considerably higher than for conventional granular bases. University of Illinois mechanistic-empirical (M-E) analyses of IDOT FWD data for the Pesotum I-57 Interstate sections also show the rubblized PCCP layer modulus is superior to that of a high-quality granular subbase.

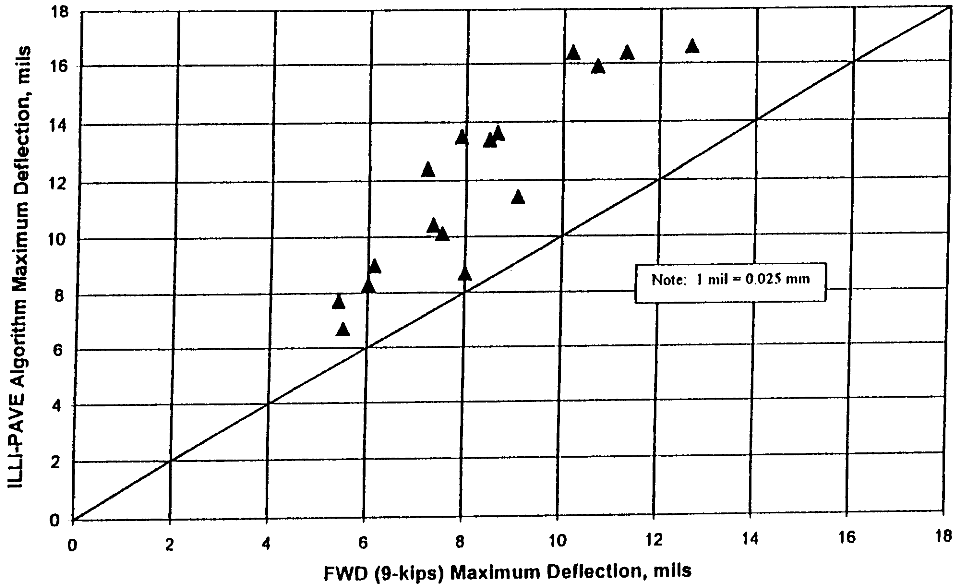


FIGURE 1 Conventional flexible pavement deflection estimates—FWD deflections for Pesotum rubblized PCCP sections.

Granular materials display "stress hardening" behavior. Their moduli increase as stress state increases. Limited IDOT FWD data for the HMA OL–rubblized PCCP section on I-57 in Union County demonstrated that for load levels ranging from 27 kN to 40 kN (9 kips) to 53 kN (12 kips) on a 300-mm-diameter load plate, the PCCP layer did not display stress-dependent behavior. The rubblized PCCP layer is not a conventional stress-hardening granular material.

It is apparent that a rubblized PCCP cannot be considered a traditional material. Thus, new structural analysis and design approaches must be considered.

HMA OVERLAY THICKNESS

General

HMA fatigue considerations control OL thickness requirements for rubblized PCCP. The critical HMA strain occurs at the bottom of the HMA OL. In contrast, HMA fatigue is not the failure mode for a traditional HMA OL constructed over a rehabilitated PCCP. Rubblized PCCP, subbase, and subgrade rutting generally are not controlling design criteria. HMA rutting, thermal cracking, and durability (weathering and stripping) are important mixture design considerations.

HMA OLs for rubblized PCCPs are thicker (generally in the 150-mm to 250-mm range) than those used to surface PCCPs rehabilitated by using traditional PCCP patching and repair procedures. The thicker OLs are used on increased design ESAL traffic sections. Considerably thicker HMA OLs also have been constructed. The Ohio Department of Transportation has constructed some rubblized PCCPs (design ESALs of approximately 42 million) with HMA OLs ranging from 338 to 363 mm (13.5 to 14 in.). In 1998 on I-65 near Lafayette, Indiana, INDOT built a 325-mm HMA OL on a rubblized PCCP project, with design ESALs of 39 million.

Constructability

Adequate in situ subgrade strength is essential for successfully supporting rubblization and high-quality HMA OL construction operations. During HMA OL construction, the subgrade is protected by the combined thickness of rubblized PCCP and subbase. Field experience has shown that rubblized PCCPs with larger PCCP segment sizes provide more effective cover than do smaller PCCP segments. IDOT's Subgrade Stability Manual (*7*) is used to consider constructability issues. If subgrade strength during construction is not adequate, the PCCP segment size can be increased (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 PCCP rehabilitation option.

Current Procedures

HMA OL thickness design procedures for rubblized PCCP have been proposed by NAPA (*3*) and the Asphalt Institute (*8*). Both include procedures based on AASHTO (*9*) principles that relate layer coefficient (layer thickness in inches) to material modulus. The Asphalt Institute recommends an AASHTO layer coefficient of 0.2 for rubblized PCCP. In the NAPA procedure, the recommended layer coefficients range from 0.34 (75 percent reliability), 0.30 (85 percent reliability), 0.29 (90 percent reliability), 0.26 (95 percent reliability), to 0.2 (99 percent reliability). The 1993 AASHTO guide (*9*) recommended coefficients for rubblized PCCP range from 0.14 to 0.30. A recent INDOT study (*6*) concluded a coefficient of 0.22 "represents a conservative value." The wide range of coefficients for rubblized PCCP obviously will produce a large range of HMA OL thicknesses!

The Asphalt Institute also suggests that the rubblized PCCP and subbase can be converted to an "equivalent granular base thickness" and suggests that the required HMA OL thickness be based on the institute's MS-1 procedure (*10*) for conventional flexible pavement design. The procedure considers HMA fatigue. This is a very conservative approach, because it has been demonstrated that rubblized PCCP demonstrates better load distribution characteristics than typical granular base layers.

A previous University of Illinois–IDOT study (*11*) demonstrated the inadequacy of the AASHTO-based structural number-layer coefficient design procedure for conventional (HMA surface-granular base) and full-depth HMA flexible pavements. Similar findings emerged from early analyses of FHWA LTPP program data. AASHTO-based procedures probably are even less applicable for rubblized PCCP–HMA OL pavements for which limited performance data are available.

Illinois Pavement Design Concepts

IDOT has adopted M-E flexible pavement design concepts for full-depth (*12,13*) and conventional flexible pavements (*14*). The M-E design concepts for these pavement types show that for thicker HMA surfaces, HMA modulus and thickness are the major factors that influence the tensile strain at the bottom of the HMA layer and thus HMA fatigue performance.

In previous University of Illinois research (*15*) an ILLI-PAVE full-depth HMA database was analyzed to develop a relation between the area under pavement profile (AUPP) FWD deflection basin parameter (see Figure 2) and the tensile strain (ϵ_{AC}) at the bottom of the HMA layer. (Note that the FWD testing is on the surface of the HMA layer.) The relation is

$$\text{Log } \epsilon_{AC} \text{ (microstrain)} = 1.001 + 1.024 \text{ (Log AUPP [inches])} \quad (2)$$

Note that the HMA thickness is not a required input in Equation 2.

University of Illinois analyses of a similar ILLI-PAVE conventional flexible pavement (HMA surface-granular base) database (*5*) indicated the validity of the ϵ_{AC}-AUPP algorithm. University of Illinois–IDOT research project IHR-535 analyses (*16,17*) of MnROAD data (FWD response and HMA ϵ_{AC} measurements) from instrumented full-depth asphalt and conventional flexible pavement sections also validate the efficacy of the ϵ_{AC}-AUPP algorithm.

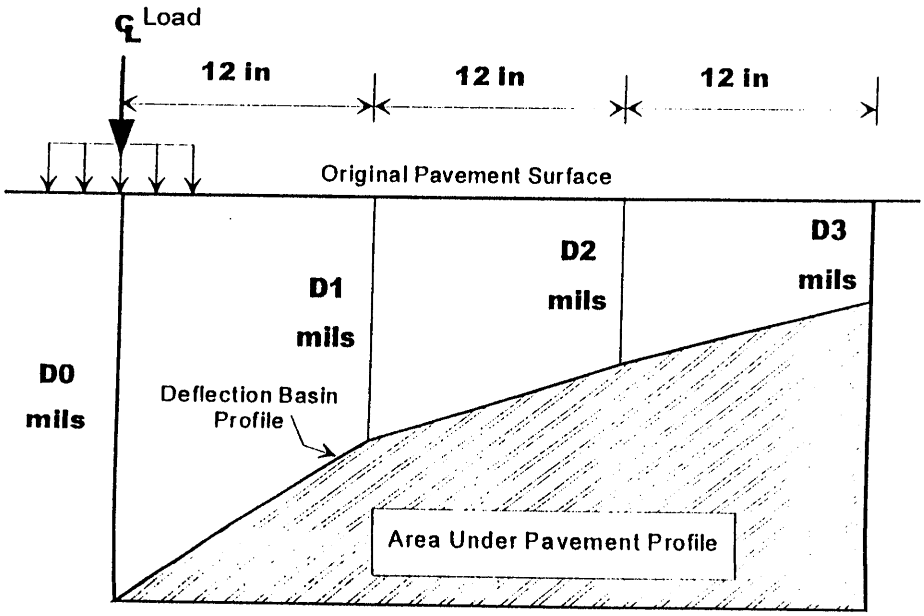
Thus, AUPP FWD data for HMA OLs over rubblized PCCPs can be used to estimate ϵ_{AC} without using backcalculation and structural modeling procedures. For HMA thicknesses greater than about 125 to 150 mm, HMA modulus and HMA thickness are the dominant factors that influence AUPP and ϵ_{AC}. For full-depth asphalt and conventional flexible pavements, AUPP is relatively insensitive to base-subbase-subgrade conditions.

Hill and Thompson (*15*) demonstrated that HMA layer "flexural rigidity" [FR = ET^3 (E = HMA modulus; T = HMA thickness)] is an excellent indicator of the FWD AUUP deflection basin shape parameter. As ET^3 increases, AUPP and HMA OL strain decrease.

M-E–BASED HMA OL DESIGN CONCEPTS FOR RUBBLIZED PCCP

General

Concepts similar to those developed by the University of Illinois for IDOT (*12–14*) are being used to extend M-E–based concepts to the analysis and design of HMA OLs for rubblized PCCPs. The structural behavior (characterized by FWD data) and HMA OL fatigue performance of the I-57 rubblized pavements (150 mm and 200 HMA OL sections) near Pesotum, Illinois, were used in initial development activities. FWD results from other Interstate rubblized PCCP projects are providing data and information for refining and validating the concepts.



AUPP = (Area Under Pavement Profile) /12 = (5*D0 - 2*D1 - 2*D2 - D3) /2

FIGURE 2 FWD AUPP deflection basin parameter (AUPP in inches, D in mils).

Pesotum Sections

A summary of the periodic FWD testing for the Pesotum sections is shown in Table 1. The HMA moduli (E_{AC}) and subgrade E_{Ri} values were backcalculated from previously developed algorithms (*18*) for full-depth HMA pavements that are routinely used by IDOT.

The rubblized PCCP modulus (E_{RUBB}) was calculated by using the ILLI-PAVE stress-dependent finite element model in a trial-and-error procedure. The backcalculated E_{AC} and subgrade E_{Ri} values are known. The rubblized PCCP layer was modeled as a 250-mm-thick constant modulus layer. Rubblized PCCP moduli (E_{RUBB}) were assigned until an FWD deflection basin (40-kN loading, 300-mm-diameter plate) match was obtained.

To provide a check, the WESDEF procedure was used to backcalculate the rubblized PCCP modulus. The E_{AC} for WESDEF was assigned the same value used in the ILLI-PAVE analyses. The WESDEF modulus calculations were for the rubblized PCCP and subgrade.

The ILLI-PAVE and WESDEF E_{RUBB} values are shown in Table 1. The values are similar. It is apparent that E_{RUBB} is not constant but is varied for different test times. Figure 3 shows ILLI-PAVE E_{RUBB}–$E_{AC}T^3$ data. There is no consistent HMA OL thickness effect. Average values for the two thicknesses are about the same. For a given OL thickness, larger E_{RUBB}'s tend to be related to increased $E_{AC}T^3$ values.

Figure 4 shows the relation between AUPP and $E_{AC}T^3$ for the Pesotum sections. An improved AUPP estimate (see Figure 5) is achieved if both the $E_{AC}T^3$ and the E_{RUBB} terms are included in the regression equation.

The flexural strain at the bottom of the HMA overlay is available from the Pesotum ILLI-PAVE results and data used to backcalculate E_{RUBB}. A favorable comparison (shown in Figure 6) between the ILLI-PAVE AC strain with the AC strain estimated from the FWD AUPP data (Equation 2) was obtained for the Pesotum data.

Additional FWD data were available from other Interstate rubblized PCCP sections constructed in Illinois since 1996. The Pesotum data and some data for the other sections are plotted in Figure 7. It does not appear that there is a unique relation between AUPP and $E_{AC}T^3$. Recall that the other sections included different PCCP pavement types and subbases. Backcalculated E_{RUBB} values for the other sections showed considerable variability and some were as low as 20–30 ksi.

In the Union County I-57 project, a coarse rubblization specification section (75 percent of the PCC segments are 225 mm or less in size and are a maximum size of 300 mm) had a considerably smaller AUPP [18 cm (7 in.) versus 29 cm (11.4 in)] than the project "finer" rubblization specification (PCC segment sizes less than 75 mm above the steel and less than 225 mm below the steel). Similar results have been noted on other rubblization projects throughout the country.

The segment size in the rubblized PCCP influences the structural response. Larger segment sizes are associated with reduced deflections and decreased AUPPs (reduced HMA OL strain, longer HMA fatigue life). As PCCP segment size further increases, and approaches segment sizes typical of break-crack-seat dimensions, further reductions are achieved, but reflection crack control effectiveness will diminish.

Predicting Fatigue Life

HMA fatigue is the controlling overlay thickness design criterion for practically all rubblized PCCPs. Fatigue life can be predicted

TABLE 1 Pesotum Test Section Data

			Average FWD Deflections, mils						Average ILLI-PAVE Algorithms		E_{rubb}, ksi	
Test Date	HMA Temp., °F	HMA Thick., in	D0 @ 0"	D1 @ 12"	D2 @ 24"	D3 @ 36"	AUPP, in	AREA, in	Eac, ksi	Eri, ksi	ILLI-PAVE	WESDEF
5/20/97	71	6	7.9	6.1	4.4	3.2	7.7	24.4	709	10.5	135	167
5/13/93	73	6	8.5	6.6	4.7	3.4	8.2	24.4	737	10.1	110	139
5/10/96	50	6	7.2	5.7	4.2	3.1	6.6	25.1	924	11.0	150	192
6/29/92	88	6	11.3	8.0	5.2	3.6	13.3	21.9	375	9.6	73	86
6/18/98	83	6	8.6	6.4	4.3	3.1	9.4	23.0	655	11.1	120	115
7/10/91	94	6	12.6	9.3	6.1	4.0	14.3	22.5	401	8.2	57	68
7/11/95	88	6	10.2	7.1	4.8	3.3	12.0	21.9	346	10.6	80	114
8/3/94	88	6	10.7	7.9	5.3	3.7	11.7	22.9	440	9.1	100	100
5/13/93	73	8	6.0	4.8	3.7	2.9	5.0	26.1	1,024	11.7	130	187
5/10/96	50	8	5.4	4.4	3.4	2.6	4.4	26.2	1,192	12.6	150	209
6/29/92	88	8	8.0	6.0	4.3	3.2	8.1	23.9	574	10.7	80	106
6/18/98	83	8	6.1	4.7	3.5	2.7	5.7	24.7	865	12.4	135	160
7/10/91	93	8	9.1	6.8	4.9	3.6	9.1	24.0	484	9.4	65	98
7/11/95	88	8	7.3	5.5	4.0	3.0	7.3	24.0	553	11.4	90	140
8/3/94	88	8	7.5	5.8	4.3	3.2	7.1	24.6	619	10.8	110	136
										Average 6" sections:	***103***	***123***
										Average 8" sections:	***109***	***148***
										Average for all sections:	***106***	***134***

Note:
1 inch = 25 mm
1 mil = 0.025 mm
1 ksi = 6.9 MPa

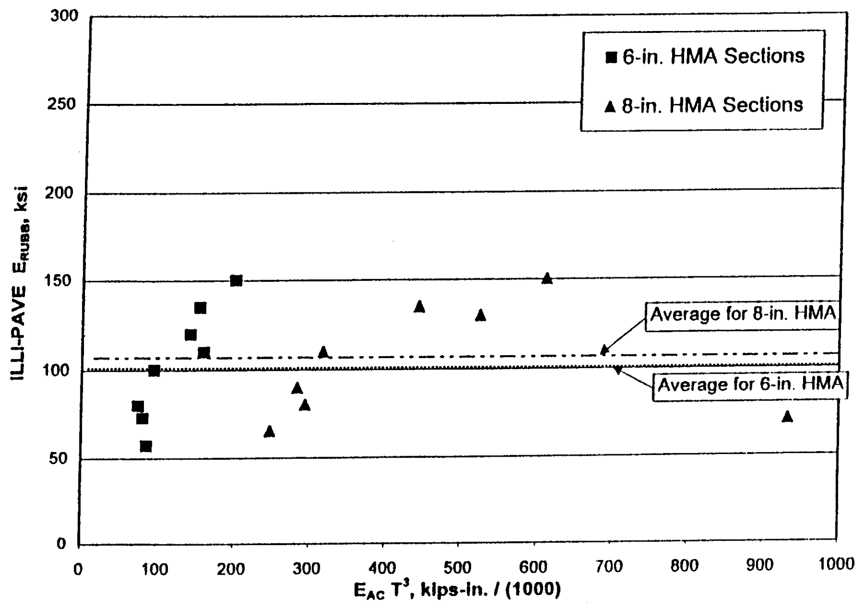


FIGURE 3 $E_{RUBB}-E_{AC}T^3$ data for Pesotum rubblized PCCP sections.

on the basis of the estimated HMA strain and an HMA fatigue design algorithm.

The Illinois M-E design approach is based on the concept of "Design Time" AC temperature/modulus. HMA fatigue consumption for Design Time conditions is equal to that estimated from considering monthly seasonal effects.

The IDOT (12) M-E HMA fatigue algorithm is as follows:

$$N = (5 \times 10^{-6})(1/\text{HMA Strain})^{3.0} \quad (3)$$

where N is the number of 18-kip ESALs and HMA Strain is the HMA tensile strain at the bottom of the HMA layer.

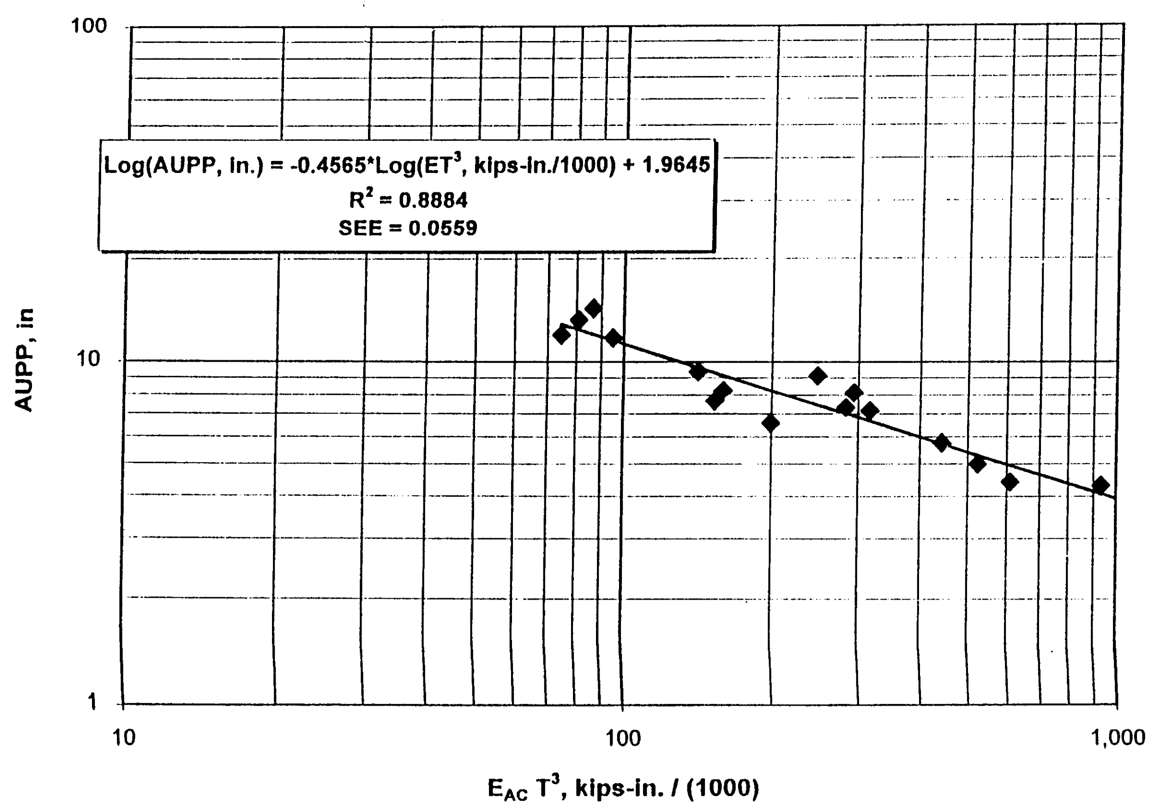


FIGURE 4 AUPP$-E_{AC}T^3$ relation for Pesotum rubblized PCCP sections.

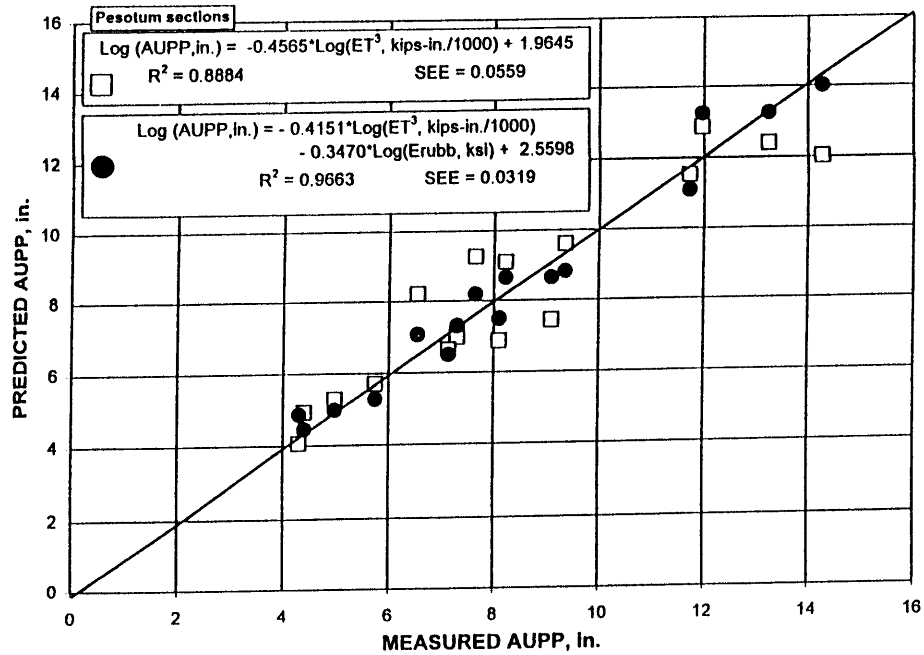


FIGURE 5 AUPP comparisons for $E_{AC}T^3$ and $E_{AC}T^3/E_{RUBB}$ algorithms.

The HMA strain (and the AUPP) required to accommodate the design ESALs can be established from the fatigue algorithm and Equation 2. For a given Design Time E_{AC}, a AUPP–$E_{AC}T^3$ algorithm (as shown in Figure 4 for the Pesotum project) can be used to establish the required HMA OL thickness. If an E_{RUBB} estimate is available (or can be assigned), an algorithm including $E_{AC}T^3$ and E_{RUBB} (see Figure 5) also can be used.

An alternative approach is to extrapolate the performance of existing rubblized PCCP pavements to new design conditions. The Design Time AUPP (and HMA strain) can be established from periodic FWD testing (over a range of temperatures) on an existing "intact" pavement. The relative estimated fatigue life of a new pavement (and related Design Time E_{AC}, HMA strain, and AUPP) is the ratio of the HMA strains (existing pavement/"new"

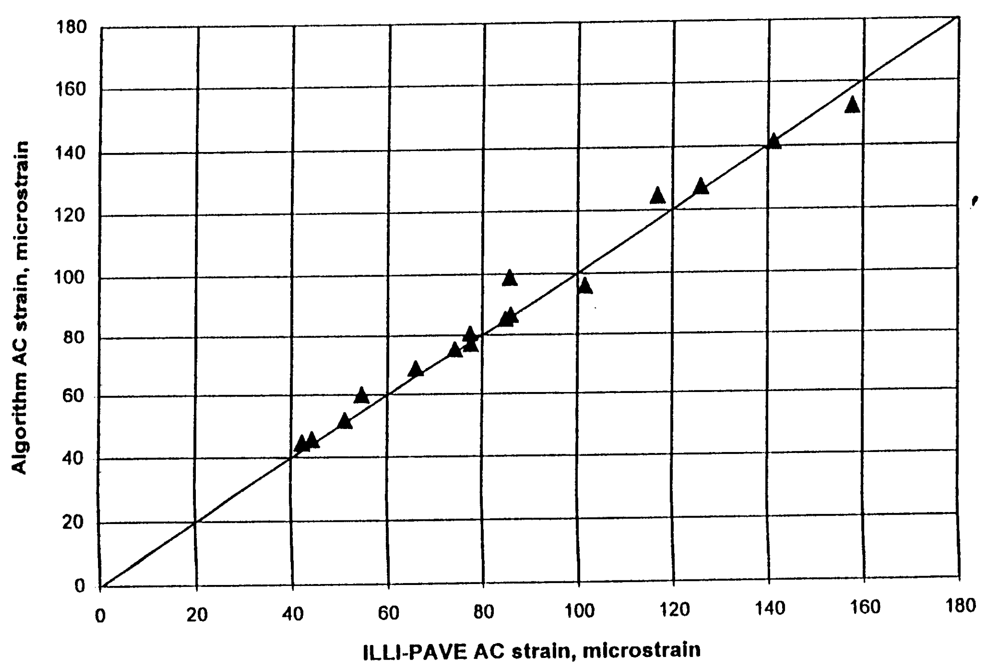


FIGURE 6 AUPP algorithm–ILLI-PAVE strain comparison.

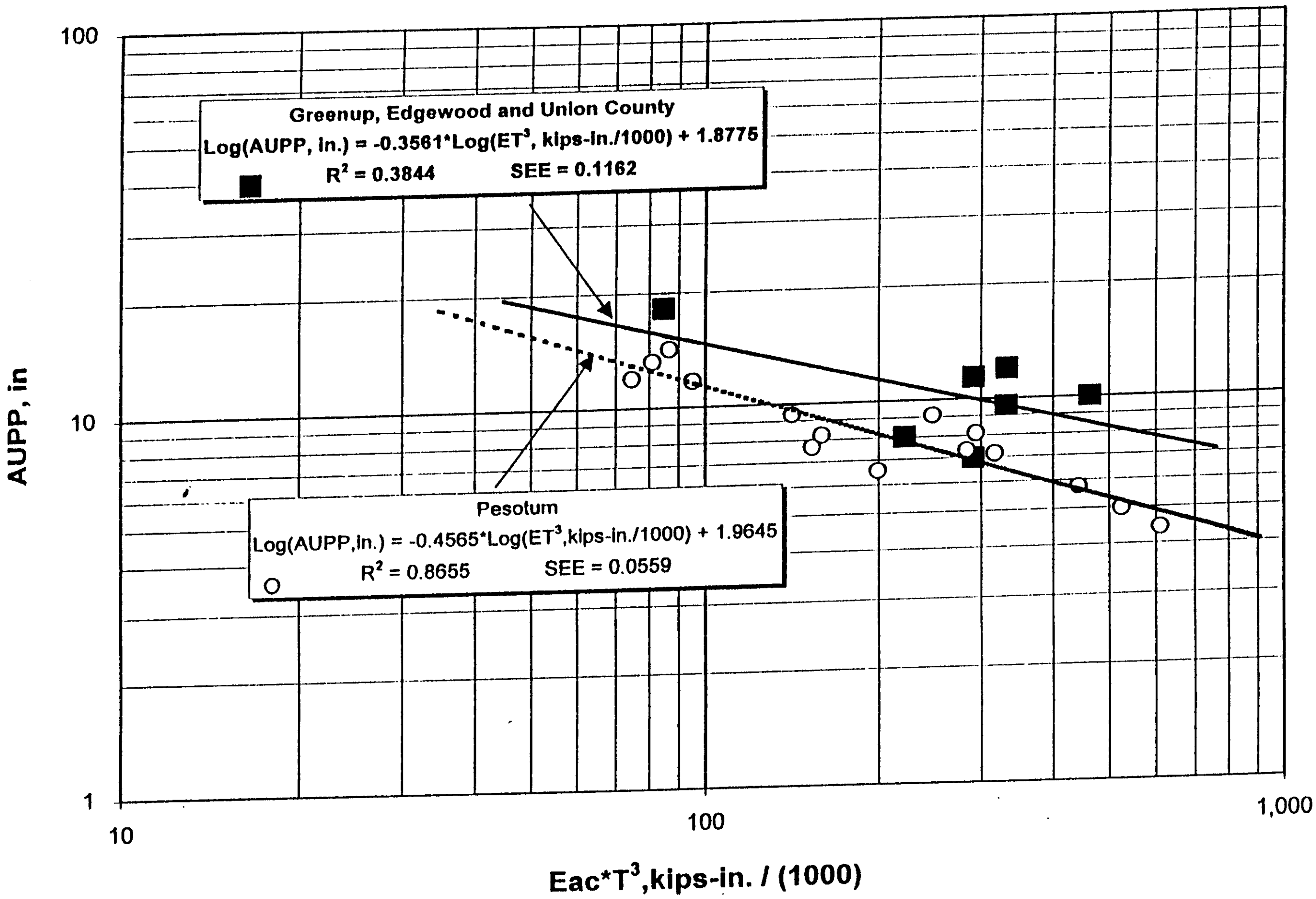


FIGURE 7 AUPP–$E_{AC}T^3$ relations for Pesotum and other rubblized PCCP sections.

pavement) raised to the HMA strain power term in HMA fatigue algorithms of the form

$$N = k\,(1/\text{HMA strain})^n$$

$$N = k'\,(1/E_{AC})^m(1/\text{HMA strain})^{n'}$$

The two approaches are demonstrated in the following with the Pesotum data.

The Design Time temperature (*12*) for Pesotum is 27°C (81°F) Regression analysis of the E_{AC} - pavement temperature data in Table 1 indicated the Design Time E_{AC} for the Pesotum sections is 4.4 GPa (634 ksi). Estimated HMA strains (based on the AUPP-$E_{AC}T^3$ relation in Figure 2 and Equation 2) for the 150-mm and 200-mm HMA OLs are 103 microstrain and 69 microstrain, respectively. The corresponding fatigue lives (Equation 3) are 4.6 million ESALs for the 150-mm section and 15.2 million ESALs for the 200-mm section. Because the 150-mm HMA OL section already has sustained more than 7.5 million ESALs without fatigue failures, the IDOT fatigue algorithm is conservative.

The alternative "Relative Life" approach also can be used to extrapolate the 150-mm HMA OL performance to other conditions. The fatigue life of the 150-mm Pesotum section is greater than 7.5 million. The strain ratio for the 150-mm and 200-mm HMA OLs is 1.49 (103/69). The estimated relative fatigue life of the 200-mm OL is 1.49 raised to the HMA strain power term in the fatigue algorithm. The sensitivity of the relative fatigue life to the HMA strain power term is shown in the following table for the Pesotum strain ratio of 1.49.

Power Term	*Relative Life*
3.0	3.3
3.5	4.0
4.0	4.9
5.0	7.3

For a Power Term of 3.0, the estimated fatigue life of the 200-mm HMA OL section is 25 million ESALs (3.3 × 7.5 million).

Note the sensitivity of Relative Life to Power Term. Assuming similar HMA fatigue properties, fatigue lives for other projects can be estimated from existing rubblized PCCP projects.

OBSERVATIONS AND SUMMARY

As more HMA OL–rubblized PCCP response and performance data and HMA fatigue information become available, the analysis and design concepts presented in this paper will be refined.

The following observations should be helpful in future HMA OL thickness design activities: