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Transportation Research Record 1374

Contents

Foreword

Mahoney et al. overview the study of the performance of granular overlays in Washington. Hall and Elliott describe an overlay design system (ROADHOG) based on the 1986 *AASHTO Guide for the Design of Pavement Structures*. Witczak and Rada present the general approach and the analysis of performance and structural data for a nationwide study of the fractured slab approach for concrete pavement rehabilitation. In another paper Witczak and Rada present design procedures for fractured concrete pavement slab overlaid by asphalt concrete. These procedures are based on their national study and are in accordance with the flexible pavement methodology presented in the 1986 *AASHTO Guide for the Design of Pavement Structures*.

Hall et al. describe revisions to the AASHTO overlay design procedures. In a second paper Hall et al. report the results of testing of their revised overlay design procedures using data from in-service pavements located throughout the nation and found that they produced reasonable results. Sidess et al. present a methodology for flexible pavement rehabilitation and development of overlay thickness design curves for pavement maintenance management systems. Rmeili et al. present results from a project to evaluate and develop maintenance and rehabilitation strategies.

Ross et al. describe the development of a computerized integrated system (PARES) to evaluate and design rehabilitation schemes for flexible pavements. Chen and Frederick report on the evaluation of stress-relieving interlayers used to reduce reflection cracking in asphalt concrete overlays of asphalt concrete pavements.

Evaluation of Granular Overlays in Washington State

JOE P. MAHONEY, NEWTON C. JACKSON, AND DANIEL J. O'NEIL

Granular overlays have been used by the Washington State Department of Transportation (WSDOT) for about 30 years. Since the mid-1980s and along with the full implementation of the WSDOT Pavement Management System, WSDOT has been interested in examining the performance of granular overlays. It is believed by WSDOT that the performance of this rehabilitation treatment is better than might reasonably be expected. Further, past practice in Washington occasionally required that the preexisting surfacing (often several bituminous surface treatment layers) be scarified before placement of the crushed rock layer. The study and conclusions are reviewed.

The granular overlay system (hereafter referred to as "granular overlay") is an alternative type of overlay for rehabilitating mostly low-volume, rural roads. The overlay consists of a layer of densely compacted, crushed rock overlain by a generally thin surface layer. Figures 1a and 1b show typical granular and asphalt concrete (AC) overlays.

Granular overlays have been used throughout the world as a pavement rehabilitation treatment. The reasons for their use fall into four primary categories:

1. To reduce reflective cracking from a preexisting pavement structure,
2. To add extra pavement structure thickness to combat frost-related effects,
3. To improve the cross-slope road profile (and ride in general), and
4. To strengthen the pavement structure.

The last category will be the primary focus of this paper.

Granular overlays have been used by the Washington State Department of Transportation (WSDOT) for about 30 years. Since the mid-1980s, and along with the full implementation of the WSDOT Pavement Management System (WSPMS), WSDOT has been interested in examining the performance of granular overlays. One reason is that WSDOT engineers have found that the performance of this rehabilitation treatment has been better than could be reasonably expected. Further, past practice in Washington State occasionally has required that the preexisting surfacing [often several bituminous surface treatment (BST) layers] be scarified before placement of the crushed rock layer.

One view of why the granular overlays have worked well structurally is that they take advantage of the stress stiffening behavior of granular materials. When a crushed rock layer is subjected to a confining pressure, its stiffness increases. Since the old pavement surface and the new surfacing confine the crushed rock layer in a granular overlay, traffic loads can provide high confining stresses, which, in effect, increase the stiffness of the crushed rock layer.

As the use of the granular overlay increased in Washington State, WSDOT realized that to improve and continue to use granular overlays, it needed to better understand how they worked, where they were appropriate, and how best to design and build them.

In cooperation with WSDOT, two initial studies were undertaken at the University of Washington (*1,2*). The results of these studies were encouraging. They led WSDOT and the associated Washington State Transportation Center (TRAC) at the University of Washington to enter into an agreement with the Federal Highway Administration to prepare a report on this topic, which was the source information for this paper (*3*).

METHODOLOGY

The study examined granular overlays in three ways. First, previous research on the behavior of confined crushed rock layers was reviewed. These studies provided information concerning the stiffnesses that have been found in crushed rock layers, the actions that can be taken to improve the crushed rock layer, and the problems that have been encountered in working with confined crushed rock layers. Next, the usable life of the granular overlay was compared with that of other types of pavement resurfacing, including AC overlays and BST. Finally, the granular overlays were field tested to determine their properties and to measure the effect of different designs on their performance.

LITERATURE REVIEW

The behavior of granular overlays depends on the condition of the crushed rock layer. Both the surface and the old pavement serve to protect this layer and to confine it. The crushed rock layer can provide much of the "strength" of the overlay. When crushed rock is used as a base course, it generally has a modulus of elasticity of about 15 to 30 ksi (100 to 200 MPa) (*4*). When it is subjected to a confining pressure of 125 psi (0.9 MPa), its modulus of elasticity can exceed 100 ksi (690 MPa) or more (*5*).

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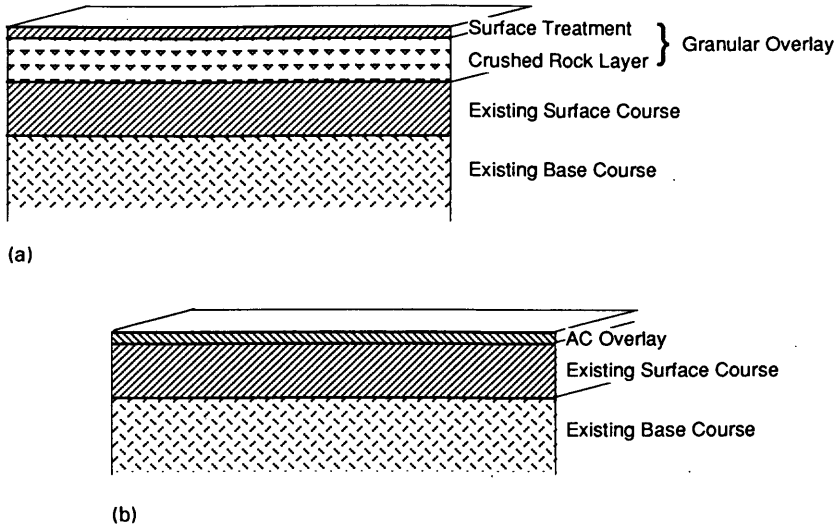

FIGURE 1 (*a*) Typical granular overlaid pavement and (*b*) typical AC overlaid pavement.

The stress sensitivity of a granular material will, in general, follow Equation 1:

$$E = K_1\theta^{K_2} \quad (1)$$

where

E = modulus of elasticity (psi),
K_1, K_2 = constants,
$\theta = \sigma_1 + \sigma_2 + \sigma_3$ = bulk stress, and
σ_1, σ_2, σ_3 = principal stresses.

A study by WSDOT and the University of Washington found that the crushed rock WSDOT normally uses (crushed surfacing top and base course) has the following "typical" constants (*b*): K_1, 8,500 (mean), 2,300 (standard deviation); K_2, 0.375 (mean), 0.067 (standard deviation). The laboratory tests used to obtain these constants were conducted at bulk stresses ranging from 4 to 28 psi.

In a traditional pavement system, the confining stresses on the crushed rock base depend on a number of factors, including the stiffness of the subgrade. Since the granular overlay is sandwiched between two stiff pavement layers, it will be subjected to higher confining pressures.

Equivalency Factors

The stiffness of a granular overlay is provided largely by the crushed rock layer (assuming that the surfacing is relatively thin). One method for comparing granular and AC overlays is to determine the thickness of a granular overlay that would provide the same life as a thickness of AC overlay. This is the technique used by Sibal (*2*) and Deoja (*1*) in their studies of granular overlays.

Sibal used two elastic layer programs to model the behaviors of AC and granular overlays: ELSYM5 and EVERSTR (*2*). The ELSYM5 program treats all layers as linearly elastic. EVERSTR treats the granular layers as nonlinear. Sibal determined the thicknesses of the crushed rock layer in a granular overlay that would provide the same pavement performance as different thicknesses of AC. In both the granular and AC overlay analyses, he varied the moduli of the subgrade and the granular overlay crushed rock layer.

Sibal considered three modes of failure: fatigue cracking of the surface, fatigue cracking of the preexisting pavement surface layer, and rutting. He determined which of the three modes of failure was critical for each model and used the corresponding number as the number of loads that would cause failure for the pavement. Finally, he compared the number of loads that would cause failure for each of the models to determine the equivalent thicknesses of granular overlays and AC overlays. The 1.0 in. of AC on top of the crushed rock layer was not calculated in the equivalency factor. For example, if a 4-in. AC overlay was to be converted to a granular overlay with an equivalency factor of 1.70, the conversion would be as follows: 4-in. AC = 1.0-in. AC + 3.0 in. of AC = 1.0-in. AC + 3.0 × 1.70 of crushed rock = 1.0-in. AC + 5.1 in. of crushed rock (or 6.1 in. total thickness).

Sibal's analyses are shown in Figure 2. His results suggest equivalency factors of about 2.0 for the stiffer crushed rock moduli.

Crushed Rock Layer in Inverted Pavements

A series of South African studies (*5,7–9*) and related data (*10*) investigated the effects of different parameters on the behavior of the crushed rock layer in inverted pavements. These studies verified that the modulus of the crushed rock layer can be high and offered insight into improved designs for this layer and optimum gradation.

Horak et al. noted that the gradation specifications are important for achieving the high densities required for optimal performance of confined crushed rock layers (*8*) and published a paper that dealt with the effects of tightening the

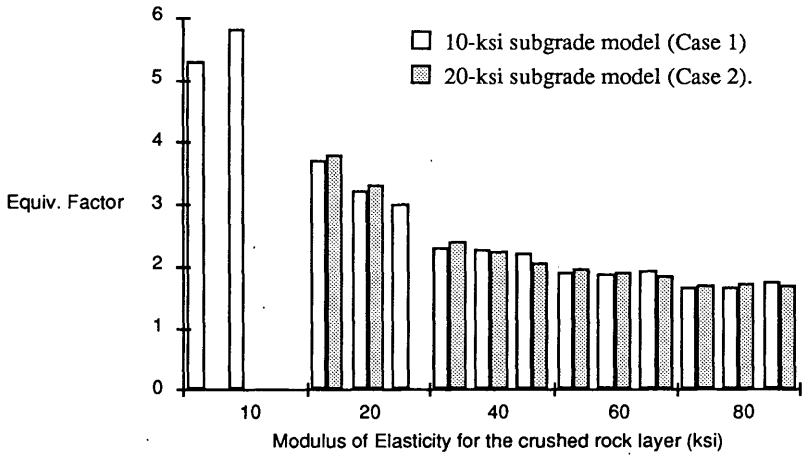

FIGURE 2 Equivalency factors versus the modulus of elasticity for the crushed-rock layer based on Sibal's calculations (*2*).

grading specifications beyond those normally required for the G1 (South African) base (*9*). Although the importance of the strength, durability, shape, and Atterberg limits of the aggregate was mentioned, the report focused on changes to the specifications to produce a better compacted base.

For convenience, the gradation bands for the South African G1 material, as well as somewhat similar gradations from AASHTO M147 (Gradings A and B) and WSDOT (Crushed Surfacing Top Course and Base Course), are given in Table 1. The gradation band for the G1 material was obtained directly from a figure in Horak et al. (*9*). It appears that the most similar gradations to the G1 are AASHTO Grading B and the WSDOT Crushed Surfacing Base Course (*11*). (The majority of granular overlays constructed by WSDOT to date have used the Crushed Surfacing Top Course grading.)

The base course on which Horak et al. reported was compacted with 99 to 103 percent of modified AASHTO and had a gradation that fell within the specifications for a G1 base, but with a few additional requirements (*9*). They found that the greater effort required to set up the crusher and obtain the correct gradation was more than offset by the increased ease in compacting the material to a higher density.

As was previously stated, the crushed rock layer is stiffer and more durable if it is well constructed. The compaction and integrity of the crushed rock layer is very important. The material for the crushed rock layer must be durable. As was mentioned by Horak et al., the easiest way to obtain the highest compaction is to use an optimum gradation (*8*). Interviews during 1990 with WSDOT project engineers on granular overlay construction projects indicated that the moisture content is also important (*12–14*). Because the crushed rock is spread in a thin layer, the moisture from the rock tends to evaporate rapidly.

A significant problem that WSDOT has encountered in the construction of the granular overlay is traffic. When there is no possible diversion, the traffic has to pass over the crushed

TABLE 1 Gradation Bands for Various Crushed Rock Specifications

Sieve Designation		Percent Passing				
		South African G1*	AASHTO M147-65		WSDOT 9-03.9 (3) Crushed Surfacing	
Standard	mm		Grading A	Grading B	Top Course	Base Course
2 in.	50	100	100	100		
1 1/4 in.	32	93 - 97				100
1 in.	25	82 - 92		75 - 95		
3/4 in.	19	72 - 85				
5/8 in.	15.9	64 - 78			100	50 - 80
3/8 in.	9.5	50 - 67	30 - 65	40 - 75		
1/4 in.	6.35	40 - 57			55 - 75	30 - 50
No. 4	4.75	35 - 52	25 - 55	30 - 60		
No. 10	2.00	23 - 39	15 - 40	20 - 45		
No. 30	0.600	14 - 26				
No. 40	0.425	11 - 24	8 - 20	15 - 30	8 - 24	3 - 18
No. 200	0.075	5 - 12	2 - 8	5 - 20	10 max	7.5 max.

* South African G1 grading taken from plotted gradation band (10)

rock layer during construction. This frequently causes washboarding. Therefore, the granular surface must be rebladed immediately before the surface layer is placed.

Improvement in BST Construction

A recent WSDOT study examined the effects that construction practices have on the problems associated with BSTs (*15*). The main problems that were investigated included flushing of excess asphalt, windshield damage due to loose rock, and aggregate loss due to poor embedment. Through a review of the use of BSTs in other western states and an examination of BST construction projects, a series of design and construction guidelines was developed.

The study also recommended several guidelines for the proper choice of roads to be overlaid with BSTs. The report suggested that the BST be applied only to roads that were not considered a high traffic risk [i.e., roads with average daily traffic (ADT) counts in excess of 5,000]. If a BST surface was used on granular overlays, these same limitations were applicable. (WSDOT mostly requires BSTs on routes with ADTs of 2,000 or less and discourages the use of BSTs on routes with ADTs of 5,000 or more.)

Existing Pavement

Although granular overlays reduce the rate of reflective cracking, they are also sensitive to moisture infiltration through breaks in the existing pavement surface. Infiltration of moisture lowers the stress sensitivity of the crushed rock layer (in terms of K_1, hence overall stiffness).

To circumvent the problem of an inconsistent base for the overlay, the highway department in Zimbabwe rips and spreads the existing base and shoulder, then stabilizes with 2 percent cement or lime (if required) and recompacts it. In this manner, the granular overlay is assured of having a solid base (*16*). Normally, all that is required is to patch any holes and seal any cracks in the existing pavement surface.

Other Advantages and Limitations

Insulation

An advantage of the granular overlay is that it protects the existing pavement from daily extremes in temperature. This is important during hot summer months and in tropical countries where pavement surface temperatures can approach and exceed 160°F (70°C) (*16*).

Increased Frost Resistance

Frost heave and thaw weakening problems are often a result of subgrade freezing, so if the subgrade is insulated from the cold, these problems are reduced. Several states, including Alaska, Iowa, Oregon, and Washington, use frost protection in their pavement thickness design calculations (*17*). Most of these states design the pavement thickness to be at least 50 percent of the total expected frost depth. In this manner, the subgrade is at least partially insulated against frost.

In Washington State, designing the overall pavement depth to be equal to 50 percent of the total frost depth has worked well for controlling all but the most severe frost problems (*18*). Unfortunately, many of the state's low-volume rural roads were built before this design procedure was adopted. These roads frequently consist of only a thin BST over 6 to 9 in. (150 to 225 mm) of base course when the frost design thicknesses are 15 to 24 in. (380 to 610 mm). Rehabilitating these roads typically means adding a granular overlay with a minimum of 4.2 in. (107 mm) of crushed rock.

To prevent the crushed rock layer from contributing to frost heave, the amount of material that passes the No. 200 sieve (0.075 mm) may have to be limited. Researchers have observed that the finest content of soils is an important indicator of frost-susceptible material (*19*). The pavement agencies surveyed specified that the maximum percentage of material that passed the No. 200 sieve be 5 to 15 percent. The lower range of this specification is lower than the range suggested by Horak et al. for obtaining the maximum compaction (*9*).

Change in Road Geometry

By adding 3 to 6 in. (75 to 150 mm) to the overall pavement structure, the granular overlay can alter the road geometry. It can be used to increase drainage, improve the road profile, and level off inconsistencies in the pavement. The additional height makes it unusable in areas where the road geometry is restricted by curb height or other considerations.

Resistance to Shear

Little information could be found concerning the ability of confined crushed rock layers to resist shear. However, this is a potential problem. Although granular materials are very strong in compression, they have little resistance to tension. When subjected to a high confining pressure, the crushed rock particles can distort, be crushed, shift, roll, or slide. The amount of movement caused by any of these actions is directly proportional to the confining pressure. If the shear stress becomes sufficiently large, the combined movement from these actions will result in a shear failure (*20*).

Design of Granular Overlays

There are two basic techniques for designing granular overlays. One is based on mechanistic-empirical calculations. The studies done by Sibal (*2*) and Deoja (*1*) are examples. Another technique is based on practical construction considerations.

The second design approach is to use the maximum thickness of granular material that can be easily compacted in one lift. This suggestion was offered in a report by Maree et al., in which the authors found that a crushed rock layer 12.6 in. (320 mm) thick did not perform significantly better than one 5.5 in. (140 mm) thick (both crushed rock layers were on cement-stabilized subbases) (*7*). Maree et al. concluded that the crushed rock layer need not exceed the maximum thick-

ness that can be placed and compacted in one lift [6 in. (150 mm)].

A study by Otte and Monismith produced similar conclusions (*21*). They used a layer elastic program, PSAD2A, to model the behavior of several inverted pavement structures. The computations were made for 9.0-kip (40-kN) dual wheels, 13.0 in. (330 mm) apart. The pavement that was simulated was an inverted pavement with a 1.4-in. (35-mm) BST surface. The thicknesses of the crushed rock layer and the cement-stabilized base and subbase were varied over a wide range of thicknesses.

The authors found that the primary stresses in an inverted pavement were on the surface course and cement-stabilized layers. They found that because of stress stiffening, as the thickness of the crushed rock layer increased from 5 to 20 in. (125 to 500 mm), the equivalent elastic modulus of the granular base declined about 30 percent. Otte and Monismith recommended the following (*21*):

1. The bituminous surfacing for inverted pavement designs should not exceed 1.2 to 1.4 in. (30 to 35 mm).
2. For typical highway traffic loads, the granular layer should have a thickness of about 5.0 to 6.0 in. (125 to 150 mm).
3. The cement stabilized layers supporting the granular layer should be (*a*) two layers, each 6 in. (150 mm) thick, if the subgrade has a CBR of 15 or better, or (*b*) one layer, 6 in. (150 mm) thick, for light traffic (rural).

The one thickness approach is also used in Zimbabwe (*16*). The highway department in Zimbabwe has found that the practical range for the construction of the crushed rock layer in an overlay is 5 to 6 in. (120 to 150 mm). Thinner layers tend to shear under a roller. If a thicker layer is needed, the road probably needs to be reconstructed.

In Washington State, granular overlays are generally built with thicknesses of 3 to 6 in. [most with a 4.2-in. (107-mm) thickness].

Costs

Typical WSDOT project specific costs for granular overlays with both AC and BST surfaces were estimated. In general, granular overlays surfaced with AC are about twice as expensive as those surfaced with a BST (about \$7.00/yd^2 for AC versus \$3.50/yd^2 for BST).

SURVIVAL AND PERFORMANCE STATISTICS

One definition of the survival life of a pavement is the amount of time between a pavement's construction and its resurfacing. The performance period is the amount of time from its initial construction to the time it reaches a minimally acceptable level. Both survival lives and performance periods provide valuable estimates of pavement life. In this portion of the report, the survival lives and performance periods of AC overlays, BST resurfacings, and granular overlays are estimated and compared.

This part of the study considered only the ages of the different types of surfaces. Other factors, such as traffic loadings, weather conditions, and soil support, affect the amount of time that a pavement lasts, but these were not directly considered. Since the study was conducted within a small geographic area, the weather and subgrade effects were assumed to be constant for all roads. The effects of higher traffic loadings were expected to be somewhat canceled by the thickness of the resurfacing.

Roads overlaid by the different techniques had different characteristics. In general, roads with AC surfaces had higher traffic counts than roads with BST surfaces. In addition, granular overlays were often used as a method for repairing badly distressed roads. Most roads that received a granular overlay had suffered from either thermal cracking or roughness problems and therefore had required special treatment.

In comparing the survival times and performance periods, note that the sets of data did not come from the same time period. Since the performance life equations were only calculated for the present road surface, they represented only resurfacings built since the late 1970s. The data for calculating the survival lives, on the other hand, were equally spread from the 1980s to the 1960s, with some dating as far back as the 1940s. Because of the problems previously mentioned with the "old" survival life data, the old information tended to increase the average survival life slightly.

Source of Data

The source of data for this analysis was the WSPMS (*22*). The WSPMS contains records of work done on the roads and the pavement condition analyses. Data from the WSPMS were spot checked against as-built plans and pavement conditions and were found to be accurate.

The estimation of survival lives and performance periods was restricted to WSDOT Districts 2 and 6 (eastern Washington). These two districts are generally rural areas where the topography ranges from mountainous to rolling hills. The average annual precipitation is about 17 in. (432 mm) in Spokane (*23*), but the area has severe frost in the winter. These are also the two districts that contain the majority of roads with granular overlays.

The WSPMS was searched to locate all roads containing BST, AC, and granular overlay resurfacings. First, the actual survival time for the different layers of pavement surfaces was calculated by subtracting consecutive resurfacing construction years from the previous years. Next, pavement condition rating (PCR) (100 = no distress, 0 = extensive distress) data on the most recent resurfacing were examined to determine whether the regression equation in the model represented a "true" regression equation. Finally, the survival lives and the performance periods were compared for each type of resurfacing and among the types of resurfacings.

Bituminous Surface Treatments

The survival times for BST resurfacings are approximately normally distributed with a median survival time of 8.0 years and a mean of 9.2 years (standard deviation = 5.1 years for a sample size of 1,310).

Next, the PCR equations were examined to determine the performance periods. Although more than 200 road sections

TABLE 2 Statistics on Performance Periods

	PCR Level	Median	Mean	Std. Dev
BSTs[1]	PCR 40	6.1	7.0	2.5
	PCR 20	7.4	8.2	2.4
	PCR 0	8.7	9.3	2.4
AC Overlays[2]	PCR 40	10.6	10.2	1.7
	PCR 20	11.4	11.3	1.7
	PCR 0	12.2	12.1	1.8
Granular Overlays[3] (All Surfaces)	PCR 40	8.7	9.2	1.8
	PCR 20	10.7	10.8	2.1
	PCR 0	12.2	12.2	2.6
Granular Overlays (BST Surfaces)	PCR 40		7.5	
	PCR 20		8.3	
	PCR 0		11.0	
Granular Overlays (AC Surfaces)	PCR 40		9.5	
	PCR 20		11.3	
	PCR 0		11.8	

[1]Based on 21 data points
[2]Based on 29 data points
[3]Based on 17 data points

were analyzed, only 21 had usable regression equations (sections less than 5 years since last resurfacing were not used). These results are summarized in Table 2.

A comparison of the values for the actual survival times and the performance periods reveals that WSDOT resurfaces BST roads when their PCR is between 20 and 0.

AC Overlays

The survival times for AC overlays were essentially normally distributed. The survival times based on AC overlays with thicknesses of less than 1.2 in. (30 mm) were separated out. According to the 1988 WSDOT specifications, only AC overlays thicker than 1.2 in. (30 mm) are subject to compaction control. The survival times for AC overlays less than 1.2 in. (30 mm) were, again, normally distributed with a median survival time of 8.0 years and a mean of 8.7 years (standard deviation = 2.8 years). For AC overlays greater than 1.2 in. (30 mm), these times were 10.0, 9.7, and 4.5 years, respectively.

There were also a significant number of survival times of 5 years and less. If only the survival times of AC overlays thicker than 1.2 in. (30 mm) and lasting more than 5 years are considered (thus eliminating construction-related and underdesigned factors), the average survival life increases to 11.5 years.

The predicted performance periods are summarized in Table 2.

Granular Overlay

Unlike the other two resurfacings (or overlays) that were examined, there are not a large number of granular overlays on the WSDOT route system; however, projects tend to have long lengths (thus a large amount of mileage). In addition, because granular overlays had only been used with greater frequency since the mid-1980s, few data were available concerning survival times. Therefore, survival times were not analyzed. This left the examination of the performance periods. The data are summarized in Table 2.

Performance Summary

Although both methods used for calculating the usable life of the resurfacings have uncertainties, the two methods together provide reasonable estimates of usable life.

A comparison of BST resurfacings with granular overlays with BST surfaces shows that the difference in predicted performance is relatively small (7 percent) at a PCR of 40 but increases (18 percent) at a PCR of 0 (the granular overlays in both cases last longer than a simple BST). However, for AC, the granular overlays surfaced with AC do not last as long as a conventional AC overlay (7 percent less at a PCR of 40 and 2 percent less at a PCR of 0).

These comparisons show that the BST-surfaced granular overlay performs better than simple BST resurfacings (without the crushed-rock layer) and AC-surfaced granular overlays not quite as well as plain AC overlays. Granular overlays are generally used to repair pavement structures with significant distresses. BST and AC overlay resurfacings are normally placed on pavements early in the distress cycle (usually due to fatigue cracking). If the granular overlays had been used on pavements in better condition, these comparisons would have probably been different.

NDT TESTING AND EVALUATION

To assess the actual performance of roads with granular overlays more than 50 centerline mi (80 km) of roads were tested with a Dynatest falling weight deflectometer (FWD) Model 8000. These roads were located throughout WSDOT Districts 2 and 6 and had a variety of structures, ages, and conditions. The tests were designed to provide evidence about the comparative performance and stiffness of granular overlays.

All of the selected roads were in rural eastern Washington. Traffic on these roads is mostly local, with occasional long-haul trucks. The topography of eastern Washington ranges from mountainous to rolling hills, and most of the land is either dry land wheat farms or scrub fields.

A sample of roads with enough different characteristics to be representative of all the other roads in these two districts was sought. Characteristics that were considered included age, pavement structure, traffic flow, and road location. The roads that were selected are summarized in Table 3. Where more than one section was tested on a specific road, each section was designated by a letter.

The analysis of these test sections was done in two fundamental ways. First, basic deflection parameters were calculated for each test section (subgrade modulus, D_0, and area parameter). These measured parameters were then compared with calculated parameters for typical AC-surfaced sections with similar subgrade moduli (modeled with the ELSYM5 linear elastic program). Second, the layer moduli were backcalculated using the program EVERCALC 3.0. The overall goal of this second effort was to estimate granular overlay moduli by use of FWD deflection data. The backcalculation process was difficult and the results contain uncertainties; however, reasonable convergence errors of 1.5 percent or less were used (RMS basis).

On the basis of the analyses of test sections on SR28A and SR17, the modulus of elasticity for the crushed rock layer of the granular overlay was estimated to be approximately 80 ksi (551.2 MPa) under a 9.0-kip load. The South African studies showed elastic moduli ranging from 29.0 to 75.4 (and higher). Presumably, one of the reasons for the higher modulus of elasticity was that the bulk stress in the crushed rock layer in the granular overlay was higher than in the inverted pavement.

CONCLUSIONS

The following conclusions are based on the entire study of granular overlays (including reviewed literature).

1. Granular overlays are effective at reducing reflection cracking, insulating the old pavement surface against extremes in temperature, and improving the road geometry.
2. BSTs are more appropriate surfacings for granular overlays than are AC overlays (on the basis of observed performance).
3. The crushed rock layer should have a maximum recommended thickness of 6 in. (150 mm) (on the basis of structural considerations only) and a minimum value of 3.0 in. (75 mm).
4. The AC equivalency factor for the confined crushed-rock layer that is properly constructed and well protected can be about 2.0 (refer to Figure 2), but in-service WSDOT pavements suggest this value is probably higher (i.e., less support).
5. Consideration should be given by WSDOT to using crushed surfacing base course (maximum aggregate size = 1¼ in.) for the crushed rock portion of the granular overlay on some projects and evaluating its performance. The gradation is similar to (but not the same as) the South African G1 material specification.
6. The pavement surfacing (before granular overlay) should be left in place if project conditions permit. This enhances the stress stiffening of the crushed-rock layer in the granular overlay system.

The full study documentation is contained in the report by O'Neil et al. (*3*).

ACKNOWLEDGMENTS

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TABLE 3 Road Sections Tested with FWD

Location	Surfacing Type	Granular Thickness in.	Granular Thickness mm	Age at testing (years)	Test Length miles	Test Length km	Average Daily Traffic[1]	Annual 18kip (80 kN) ESAL[2]
District 2								
SR17	BST	2.4-12[3]	60-300	6	8.0	12.8	960	107,000
SR 24	BST	3.6	90	4	11.0	17.6	920	26,000
SR 28A	AC	3.0	75	6	5.0	8.0	4,700	180,000
SR 28B	BST	3.6	90	6	5.8	9.3	640	29,000
District 6								
SR21A	AC	4.2	107	11	5.0	8.0	330	28,000
SR21B	BST	3.0	75	10	5.0	8.0	280	13,000
SR231A	BST/AC	4.2	107	11	5.0	8.0	180	5,700
SR231B	BST/AC	4.8	122	8	5.0	8.0	230	6,400

[1]WSPMS, 1990 [2]Calculation is based on WSDOT's estimate of truck count (MIDAL, 1990)
[3]Thickness varies

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ROADHOG—A Flexible Pavement Overlay Design Procedure

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ROADHOG, a practical, easy-to-use system for selecting flexible pavement overlay thicknesses, was developed for the Arkansas State Highway and Transportation Department. ROADHOG is an NDT-based structural design procedure that follows the 1986 *AASHTO Guide* structural deficiency approach to overlay design with structural capacities expressed as structural numbers (SNs). Major departures from the 1986 *AASHTO Guide* are the methodologies for determining the effective structural number of the existing pavement (SN_{eff}) and for estimating the in situ subgrade resilient modulus (M_r). SN_{eff} is determined by a relationship between two falling weight deflectometer (FWD) surface deflections: the deflection at the center of loading and the deflection at a distance from loading equal to the pavement thickness. M_r is estimated using a regression algorithm developed from the ILLI-PAVE finite element structural pavement model. ROADHOG is contained in a user friendly, stand-alone (executable) computer program that directly uses the FWD field data files. In addition to determining overlay thicknesses, the program can subdivide a project into statistically similar analysis units on the basis of the overlay thickness required at each NDT test site.

In recent years, highway programs nationwide have shifted their emphasis from new construction to rehabilitation, maintenance, and preservation. With this shift, a major deficiency in pavement design technology became more significant. That deficiency was the lack of practical, proven design procedures for selecting the thickness of pavement overlays. The need for a flexible pavement overlay design procedure was particularly significant in Arkansas, where, except for the Interstates, most highways have flexible pavements. Research Project TRC-8705 was initiated by the Arkansas Highway and Transportation Department (AHTD) and the University of Arkansas to correct this deficiency.

From the beginning, the major objective of the project was to develop a practical, easy-to-use design procedure that was compatible with other AHTD pavement design practices and that consistently produced reasonable design thicknesses. AHTD designs pavements using the AASHTO Guide (*1*). When the 1986 guide became available, AHTD adopted it in place of the previous guide. Unlike the previous guide, the 1986 guide contained procedures for overlay design. These were not complete but did provide a framework around which a complete design procedure could be developed that would be compatible with the new pavement portions of the guide and, thus, be compatible with AHTD's other pavement design practices.

The completed design procedure developed under TRC-8705 was named ROADHOG to designate that it is a roadway design tool developed at the University of Arkansas (the "Hogs"). The computerized version of the design procedure is a stand-alone (executable) program that is extremely flexible and user friendly. The programming was done in CLIPPER, a data base management language accessible to data base file formats, to facilitate the handling of nondestructive testing data. (All product names are registered trademarks of their respective development corporations.) AHTD stores and manipulates field NDT results using the dBASE data base management software. An attractive feature of the ROADHOG program is its modular construction. Each major function of the procedure is contained in a separate program module. This will facilitate upgrading the program as technology improves with little visible effect on the system as a whole.

OVERLAY DESIGN METHODOLOGY

The AASHTO approach to flexible pavement design uses a structural number (SN) to reflect the combined structural contribution of all the pavement layers (surface, base, and subbase). SN is defined by

$$SN = a_1 * D_1 + a_2 * D_2 + \ldots + a_n * D_n \quad (1)$$

where a_n is the layer coefficient of layer n and D_n is the thickness of layer n.

The 1986 guide uses SN in a "structural deficiency" approach to overlay design. In its simplest terms, the structural deficiency approach states that the overlay required is the difference between the total structure needed and the structure that currently exists. The guide expresses this with the following equation:

$$SN_{ol} = SN_y - F_{rl}(SN_{eff}) \quad (2)$$

where

SN_{ol} = required structural number of overlay,
SN_y = structural number required to carry future projected traffic,
F_{rl} = remaining life factor, and
SN_{eff} = effective structural number of existing pavement.

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Determination of the required overlay thickness involves converting the required structural number of the overlay into a thickness of asphalt concrete using the appropriate material coefficient:

$$D_{o1} = SN_{o1}/a_{ac} \quad (3)$$

where

D_{o1} = thickness of overlay,
SN_{o1} = structural number of overlay, and
a_{ac} = material coefficient of asphalt concrete.

Within this general approach, the major components lacking for a complete, workable design procedure are specific methodologies for determining SN_{eff} and the subgrade resilient modulus (M_r) needed for calculating SN_y. Although ROADHOG uses the general AASHTO approach to overlay design, the methods used by ROADHOG for determining SN_{eff} and M_r are radically different.

In the AASHTO method the values for SN_{eff} and M_r are interdependent. Both values are determined on the basis of backcalculated moduli from the NDT deflection data. M_r is determined from the deflection at a point some distance from the center of NDT loading. This value is then used in the determination of modulus values for the pavement layers. SN_{eff} is calculated using these moduli. As a result, an error in the determination of M_r will result in an error (although opposite in sign) in SN_{eff}.

In ROADHOG the determination of SN_{eff} and M_r are independent. As in the AASHTO method, SN_{eff} is assumed to be a function of pavement stiffness. However, instead of backcalculating layer moduli (or a single overall equivalent modulus), ROADHOG uses a relationship between SN and a deflection differential called delta D. Delta D is the difference between the deflection measured at the center of loading and the deflection at a point equal to the total pavement thickness (surface + base + subbase). M_r is determined from the deflection measured 36 in. from the center of loading using an analysis algorithm developed by Elliott and Thompson (*2*) using the ILLI-PAVE finite element pavement model.

In addition to independence, these methods of SN_{eff} and M_r determination have other advantages over the AASHTO methods. SN_{eff} determination is relatively independent of depth to bedrock, a major concern in a backcalculation scheme such as that used by AASHTO. Also, the M_r value determined by ROADHOG is consistent with the value used in the *AASHTO Guide* design equation to represent the AASHO Road Test subgrade. As discussed by Elliott (*3*), the M_r value backcalculated by the AASHTO method must be modified to be consistent with the AASHO Road Test subgrade value. The methods of SN_{eff} and M_r determination are discussed in greater detail later.

Another component of the AASHTO design methodology modified in the ROADHOG procedure was F_{r1}, the remaining life factor. F_{r1} is an adjustment factor for the effective structural capacity of the existing pavement. The factor attempts to "reflect a more realistic assessment of the weighted effective capacity during the overlay period" (*1*). Elliott (*4*) investigated the concept and application of remaining life in overlay design as presented in the *AASHTO Guide*. He demonstrated that the F_{r1} relationship is flawed. Elliott's analyses demonstrated that for all practical purposes, 1.0 is the appropriate value for F_{r1}. The concept of remaining life should not be completely ignored in a comprehensive overlay design methodology; however, its inclusion within the structural deficiency design approach used in ROADHOG was not found to be reasonable or practical. Thus, for ROADHOG Equation 2 is reduced to

$$SN_{o1} = SN_y - (SN_{eff}) \quad (2a)$$

OVERVIEW OF THE ROADHOG PROCEDURE

The basic framework for a structural deficiency overlay design procedure is summarized in the following steps. The ROADHOG procedure was constructed upon such a framework.

1. Analysis unit delineation,
2. Traffic analysis,
3. Materials and environmental study,
4. Effective structural capacity analysis (SC_{xeff}),
5. Future overlay structural capacity analysis (SC_y), and
6. Overlay thickness selection.

Step 1, analysis unit delineation, identifies subsections of an overlay project (analysis units) with similar features such as cross section, subgrade support, and pavement condition. Unit delineation helps to optimize an overlay design by identifying varying overlay requirements along a project, rather than recommending a single "average" overlay requirement. Unit delineation should be performed both before the overlay thickness selection on the basis of existing conditions and after the required overlay is calculated for each NDT point along the project. ROADHOG includes methodology for the second unit delineation; that is, methodology for breaking the project into units based on the overlay thicknesses. Unit delineation based on existing conditions needs to be done by the designer before using ROADHOG. Further discussion of this is given later.

Steps 2 and 3, traffic analysis and materials and environmental study, respectively, generate input parameters for use in subsequent steps. These parameters may include design (future) traffic, traffic history, material coefficients, and data concerning current material condition. The designer must have completed these tasks before using the ROADHOG structural design procedure.

Steps 4 and 5 determine the values of the variables (SN_y and SN_{eff}) appearing in Equation 2a. From the values generated in these steps, the required structural capacity (SN_{o1}) of the overlay is calculated.

Step 6, overlay thickness selection, finalizes the overlay design. Equation 3 is used in this step to determine overlay thicknesses for the project.

ROADHOG uses nondestructive testing (NDT) data to evaluate the existing pavement system (pavement and subgrade). NDT-based design procedures have several potential advantages over other procedures, including speed at which data may be obtained, cost of obtaining data, and the amount of data available for a particular project. In addition, NDT-based procedures attempt to evaluate existing pavement systems in situ instead of attempting to relate laboratory test results to field conditions.

The ROADHOG overlay design procedure uses NDT data generated by a falling weight deflectometer (FWD). The FWD applies a load pulse to a pavement and measures the resulting surface deflection. Details of the setup and operation of the FWD are available elsewhere (*5*). The FWD attempts to simulate the effect of a moving wheel load on the pavement surface (*6*). Studies have indicated that the FWD generates load responses comparable with those produced by moving wheel loads (*5,7*). This effect is a major advantage over static tests when attempting to evaluate the pavement as it exists in the field.

ROADHOG uses NDT data taken directly from the FWD field storage file. The program reads the file from the floppy disk and creates a data base file that holds both the field data and the calculated overlay design parameters. Each record in the data base represents a single FWD test performed for the overlay project. A required overlay thickness is determined for each FWD test along the project. The project may be (designer's option) divided into recommended analysis units based on the required overlay thickness at each NDT site. Output options allow the designer to choose whether to use the recommended analysis units and to choose any combination of available data stored in the data base file. User inputs, in addition to the NDT data file from the FWD, include the existing thickness of asphalt concrete surface, the total existing pavement thickness, new pavement design parameters (reliability, standard deviation, delta PSI, and design traffic), asphalt concrete (overlay) material coefficient, and the minimum acceptable length of an analysis unit.

Figure 1 is a flow diagram showing the primary modules of the ROADHOG design procedure. A brief synopsis of the ROADHOG modules follows. Subsequent sections detail the procedures and algorithms used by each respective module.

1. XFORM—The FWD device stores data generated during a nondestructive test in ASCII format on a floppy disk; XFORM transforms these data into a data base file (dBASE format) for use in later modules.
2. SNEFF—The SNEFF module uses a relationship between the deflection basin and structural capacity of a pavement system to generate SN_{eff} for each FWD deflection basin.
3. MRCALC—The MRCALC module uses the deflection measured at 36 in. from the load to estimate the in situ M_r of the subgrade at each test point.
4. NEWFLEX—The NEWFLEX module uses designer input and the subgrade M_r determined in MRCALC to calculate the structural number required to carry future traffic (SN_y).
5. OVLTHICK—The OVLTHICK module calculates the overlay thickness required to strengthen the existing pavement to carry future projected traffic.
6. UNIDEL—The UNIDEL module uses the cumulative difference approach with the required overlay thickness at each FWD test site to subdivide the overlay project into analysis units.
7. OUTPUT—The OUTPUT module allows the user to see the results of the design procedure, sending the results to either the screen, a printer, or a file.

EFFECTIVE STRUCTURAL CAPACITY ANALYSIS

A number of methods currently exist to estimate the effective structural capacity of a pavement, primarily falling into three categories: (*a*) component analysis procedures, (*b*) deflection-based procedures, and (*c*) analytically based, or mechanistic, procedures. An excellent synopsis of each type of procedure is available elsewhere (*8*).

ROADHOG uses a deflection-based procedure in which the effective structural capacity of the existing pavement system is related to the deflection basin generated and measured by the FWD. The ROADHOG module that performs the effective structural capacity analysis is termed "SNEFF." The algorithm forming the basis of SNEFF was developed at the University of Arkansas by Kong (*9*). The salient features of Kong's procedure are reproduced here.

The development of the SN_{eff} algorithm began with the concept that at sufficient distances from the center of loading the surface deflection is almost entirely due to deformation within the subgrade. As shown in Figure 2, the zone of influence due to loading extends with depth. Directly below the loading plate, all materials "feel" the effect of the load and deform. At locations beyond the loading plate, only those materials within the zone of influence are deformed. At some distance, only the subgrade deforms. This concept serves as

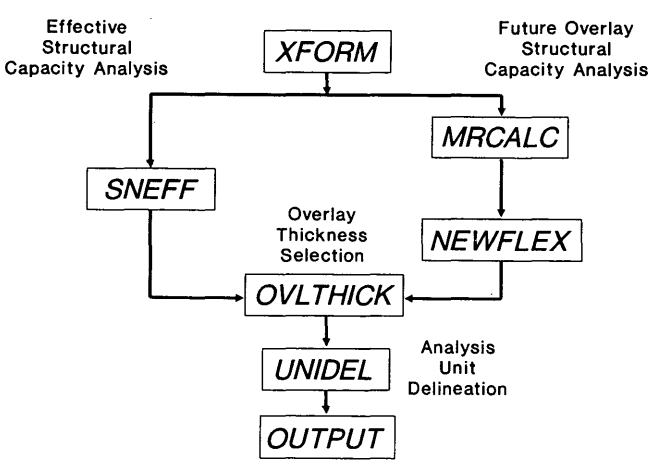

FIGURE 1 ROADHOG flow diagram.

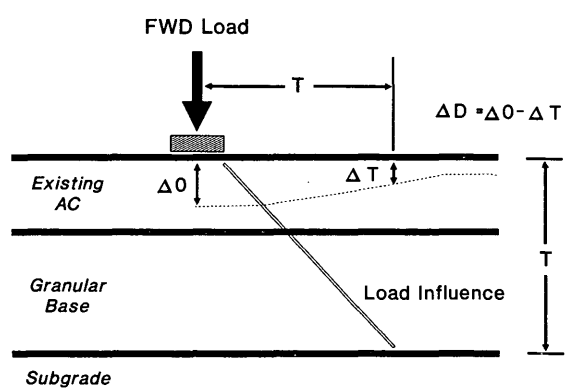

FIGURE 2 Conceptual basis for SNEFF.

the basis for most subgrade resilient modulus backcalculation methods.

Viewed from the perspective of the pavement, this concept suggests that the difference between two deflections could be used as measure of the pavement stiffness. Using the AASHTO assumption that SN_{eff} is a function of stiffness, the deflection difference becomes a measure of SN_{eff}. If the deflection at distance *T* in Figure 2 is due to subgrade deformation and the deflection at the center of loading is due to pavement and subgrade deformation, the difference between the two deflections, delta D, should represent the deformation within the pavement alone. If the zone of influence spreads at an angle of about 45 degrees, the distance *T* would be equal to the pavement thickness.

A relationship between the pavement stiffness (delta D) and the effective structural capacity of the pavement (SN_{eff}) was developed using elastic layer theory. Deflection basins were generated for a variety of pavement cross sections using the elastic layer program ELSYM5 (*10*). Delta D was calculated for each deflection basin and plotted against the structural number of the associated pavement cross section. The structural number of a pavement was calculated using Equation 1. Layer coefficients for new pavements were used in the determination: AC coefficient = 0.44; crushed stone base coefficient = 0.14. Plots of delta D versus SN_{eff} resulted in curves like those shown in Figure 3.

Total pavement thicknesses were 8, 12, and 24 in. Asphalt thicknesses ranged from 1 to 17 in. The elastic modulus values used in ELSYM5 to represent the asphalt and granular materials were 500 ksi and 30 ksi, respectively. These represent typical values for AC at about 70°F and dense graded granular base. They also are consistent with the layer coefficients and modulus relationships used by AASHTO. Subgrade resilient modulus values of 3.5 ksi, 7 ksi, 14 ksi, and 21 ksi were used for the analyses. These were selected as representative of the range of values for Arkansas subgrades expected on the basis of previous work (*11*). The results of the analyses (Figure 3) show the delta D–SN_{eff} relationship to be reasonably independent of the subgrade modulus.

These analyses, however, incorporated the standard elastic layer assumption of a semi-infinite depth of subgrade. Subgrade thickness (depth to bedrock) is believed to be one of the complicating factors in the backcalculation of subgrade resilient modulus (*6*). Additional analyses were performed to determine whether this factor might also be significant relative to the delta D–SN_{eff} relationship. Subgrade depths ranging from 8 ft to semi-infinite were considered. The delta D–SN_{eff} relationship was found to be reasonably independent of the subgrade depth (Figure 4). These findings indicate that this approach provides a practical method for the determination of SN_{eff} that is independent of the subgrade.

For the method to be complete, a means was needed for temperature adjustment. Asphalt concrete is quite temperature sensitive, exhibiting modulus increases at lower temperatures and modulus decreases at higher temperatures. As a result, delta D is also temperature sensitive. The elastic modulus used in the delta D–SN_{eff} analyses was selected as typical of the resilient modulus of an Arkansas asphalt concrete at 70°F.

Additional ELSYM5 analyses were conducted to examine the effect of other AC temperatures on delta D. The AC-modulus temperature relationship shown in Figure 5 was used to select modulus values for other temperatures. From these analyses temperature adjustment curves were established. The temperature correction factor is the ratio of delta D at a given temperature to delta D at 70°F. For testing temperatures other than 70°F, delta D from the given test is multiplied by the temperature correction factor to yield a corrected value of delta D. The corrected delta D value is used with the curves in Figure 3 to estimate SN_{eff} of the existing pavement. The temperature adjustment was reasonably independent of the subgrade but depended on both total pavement thickness and AC thickness. The temperature adjustment factors for an 8-in. pavement are shown in Figure 6.

The SNEFF module uses a second-order polynomial equation (r^2 = 0.98) to approximate the delta D/SN_{eff} relationship. For pavement thicknesses other than those shown in Figure 3, SNEFF uses linear interpolation to generate the points necessary to define a delta D/SN_{eff} curve. For the temperature adjustment, SNEFF approximates each curve with two straight-line segments joined at the 70°F point. For pavement thicknesses other than those analyzed by Kong, SNEFF calculates a temperature correction using linear interpolation.

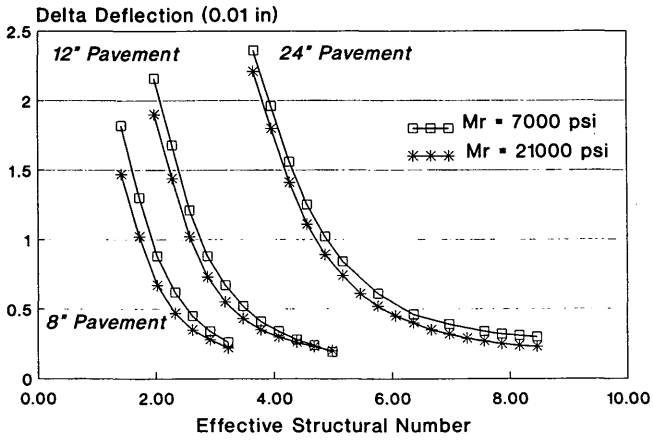

FIGURE 3 Delta D versus effective structural number.

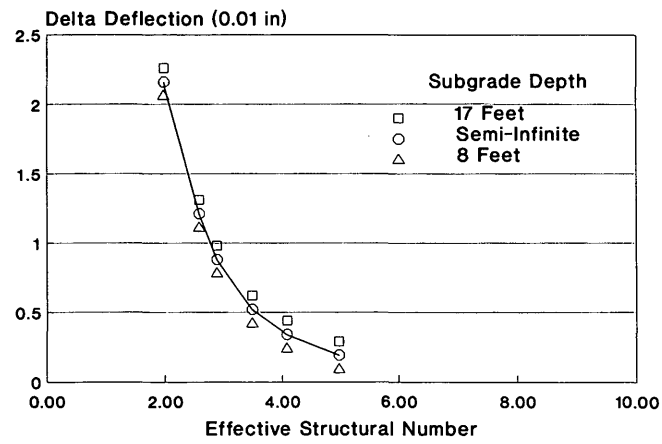

FIGURE 4 Effect of subgrade depth on the delta D–SN_{eff} relationship for a 12-in. pavement.

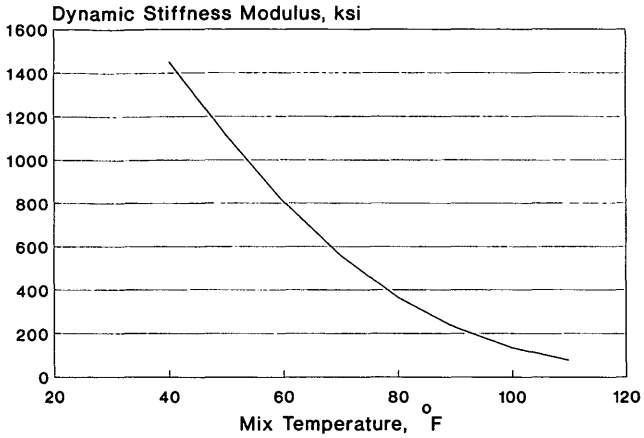

FIGURE 5 AC temperature modulus relationship used in SNEFF (*2*).

The delta D/SN_{eff} relationship developed using elastic layer theory (e.g., ELSYM5) was verified using an alternate approach—the ILLI-PAVE finite element method (*12*). The relationship between delta D and SN_{eff} generated using ILLI-PAVE was virtually identical to that generated using ELSYM5.

FUTURE OVERLAY STRUCTURAL CAPACITY ANALYSIS

In the ROADHOG design procedure, future overlay structural capacity analysis consists mainly of two steps: (*a*) determination of the in situ subgrade resilient modulus and (*b*) calculation of the structural capacity required to carry future traffic. The subgrade resilient modulus, used in the structural capacity calculation, is determined from NDT data. The required structural capacity calculation is identical to a new pavement design. A discussion of the procedure used by ROADHOG in each of these steps follows.

Estimation of elastic properties (e.g., modulus) of pavement layers and subgrade from NDT data has received much attention. A number of procedures for backcalculating the subgrade resilient modulus from deflection data have been developed (*5,7,8,13*). As part of the ROADHOG development effort, Morrison (*6*) studied the types of backcalculation procedures available and their applicability to the soils and environmental conditions found in Arkansas. Morrison identified three general backcalculation procedures for determining subgrade modulus: iterative techniques, direct solution, and empirical response algorithms. Each type of backcalculation technique has strengths and weaknesses.

The primary goals is selecting a procedure for use in ROADHOG included accuracy, simplicity, and speed. Some published backcalculation techniques are extremely elegant but were not yet considered practical for use in an everyday design procedure due to the equipment and time necessary to run the analyses. In addition, the inherent variability of resilient modulus associated with in situ soils makes the expenditure of large amounts of time and energy to backcalculate a modulus value to the nearest psi seem unproductive. The value of subgrade resilient modulus backcalculated using data from an FWD test represents the state of the subgrade soil at that particular point along the project and at prevailing moisture and stress-state conditions. Because of the variability of soil properties in horizontal construction, the procedure selected for estimating E_{ri} for overlay design should be practical and yield reasonable E_{ri} estimates.

Another consideration in selecting a backcalculation procedure is the appropriateness of using the modulus in the AASHTO design equation for flexible pavements. The original performance equations developed from AASHO Road Test data did not include any measure of soil support. To modify the AASHO performance equation for design, the 1986 guide incorporated a subgrade resilient modulus function in which a value of 3,000 psi was assigned to the subgrade at the AASHO Road Test site. This value seems to agree with the breakpoint resilient modulus values obtained by Thompson and Robnett (*14*) using Road Test soils. The breakpoint resilient modulus is defined as the point at which the slope of the resilient modulus–repeated deviator stress curve (Figure 7) typical of a fine-grained soil "breaks," or changes.

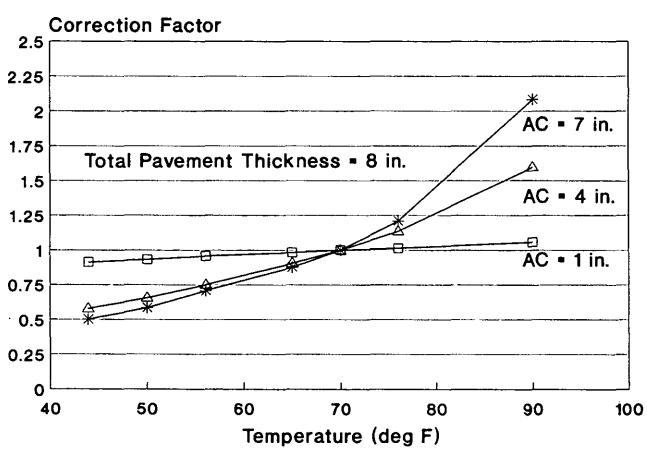

FIGURE 6 Temperature adjustment factors for 8-in. pavement in SNEFF.

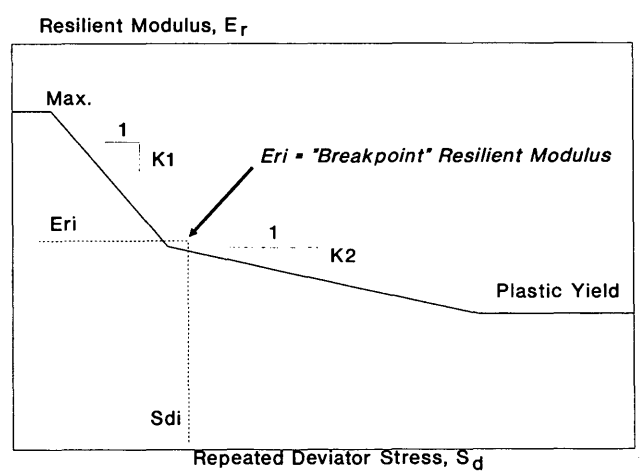

FIGURE 7 Typical representation of the resilient modulus–repeated deviator stress relationship for fine-grained soils (*2*).

To be consistent with the development of the AASHTO design equation, the subgrade modulus value used in the equation should be E_{ri}, the breakpoint resilient modulus. Of the backcalculation methods available, only some empirical response algorithms based on the finite element pavement model ILLI-PAVE calculate the breakpoint subgrade resilient modulus. Modulus values calculated by other methods must be adjusted to remain consistent with the AASHTO design equation (*3*).

ROADHOG uses an empirical response algorithm developed by Elliott and Thompson (*2*) to estimate the subgrade resilient modulus. The algorithm was developed from data generated by the finite element structural pavement model ILLI-PAVE. Finite element–based backcalculation procedures have several advantages over elastic layer– and numeric-based procedures (*6*). However, a major obstacle to using a finite element analysis in routine design is the complexity of the calculations involved. To date, mini- or mainframe computers are needed to fully exploit the advantages gained by using the finite element method. Empirical response algorithms like those developed by Elliott and Thompson help to bring finite element methods to the routine design level.

A finite element model used to generate a response algorithm must be valid for the conditions under which it is used. The data base used to develop the empirical response algorithm discussed here was comprehensive, covering a wide range of asphalt concrete and granular base thicknesses and subgrade strengths. Elliott and Thompson actually developed three equations for estimating the subgrade modulus beneath existing flexible pavements: (*a*) surface treatments—asphalt concrete thickness equal to 0.0 in., (*b*) conventional flexible pavements—asphalt concrete thicknesses ranging from 3 to 16 in., and (*c*) full-depth pavements—asphalt concrete ranging from 4 to 16 in. The three equations are nearly identical and produce practically the same subgrade modulus prediction. Therefore for practical purposes the equation selected for use in ROADHOG covers the range of AC thicknesses from 0 to 16 in. The ranges of material properties used in the analysis agree with observed material properties in the state of Arkansas, validating the use of the response algorithm for the ROADHOG procedure.

The calculation of in situ subgrade resilient modulus for the ROADHOG procedure is contained in the module MRCALC. The finite element–based response algorithm uses a single measured deflection to estimate the subgrade resilient modulus. The regression equation takes the form

$$E_{ri} = 25.0346 - 5.2454D_3 + 0.2864D_3^2 \quad (4)$$

where E_{ri} is the breakpoint subgrade resilient modulus and D_3 is the surface deflection at 3 ft from load.

Deflection data from sensors spaced at 0, 1, 2, and 3 ft from the load were analyzed during the development of the response algorithms. The data from the sensor at 3 ft had the highest correlation coefficient (0.99) with the calculated resilient modulus. The standard error of estimate for the response algorithm was 0.64 ksi. The comprehensive data base and an excellent fit make the algorithm a powerful computational tool.

The determination of the structural capacity (structural number) required to carry future traffic is identical to new pavement design. ROADHOG uses the AASHTO flexible pavement design procedure in the module NEWFLEX to determine structural requirements (*1*). User input for the NEWFLEX module includes the design reliability, design standard deviation, design change in serviceability index (delta PSI), and the number of ESALs for the design period. The subgrade resilient modulus, calculated by the MRCALC module, is supplied by ROADHOG.

OVERLAY THICKNESS SELECTION

Thickness selection of a flexible (structural) overlay for an existing flexible pavement is straightforward. The ROADHOG procedure uses the structural number relationships shown in Equations 2a and 3 for thickness selection.

In the ROADHOG procedure, overlay thickness selection is performed by the module OVLTHICK. OVLTHICK obtains the values of effective and required structural number from the data file and prompts the user for the asphalt concrete material coefficient. A value of a_{ac} equal to 0.44 is recommended to the user; however, the user may elect to use another value if it is deemed to be more appropriate. The 0.44 value is the average material coefficient for asphalt concrete as determined from the AASHO Road Test.

ANALYSIS UNIT DELINEATION

Analysis unit delineation is a process by which a length of pavement slated for rehabilitation (e.g., overlay) is subdivided into homogenous sections. Homogeneous sections or analysis units have been defined as "sections of pavement that can be considered nearly alike in terms of performance, age, traffic, structural capacity, etc., and for which a single treatment is appropriate" (*8*). Subdividing a project into analysis units can greatly increase the efficiency and cost-effectiveness of an overlay design. The use of analysis units can help to ensure that the optimum amount of overlay is placed where it is needed.

The subdivision of an overlay project into analysis units may be performed at a number of occasions in the overlay design process. Unit delineation should be performed before any pavement testing or design analysis based on construction records, visible pavement distress, known subgrade conditions, and so forth The overlay design would then be performed separately on each predetermined analysis unit. Analysis units may also be defined by a material sampling program or NDT data such as maximum deflection under load, and the overlay design performed separately on each predetermined analysis unit. ROADHOG performs analysis unit delineation on the basis of the actual required overlay thickness determined at each NDT test site, making unit delineation the final step in the ROADHOG overlay design procedure.

The ROADHOG procedure uses the "cumulative difference method" outlined in the 1986 *AASHTO Guide* to perform unit delineation in the module UNIDEL. A full discussion of the statistical method is contained in Appendix J of the *AASHTO Guide* (*1*). ROADHOG uses the required overlay thickness calculated for each measured deflection basin as the response variable in the procedure. The actual required

overlay thickness is the most reasonable estimate of the structural deficiency of the pavement at each NDT test site.

The UNIDEL module allows the designer to set the minimum length of an analysis unit. For long projects, a recommended minimum length of analysis unit is 1,000 ft (*8*). The minimum length should be based on economics and practicality. UNIDEL establishes "calculated analysis units" based solely on the statistical procedure outlined above. "Recommended analysis units" are determined by combining calculated units shorter than the minimum with adjacent units. After recommended units are determined, UNIDEL assigns each station along the project a unit number. Output of results according to analysis units is based on the assigned unit numbers.

DESIGN RELIABILITY

One difficulty in making meaningful comparisons between ROADHOG and other overlay design procedures is the method of applying a reliability level to the design. Reliability is the probability that a design will perform as intended for the design period. Thus a design with "50 percent reliability" has a 50 percent chance of performing satisfactorily; conversely, the design has a 50 percent change of failing during the design period.

In NDT overlay design, a level of reliability can be applied to the required thickness at each individual NDT test point, to the overall average required thickness, or to both. However, the meaning of applying a reliability to the average required thickness from thicknesses already determined at a reliability level is unclear. An in-depth study is needed to determine a meaningful method of handling reliability in overlay design.

IMPLEMENTATION

ROADHOG was completed in May 1990 and turned over to AHTD for trial implementation. For approximately 1 year, the AHTD research staff evaluated ROADHOG by using it together with other overlay design approaches to develop thickness recommendations for the Roadway Design Division. After 1 year, ROADHOG was released to Roadway Design to be used as the primary, routine overlay design tool. In a recent meeting, design engineers using ROADHOG expressed satisfaction with the procedure. The only reservations expressed were relative to some very thin overlay thicknesses from a few projects. However, a review of these projects revealed that, whereas the pavements needed rehabilitation, this need was not due to structural inadequacy; and ROADHOG, like other NDT-based procedures, only addresses structural inadequacy.

As stated in previous sections, methods used for SN_{eff} and M_{ri} estimation are based on material properties representative of conditions encountered in Arkansas. To implement the ROADHOG procedure in other areas, care must be taken to ensure that the material properties used by ROADHOG are representative of local conditions. If local material properties vary significantly from those used in ROADHOG modules, additional analyses (similar to those used in developing the algorithms used by ROADHOG) should be performed with the local material properties.

CONCLUSION

ROADHOG has proven to be a practical, easy-to-use design procedure for determining the overlay thickness needed to correct structurally deficient flexible pavements in Arkansas. The procedure follows the general design approach contained in the 1986 *AASHTO Guide* but incorporates some improved features. SN_{eff} is determined by a method that is independent of the subgrade resilient modulus (M_r) and the depth to bedrock, and M_r is determined in a manner consistent with the AASHTO design equation and not requiring the modification needed by other backcalculation methods. The unit delineation method used in ROADHOG assists the designer in optimizing the design by identifying areas needing different overlay thicknesses.

ACKNOWLEDGMENTS

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DISCUSSION

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The 1986 *AASHTO Guide* (*1*) incorporates a remaining life concept in overlay thickness design and expresses the overlay structural requirement in the following form (Equation 2 in the paper):

$$SN_{OL} = SN_O - F_{RL}(SN_{xeff}) \quad (5)$$

Elliott (*2*) demonstrated that the overlay thickness computations using AASHTO design curves for F_{RL} produced inconsistent results and recommended that "the AASHTO overlay design approach be revised to exclude remaining life considerations." However, subsequent work by Easa (*3*) and Fwa (*4*) illustrated that the AASHTO concept of remaining life is fundamentally correct and that consistent results are obtained using corrected F_{RL}. Unfortunately, the current paper continues to adopt the view expressed by Elliott (1989) and states that the inclusion of the concept of remaining life "was not found to be reasonable or practical." It is shown in this discussion that reverting to the use of the traditional overlay equation $SN_{OL} = SN_O - SN_{xeff}$ (Equation 2a in the paper, setting $F_{RL} = 1.0$), as recommended by Elliott (*2*) and the authors, is conceptually unsound.

SIGNIFICANCE OF REMAINING LIFE CONCEPT

The basic difference between the AASHTO overlay equation (Equation 5) and the traditional overlay equation lies in the fact that the traditional equation computes SN_{OL} required at the time of overlay construction, and no check is made to ensure that the overlay provided will be adequate during the entire design period. It can be shown (*1,4*) that, depending on the structural deterioration rate of an existing pavement after overlay application, the overlay requirement at a later stage of the service life of the pavement may exceed the value computed by the traditional overlay equation. Using different deterioration curves for old pavements after overlay, Easa (*3*) and Fwa (*4*) showed that the overlay thickness requirement varied with the remaining lives of the old pavements. Including the remaining life consideration in overlay design is therefore a refinement and improvement of the traditional overlay equation.

SIGNIFICANCE OF REMAINING LIFE FACTOR F_{RL}

The remaining life factor can be derived and shown to be a function of the structural capacities of existing and overlaid pavements and hence a function of the remaining lives of the pavements (*1,4*). Referring to Appendix CC of the 1986 *AASHTO Guide*, where an excellent description of the remaining life concept is presented, it is obvious that the correct F_{RL} value to be used in Equation 5 should be determined by considering the overlay requirements at all stages in the overlay design life. A detailed explanation of how this can be done is given elsewhere (*4*), where a procedure for selecting the governing F_{RL} value is described. The F_{RL} value so determined will lead to the choice of SN_{OL} from Equation 5 that provides an adequate overlay thickness for the entire period of the design service life. It is apparent that this important aspect of remaining life factor computation is not considered by Elliott (*2*) and the authors in their analyses of overlay design. Not realizing this significance of the concept has probably resulted in their call for exclusion of remaining life consideration from overlay design and their doubt of the statement that factor F_{RL} would "reflect a more realistic assessment of the weighted effective capacity during the overlay period."

BASIS OF AUTHORS' RECOMMENDATIONS

The authors state that "for all practical purposes, a value of 1.0 is the appropriate value for F_{RL}," and that the inclusion of the remaining life concept "within the structural deficiency design approach used in ROADHOG was not found to be reasonable or practical." The only basis for these recommendations was the work reported by Elliott (*2*). An examination of Elliott's paper shows that there is little justification for the strong recommendations. The recommendations were based solely on an analysis using a "simple scale transformation" that relates R_{Lyx} to R_{Ly} for a given R_{Lx} as follows:

$$\left\{\frac{R_{Lx}}{1}\right\} = \left\{\frac{R_{Lyx}}{R_{Ly}}\right\} \quad (6)$$

Substituting Equation 6 into the AASHTO equation for calculating F_{RL}, Elliott (*2*) concluded that the appropriate value for F_{RL} was 1.0. Three points can be raised regarding Elliott's analysis:

1. The F_{RL} value was computed for only one point (i.e., at the end of the service life), which is not a correct way of selecting a design F_{RL} value. For example, F_{RL} values greater than 1.0 were mentioned. This would not be the case if the proper procedure of selecting a design F_{RL} value according to the concept of remaining life is followed. At the time of overlay construction, F_{RL} would be 1.0 if evaluated then. This effectively eliminates one from choosing an F_{RL} value greater

than 1 when the overlay requirements at other times during overlay service life are checked.

2. No physical meaning or practical significance was given to justify why the relationship in Equation 6 was used. The "philosophy" given by Elliott (*2*) was "the concept of the man who each day walks halfway to his destination." This writer finds it difficult to relate the philosophy to overlay performance. It is, however, easy to show that Equation 6 has a highly controversial implication not stated by the authors or Elliott (*2*): for a given pavement with known R_{Lx}, the rate of change in R_{Lyx} is proportional to the rate of change in R_{Ly}. This underlying assumption of Equation 6 is severely restrictive in application, and it is not in agreement with common understanding of how old and new pavements deteriorate. Easa (*3*) and Fwa (*4*) have shown that there are many other more meaningful deterioration relationships that would produce consistent overlay designs according to the AASHTO remaining life design concept, and that the values of F_{RL} varied from about 0.5 to 1.0 for the cases they considered.

3. It is physically meaningful to derive the pavement performance relationship in terms of pavement conditions or structural capacity. Equations involving multiplication and division of remaining life fractions R_L of different pavements are difficult to interpret physically. This is because R_L is a nonlinear function of pavement structural condition, and it is a fraction of load repetitions N_f, which are different for different pavements. Fwa (*4*) has shown that putting different R_L values in an equation without paying attention to the different base N_f values can lead to erroneous results. For example, the "simple" transformation of (R_{Lx}, R_{Lyx}) to (1, R_{Ly}) as shown in Equation 6 is deceptively straightforward. However, expressed in terms of pavement structural condition or load repetition capacity, the "walking man philosophy" may not make any sense.

SUMMARY REMARKS

This discussion deals with the overlay design methodology of the paper. Explanation and findings of other studies have been presented to show that the AASHTO remaining life concept is technically sound and that the recommendations by the authors concerning the use of F_{RL} = 1.0 and the exclusion of the remaining life concept in overlay design are misleading and not justified.

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AUTHORS' CLOSURE

We recognize the validity of remaining life as a concept that needs to be considered in overlay design. However, its application as presented in the 1986 *AASHTO Guide* is flawed and adds complications to the design process that are not warranted. Because of this, the 1986 approach to remaining life was not included in ROADHOG. The concept adopted for ROADHOG is that SN_{eff} should represent the contribution of the pavement to the future performance after it is overlaid. If SN_{eff} is selected properly, the application of the F_{RL} term represents a double penalty for the effects of past traffic. Research is needed to determine whether the method of selecting SN_{eff} developed for ROADHOG selects the appropriate value.

We are familiar with the papers by Fwa and Easa. These papers acknowledge the flaw in the 1986 guide remaining life first observed by Elliott. Both papers attempt to remedy the flaw; however, the remedies further complicate a process that is already more complex than is warranted within the empirical AASHTO approach to pavement design. The AASHTO approach has served for many years as a useful design tool. However, it is a tool that has been extrapolated far beyond its original data base, often with little justification other than engineering judgment. Its application to overlay design represents further extrapolation. The addition of a complicated, sophisticated approach to remaining life simply is not justified.

In this respect, the decision to not include the F_{RL} term in ROADHOG was not solely because of its flaws. Even without the flaws there are reasons and sentiment for its removal. Other reasons are the removal of unwarranted complications as cited above and the elimination of confusion and lack of understanding generated by the remaining life factor when it was introduced in 1986.

The introduction of the F_{RL} term into overlay design created much confusion. Designers did not understand the term or its application. Even veteran pavement researchers had trouble accepting and understanding remaining life as it is presented in the 1986 guide. This is perhaps best demonstrated by the fact that the F_{RL} term was introduced in 1986 (and reviewed and questioned by knowledgeable pavement engineers before that), yet the flaws in the concept were not noted until 1989.

The development of the F_{RL} methodology and the modifications proposed by Fwa and Easa suggest a lack of understanding of the limitations of the AASHO Road Test performance equations that serve as the basis for the AASHTO pavement design procedures. These equations are best-fit regression equations developed to predict the performance of the pavement sections at the Road Test. Strictly speaking, they are only valid within the very limited context of the pavement types, axle loads, subgrade, environment, time, and so forth of the AASHO Road Test. To use them to develop a sophisticated concept of remaining life for overlay design represents a gross extrapolation, far beyond the original data base or intent of the equations. Such use suggests that the Road Test equations are fundamental behavioral relationships. They are not. To use these empirical equations in this manner is an interesting academic exercise, but it is not valid.

There is also a danger in incorporating procedures developed in such a manner into routine engineering practice, especially when such procedures are complicated and sophisticated in appearance. The incorporation and sophistication suggest a legitimacy that does not exist. Once accepted into practice, the procedures can become "etched in stone" and very difficult to change or correct when more advanced technology becomes available.

Nationwide Evaluation Study of Asphalt Concrete Overlays Placed on Fractured Portland Cement Concrete Pavements

MATTHEW W. WITCZAK AND GONZALO R. RADA

Historically, agencies responsible for pavement rehabilitation have tried a wide variety of materials, processes, and construction methods to eliminate or minimize reflective cracking of asphaltic concrete overlays placed on existing portland cement concrete (PCC) pavements. Over the last 10 years, the fractured slab approach using rubblize, crack and seat, and break and seat has gained increased acceptance. Because the fractured slab approach has gradually evolved through field demonstration and actual projects, very little fundamental knowledge concerning design, construction, and performance models is available. Understandably, performance to date has been variable. To improve the state of the art and develop a better understanding of these techniques, a nationwide study was undertaken. A literature review resulted in the location of nearly 500 highway projects throughout the United States. From this generalized data base, approximately 100 sites were selected for detailed field studies. Field crews conducted visual distress surveys to assess pavement performance and nondestructive deflection testing to assess the in situ characteristics of the pavement layers. The general approach used for the research study and the analysis of field performance and structural data obtained is presented. Performance predictive equations are presented along with the evaluation of the back-calculated effective moduli of fractured PCC slabs for each technique. Analysis of within and between project variability is presented.

The selection of optimal rehabilitation procedures and strategies for deteriorating highway pavements requires a knowledge of the type and cause of the distress, determination of candidate rehabilitation procedures, and selection of an optimal strategy based on economic and other considerations. For portland cement concrete (PCC) pavements, the array of possible rehabilitation procedures includes nonoverlay methods such as undersealing, grinding of the surface, and removal and replacement of distressed areas; full reconstruction by replacement or recycling; PCC overlays; and asphaltic concrete (AC) overlays.

Review of current practice indicates that AC overlays are the most commonly used PCC rehabilitation procedure, with about $1 billion of AC overlays placed each year, and this amount will likely increase in the future (*1*). Even though they are commonly used, the performance of AC overlays on PCC pavements is often hampered by the occurrence of reflection cracks over existing joints and cracks. This type of distress constitutes the most frequent cause of the loss of performance for AC overlays.

Reflection cracks in the AC overlays are caused by a combination of thermal and traffic-induced stresses. Expansion and contraction of the PCC pavement results in horizontal movements that produce strains in the AC overlay exceeding its tensile strength. Traffic loads can cause vertical differential movements at the location of joints and working cracks in the PCC slab and induce critical shear stresses at the bottom of the AC layer. The overlay immediately over the joints and working cracks in the PCC is not able to accommodate these localized movements, resulting in the development of reflection cracks.

A wide variety of rehabilitation techniques aimed at preventing the formation of, or minimizing, reflection cracking have been attempted. They include thick (conventional) overlays, crack relief layers, the saw and seal technique, special overlay and interface materials, and the fractured slab approach. Of these, the technique that has been used increasingly over the last 10 years has been the fractured slab approach.

The major objective of the fractured slab approach is to reduce the effective in situ slab length before the overlay is placed. If this is effectively accomplished, the likelihood of having reflective cracks appear is significantly reduced or eliminated. The probability of reflective cracking is proportional to the horizontal movement at joints and cracks, which in turn is directly proportional to the spacing between joints and cracks.

The fractured slab category is generally subdivided into three major types of rehabilitation: rubblize, crack and seat, and break and seat. Rubblize is a fractured slab process intended to transform the existing PCC layers into fragments having textural and gradational characteristics similar to those of a large aggregate size crushed stone base. It is most effectively accomplished with a resonant pavement breaker, which has been successfully used on all types of existing PCC pavements [i.e., jointed plain (JPC), jointed reinforced (JRC), and continuously reinforced (CRC) concrete pavements].

Crack/seat and break/seat are fracture techniques intended to produce very short rigid slabs whose effective lengths vary from 12 to 48 in. The techniques are similar, with guillotines or spring-arm (whip) hammers being used to develop reduced crack spacings in the existing PCC pavement. There is, however, a significant distinction between the two techniques. Crack/seat is associated with the fractured slab process conducted solely on JPC pavements. For these pavements, the

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objective of the crack/seat process is to develop closely spaced, tight cracks that permit load transfer across the crack through aggregate interlock with little loss of structural value. Fracture or cracking through the entire depth of the PCC layer is the ultimate goal.

Break/seat is associated with the fractured slab process on JRC pavements. The ultimate objective of this technique is to physically fracture the distributed steel or completely debond the steel from the concrete. Whereas cracking may result through the entire PCC layer depth, if steel fracture or debonding is not accomplished, the effective slab length is not reduced in the construction process, and what remains is a series of smaller slabs tied together into a longer effective slab by the bonded distributed steel.

A corequisite to the slab fracturing process is the seating portion of the construction. For both cracking and breaking, it is customary to have five to seven passes of a 35- to 50-ton rubber-tired roller seat the fractured slab fragments. This provides a relatively smooth and uniform grade for paving operations and serves as an excellent means of proof-rolling before the AC overlay is placed. For rubblized projects, steel vibratory rollers (generally 10 tons) are normally used for the compaction or seating process.

NATIONAL EVALUATION STUDY

Objectives and Overview

Over the past 10 years, there has been a dramatic increase in the use of the fractured slab techniques for the rehabilitation of deteriorating PCC pavements. Much field experience has been gained during this time. However, little technical guidance, relative to the design and construction of these techniques, is available to adequately predict their performance in minimizing reflective cracking under specific traffic and climatic conditions for a particular pavement structure and existing condition.

In recognition of the critical need for a sound technical basis to support the use of the fractured slab approach, a major state-of-the-art research study was undertaken. The overall objective of the study was to develop national guidelines and methodologies for the use of these three rehabilitation techniques.

This paper presents the general approach used for the research study and concentrates on the analysis of field performance and structural data obtained. Performance predictive equations, using the pavement condition index (PCI), are presented along with the evaluation of the backcalculated effective moduli of fractured PCC slabs for each rehabilitation category. Analysis of within-project and between-project variability is presented and forms the basis for an overall design methodology described in a companion paper in this Record by Witczak and Rada.

Synthesis of Current Practice and Data Base Development

Sources of information initially collected and evaluated were obtained from various state asphalt pavement organizations, federal and state highway agencies, the Transportation Research Board, the National Asphalt Pavement Association, the Asphalt Institute, and other highway-oriented organizations. During the conduct of this study, several extremely relevant studies were completed, including *NCHRP Synthesis 144* (*2*), FHWA Contract DTFH 61-86-C-00079 (*3,4*), and individual state highway agency investigations. A complete list of the information sources may be found in the final report of this study (*5*).

The collected information was used to prepare a synthesis of current practice. This included design, construction, specifications, costs, and performance experience for projects using the crack-controlling techniques being studied. The data base was also used to identify AC overlay rehabilitation projects that used slab fracture techniques to control reflection cracking as candidates for the field investigations.

A total of 454 field projects in 34 states were identified for which at least partial design, construction, and performance information was available. Of the 454 projects, 250 were crack/seat, 150 break/seat, 19 rubblize, and 35 unknown (steel information was not available; thus, they could not be grouped into the crack/seat or break/seat categories). The geographic distribution of the projects by rehabilitation type is shown in Figure 1. In general, crack/seat projects were concentrated in the upper midwest and western states, break/seat projects in the northeastern portion of the country, and rubblize projects in the eastern half of the country.

Of interest from a practical viewpoint is a summary of several key variables given in Table 1 for each of the three re-

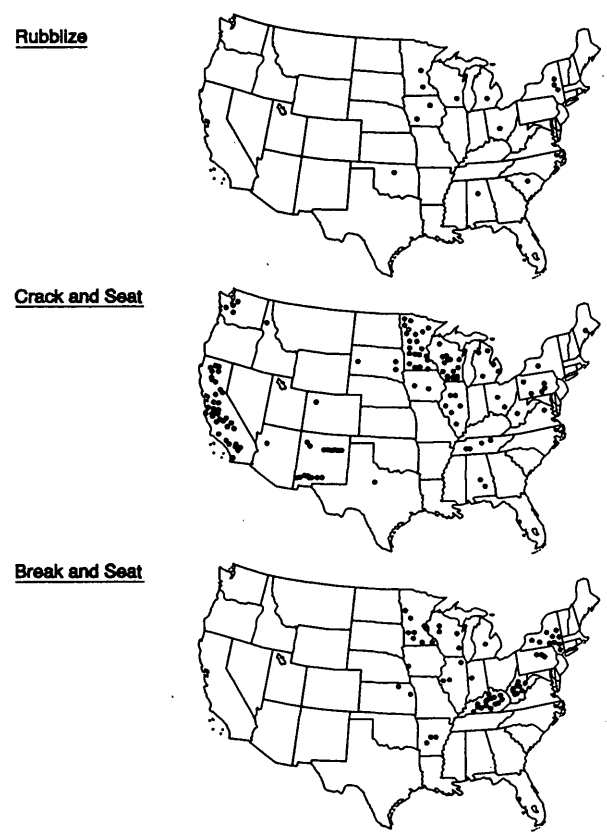

FIGURE 1 Geographical distribution of projects by rehabilitation type.

habilitation types. As can be observed, the average dates of the rehabilitation options clearly show the relative "youthfulness" of the methods discussed, particularly for the rubblize technique. The table also indicates that the average PCC thicknesses range between 8 and 10 in., which are typical of highway pavements. Finally, the resulting average statistics for the AC overlay thicknesses show that the rubblize technique had the largest average overlay thickness. This is consistent with the fact that the rubblization process is intended to truly transform an existing rigid layer into a conventional flexible layer.

A computerized data base was produced by compiling information on these projects from available documents and reports plus follow-up contacts with state highway agency personnel to search files for additional information. In this study, two types of data bases were developed: (*a*) general data base and (*b*) detailed data base. The general data base contains available data and information found previously through the literature search. Wherever possible, this information was used during the analysis portion of the study. However, the major use of this data base was to select pavement test sections for the field investigation program. These projects formed the detailed data base.

A primary consideration in the selection of the field test sections was to include a range of variables that influence pavement performance—for example, climate, AC overlay thickness, rehabilitation age, and age of existing PCC pavement before overlay. A total of 93 pavement sections—17 rubblize, 35 crack/seat, and 41 break/seat projects—were eventually selected from the general data base for the field investigation program. An additional 54 sections were included in the detailed data base as a result of deflection testing and other pertinent data provided by the Kentucky, Illinois, and Michigan highway agencies.

Field Investigations

Because of the limited availability of documented field performance for the fracture slab techniques, a primary activity of this research study was a field investigation of existing highway pavement sections where these techniques had been used. The two major field testing activities were visual distress surveys made in accordance with the PCI highway methodology and nondestructive deflection testing (NDT) using a falling weight deflectometer (FWD).

The test sections generally consisted of a 1,000-ft strip in one direction. For the PCI data collection effort, this strip was divided into five 200-ft segments. The segments in the driving lane, as well as the adjoining five segments in the passing lane, made up the 10 sample units used for recording PCI data. For each sample unit, the types of distress present were identified and their severity and extent were quantified.

The deflection testing was conducted every 25 ft for the 1,000-ft length, in the outside wheelpath of the driving lane. Each test point consisted of three drops at a target 9,000-lb load with a Phonix M10000 FWD. Deflections were measured for every drop by means of six geophones located at distances ranging from 0 to 60 in. from the center of the load plate. Deflection data were also received for 42 test sections in Illinois, 8 test sections in Michigan, and 4 test sections in Kentucky. In all, 4,700 NDT test points on 140 sections were obtained and subsequently used in the structural analyses—1,019 points on 24 rubblized sections, 1,776 points on 64 crack/seat sections, and 1,905 points on 52 break/seat sections.

Data Analysis and Interpretation

Once the field testing was completed on all test sections and data incorporated into the detailed data base, an extensive analysis was initiated. The major thrust of the analysis was to evaluate the rehabilitated pavements from both a functional and structural point of view. The results of the PCI visual condition surveys were used to estimate performance trends. Nondestructive deflection data, coupled with available cross section information, were used to backcalculate the effective in situ modulus for the pavement section layers.

The PCI values were determined according to the U.S. Army Corps of Engineers procedure (*6*). In addition, a new index value was introduced into the analysis: PCI_{lt}. The PCI_{lt} value reflects the PCI value due only to the presence of longitudinal/transverse cracking in the overlay—it excludes all distress types other than the cracking. Because prevention of reflective cracking is a major concern to the efficiency of the rehabilitation activity investigated, it was believed that the PCU_{lt} value would perhaps be another appropriate statistic to analyze.

Effective moduli of the pavement layers were backcalculated from elastic layer theory and the measured deflection basins for each specific test location. One of the major underlying hypotheses of the backcalculation study deals with the fundamental concept that a direct relationship exists between the fractured PCC modulus (E_{PCC}) value and the overall effectiveness of the construction operation in reducing the effective in situ moduli. Thus, the lower the E_{PCC} value, the greater the effectiveness of the construction operation in min-

TABLE 1 Summary of Average Statistics of Field Test Sections

Rehabilitation Type	Average Date of Rehabilitation	Average AC Overlay, in.	Average PCC Thickness, in.
Crack and Seat	1984	4.4	8.3
Break and Seat	1985	5.6	9.4
Rubblize	1986	6.0	8.9
Saw and Seal	1983	3.4	8.3

imizing the potential for eventual reflective cracking of the AC overlay.

All sections were analyzed as three-layer structures, with the bottom (subgrade) layer being of semi-infinite thickness. These structures were analyzed with a PCS/LAW in-house program called EMOD. This program uses the Chevron N-Layer elastic solution as a subroutine within the backcalculation analysis, and the respective moduli given in the output are those that result in the minimum cumulative residual square error at all deflection readings. Required thickness data were obtained from the respective state DOTs. Poisson's ratios were assumed to be 0.35 for the AC overlay, 0.30 for the fractured PCC layer, and 0.4 for the subgrade.

On the basis of the resulting PCI, PCI_{lt}, and fractured PCC modulus (E_{PCC}) data, three analyses were conducted: (*a*) development of PCI and E_{PCC} predictive models, (*b*) investigation of the influence of crack spacing on the E_{PCC}, and (*c*) investigation of the variability of the E_{PCC} values between and within projects. This paper cannot address all of the results and findings that were generated from these analyses, but several key results are presented that form the basis for the design methodology described in the companion paper in this Record.

MAJOR STUDY RESULTS

PCI Predictive Equations

One of the major objectives of the data analysis was the development of equations to predict both the pavement condition index (PCI and PCI_{lt}) and the in situ modulus of the fractured PCC layer (E_{PCC}). To accomplish this, multiple regression techniques were used to develop hundreds of models, using as many as 15 independent variables. Only the best regression models are presented in this paper. The ultimate criteria used to select the best models were the correlation coefficient (R^2) value and engineering reasonableness of the significant parameters (independent variables) coupled with the respective sign of the coefficients.

Before discussing the results of the multivariate PCI predictive equations, it is both important and revealing to present global trends of PCI versus time from the rehabilitation construction. Table 2 summarizes the PCI_{lt}-time predictive equation developed for each of the three fractured slab techniques studied and the average time to reach typical failure conditions. The rubblized and crack/seat techniques appear to be the best-performing systems, whereas the typical break/seat sections do not respond quite as well. On the basis of these results, it can be stated that the ranking of techniques, in order of decreasing typical performance life, appears to be rubblization (best), crack/seat, and break/seat (worst).

The multivariate PCI predictive equations indicate that relatively good models were developed for each rehabilitation technique except the break/seat. The recommended models are summarized in Table 3 for both PCI and PCI_{lt}. As can be observed, the R^2 values for the rubblize and crack/seat equations are good. However, this fact must be tempered with the small data base coupled with the relative youth of these techniques. Furthermore, the use of these equations in a design sense is not recommended because of the significant degree of extrapolation outside the data variable levels that would be required.

Whereas the rubblize and crack/seat equations are limited in use, the rationality and sign of the coefficients for both time and overlay thickness are reasonable; PCI decreases with time and increases with overlay thickness. The subbase modulus is likewise considered to be a logical variable, reflecting a portion of the existing pavement support capacity. Because of the small data base, only a linear variable in time was found, although it is reasonable to expect that a nonlinear time variable would best model the behavior. Attempts to incorporate traffic and other variables proved fruitless due to the lack of accurate information in the data base. Finally, the presence of the precipitation and temperature terms in the models cannot be conclusively rationalized because they may either be false indicators of other more important geographical factors not studied or they may indeed reflect the significant influence of the environment on the performance of these sections. Their inclusion, however, was found to significantly increase the overall R^2 value of the predictive models.

For the break/seat technique, the models given in Table 3 were the best models developed. However, they are considered to be poor performance predictors, as judged by the low R^2 values obtained. A significant effort was expended toward obtaining models with greater predictive accuracy, but this effort proved fruitless, even though the maximum performance period and number of sections within the data base were the greatest for this technique. Several equations were developed with R^2 values in the 0.40 to 0.57 range, but they were rejected due to the unreasonable coefficient signs. Thus, the high variability of the performance could not be explained in an analytical manner.

TABLE 2 Global PCI-Time Models and Times to Failure

Rehabilitation Type	General PCI-Time Model	Time to PCI = 50	Time to PCI = 40
Rubblize	$PCI_{lt} = 100 - 1.613t + 0.092t^2$	*	*
Crack/Seat	$PCI_{lt} = 100 - 0.343t - 0.136t^2$	18.0 years	19.8 years
Break/Seat	$PCI_{lt} = 100 - 0.050t - 0.316t^2$	12.5 years	13.6 years
All Fractured Slabs	$PCI_{lt} = 100 - 0.149t - 0.252t^2$	13.8 years	15.2 years

(*) Unable to project time as PCI>90 at t=8 to 10 years.

TABLE 3 PCI Predictive Equations

Rehabilitation Type	PCI Prediction Model	R^2
Rubblize	$PCI = 173.77 - 0.878t + 0.389h_o - 0.744P - 0.719T$	0.753
	$PCI_{lt} = 217.28 - 0.615t + 0.310h_o - 1.14P - 1.13T$	0.665
Crack/Seat	$PCI = 127.02 - 3.03t + 2.4h_o - 0.34P - 0.44T + 0.09E_{SUB}$	0.755
	$PCI_{lt} = 116.8 - 1.42t + 0.658h_o - 0.13P - 0.25T + 0.001E_{SUB}$	0.511
Break/Seat	$PCI = 100.0 - 0.249t^2 - 0.0027(T*JS) + 0.072E_{SUB} + 0.465h_o$	0.393
	$PCI_{lt} = 100.0 + 5.61t - 0.77t^2 - 0.0032(T*JS) + 0.81h_o$	0.443

Note: t = time in years; h_o - HMA overlay thickness (inches); P = annual average precipitation (inches); T = annual average temperature range (F°); E_{SUB} = subgrade (subbase) modulus (ksi); JS = joint spacing (feet).

Fractured PCC Modulus Predictive Equations

Like the PCI analysis, reasonable predictive models for the fractured PCC modulus (E_{PCC}) were obtained for the rubblized and the crack/seat techniques. The recommended model for the rubblized technique is

$$E_{PCC} = 1690 - 15.4P - 17.34T + 2.2E_{SUB} + 1.9JS + 34.7CS \qquad R^2 = 0.603 \tag{1}$$

whereas that for the crack/seat technique is

$$E_{PCC} = -968.39 + 20.34E_{SUB} + 34.89CS + 5.37SL \qquad R^2 = 0.776 \tag{2}$$

where

P = annual average precipitation (in.),
T = annual average temperature range (°F),
E_{SUB} = subgrade (subbase) modulus (ksi),
CS = crack spacing (in.), and
SL = seating load (tons).

For the rubblized data, the relatively good R^2 value probably reflects the small range of E_{PCC} values (200 to 700 ksi). The presence of the precipitation and temperature variables cannot be conclusively rationalized; they may indirectly reflect other geographic or environmental variables not considered in the analysis. The joint spacing variable is not of real significance as indicated by the small coefficient. The subbase modulus variable is believed to be reasonable and important; it implies that as the subbase becomes stiffer, more energy is required to achieve a given E_{PCC} value. Finally, the influence of the crack spacing appears to be significant at first glance, but a closer look at the data reveals that this variable only ranges from 6 to 12 in.

On the basis of the significant data base size and the relatively high R^2 value obtained, it is believed that the predictive model developed for the crack/seat technique is quite good. The model indicates that the three primary variables influencing the modulus are the subbase stiffness, crack spacing, and seating load used in the construction process. As one would expect, the E_{PCC} value increases as these variables are increased.

Finally, the statistical analysis to develop an E_{PCC} predictive model for the break/seat technique did not result in a positive conclusion. More than 35 model forms were tried, but in no case was a model found with an R^2 value greater than 0.20. This finding is consistent with the large scatter and range of modulus values determined in the study. It was concluded that the primary cause of this problem is the large variability within the construction process to fully achieve steel debonding or steel fracture, or both.

Influence of Crack Spacing on the Fractured PCC Modulus Values

The effective modulus of a fractured slab is a function of the nominal fragment size actually achieved during the construction process. In practice, this concept has been applied to specifications that substitute the proposed crack spacing to be achieved as an indirect indicator of the effective modulus of the fractured layer. Whereas general correlations exist between crack spacing and modulus, the use of visual assessments is not always accurate for estimating the fractured moduli of the PCC layer. As the spacing is reduced, the PCC layer behaves less like a slab having a sound modulus of approximately 5,000 ksi and more like a flexible layer with a significantly lower modulus value.

It is common to denote the relationship between spacing and reduced effective PCC modulus by the use of a modular ratio parameter (E_r):

$$E_r = \frac{E_{PCC}}{E_{sound}} \tag{3}$$

where E_{PCC} is the effective PCC modulus of fractured slab and E_{sound} is the modulus of elasticity of sound PCC (E = 5,000 ksi).

Using the fractured PCC moduli found in this study, average E_r values as a function of crack spacing were developed and compared with the recommended 1986 AASHTO relationship (7). Figure 2 shows that relatively good agreement exists for the rubblized and crack/seat sections. However, major differences are present for the break/seat sections. As shown, the modular ratio results are approximately two to three times as large as those for the crack/seat sections at the same specified crack spacing. This fact, reinforcing previous results and analyses, led the authors to conclude that the break/seat process may not effectively achieve full debonding of the steel from the PCC or successful fracturing of the steel. Because of this, the actual effective slab length after fracturing is much greater than one would conclude by looking at the actual crack spacing.

In the previous section, predictive equations for the E_{PCC} values were presented for both rubblized and crack/seat sections. These models illustrate the importance of crack spacing on the fractured PCC modulus. As the spacing is increased, E_{PCC} increases. In the case of the crack/seat technique, historic information has shown that crack spacings typically range from 12 to 60 in. However, on the basis of the results of this study (i.e., E_{PCC} predictive equation), the following crack spacings as a function of the foundation type are recommended: 30 in. for subgrade soils, 24 in. for granular subbase, and 12 in. for stabilized subbase. Target spacings decrease as the stiffness of the underlying foundation is increased. Rubblization generally results in fragment sizes in the range of 6 to 12 in., which appears satisfactory for this technique.

Variability of Fractured PCC Modulus Values

The analysis of the deflection test data consisted of 4,700 backcalculated estimates of the in situ fractured PCC modulus (E_{PCC}) on 140 sections (64 crack/seat, 52 break/seat, and 24 rubblize). For each section, the average E_{PCC} value, standard deviation, and other statistics were calculated. Whereas the results appeared to be highly variable at first glance, further detailed analyses of the data led to important conclusions regarding the two major forms of variation encountered: "between" and "within" project variability.

The between project variability reflects the variation between the average project predicted E_{PCC} values. As such, the standard deviation (σ_b) or variance (σ_b^2) reflects the variations attributable to each construction process on a national scale. Specifically, factors such as the type of equipment, specific breaking energy, specified crack spacing, and the specific site factors and pavement cross section are all reflected within the σ_b (σ_b^2) parameter.

The analysis of the statistical frequency distribution patterns of the average project E_{PCC} value was used to assess the variability (σ_b or σ_b^2) of a particular rehabilitation type for the spectrum of construction projects examined across the United States. Figure 3 shows the between project E_{PCC} frequency distributions for each of the three techniques in question. Table 4 summarizes the between project E_{PCC} statistics.

Figure 3 shows that the rubblized E_{PCC} project distribution is somewhat normal (actually bimodal) and contained within a relatively small range of E_{PCC} values. Similarly, the frequency distribution for the crack/seat sections investigated is normally distributed and is contained within a small range of E_{PCC} values. There are, however, several outliers of data found at high modulus values.

Unlike the rubblized and crack/seat projects, the break/seat E_{PCC} values are widely distributed and highly indicative of the variable success in fracturing or debonding distributed steel in the concrete. Typical values of as low as 250 ksi to as large as 2,750 ksi were obtained. The distribution appears to be more uniform across these limits rather than normally distributed.

In contrast to the between project variation, the within project variability (σ_w or σ_w^2) reflects the variation of the E_{PCC} values obtained within a given rehabilitation project. As such,

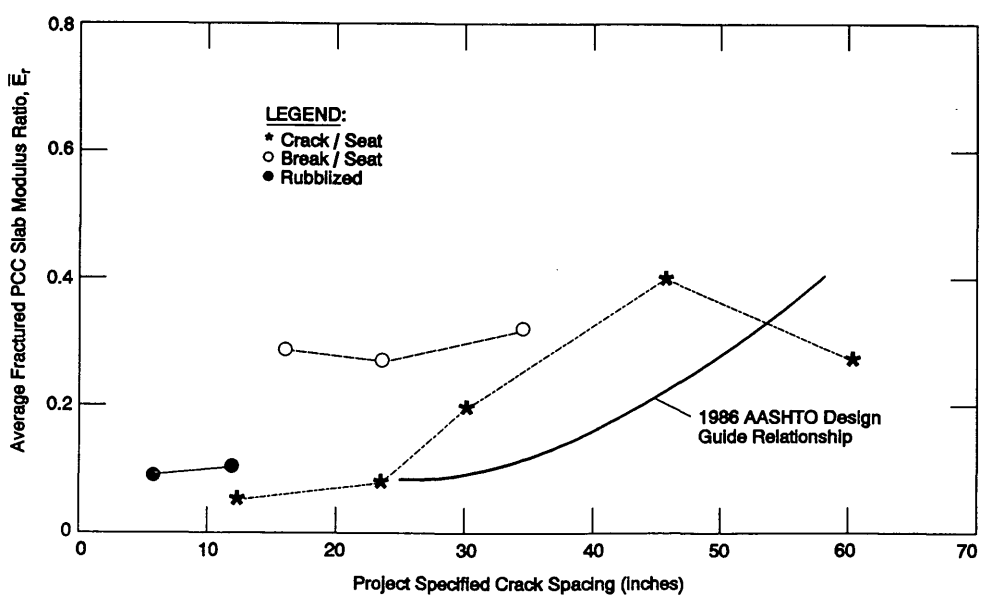

FIGURE 2 Relationship of average PCC modulus ratio versus specified project crack spacing for fractured slab techniques.

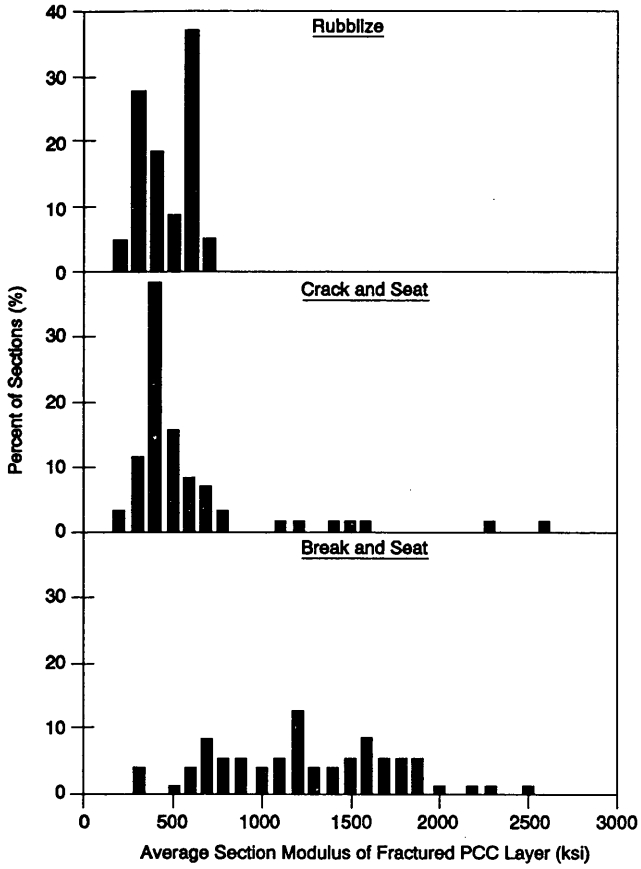

FIGURE 3 Frequency distribution of in situ PCC modulus values by rehabilitation values.

the magnitude of this variation within a given site reflects the ability of the contractor to develop a uniform fractured slab "product" after cracking, breaking, or rubblization has taken place.

To determine whether meaningful trends in the magnitude of the within project variation were evident, studies examining both the standard deviation (σ_w) and coefficient of variation (CV_w) were undertaken. The results indicated that a wide range of σ_w values existed for all three rehabilitation types and that no single value of the σ_w was typical for a particular rehabilitation activity.

The results comparing the within project standard deviation (σ_w) to the average E_{PCC} (project) modulus value for all of the fractured slab project data are shown in Figure 4. The line passing through the origin represents the average best-fit relationship to the data points. In turn, the slope of this line represents the average coefficient of variation (CV_w) for the within project variation.

As observed, a typical coefficient of variation of CV_w = 40 percent may be viewed as appropriate for all construction projects studied. Although not shown, the distribution of these values was also normally distributed for all three rehabilitation types. These important findings gave way to defining guidelines for project construction uniformity for all fractured slab techniques, regardless of the actual average E_{PCC} value achieved at the project site.

Recommended construction control categories for various levels of project uniformity are given below:

Category	*Range*
Good to excellent	$CV_w < 30$ percent
Fair to good	30 percent $\leq CV_w \leq$ 50 percent
Poor to fair	$CV_w > 50$ percent

For the "good to excellent" and "poor to fair" categories, the percentage of the projects evaluated was approximately 22 percent. The "fair to good" CV_w values contained about 56 percent of all computed within project CV_w values found.

SUMMARY AND CONCLUSIONS

This paper presented the results of a nationwide study on three new and innovative methodologies for rehabilitation of PCC pavement involving the fracturing of the slabs before the placement of an AC overlay. The major objective was to develop guidelines to eliminate or minimize the occurrence of reflective cracks in the overlay. The specific techniques evaluated were rubblization, crack and seat, and break and seat.

On the basis of the results of this study, the following major observations were made:

- The relative ranking of the fracturing techniques, in order of decreasing performance life, appears to be rubblization (best), crack/seat, and break/seat (worst). Reasonable PCI predictive models were developed for the first two techniques.

TABLE 4 Summary of Between Project E_{PCC} Statistics

Type of Rehab	No. of Sections	Between Project Results			Remarks
		E_{PCC}	σ_b	CV_b(%)	
Rubblized	22	412.5 ksi	154.4 ksi	37.4%	Recommended (excludes 2 outliers)
	24	501.8 ksi	338.9 ksi	67.5%	All data
Crack/Seat	46	409.0 ksi	140.7 ksi	34.4%	Recommended (excludes all values greater than 1000 ksi; i.e., Crack spacings greater than 48")
	64	780.6 ksi	665.6 ksi	85.3%	All data
Break/Seat	52	1271.5 ksi	548.7 ksi	43.2%	All data; Recommended
All Fractured Slabs	120	783.4 ksi	377.4 ksi	48.2%	All Recommended results
	140	915.1 ksi	578.0 ksi	63.2%	All data

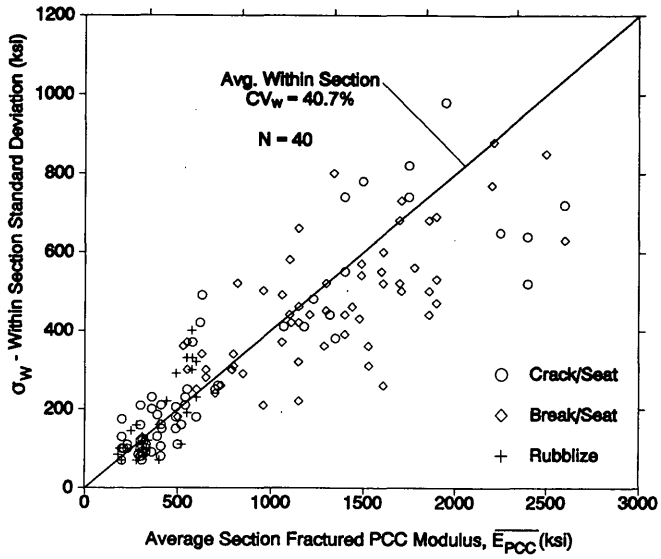

FIGURE 4 Within section variability of fractured PCC slab modulus.

- Reasonable predictive models for the fractured PCC modulus were obtained for the rubblized and crack/seat techniques. These models clearly show the importance of crack spacing and the foundation support of the existing PCC pavement. As both of these variables are increased, the E_{PCC} value of the fractured slab increases. The development of a similar model for the break and seat technique was not possible because of extreme variation in PCC values apparently due to inefficient fracturing or debonding of the distributed steel.
- Some of the most significant and important findings of the study involve the statistical frequency distributions of the effective E_{PCC} values for each rehabilitation technique. Both between project and within project variability were analyzed.
- For the crack/seat and rubblized pavement sections, the resulting frequency distributions of the project mean E_{PCC} value were quite similar: average E_{PCC} = 400 to 500 ksi and a between project coefficient of variation value of approximately 35 percent.
- In contrast, the break/seat distribution was uniformly distributed across a wide range of E_{PCC} values (i.e., 250 to 2,750 ksi). This clearly reinforces the conclusion that the break/seat process on JRC pavements is not uniformly efficient in fully debonding or fracturing the distributed steel.
- On the basis of the analysis results of the within project variability, guidelines for project uniformity were developed.

From these and other observations, the following major conclusions and recommendations were developed:

- Rubblization of deteriorating PCC pavements followed by an AC overlay is an excellent rehabilitation method that is equally effective for all types of existing PCC pavements. This technique is the preferred rehabilitation method for all types of PCC pavements.
- The crack and seat technique followed by an AC overlay is a very effective rehabilitation method for deteriorating JPC pavements. Improvements in equipment over the past years, coupled with the tendency to use smaller crack spacings, should result in improved performance. However, it is strongly recommended that the suggested minimum crack spacing guidelines presented in the paper be followed.
- The currently used construction techniques for break and seat rehabilitation of JRC pavements result in a high degree of variability in effective moduli, indicating inadequate breaking or debonding of the reinforcing steel. Until improvements are made in the breaking operations, this rehabilitation option should be used with extreme caution and coupled with field quality control measures based on deflection testing during the construction process unless local experience shows otherwise.

Finally, whereas much useful information was obtained from this initial nationwide study, additional research is required to further refine and improve the recommended guidelines and methodologies. This can only be accomplished through a combined process of data collection and periodic analysis of long-term pavement performance information. The detailed data base developed in this study can serve as the basic framework upon which additional projects can be added to expand the total number of experimental sections and performance data.

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All opinions, conclusions, and recommendations reported in this paper are strictly those of the authors and do not necessarily represent the views of NAPA or SAPAE.

Asphalt Concrete Overlay Design Methodology for Fractured Portland Cement Concrete Pavements

MATTHEW W. WITCZAK AND GONZALO R. RADA

Little technical information or guidance is presently available for engineers to properly design asphalt concrete (AC) overlays over existing PCC pavements that have been fractured to minimize or eliminate the problem of reflective cracking. Such construction techniques as crack and seat, break and seat, and rubblization have been used by the industry at an increasing rate over the last 10 years. However, most design procedures have been highly subjective, extremely conservative, and based on a lack of engineering principles related to the actual construction process used as well as an accurate assessment of the in situ physical properties of the fractured slab. To improve the state of the art and develop a better understanding of these rehabilitation techniques, a nationwide evaluation study of these rehabilitation types was conducted. The study led to the field evaluation of performance and structural in situ properties of more than 100 actual construction projects where these techniques had been used with AC overlays. On the basis of the study results, design procedures were developed for highway pavements and are presented. The design procedures are based on the flexible pavement performance methodology presented in the 1986 *AASHTO Guide*.

In many respects, the rehabilitation of pavement systems is a more complex engineering task than the design of new ones. Many factors within the rehabilitation process are beyond the current state of the art. As a result, the engineer must use a great deal of judgment in the overall design process. The engineer should also approach the rehabilitation process with the viewpoint that several technically feasible solutions may be present for any given project.

From a general viewpoint, there are several major categories of possible rehabilitation activity available to the engineer dealing with existing portland cement concrete (PCC) pavements. They include do nothing, concrete pavement restoration, PCC overlays, asphaltic concrete (AC) overlays, and reconstruction. Because the overall objective of this paper concerns the rehabilitation of rigid pavements using AC overlays, details regarding the other techniques are not addressed.

AC overlays have been used for many years to rehabilitate existing PCC pavements, but their successful design affords a more difficult challenge to the engineer compared with the rehabilitation of existing flexible pavement systems. Placing an AC overlay on an existing PCC pavement, even if significantly cracked, does not necessarily make the overlaid pavement perform in a flexible manner. Because of this, use of overlay methodologies based on flexible pavement performance models may be erroneous.

Use of flexible models for the overlay process will lead one to conclude, in the vast majority of design situations, that no structural overlay is required. Whereas this is technically true from the viewpoint of protecting the subgrade against excessive repetitive shear displacements, the use of thin AC overlays will invariably lead to the major problem of reflective cracking in a relatively short time. The design of an AC overlay over an existing PCC pavement must always take into account the potential to minimize or eliminate the reflective cracking problem.

Various AC overlay techniques used on existing PCC pavements with the primary view of the reflective crack problem include thick (conventional) overlays, crack relief layers, the saw and seal technique, special overlay and interface materials, and the fractured slab approach—rubblize, crack/seat, and break/seat. This paper focuses on the design of AC overlays for fractured PCC pavements.

At present, little technical information or guidance is available for engineers to properly design AC overlays over existing PCC pavements that have been fractured to minimize or eliminate the problem of reflective cracking. Such construction techniques as crack and seat on plain PCC pavements, break and seat on reinforced PCC pavements, and rubblization have been used at an increasing rate in the last 10 or more years. However, most design procedures have been highly subjective, extremely conservative, and based on a lack of engineering principles related to the actual construction process used as well as an accurate assessment of the in situ physical properties of the fractured slab.

To improve the state of the art and develop a better understanding, a nationwide study of these rehabilitation techniques was undertaken. The study led to the field evaluation of performance (distress) and in situ structural properties of more than 100 actual construction projects where the fracture techniques had been used with AC overlays. On the basis of the study results, design procedures were developed for highway pavements and are presented in this paper.

The design procedures are based on the use of the flexible pavement performance methodology presented in the 1986 *AASHTO Guide* (*1*). An analysis of the effective distribution of the in situ fractured PCC slab modulus results has led to the selection of realistic AASHTO layer coefficients (a_i values) in determining overlay requirements. The selection of

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the appropriate a_i value is based on reliability values established from an analysis of the expected variability within and between projects. In addition, an alternate design approach using nondestructive deflection testing (NDT) equipment for quality control/assurance (QA/QC) during the construction (fracturing) phase is presented. Finally, example problems are used to illustrate both design approaches.

INITIAL DESIGN CONSIDERATIONS

Whereas many factors influence the final rehabilitation technique selected for any given project, two of the most fundamental and important considerations are the pavement type and its existing condition. The fractured slab techniques are generally recommended for pavements in fair to failed condition—present serviceability index (PSI) $\leq$ 2.5, pavement condition index (PCI) $\leq$ 50, AASHTO structural condition factor (C_x) $\leq$ 0.78, or pavement remaining life (R_{LX}) $\leq$ 20 percent. However, the engineer should not completely rule out the potential economy of these options for pavements in fair to moderate condition. Whereas these options may be economically unfeasible for this condition category, detailed studies should be conducted within a life cycle approach to ensure that this is the case.

The process of rubblization is the only form of slab fracture recommended for all PCC pavement types: jointed plain (JPC), jointed reinforced (JRC), and continuously reinforced (CRC) concrete pavements. The crack/seat option is only applicable to JPC pavements, whereas the break/seat technique is only recommended for JRC pavements. For each technique, two alternative design approaches have been developed from the nationwide study: office and field design methods.

In the first approach, the design methodology should be viewed as an office type of design in which fairly typical values of the postfractured PCC layer are selected with a fair degree of certainty. Thus, without the precise knowledge of the effectiveness of the fracturing operation, AC overlay designs can be developed on the basis of the results of this study. In contrast, field approach designs imply the absolute need to measure the as-constructed effectiveness of the construction operation to determine the required thickness of the AC overlay. Whereas either method can be used for both the rubblization and crack/seat options, the use of only the field approach is currently recommended for the break/seat option because of the highly variable field results of the national study.

BASIC DESIGN PHILOSOPHY

The term "fractured slab techniques" relates to those rehabilitation options directly associated with the reduction of the original PCC slab lengths to smaller effective lengths, to minimize or eliminate the reflective crack problem. A fundamental relationship governing these techniques is that as crack spacing decreases, the likelihood of reflective cracking decreases. In companion papers (*2*; paper by Witczak and Rada in this Record) it has been shown that regardless of the type of rehabilitation considered, general relationships exist between the effective in situ fractured PCC modulus (E_{PCC}) and nominal fragment size. This fact has been shown by numerous other researchers as well. Because of this, the selection and use of the E_{PCC} must likewise be directly related to the probability of reflective cracking.

This consideration is shown in Figure 1. As the E_{PCC} of the pavement is decreased, the probability of obtaining reflective cracking at any thickness of AC overlay, h_{ol}, is also decreased. In addition, as the thickness of overlay is increased, at any unique value of the E_{PCC}, the probability of reflective cracking

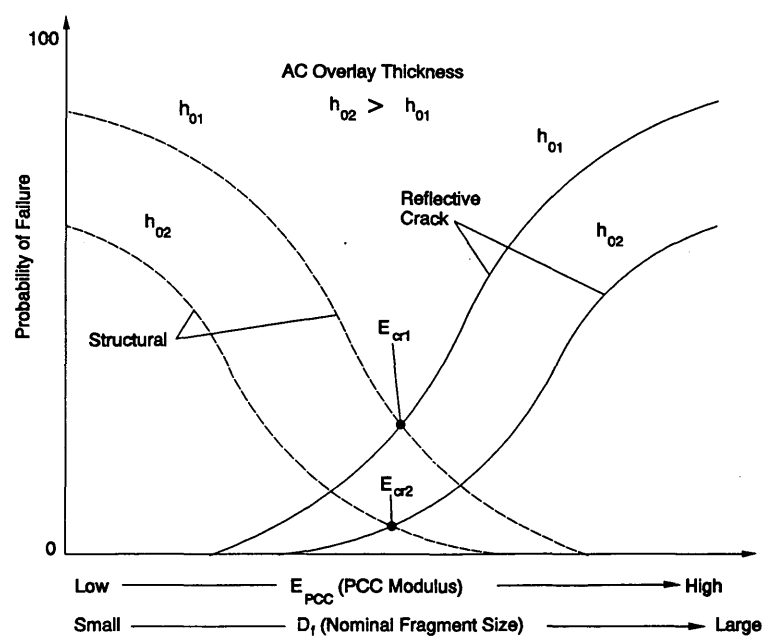

FIGURE 1 Influence of fractured PCC modulus and AC overlay thickness on structural and reflective crack failure.

must decrease. This implies that the best solution for the reflective crack problem is to ensure that the smallest possible effective slab length, nominal fragment size, and effective PCC modulus exist.

There is, however, another major consideration in the design and construction process. It should be clearly recognized that as PCC pavements are fractured, they become and act more like flexible pavement systems rather than PCC pavements designed for rigid slab action. The implication of this fact is also shown in Figure 1. An opposite relationship exists to the reflective crack problem in that as the E_{PCC} is increased, the structural capacity of the existing pavement is increased, at any thickness of overlay h_{ol}. As a result, the probability of structural failure is decreased. Likewise, at any given E_{PCC} value, as the AC overlay thickness is increased, the structural failure probability must likewise decrease.

If both of these considerations are viewed together, an important fact concerning fractured slab techniques is revealed. Figure 1 shows that at a given thickness of overlay, the intersecting point of the two relationships (reflective cracking and structural distress) yields a critical E_{PCC} value, which minimizes both possible distress modes. This critical effective modulus (E_{cr}) represents a threshold minimum modulus of the fractured slab such that the probability of both potential distress modes is the minimum possible for any given project. In the development of the design methodologies, a provisional critical modulus value of E_{cr} = 1,000 ksi was established independent of the AC overlay thickness. Furthermore, to incorporate the influence of the normal project variation, it is recommended that no more than 5 percent of the project's E_{PCC} values be greater than the E_{cr} = 1,000 ksi value.

At this point, it is important to recall the major findings and conclusions that were presented in the companion paper (*2*) regarding the importance of between and within project variation of the backcalculated E_{PCC} values found from the field NDT study. Figure 2 shows these results for the between project variability and the frequency distribution of the within project coefficient of variance. Because the within project variation is highly indicative of the project uniformity or ability of the contractor to provide a uniform product, the zones shown in Figure 2b are indicative of the three types of degree of uniformity that were found from the national study.

The combination of both variability forms must be jointly viewed to gain full appreciation of the design methodology that will be presented. In Figure 2a, the average project E_{PCC} means for two typical projects (E_{p1} and E_{p2}) are shown. For each project mean, a range of within project CV_w values may affect the actual distribution of the E_{PCC} values for any given project. For purposes of the following explanation, it is assumed that three levels of the within project variability (CV_w) exist: CV_{ge}, good to excellent control; CV_{fg}, fair to good control; and CV_{pf}, poor to fair control. The actual project E_{PCC} frequency distributions for the six possible combinations are shown in Figure 3.

For Project 1, the three frequency distributions reflecting the range of project uniformity are shown in relation to the critical E_{cr} level for reflective cracking. Because the average E_{p1} is small, the probability of any combination of within project variation exceeding the critical threshold E_{cr} value is very remote. However, if the resulting frequency distributions for the second project (E_{p2}) are observed, as the project nonuniformity is increased, a significant area for Curve 3 exceeds

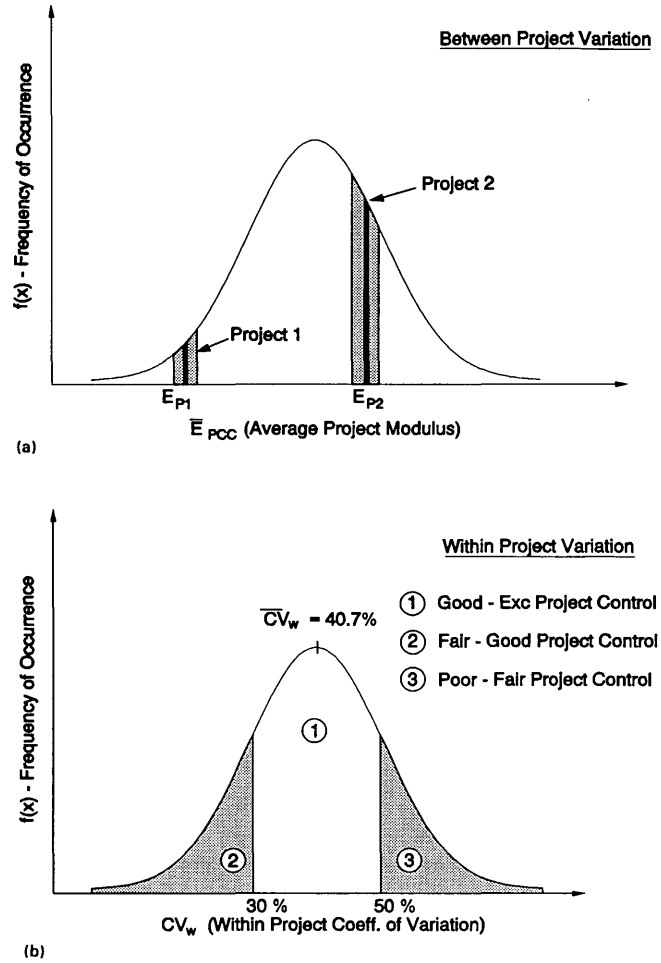

FIGURE 2 Between and within project E_{PCC} variability: (*a*) frequency distribution of average project means; (*b*) frequency distribution of within project CV value.

the E_{cr} level. It can therefore be concluded that the ability of a given project to satisfy the E_{cr} criteria is not only a function of the project average E_{PCC} value but also highly dependent on the within project variation attained in the construction process by the contractor. From a structural viewpoint, a greater thickness of AC overlay would be required for Project 1 than for Project 2 because Project 1 has a lower modulus (E_{PCC}).

Whereas the previous discussion has primarily focused on the E_{PCC} distributions and their within project variability relative to the critical E_{cr} for minimizing or eliminating reflective cracking, implications for the E_{PCC} distribution must also be considered relative to the structural overlay design. As discussed later in this paper, the overlay methodology is based on the use of the AASHTO structural number (SN) concept for flexible pavements (*1*). An important parameter in SN computations is the structural layer coefficient (a_i).

Analytically, the a_i value can be related to the elastic modulus of a material (E_i) through the following relationship:

$$a_i = a_s \sqrt[3]{\frac{E_i}{E_s}} \tag{1}$$

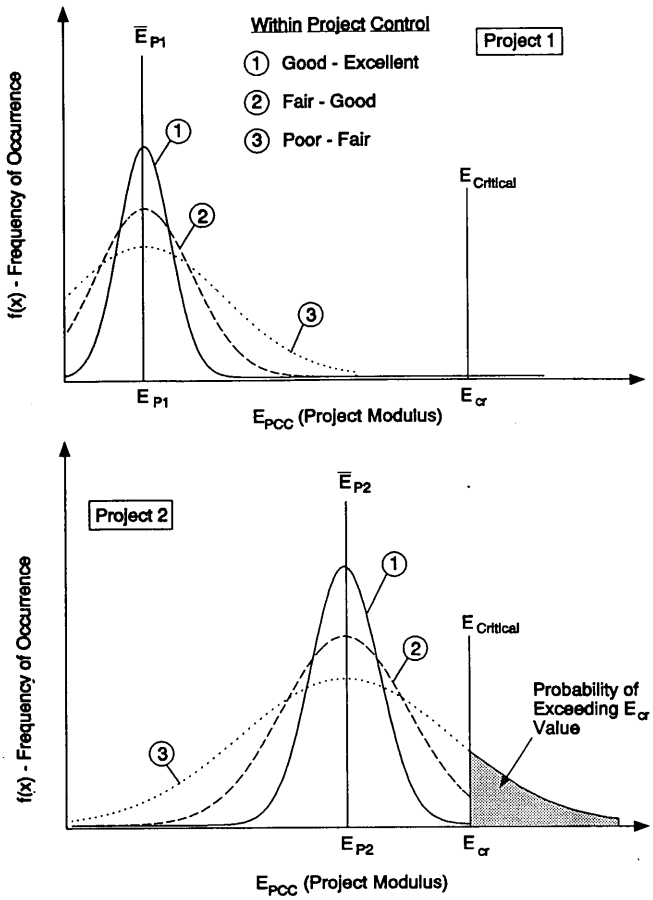

FIGURE 3 Fractured slab modulus frequency distributions reflecting between and within project variability.

where the subscript i represents the material in question and the subscript s represents an arbitrary standard material whose a_s and E_s were established for AASHO Road Test materials.

Using a dense-graded crushed stone base as the standard, it has been found that $a_s = 0.14$ and $E_s = 30{,}000$ psi. Substituting these values into Equation 1 yields

$$a_i = 0.0045\sqrt[3]{E_i} \tag{2}$$

Thus, a direct transformation between the in situ fractured modulus (E_i or E_{PCC}) and the AASHTO layer coefficients (a_i) for the fractured material can be easily made.

Figure 4 shows a typical frequency distribution of the a_i value for a given project, which was developed using the E_i to a_i transformation. Also shown on Figure 4 are two separate E_i (a_i) values. The first (E_{cr} or a_{cr}) has been fully discussed as the critical E_{PCC} for reflective crack control. The second value (a_d) represents a design value selected by the engineer for the structural overlay process; the area under the curve that is less than this value (i.e., a_d) represents the probability of structural failure for the project. Clearly, as the engineer desires a higher design reliability, a smaller value of a_d must be selected. This, in turn, will result in thicker AC overlays being required as the reliability level is increased. It can therefore be concluded that within project variability is a very significant parameter influencing both the probability of reflective cracking and the probability of structural failure.

GENERAL OVERLAY DESIGN PRINCIPLES

All three rehabilitation options within the fractured slab category behave more like flexible systems than rigid ones. The

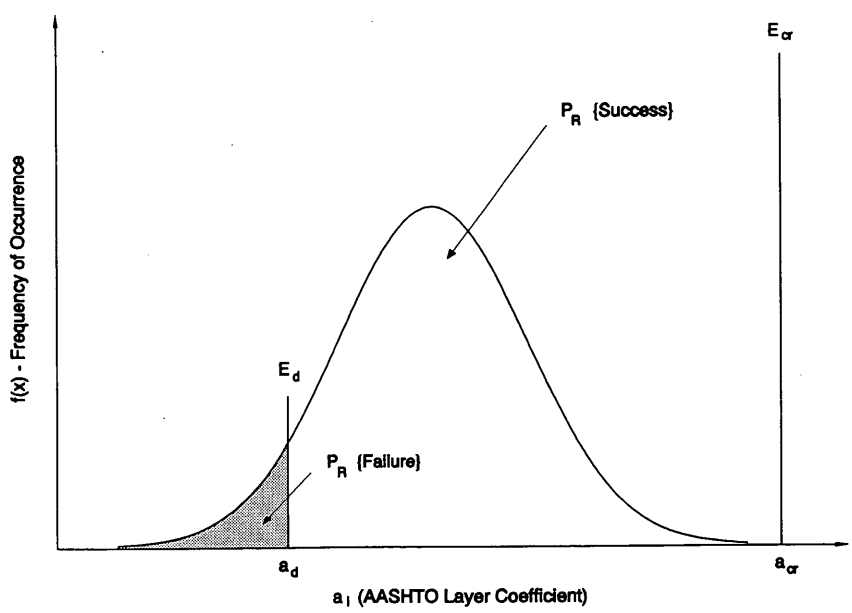

FIGURE 4 Frequency distribution of the AASHTO layer coefficient for fractured PCC slabs.

classical flexible pavement performance models are therefore more applicable and accurate as the basis of any overlay methodology. Another important consideration is the fact that the fractured slab process turns the existing rigid pavements into new flexible pavements. The new pavement, in turn, can be viewed as being the AC overlay equivalent of the placement of an AC surface course on new construction. Because of this, it is believed that the use of the remaining life factor (F_{RL}) present in the AASHTO guide (*1*) is not applicable to the fractured slab process (i.e., $F_{RL} = 1$).

The AC overlay methodology on which the rehabilitation of fractured slabs is presented is based on the well-known and widely used structural capacity deficiency approach. The AASHTO guide (*1*) flexible performance models using the SN approach are used as the equivalent parameter of the structural capacity. Thus, the general overlay equation is based on the simple expression

$$SN_{ol} = SN_y - SN_{xeff} \tag{3}$$

where

SN_y = future structural capacity required of a new flexible pavement constructed over the existing subgrade to accommodate the traffic within the life of the overlay,
SN_{xeff} = effective capacity of the existing pavement structure after fracturing has taken place, and
SN_{ol} = additional structural capacity that will be required from the AC overlay.

Recognizing that

$$SN_{ol} = a_{ol}h_o \tag{4}$$

and using the commonly accepted $a_{ol} = 0.44$ for AC, the required overlay thickness can be expressed by

$$h_o = \frac{SN_y - SN_{xeff}}{0.44} \tag{5}$$

The solution of the h_o value involves the solution of the two variables: SN_y and SN_{xeff}. The solution of SN_y is very direct because it is based solely on the AASHTO guide solution for new flexible pavements (*1*). The computation of the SN_{xeff} value should incorporate not only the fractured slab but also any subbase layers present in the existing pavement. Thus

$$SN_{xeff} = a_d D_o + a_{sb}h_{sb} \tag{6}$$

where

a_d = design layer coefficient of the fractured PCC layer,
a_{sb} = layer coefficient of any existing subbase layer material,
D_o = original thickness of the PCC slab, and
h_{sb} = subbase layer thickness.

The reader is referred to the AASHTO guide (*1*) for further details regarding the selection of the appropriate a_{sb} values for a variety of materials that may be present. Because layer thicknesses (D_o and h_{sb}) can usually be found from historic construction data or obtained from drilling/coring operations, the most significant factor to be determined is the a_d value for the fractured slab.

STRUCTURAL DESIGN LAYER COEFFICIENT, a_d

The selection of the appropriate a_d value is a critical and sensitive part of the overlay analysis. This parameter relates to the structural failure of the overlaid pavement, and thus it is necessary to apply design conservatism to the design process. However, the within project variability (CV_w) also plays a key role in the selection of a_d in that the optimum construction process should yield an average E_{PCC} as large as possible, with as low a CV_w value as possible, to ensure that the E_{cr} level is met.

As with any design analysis, the engineer must select a typical design value in the absence of site-specific data. This "office design" must obviously be based on a relatively conservative approach which, in turn, is based on a high degree of reliability. This classical engineering design approach is referred to as the office design method in this paper.

An alternative approach is to use a methodology based on the site-specific construction information obtained through deflection testing. This information, when analyzed, can serve as a QA/QC measure and provide actual in situ response for the fractured slab process to develop dynamic design values. Whereas there may be practical restraints on the implementation of this design approach, the potential for saving considerable money in the rehabilitation process should not be overlooked by the design engineer. This approach is referred to as the field design method in this paper. The following sections define each of these recommended design methods in further detail.

Office Design Method

The office design method represents the development of a typical a_d value for design of pavements without site-specific information. In determining this value, the results of all E_{PCC} values obtained in this national study, for a particular type of fractured slab analysis, were used. The procedure described is based on not only the between project variability but the within project variability as well. The overall design reliability (R) associated with the particular a_d found from the ensuing analysis is related to the joint probabilities associated with each frequency distribution.

Using the principle of normal probability, the between project distribution of the E_{PCC} value can be characterized by

$$E_p = \overline{E_{PCC}} - k_{\alpha b}\sigma_b \tag{7}$$

where

E_p = average E_{PCC} for a given project,
E_{PCC} = average of all project means,
$k_{\alpha b}$ = standardized normal deviate associated with between project probability of failure (α_b), and
σ_b = between project standard deviation.

Likewise, the following relationship exists for the within project distribution:

$$E_d = E_p - k_{\alpha w}\sigma_w \quad (8)$$

where

E_d = design E_{PCC} for a given project,
$k_{\alpha w}$ = standardized normal deviate associated with the within project probability of failure (α_w), and
σ_w = within project standard deviation.

For the within project variability, the coefficient of variation (CV_w) was a constant value regardless of the E_p value (*2*). Therefore

$$CV_w = \frac{\sigma_w}{E_p} \quad (9a)$$

or

$$\sigma_w = CV_w E_p \quad (9b)$$

Substituting Equation 9b into Equation 8 yields

$$E_d = (\overline{E_{PCC}} - k_{\alpha b}\sigma_b)(1 - k_{\alpha w}CV_w) \quad (10)$$

The value of E_d represents the design value of the fractured slab technique existing at an overall design reliability, R, defined by

$$R = 1 - (\alpha_b)(\alpha_w) \quad (11)$$

For each specific fractured slab process, the E_d value can be computed from three E_{PCC} distribution parameters: E_{PCC}, α_b, and CV_w at any desired reliability level, R. In turn, the design layer coefficient (a_d) can be computed from E_d by means of Equation 2. Therefore, it is possible to develop relationships of a_d as a function of the overall reliability for the results of this study.

Table 1 summarizes the key between and within project statistics found for the fracture techniques; the reader is referred to the companion paper for a more detailed discussion of these parameters. Using these statistics as input into the equations presented earlier, E_d and a_d values were developed as a function of the design reliability for rubblize and crack/seat projects. Table 2 summarizes these computations. A comparison of these results indicates that the a_d values are practically identical for the rubblize and crack/seat techniques. Accordingly, the final recommended a_d relationship is shown in Figure 5.

For typical values of design reliability encountered in practice, a value of a_d = 0.28 is recommended. This is equivalent to a reliability value slightly in excess of 90 percent. However, the engineer must use judgment in selecting the appropriate design reliability level for any given project; as the relative importance of the project increases, a higher R value and hence lower a_d value may be selected.

Whereas Table 1 also summarizes the key project statistics for the break/seat projects, the office design method is not recommended for this rehabilitation technique—the analysis of both performance data and in situ structural properties obtained from the field study indicates that a wide range of breaking efficiency actually occurs. This finding strongly sup-

TABLE 1 Summary of E_{PCC} Statistics

BETWEEN PROJECT VARIABILITY

Type of Rehab	No. of Sections	Between Project Results			Remarks
		E_{PCC}	σ_b	CV_b(%)	
Rubblized	22	412.5 ksi	154.4 ksi	37.4%	Recommended (excludes 2 outliers)
	24	501.8 ksi	338.9 ksi	67.5%	All data
Crack/Seat	46	409.0 ksi	140.7 ksi	34.4%	Recommended (excludes all values greater than 1000 ksi i.e., Crack spacings 48" or greater)
	64	780.6 ksi	665.6 ksi	85.3%	All data
Break/Seat	52	1271.5 ksi	548.7 ksi	43.2%	All data; Recommended
Combined	120	783.4 ksi	377.4 ksi	48.2%	All Recommended results
	140	915.1 ksi	578.0 ksi	63.2%	All data

WITHIN PROJECT VARIABILITY

Type of Rehab	No. of Sections	Within Project Results			Remarks
		CV_w	σ_{CVw}	CV_{CVw}	
Rubblized	24	44.4%	12.9%	29.1%	All data
Crack/Seat	64	41.2%	12.8%	31.0%	All data
Break/Seat	52	38.4%	12.6%	32.7%	All data
Combined	140	40.7%	12.7%	31.3%	All data

TABLE 2 Computational Summary for Reliability-Based a_d Values

RUBBLIZED PROJECTS

Between Project			Within Project			Design Values			
α_b	$K_{\alpha b}$	$E_p = E_{PCC} - K_{\alpha b}\sigma_b$	α_w	$K_{\alpha w}$	CV_w, %	E_d	a_d	LE_d	R, %
0.50	0.000	412.5	0.50	0.000	44.4	412.5	.335	2.39	75.00
0.25	0.671	308.9	0.25	0.671	44.4	216.8	.270	1.93	93.75
0.15	1.036	252.5	0.15	1.036	44.4	136.3	.232	1.66	97.75
0.10	1.282	214.6	0.10	1.282	44.4	92.4	.203	1.45	99.00
0.05	1.645	158.5	0.05	1.645	44.4	42.7	.157	1.12	99.75

E_{PCC} = 412.5 ksi; σ_b = 154.4 ksi; CV_w = 44.4%

CRACK/SEAT PROJECTS

Between Project			Within Project			Design Values			
α_b	$K_{\alpha b}$	$E_p = E_{PCC} - K_{\alpha b}\sigma_b$	α_w	$K_{\alpha w}$	CV_w, %	E_d	a_d	LE_d	R, %
0.50	0.000	409.0	0.50	0.000	41.2	409.0	.334	2.38	75.00
0.25	0.671	314.6	0.25	0.671	41.2	227.6	.275	1.96	93.75
0.15	1.036	263.2	0.15	1.036	41.2	150.9	.239	1.71	97.75
0.10	1.282	228.6	0.10	1.282	41.2	107.9	.214	1.53	99.00
0.05	1.645	177.5	0.05	1.645	41.2	57.2	.173	1.24	99.75

E_{PCC} = 409.0 ksi; σ_b = 140.7 ksi; CV_w = 41.2%

ports the fact that this approach may yield highly variable and uncertain performance. Because of this, extreme care must be exercised during construction to ensure that a minimum effective PCC modulus of the fractured slab occurs. Without such field verification, the technique of rubblization is currently recommended rather than break/seat pending future studies. Alternatively, where states have had successful prior experience with break/seat techniques, this process should be viewed as a viable rehabilitation approach.

Field Design Method

The field design approach is based on the use of deflection basin data collected during the construction operation to en-

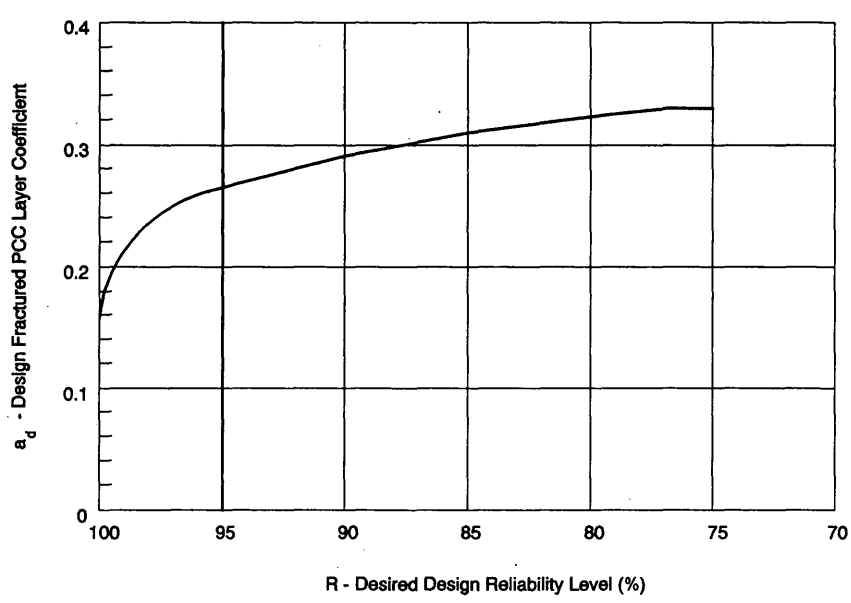

FIGURE 5 Recommended AASHTO structural layer coefficient for rubblized and crack/seat PCC layers as a function of desired reliability.

sure that both design criteria (a_d and E_{cr}) are met in the fracturing process before placement of the overlay. Though this approach may have some initial practical implementation problems, its use on all future fractured slab projects is highly recommended because of the significant advantages that can be gained in the design and construction process. The major advantages include the potential for increased project uniformity, better future pavement performance and life, significant cost savings in the initial overlay construction, and a better procedure to more accurately assess whether the slab fracturing process is as efficient as desired (i.e., compared with either visual crack studies or limited coring). This methodology is the same for all types of fractured slab rehabilitation options and is applicable to all types of existing rigid pavements (JPC, JRC, and CRC).

The general approach to the design method is as follows. Immediately after the contractor has completed his initial round of "slab fracture" on a defined section, deflection readings should be taken on at least 30 random points within the section limits. The deflection basin data should then be used to calculate the in situ E_{PCC} value for each test point, the project average E_{PCC} value, and the within project standard deviation, σ_p.

Next, check to see whether the E_{cr} has been met. This is done by finding

$$K_\alpha = \frac{(1{,}000 - \overline{E_{PCC}})}{\sigma_p} \tag{12}$$

Using this value as input into the normal probability table contained in most statistical and probability textbooks, the α value or probability of exceeding the E_{cr} = 1,000 ksi criterion can be determined. If the computed α value is greater than 5.0 percent, the E_{cr} criterion has not been met, and the contractor should be instructed to refracture the area. If this is done, the sequence goes back to the beginning.

On satisfying the E_{cr} criterion, the next step is to check the design a_d value. Using the normal probability table, a value of K_β can be selected for any given design level of reliability (e.g., K_β = 1.037 for R = 85 percent). The field-derived a_{df} value can be then computed from

$$a_{df} = 0.0045 \sqrt[3]{(\overline{E_{PCC}} - K_\beta \sigma_p)} = 0.0045 \sqrt[3]{E_d} \tag{13}$$

The final step deals with the comparison of the "field" a_{df} value with the "office" a_{do} value used to establish the preliminary AC overlay design thickness. If $a_{df} > a_{do}$, the engineer has two options. First, because this condition is conservative relative to the original AC design, a "do nothing" option may be selected. However, it is possible to compute the possible reduction in AC overlay thickness (Δh_o) that may be implemented directly in the field. This is accomplished by

$$\Delta h_o = \frac{(a_{do} - a_{df})}{0.44} \tag{14}$$

If Δh_o is greater than 1.0 in. or more, every consideration should be given to adjusting the initial design recommendation of h_o (overlay thickness) by the Δh_o value. Conversely, if $a_{df} < a_{do}$, the Δh_o equation can be used to determine how much more overlay would be necessary for the actual fractured conditions achieved in the field.

EXAMPLE PROBLEMS

Example 1

An example of the rubblized overlay rehabilitation option is presented to summarize the design methodology recommended. For this example project, an existing JPC, with existing joint spacing of 20 ft, has PSI = 2.1. More than 25 percent of the slabs exhibited extensive cracking indicating fair to poor pavement condition. The existing PCC pavement is 9.0 in. thick and has a subbase (unbound) of 6.0 in. The AASHTO layer coefficient for the subbase has been found to be a_{sb} = 0.09. The use of the AASHTO new flexible pavement performance model for the overlay life and traffic has indicated that a SN_y = 4.82 will be required. An office design solution is desired for a rubblized AC overlay.

From the problem description, the following values are known: SN_y = 4.82, a_{ol} = 0.44, D_o = 9.0, a_d = f(reliability level, R), and h_{sb} = 6.0. Substituting these inputs into Equation 5 yields

$$h_o = 9.73 - 20.45 a_d$$

Because a_d is a function of the design reliability level, the solution of h_o is presented in Table 3 for several levels of R as well as the recommended values of a_d = 0.28. It can be observed that the design h_o is affected by the selection of the desired R value. For typical reliability levels between 85 and 95 percent, the overlay thickness requirements vary between 3.5 and 4.5 in. The typical recommended value of a_d = 0.28 results in a design h_o = 4.0 in.

TABLE 3 Required Overlay Thickness as a Function of Reliability Level—Example Problem 1

Reliability Level	Layer Coefficient, a	Overlay Thickness, h (inches)
75%	0.34	2.8
85%	0.30	3.6
90%	0.29	3.8
95%	0.26	4.4
99%	0.20	5.6
(Recommended)	0.28	4.0

Example 2

The following example is based on the field design method applied to a break/seat project on an existing JRC pavement. The pavement to be rehabilitated has a 40.0-ft joint spacing and is 10.0 in. thick. It rests on a 4-in. cement-treated base having an AASHTO layer coefficient of a_{sb} = 0.15. The PSI of the pavement is 2.3, and it is in fair to poor condition. The required future structural capacity needed in the overlay period has been found to be SN_y = 5.95. Because the facility receives heavy traffic, the engineer has selected a design reliability of 95 percent for the project. For the preliminary design, an h_o value of 5.0 in. was selected by the design team on the basis of experience.

After the contractor conducted a preliminary breaking of a given section of the project, NDT testing was used to determine the statistics associated with the E_{PCC} values. They were E_{PCC} = 1196 ksi, σ_w = 385 ksi, and CV_w = 32.2 percent.

These results indicate that the "broken" section does not satisfy the E_{cr} criterion of having less than 5 percent of the E_{PCC} values exceed the threshold limit of 1,000 ksi because the average E_{PCC} is much greater than the threshold.

The contractor was then instructed to conduct further breaking. The NDT backcalculated E_{PCC} statistics were E_{PCC} = 526 ksi, σ_w = 129 ksi, and CV_w = 24.5 percent. For the criterion of α = 5 percent for the E_{cr} limit, the value of $K_{\alpha w}$ = 1.645 is found from the normal distribution table. Thus, the upper limit of the actual E_{PCC} distribution at a 5 percent level is given by

$$E_u = E_{PCC} + K_\alpha \sigma_w = 526 + 1.645(129) = 738 \text{ ksi}$$

Therefore, the pavement meets the E_{cr} reflective crack criterion and the actual field a_{df} value can be now determined from the E_d value:

$$E_d = E_{PCC} - K_\alpha \sigma_w = 526 - 1.645(129) = 313.8 \text{ ksi}$$

and

$$a_{df} = 0.0045E_d^{0.333} = 0.0045(313{,}800)^{0.333} = 0.31$$

Once the a_{df} value has been established, the required overlay thickness check can be performed:

$$h_o = \frac{SN_y - (a_{df}D_o + a_{sb}h_{sb})}{a_{ol}}$$

Thus, the actual broken JRC pavement would require an overlay of h_o = 5.1 in. Because the preliminary design was based on h_o = 5.0 in., no modification (either + or $-\Delta h_o$ adjustment) is required for the final design cross section.

SUMMARY AND CONCLUSIONS

In this paper, AC overlay design procedures for fractured PCC pavements were presented. These procedures were developed from the results of a nationwide evaluation study and are based on the use of the AASHTO flexible pavement performance methodology.

The basic design philosophy is that as fractured slab fragments become smaller, the E_{PCC} value becomes less. This has two important ramifications. To minimize or eliminate reflective cracking, it is desirable to have the effective E_{PCC} value as small as possible. However, in so doing, the strength of this fractured layer decreases, which in turn requires a thicker overlay. As a consequence, the overall philosophy of the fracture techniques should be to obtain as large an in situ E_{PCC} value possible to minimize the required overlay thickness but ensure that there is a small probability of having within project E_{PCC} values exceed a certain upper or critical value (E_{cr}).

In development of the overlay methodologies, reliability levels of 95 percent have been used as the basis for the recommendations. In addition, the critical level of E_{PCC} to ensure that reflective cracking will not occur has been provisionally selected to be E_c = 1,000 ksi.

Two design approaches were presented in the paper: office and field design methods. The office design approach was based on the selection of a conservative estimate of the AASHTO structural layer coefficient or a_i value to be used for each rehabilitation technique. Information obtained from the between and within E_{PCC} variability studies was used to determine appropriate levels of a_i as a function of the desired design reliability for the rehabilitation.

The second approach, the field method, is predicated on the use of nondestructive deflection testing at the construction site to monitor and control the final design thickness. At a given project site, the deflection test results are used to determine the in situ frequency distribution of the backcalculated E_{PCC} values. This distribution is checked to ensure that no more than 5 percent of the E_{PCC} results exceed the critical 1,000 ksi upper limit value. Once this criterion is satisfied, the actual project E_{PCC} distribution is then used to determine the final design project a_i value so that the final AC overlay thickness can be determined.

Whereas either design approach can be used for both the rubblization and crack/seat options, the use of only the field approach is recommended for the break/seat option.

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All opinions, conclusions, and recommendations reported in this paper are strictly those of the authors and do not necessarily represent the views of NAPA or SAPAE.

Revision of AASHTO Pavement Overlay Design Procedures

KATHLEEN T. HALL, MICHAEL I. DARTER, AND ROBERT P. ELLIOTT

The AASHTO overlay design procedures have been extensively revised to make them easier to understand and use, more adaptable to calibration by local agencies, and more comprehensive. The revised overlay design procedures described use the concepts of structural deficiency and required future structural capacity determined from the AASHTO flexible and rigid pavement design equations. The procedures provide detailed guidelines on several important topics related to overlay design, including overlay feasibility, structural versus functional overlay needs, preoverlay repair, reflection crack control, and overlay design reliability level. Detailed guidelines were also developed for pavement evaluation for overlay design, including distress surveying, nondestructive testing, and destructive testing (coring and materials testing). Seven separate overlay design procedures were developed, encompassing all of the combinations of overlay and pavement types. Each of the design procedures follows a sequence of eight steps, by which the required future structural capacity for the design traffic, effective structural capacity of the existing pavement, and required overlay thickness are determined.

Chapter 5 of Part III of the 1986 AASHTO *Guide for Design of Pavement Structures* (*1*) addresses overlay design. These overlay design procedures were recently revised to make them easier to use, more adaptable to calibration by local agencies, and more comprehensive (*2–4*). The proposed procedures are currently under consideration by AASHTO.

A complete, step-by-step overlay design procedure was developed for the following combinations of pavement and overlay type:

Overlay	*Existing Pavement*
AC	AC
AC	Crack/seat, break/seat, and rubblized PCC
AC	JPCP, JRCP, and CRCP
AC	AC/JPCP, AC/JRCP, and AC/CRCP
Bonded PCC	JPCP, JRCP, and CRCP
Unbonded PCC	JPCP, JRCP, and CRCP
PCC	AC

Guidelines were also provided for overlay feasibility, preoverlay repair, reflection crack control, overlay design reliability level, and several other important considerations in overlay design. Detailed guidelines were developed for pavement evaluation for overlay design, including distress surveying, nondestructive deflection testing (NDT), and destructive testing.

The revised AASHTO overlay design procedures use the structural deficiency approach, in which the effective structural capacity of the existing pavement is subtracted from the required future structural capacity as determined from the AASHTO flexible and rigid pavement design equations. This concept was retained to maintain compatibility between Parts II and III of the guide and to keep the procedure relatively simple. Development of a more sophisticated mechanistic approach to overlay design was not within the scope of the revisions. NDT is recommended for characterization of the existing pavement, to the extent appropriate within the framework of these empirical design procedures.

Three approaches were developed for characterizing the effective structural capacity of existing pavement (SN_{eff}, D_{eff}). Not all three approaches are appropriate for all pavement types. The approaches are

1. Visual condition survey and materials testing,
2. NDT (where appropriate), and
3. Remaining life (where appropriate).

This paper presents an overview of the revised AASHTO overlay design procedures. The procedures are presented in detail by Darter et al. (*2*). The development of the procedures is documented by Darter et al. (*3*). In addition, the revised procedures were extensively tested using data from many actual in-service pavements located throughout the United States. The results of this field testing are provided by Darter et al. (*4*).

IMPORTANT CONSIDERATIONS IN OVERLAY DESIGN

Overlay design requires consideration of many important items in addition to required overlay thickness. Each of these is very briefly discussed in this section and is addressed in more detail elsewhere (*2*).

Overlay feasibility: The feasibility of any type of overlay depends on availability of adequate funds, construction feasibility (including lane closure restrictions, materials and equipment availability, overhead clearances, and other factors), and the required future performance life of the overlay.

Preoverlay repair: Much of the deterioration that occurs in overlays results from deterioration that was not repaired in the existing pavements. The amount of preoverlay repair needed is related to the type of overlay selected. If the existing pavement is severely deteriorated, selecting an overlay type that is less sensitive to existing pavement condition may be more cost-effective than doing extensive preoverlay repair.

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Reflection crack control: The revised AASHTO overlay design procedures do not consider reflection cracking in the overlay thickness design. Additional steps must be taken to reduce the occurrence and severity of reflection cracking.

Traffic loadings: The 18-kip equivalent single-axle loads (ESALs) expected in the design lane over the design life of the overlay must be calculated using the appropriate flexible pavement or rigid pavement load equivalency factors from Part II of the guide. Failure to use the correct type of ESALs will result in a significant error in the overlay design.

Subdrainage: Existing subdrainage conditions usually have a great influence on how well an overlay performs. A subdrainage evaluation should be conducted as described in Part III of the guide.

Rutting in AC pavements: The cause of rutting in an existing AC pavement must be determined before an AC overlay is designed. An overlay may not be appropriate if severe rutting is occurring because of instability in any of the existing pavement layers.

Milling AC surface: Removal of a portion of an existing AC surface frequently improves the performance of an AC overlay. Significant rutting or other major distortion should be removed by milling before another overlay is placed.

Recycling the existing pavement: Recycling a portion of an existing AC layer may be considered as an option in the design of an overlay. Complete recycling of the AC layer or recycling of a PCC slab necessitates designing the reconstructed pavement according to procedures for new pavement design.

Structural versus functional overlays: The revised AASHTO overlay design procedures provide an overlay thickness to correct a structural deficiency. If no structural deficiency exists, an overlay thickness less than or equal to zero will be obtained. This does not mean, however, that the pavement does not need an overlay to correct a functional deficiency.

Shoulders: If an existing shoulder is in good condition, the shoulder may be overlaid to match the grade of the traffic lanes, after patching of deteriorated areas on the shoulder. If an existing shoulder is in such poor condition that it cannot be patched economically, it should be removed and replaced.

Existing PCC slab durability: The durability of an existing PCC slab greatly influences the performance of AC and bonded PCC overlays. If D cracking or reactive aggregate distress exists, the deterioration of the existing slab can be expected to continue after overlay.

PCC overlay joints: Bonded or unbonded jointed concrete overlays require special jointed design that considers the characteristics of the underlying pavement. Factors to be considered in overlay joint design include joint spacing, saw cut depth, sealant reservoir shape, and load transfer requirements.

PCC overlay reinforcement: Jointed reinforced and continuously reinforced concrete overlays require an adequate amount of reinforcement to hold cracks together. Friction between the overlay slab and the base slab should be considered in the reinforcement design.

PCC overlay bonding/separation layers: Bonded overlays must be constructed to ensure that the overlay remains bonded to the existing slab. Unbonded overlays must be constructed to ensure that the separation layer prevents reflection cracks in the overlay.

Overlay design reliability level and overall standard deviation: An overlay may be designed for different levels of reliability using the procedures described in Part I of the guide for new pavements. This is done by determining the structural capacity (SN_f or D_f) required to carry traffic over the design period at the desired level of reliability.

Reliability level has a large effect on overlay thickness. Varying the reliability level used to determine SN_f or D_f between 50 and 99 percent may produce overlay thicknesses varying by 6 in. or more (*4*). On the basis of field testing, it appears that a design reliability level of approximately 95 percent gives overlay thicknesses consistent with those recommended for most projects by state highway agencies when the overall standard deviations recommended in Part I and II are used (*4*). There are, of course, many situations for which it is desirable to design at a higher or lower level of reliability, depending on the consequences of failure of the overlay. The reliability level to be used for different overlay types may vary and should be evaluated by each agency for different highway functional classifications or traffic volumes.

The designer should be aware that some sources of uncertainty are different for overlay design than for new pavement design. Therefore, the overall standard deviations recommended for new pavement design may not be appropriate for overlay design. The appropriate value for overall standard deviation may vary by overlay type as well. An additional source of variation is the uncertainty associated with establishing the effective structural capacity (SN_{eff} or D_{eff}) of the existing pavement. However, some sources of variation may be less significant for overlay design than for new pavement (e.g., estimation of future traffic).

Pavement widening: Many AC overlays are placed over PCC pavements in conjunction with pavement widening (either adding lanes or adding width to a narrow lane). This situation requires coordination between the design of the widened pavement section and the overlay so that both the existing and the widening sections will be structurally and functionally adequate.

PAVEMENT EVALUATION FOR OVERLAY DESIGN

It is important that an evaluation of the existing pavement be conducted to identify any functional or structural deficiencies and to select appropriate preoverlay repair, reflection crack treatments, and overlay designs to correct these deficiencies.

Figure 1 shows the concepts of structural deficiency and effective structural capacity. The structural capacity of a pavement when new is denoted SC_o. For flexible pavements, structural capacity is the structural number, SN. For right pavements, structural capacity is the slab thickness, D. For existing composite pavements (AC/PCC), the structural capacity is expressed as an equivalent slab thickness.

The structural capacity of the pavement declines with time and traffic. At any time that an evaluation is done for the purpose of overlay design, the structural capacity has decreased to SC_{eff}. The effective structural capacity is expressed by SN_{eff} for flexible pavements and by D_{eff} for rigid and composite pavements.

If a structural capacity of SC_f is required for the future traffic expected during the overlay design period, an overlay with a structural capacity of SC_{ol} (where $SC_f - SC_{eff} = SC_{ol}$)

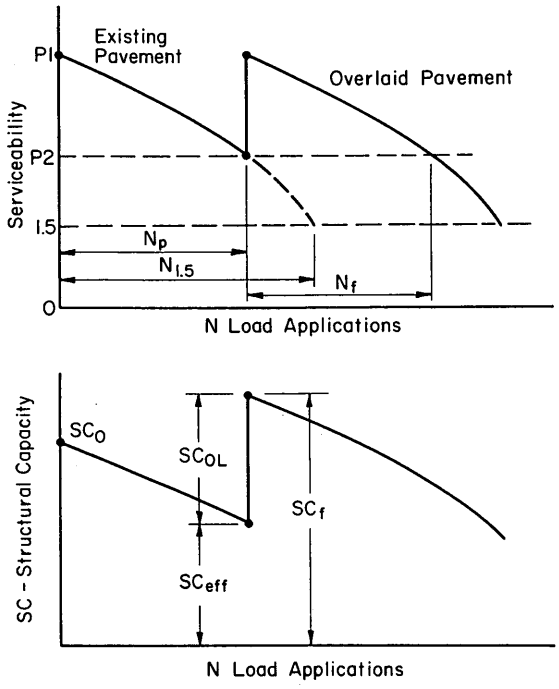

FIGURE 1 Structural capacity loss over time and with traffic.

must be added to the existing pavement structure. Obviously, the required overlay structural capacity can be correct only if the required future structural capacity and the assessment of the existing structural capacity are correct. The primary objective of the structural evaluation is to determine the effective structural capacity of the existing pavement. Three methods are described for determining effective structural capacity.

Structural Capacity Based on Visual Survey and Materials Testing

A key component in determination of effective structural capacity is observation of existing pavement conditions. In addition to information on the pavement's original design, construction, and maintenance history, information on the pavement's current condition must be obtained. A distress survey should be conducted to identify the type, amount, severity, and location of distresses present. The key distress types for each pavement type that should be considered in determining the effective structural capacity are described elsewhere (*2*). Recommendations for preoverlay repair for each overlay type are given elsewhere (*2*).

A drainage survey should be coupled with the distress survey. The objective of the drainage survey is to identify moisture-related problems and locations where drainage improvements might be effective in reducing the influence of moisture on the performance of the pavement after overlay.

A coring and testing program should be coordinated with the distress survey to verify layer thicknesses, obtain material samples for testing, and investigate the causes of the observed distress. Coring locations should be selected after the distress survey to ensure that all significant pavement conditions are represented. If NDT is done, the data from that testing should also be used to select appropriate sites for coring.

Specific recommendations for assessing effective structural capacity from distress survey and materials testing information are given elsewhere (*2*) for each overlay type.

Structural Capacity Based on NDT

NDT is an extremely valuable and rapidly developing technology. When properly applied, NDT can provide a vast amount of information with a reasonable expenditure of time, money, and effort. The analyses, however, can be sensitive to unknown conditions and must be performed by knowledgeable, experienced persons.

Within the scope of these overlay design procedures, NDT structural evaluation differs depending on the type of pavement. For PCC pavements, NDT serves three functions: (*a*) to examine load transfer efficiency at joints and cracks, (*b*) to estimate the effective modulus of subgrade rection (*k* value), and (*c*) to estimate the PCC modulus of elasticity (which provides an estimate of flexural strength). For AC pavements, NDT serves two functions: (*a*) to estimate the roadbed soil resilient modulus and (*b*) to directly estimate SN_{eff}. Some agencies use NDT to backcalculate the moduli of the individual layers of an AC pavement and then use these moduli to estimate SN_{eff}. This approach is not recommended in the revised AASHTO overlay design procedures because it implies and requires a level of sophistication that does not exist with the structural number approach to design.

Structural Capacity Based on Remaining Life

The remaining life approach to structural evaluation is based on the concept shown in Figure 1. This concept is that repeated loads gradually damage the pavement and reduce the remaining number of loads the pavement can carry. To determine the remaining life, the designer must determine the actual amount of traffic the pavement has carried to date and the total amount of traffic the pavement could be expected to carry to "failure" (when serviceability equals 1.5, to be consistent with the AASHO Road Test equations). Both traffic amounts must be expressed in 18-kip ESALs. The difference between these values, expressed as a percentage of the total traffic to failure, is the remaining life:

$$RL = 100\left[1 - \left(\frac{N_p}{N_{1.5}}\right)\right] \quad (1)$$

where

RL = remaining life (percent),
N_p = total traffic to date, 18-kip ESAL, and
$N_{1.5}$ = total traffic to pavement failure (P2 = 1.5), 18-kip ESAL.

With RL determined, the designer may obtain a condition factor (CF) from Figure 2. The remaining life method as presented in the revised AASHTO overlay design procedures makes use of a thorough examination of the relationship be-

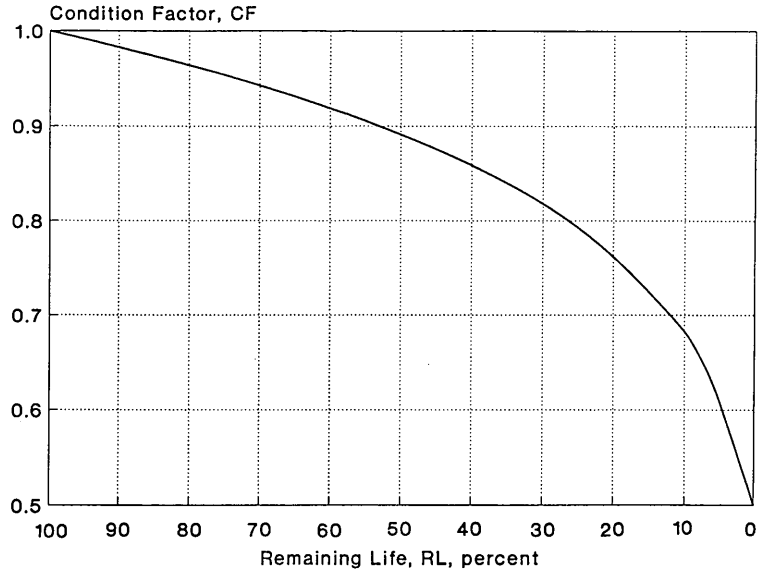

FIGURE 2 Relationship between condition factor and remaining life.

tween remaining life and condition factor done by Elliott (*5*). CF is defined by

$$CF = \frac{SC_n}{SC_o} \quad (2)$$

where SC_n is the pavement structural capacity after N_p ESAL and SC_o is the original pavement structural capacity.

The existing structural capacity may be estimated by multiplying the original structural capacity of the pavement by CF. For example, the original structural number (SN_o) of a flexible pavement may be calculated from material thicknesses and the structural coefficients for those materials in a new pavement. SN_{eff} based on a remaining life analysis would be

$$SN_{eff} = CF * SN_o \quad (3)$$

The structural capacity determined by this relationship does not account for any preoverlay repair. The calculated structural capacity should be viewed as a lower limit value and may require adjustment to reflect the benefits of preoverlay repair.

The remaining life approach to determine SN_{eff} or D_{eff} has some serious deficiencies associated with it. There are four major sources of error:

1. The predictive capability of the AASHO Road Test equations,
2. The large variation in performance typically observed even among pavements of seemingly identical designs,
3. Estimation of past 18-kip ESALs, and
4. Inability to account for the amount of preoverlay repair to the pavement.

As a result, this method of determining the remaining life of the pavement can in some cases produce erroneous results. The remaining life estimate may be very low even though little load-associated distress is present. If load-related cracking is present in small amounts and at a low severity level, the pavement has considerable remaining life, regardless of what the traffic-based remaining life calculation suggests. At the other extreme, the remaining life estimate may be very high even though a substantial amount of medium- and high-severity load-related cracking is present. In this case, the pavement really has little remaining life. At any point between these two extremes, the remaining life computed from past traffic may not reflect the amount of fatigue damage in the pavement, but discerning this from observed distress may be more difficult. If the computed remaining life appears to be clearly at odds with the amount and severity of load-associated distress present, the remaining life method should not be used to compute the structural capacity of the existing pavement.

The remaining life approach to determining structural capacity is not directly applicable, without modification, to pavements that have already received one or more overlays.

SUMMARY OF REVISED AASHTO OVERLAY DESIGN PROCEDURES

The revised AASHTO overlay design procedure actually consists of seven separate, stand-alone design procedures, one for each of the overlay/pavement combinations listed earlier. The procedures were developed in this fashion to enhance their clarity and ease of use.

The design procedure for each type of overlay begins with a description of the construction tasks involved, conditions under which that type of overlay may not be feasible, detailed preoverlay repair recommendations, and considerations for reflection crack control.

For each type of overlay, the thickness design process follows eight steps.

1. Determine existing pavement design and construction: The layer thickness and material inputs required are identified.

2. Traffic analysis: Predicted future 18-kip ESALs in the design lane over the design period are required. The type of ESALs (rigid or flexible) appropriate for the overlay/pavement combination is required. The remaining life method of determining SN_{eff} or D_{eff} also requires past cumulative ESALs.

3. Condition survey: Distress types, severities, and quantities required for determination of the effective structural capacity of the existing pavement are specified.

4. Deflection testing (strongly recommended): Specific procedures for deflection testing for determination of inputs to the overlay design procedure are described. For AC pavements, deflection testing provides an estimate of the design subgrade resilient modulus needed to determine SN_f, and also a direct estimate of SN_{eff}. For PCC pavements, deflection testing provides estimates of several parameters needed to determine D_f, including the effective k value, the PCC elastic modulus, the PCC modulus of rupture, and the J load transfer factor. A heavy-load deflection device such as the falling weight deflectometer (FWD) is recommended. Guidelines on NDT load levels, sensor locations, and testing locations are given as appropriate for each existing pavement type.

5. Coring and materials testing (strongly recommended): Guidelines for laboratory testing and visual examination of materials samples are given.

6. Determination of required structural capacity for future traffic (SN_f or D_f): Each of the inputs required to determine SN_f or D_f according to the flexible or rigid pavement design equation in Part II of the guide is described. Guidelines for determining these inputs and the ranges of their reasonable values are given. With each overlay design procedure, a worksheet is provided for determination of the future structural capacity required.

7. Determination of effective structural capacity of the existing pavement (SN_{eff} or D_{eff}): Of the three available methods for determining effective structural capacity, those appropriate for the existing pavement type are described. For AC pavements with no previous overlay, all three methods are applicable. For bare PCC pavements, the condition survey method and remaining life method are applicable. For AC/PCC pavements, only the condition survey method is applicable. With each overlay design procedure, a worksheet is provided for determination of the effective structural capacity of the existing pavement.

8. Determination of overlay thickness: In each procedure, an equation is given for the overlay thickness required to satisfy the pavement's structural deficiency and support the predicted future traffic over the design period.

Each overlay design procedure also includes a discussion of other relevant items, such as subdrainage, shoulders, and widening (for all overlay/pavement combinations), surface milling (for AC overlays of AC pavements and existing AC/PCC pavements), overlay joints and overlay reinforcement (for all PCC/pavement combinations), and bonding procedures and separation layers (for bonded and unbonded PCC overlays, respectively).

Highlights of the individual overlay design procedures are described in the following sections. This summary does not, of course, provide the details necessary to apply the design procedures. Complete information on the procedures, their development, and their field testing can be found elsewhere (*2–4*).

AC OVERLAY OF AC PAVEMENT

The required thickness of an AC overlay for an AC pavement is given by the following equation:

$$D_{ol} = \frac{SN_{ol}}{a_{ol}} = \frac{(SN_f - SN_{eff})}{a_{ol}} \tag{4}$$

where

SN_{ol} = required overlay structural number,
a_{ol} = structural coefficient for the AC overlay,
D_{ol} = required overlay thickness (in.),
SN_f = structural number required to carry future traffic, and
SN_{eff} = effective structural number of the existing pavement.

The design subgrade resilient modulus, which is required to determine SN_f, may be determined from deflection testing using the following equation:

$$\text{Design } M_R = C\left(\frac{0.24\,P}{d_r\,r}\right) \tag{5}$$

where

Design M_R = design subgrade resilient modulus (psi),
P = applied load (lb),
d_r = deflection at a distance r from the center of the load (in.),
r = distance from center of load (in.), and
C = 0.33 (recommended).

This method of determining the subgrade modulus was proposed by Ullidtz (*6,7*) and is based on Boussinesq's deflection equation (*8*). Its derivation is provided elsewhere (*3*). This equation may be applied to deflections measured at a sufficient distance from the applied load that the deflection is due only to subgrade deformation. A correction factor C no greater than 0.33 is required to make the subgrade resilient modulus consistent with the laboratory-measured value of 3,000 psi at a deviator stress of 6 psi, which was used for the AASHO Road Test soil in the development of the flexible pavement design equation. The need for this correction was verified using field and laboratory subgrade modulus data from the AASHO Road Test and other sites (*3*). The design subgrade resilient modulus may also require seasonal adjustment, in accordance with Part II of the guide. The subgrade resilient modulus may also be determined by laboratory testing or from relationships developed between resilient modulus and other soil properties.

The NDT method of SN_{eff} determination follows an assumption that the structural capacity of the pavement is a function of its total thickness and overall stiffness. The relationship between SN_{eff}, thickness, and stiffness is

$$SN_{eff} = 0.0045D\sqrt[3]{E_p} \tag{6}$$

where D is the total thickness of all pavement layers above

the subgrade (in.) and E_p is the effective modulus of all pavement layers above the subgrade (psi).

The pavement's effective modulus may be determined by trial and error using the following equation:

$$d_0 = 1.5pa \times \left\{ \frac{1}{M_R\sqrt{1+\left(\frac{D}{a}\sqrt[3]{\frac{E_p}{M_R}}\right)^2}} + \frac{\left[1-\frac{1}{\sqrt{1+\left(\frac{D}{a}\right)^2}}\right]}{E_p} \right\} \quad (7)$$

where

d_0 = deflection measured at the center of the load plate (and adjusted to a standard temperature of 68°F) (in.),
p = NDT load plate pressure (psi),
a = NDT load plate radius (in.),
D = total thickness of pavement layers above the subgrade (in.),
M_R = subgrade resilient modulus (psi), and
E_P = effective modulus of all pavement layers above the subgrade (psi).

This equation is based on Odemark's method for determination of deflection in a two-layer system (*9*), using Boussinesq's one-layer deflection (*8*) and the concept of "equivalent thickness" described by Barber (*10*). Its derivation is provided elsewhere (*3*).

The condition survey method of SN_{eff} determination involves a component analysis using the structural number equation:

$$SN_{eff} = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \quad (8)$$

where

D_1, D_2, D_3 = thicknesses of existing pavement surface, base, and subbase layers;
a_1, a_2, a_3 = corresponding structural layer coefficients; and
m_2, m_3 = drainage coefficients for granular base and subbase.

Suggested layer coefficients for existing AC pavement layer materials are given elsewhere (*2*). The values suggested are less than or equal to the values that would be assigned to the materials if new, depending on the quantity and severity of distress present, and evidence of pumping, degradation, or contamination by fines. Guidelines for selection of drainage coefficients are given in Part II of the guide. It is emphasized in the overlay design procedure that the poor drainage situation at the AASHO Road Test would be expressed by drainage coefficient values of 1.0 for granular layers.

AC OVERLAY OF FRACTURED PCC SLAB PAVEMENT

This procedure addresses the design of AC overlays placed on PCC pavements after they have been fractured by any of the following techniques: break/seat (for JRCP), crack/seat (for JPCP), or rubblize/compact (for JRCP, JPCP, or CRCP). The design procedure for AC overlays of fractured PCC slab pavements follows the same basic approach used for AC overlay of AC pavements. Deflections measured on the PCC slab before fracturing may be used to determine the subgrade modulus. A smaller *C* factor (0.25) is recommended for adjustment of the backcalculated subgrade modulus to a design value, because the stress state in the subgrade is much lower beneath an intact PCC slab than beneath an AC pavement.

The structural properties of fractured PCC slabs are difficult to characterize. Backcalculated modulus values ranging from 100,000 to 800,000 psi, and within-project coefficients of variation of 40 percent or more, have been reported for rubblized pavements (*11,12*). Backcalculated modulus values ranging from a few hundred thousand psi to a several million psi, and within-project coefficients of variation of 40 percent or more, have been reported for cracked/seated and broken/seated slabs (*11–16*).

SN_{eff} is determined for fractured PCC slabs by component analysis using the following structural number equation:

$$SN_{eff} = a_2 D_2 m_2 + a_3 D_3 m_3 \quad (9)$$

where

D_2, D_3 = thicknesses of fractured slab and base layers,
a_2, a_3 = corresponding structural layer coefficients, and
m_2, m_3 = drainage coefficients for fractured slab and granular subbase.

The recommended ranges of values for a_2 are 0.20 to 0.35 for crack/seat JPCP and break/seat JRCP and 0.14 to 0.30 for rubblized PCC of any type. Since the layer coefficient represents the overall performance contribution of that layer, it is likely that it is not related solely to the modulus of that layer, but to other properties as well, such as the load transfer capability of the pieces. The large variability of layer moduli within a project is also of concern. This extra variability should ideally be expressed in an increased overall standard deviation in designing for a given reliability level.

AC OVERLAY OF JPCP, JRCP, AND CRCP

The required thickness of an AC overlay of a bare PCC pavement is given by the following equation:

$$D_{ol} = A(D_f - D_{eff}) \quad (10)$$

where

D_{ol} = required thickness of AC overlay (in.),
A = factor to convert PCC thickness deficiency to AC overlay thickness,
D_f = slab thickness to carry future traffic (in.), and
D_{eff} = effective thickness of existing slab (in.).

The *A* factor, which is a function of the PCC thickness deficiency, is given by the following equation:

$$A = 2.2233 + 0.0099(D_f - D_{eff})^2 - 0.1534(D_f - D_{eff}) \quad (11)$$

A is used to convert PCC thickness deficiency to required AC overlay thickness. A value of about 2.5 has been used for many years in various overlay design procedures. For example, a 2-in. bonded PCC overlay is considered roughly equivalent to a 5-in. AC overlay. However, for greater PCC thickness deficiencies, using a value of 2.5 for *A* produces AC overlay thicknesses that are not realistic. This concern was addressed by an investigation of the *A* factor for design of AC overlays of PCC pavements.

Examination of the Corps of Engineers' field data (*17,18*) from which the *A* value of 2.5 was derived revealed that this value is overly conservative (*3*). To investigate further what *A* factor should be used in design of AC overlays of PCC pavements, the elastic layer program BISAR was used to compute stresses in PCC slabs with a range of PCC and AC overlay thicknesses. The *A* factor required (for an AC overlay thickness that would produce the same stress in the base slab as a given thickness of bonded PCC overlay) decreased as the PCC thickness deficiency increased. The development of Equation 11 is described elsewhere (*3*).

The overlay design procedures in the 1986 guide proposed that the effective *k* value be determined using backcalculated elastic moduli for the base and subgrade. This approach is not recommended in the revised overlay design procedures. Rather, direct backcalculation of the effective *k* value is recommended, and a simple procedure for doing so is provided.

The effective dynamic *k* value of the foundation and the elastic modulus of the PCC slab may be directly determined from the maximum deflection d_0 measured beneath an NDT load plate and the deflection basin AREA, defined as follows:

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

where d_0 is the deflection in center of loading plate (in.) and d_i = deflections at 12, 24, and 36 in. from plate center (in.).

AREA has units of length, rather than area, since each of the deflections is normalized with respect to d_0 in order to remove the effect of different load levels and to restrict the range of values obtained. AREA and d_0 are thus independent parameters, from which the two unknown values E_{pcc} and *k* may be determined for a known slab thickness. This approach to direct backcalculation of pavement and foundation moduli in two-layer pavements was first proposed by Hoffman and Thompson (*19*) and adapted to E and *k* backcalculation for PCC pavements by ERES (*20*) and Foxworthy (*21*). Further investigation of this concept by Barenberg and Petros (*22*) and by Ioannides (*23*) has produced a forward solution procedure to replace the iterative and graphical procedures used previously.

For a given load radius and sensor arrangement, a unique relationship exists between AREA and the "dense liquid" radius of relative stiffness of the pavement system (ℓ_k), in which the subgrade is characterized by a *k* value (*24*):

$$\ell_k = \sqrt[4]{\frac{E_{pcc}\, D_{pcc}^3}{12\,(1 - \mu_{pcc}^2)\, k}} \tag{13}$$

where

ℓ_k = dense liquid radius of relative stiffness (in.),
E_{pcc} = PCC elastic modulus (psi),
D_{pcc} = PCC thickness (in.),
μ_{pcc} = PCC Poisson's ratio, and
k = effective *k* value (psi/in.).

The following equation for ℓ_k as a function of AREA was developed by Hall (*12*):

$$\ell_k = \left[\frac{\ln\left(\dfrac{36 - \text{AREA}}{1812.279133}\right)}{-2.559340}\right]^{4.387009} \tag{14}$$

The effective *k* value may be obtained from Westergaard's deflection equation (*24*) using the measured maximum deflection and the ℓ_k corresponding to the computed AREA:

$$k = \left(\frac{P}{8\, d_0\, \ell_k^2}\right)\left\{1 + \left(\frac{1}{2\pi}\right)\left[\ln\left(\frac{a}{2\,\ell_k}\right) + \gamma - 1.25\right]\left(\frac{a}{\ell_k}\right)^2\right\} \tag{15}$$

where

d_0 = maximum deflection (in.),
P = load (lb), and
γ = Euler's constant (0.57721566490).

The effective static *k* value, used in the rigid pavement design equation for determination of D_f, is estimated by dividing the effective dynamic *k* value by two (*3,21,25*).

The elastic modulus of the PCC slab may be determined from the slab thickness, the *k* value, and the radius of relative stiffness. The PCC modulus of rupture may be estimated from the backcalculated PCC elastic modulus or from indirect tensile strengths of cores. For CRCP, the modulus of rupture should be determined from backcalculated E values only at points that have no cracks within the deflection basins.

For JPCP and JRCP, deflection testing is recommended to measure load transfer at tranverse joints. The overlay design procedure provides guidelines for load transfer measurement and selection of the J factor. For CRCP, a J factor of 2.2 to 2.6 is recommended for overlay design, assuming that working cracks are repaired with continuously reinforced PCC.

The effective thickness of the existing slab (D_{eff}) is computed from the following equation:

$$D_{eff} = F_{jc} * F_{dur} * F_{fat} * D \tag{16}$$

where

D = existing PCC slab thickness (in.),
F_{jc} = joints and cracks adjustment factor,
F_{dur} = durability adjustment factor, and
F_{fat} = fatigue damage adjustment factor.

The joints and cracks factor F_{jc} adjusts for the extra loss in PSI caused by deteriorated reflection cracks in the overlay

that will result from any unrepaired deteriorated joints, cracks, punchouts, and other discontinuities in the existing slab before overlay. Full-depth repair of these distresses before overlay is strongly recommended. The overlay design procedure provides guidelines for assigning F_{jc} on the basis of the number of deteriorated joints, cracks, punchouts, and other major discontinuities left unrepaired.

The durability factor F_{dur} adjusts for an extra loss in PSI of the overlay when the existing slab has durability problems such as D cracking or reactive aggregate distress. Guidelines provided for assignment of F_{dur} on the basis of the severity and quantity of durability-related distress.

The fatigue damage factor F_{fat} adjusts for past fatigue damage in the slab. Guidelines are provided for assignment of F_{fat} on the basis of the severity and quantity of load-related distress.

AC OVERLAY OF AC/PCC PAVEMENT

This procedure addresses the design of second AC overlay for JPCP, JRCP, and CRCP with an existing AC overlay. The design procedure follows the same basic approach used for AC overlays of bare PCC pavements. The equation for AC overlay thickness is the same.

The effective dynamic k value of the foundation and the elastic modulus of the PCC slab may be determined using the procedure described for bare PCC pavements, except that adjustments must be made to the measured maximum deflection d_0 and basin AREA. The compression in the AC surface under the NDT load plate must be subtracted from the maximum deflection measured at the AC/PCC pavement surface to obtain the deflection of the PCC layer. The AC compression is a function of the AC modulus, AC thickness, and AC/PCC interface condition (as determined from cores) (*12*). For AC/PCC bonded,

$$d_{0\ \mathrm{compress}} = -0.0000328 + 121.5006 \left(\frac{D_{ac}}{E_{ac}}\right)^{1.0798} \tag{17a}$$

For AC/PCC unbonded,

$$d_{0\ \mathrm{compress}} = -0.00002132 + 38.6872 \left(\frac{D_{ac}}{E_{ac}}\right)^{0.94551} \tag{17b}$$

where

$d_{0\ \mathrm{compress}}$ = AC compression at center of load (in.),
D_{ac} = AC thickness (in.), and
E_{ac} = AC elastic modulus (psi).

Direct measurement of the AC mix temperature at three or more times during deflection testing is recommended to assign an AC mix temperature to each deflection basin. A relationship between AC elastic modulus and temperature may then be used to assign an AC modulus to each deflection basin. The Asphalt Institute's equation for AC modulus as a function of temperature, mix parameters (percent fines, volume of voids, percent asphalt, and asphalt viscosity), and loading frequency (approximately 18 Hz for the FWD load duration of 25 to 30 msec) may be used for this purpose (*26*). An alternative is to conduct diametral resilient modulus testing (ASTM D4123) of cores from the AC surface at two or more temperatures to establish a curve of AC modulus versus temperature. Laboratory-measured AC modulus values must be adjusted to the frequency of the NDT device used (*3*).

The Asphalt Institute's equation for AC modulus applies to new mixes. AC that has been in service for some years may have either a higher modulus (due to hardening of the asphalt) or lower modulus (due to deterioration from stripping or other causes) at any given temperature.

The effective thickness of the existing slab (D_{eff}) is computed from the following equation:

$$D_{eff} = (D_{pcc} * F_{jc} * F_{dur}) + \left[\left(\frac{D_{ac}}{2.0}\right) * F_{ac}\right] \tag{18}$$

Guidelines are provided elsewhere (*2*) for assignment of F_{jc}, F_{dur}, and the AC quality adjustment factor F_{ac}, for existing AC/PCC pavements.

BONDED PCC OVERLAY OF JPCP, JRCP, AND CRCP

This procedure follows the same basic approach used for AC overlays of bare PCC pavements. The following equation for bonded PCC overlay thickness is used:

$$D_{ol} = D_f - D_{eff} \tag{19}$$

The k value, PCC elastic modulus of rupture, and J load transfer factor for the existing PCC pavement should be used to determine D_f. The effective thickness of the existing slab (D_{eff}) is computed from the following equation:

$$D_{eff} = F_{jc} * F_{dur} * F_{fat} * D \tag{20}$$

UNBONDED PCC OVERLAY OF JPCP, JRCP, AND CRCP

This procedure follows the same basic approach used for AC overlays of bare PCC pavements. The following equation for unbonded PCC overlay thickness is used:

$$D_{ol} = \sqrt{D_f^2 - D_{eff}^2} \tag{21}$$

The elastic modulus, modulus of rupture, and load transfer factor for the overlay PCC should be used to determine D_f. The effective thickness of the existing slab is computed from the following equation:

$$D_{eff} = F_{jc} * D \tag{22}$$

Field surveys of unbonded concrete overlays have shown that durability distress and fatigue damage in the existing slab have very little effect on the performance of the unbonded overlay. Therefore, the F_{dur} and F_{fat} factors are not used to determine D_{eff} for design of unbonded concrete overlays.

Field surveys of unbonded jointed concrete overlays have also shown little evidence of reflection cracking or other problems caused by deteriorated joints and cracks in the existing

slab. Therefore, the F_{jcu} factor, which is used for design of unbonded overlays, makes a smaller adjustment to the existing slab thickness than the F_{jc} factor, which is used for design of bonded PCC and AC overlays. Although the thickness design procedure is the same for jointed and CRC overlays, unbonded overlays are not intended to bridge areas of poor support, and in particular CRC overlays may require more preoverlay repair in some situations.

JPCP, JRCP, AND CRCP OVERLAY OF AC PAVEMENT

A PCC overlay of an AC pavement is designed using the following equation:

$$D_{ol} = D_f \tag{23}$$

The effective k value to be used for design of a PCC overlay of an existing AC pavement may be estimated from the subgrade modulus and the effective pavement modulus, determined from deflection testing as described previously, using the k value nomograph provided in Part II of the guide. This dynamic k value must be divided by 2 to obtain the static k value for use in design.

The engineer should be aware that this approach to determining the design static k value for PCC/AC design has some significant limitations. The k value nomograph in Part II of the guide was developed using an elastic layer program, without verification with field deflection data. Whereas it may yield reasonable values in some instances, it may yield unreasonably high values in other instances. Further research of the subject of support for PCC overlays, including deflection testing on in-service PCC/AC pavements and backcalculation of effective k values, is strongly encouraged.

CONCLUSIONS

The revised AASHTO overlay design procedures use the concepts of structural deficiency, structural number for flexible pavements, and future required structural capacity determined from the AASHTO flexible and rigid pavement design equations. These concepts were retained to maintain compatibility between Parts II and III of the guide.

Development of a more sophisticated mechanistic approach to overlay design was not within the scope of the revisions. NDT is recommended for use in characterizing the existing pavement to the extent appropriate within the framework of these empirical design procedures.

The AASHTO overlay design procedures were extensively revised to make them easier to use, more adaptable to calibration by local agencies, and more comprehensive. Key revisions to the overlay design procedures include the following:

1. Guidelines for overlay feasibility;
2. Guidelines for several important considerations (preoverlay repair, reflection crack control, subdrainage, AC surface milling, shoulders, AC surface recycling, AC rutting, overlay design reliability level, PCC durability, PCC overlay bonding/separation layers, pavement widening, and PCC overlay joints and reinforcement);
3. A complete step-by-step overlay design procedure for each overlay type;
4. Guidelines for pavement evaluation for overlay design, including distress surveying, nondestructive testing, and destructive testing;
5. Guidelines for selecting inputs for determination of required future structural capacity (SN_f, D_f);
6. Guidelines for characterization of effective structural capacity of existing pavement (SN_{eff}, D_{eff}) using three approaches [condition survey and materials testing, NDT testing (where appropriate), and remaining life (where appropriate)]; and
7. Improved adaptability of the overlay thickness design procedures to local conditions to produce more reasonable answers.

ACKNOWLEDGMENT

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DISCUSSION

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A major contribution of the 1986 *AASHTO Guide* (*1*) is the introduction of a remaining life concept in overlay thickness design, where the overlay structural requirement is expressed in the following form:

$$SN_{OL} = SN_y - F_{RL}(SN_{xeff})$$

or

$$D_{OL} = D_y - F_{RL}(D_{xeff}) \quad (24)$$

An excellent description of the concept is found in Chapter 5 of Part III and Appendix CC of the *AASHTO Guide*. That the concept is fundamentally correct and conceptually sound has subsequently been ascertained independently by Easa (*2*) and Fwa (*3*). Unfortunately, without giving valid justification, the revision proposed by the paper chooses to ignore this important issue and the F_{RL} factor totally. The paper adopts the traditional overlay equation $SC_{OL} = SC_y - SC_{eff}$ throughout. This writer considers the paper to be incomplete without addressing the remaining life concept related to the F_{RL} factor. The only clue to why the authors have decided to ignore Equation 24 is found in one misleading statement: "The remaining life method as presented in the revised AASHTO overlay design procedure makes use of a thorough examination of the relationship between remaining life and condition factor by Elliott." Elliott's work (*4*) did not produce a new condition factor CF expression as implied by the statement in the text. Instead, Elliott (*4*) addressed AASHTO remaining life concept and F_{RL} factor and concluded that (*a*) the appropriate value for F_{RL} is 1.0 and (*b*) the AASHTO overlay design approach should be revised to exclude remaining life considerations. This discussion will show that Elliott's sweeping conclusions are not justified and why it is not wise to discard Equation 24 and revert to the use of the traditional overlay equation.

REMAINING LIFE CONCEPT AND F_{RL}

Elliott's (*4*) recommendation to exclude remaining life consideration from AASHTO overlay design was based on the reasons that (*a*) AASHTO design produces inconsistent results and (*b*) the appropriate F_{RL} value is 1.0. The study by Fwa (*3*) shows that inconsistencies in AASHTO overlay designs were due solely to a flaw in the formula for computing F_{RL}. A corrected formula for F_{RL} was derived according to the very concept of remaining life described in the *AASHTO Guide*. Using the corrected formula for F_{RL}, it was illustrated that consistent overlay designs were obtained with Equation 24. Elliott's reason (*a*) is therefore invalid.

Elliott's claim of $F_{RL} = 1.0$ was based on an analysis using a "simple scale transformation" that relates R_{Lyx} to R_{Ly} for a given R_{Lx} as follows:

$$\left\{\frac{R_{Lx}}{1}\right\} = \left\{\frac{R_{Lyx}}{R_{Ly}}\right\} \quad (25)$$

The transformation is artificial with no clear physical meaning. It is also controversial because R_{Lx} and R_{Lyx} of the old pavement and R_{Ly} of overlaid pavement are computed from different base N_f values. Elliott said that Equation 25 was based on the "philosophy" of "the man who each day walks halfway to his destination," a "philosophy" many readers would find difficult to relate to overlay performance. Although not stated by Elliott, Equation 25 actually assumes that the rate of decrease R_{Lyx} (of old pavement) is proportional to that of R_{Ly} (of new overlaid pavement). This appears to be too strong an assumption with very restrictive application because it is common knowledge that the structural capacities and hence the remaining lives of pavements of different ages decrease at unequal (and nonproportional) rates. Incidentally, Elliott's assumption is similar to the condition for a lower bound overlay design (with $F_{RL} = 1.0$) independently identified by Easa (*2*) and Fwa (*3*) as explained in the next section. Both Easa and Fwa also illustrated that there exists an upper bound overlay design and there are theoretically many possible so-

lutions (with F_{RL} values less than 1.0) between the two bounds. Elliott's analysis is therefore applicable to a very special case, probably a highly unlikely one. Elliott's reason (*b*), based on conclusions drawn from the limited analysis, is inadequate to justify his recommendations to set F_{RL} = 1.0 and exclude remaining life consideration from the AASHTO overlay design approach.

UPPER AND LOWER BOUND OVERLAY DESIGN

Subsequent to Elliott's work (*4*) that pointed out inconsistencies in the AASHTO overlay design method, Easa (*2*) and Fwa (*3*) separately confirmed the fundamental correctness of the AASHTO overlay design approach that incorporates the concept of remaining life, and proposed different procedures to eliminate the inconsistencies caused by a flaw in F_{RL} calculation. Both Easa and Fwa defined a lower and an upper bound overlay solution. The lower bound solution corresponds to the case with F_{RL} = 1.0 (which is the maximum possible value of F_{RL}) where the rate of structural deterioration of an old pavement after overlay is assumed to be the same as that of a new pavement. The upper bound solution is one with $F_{RL} \leq 1.0$ where the old pavement after overlay is assumed to continue to deteriorate at a rate as if no overlay were applied. It is easy to see that the overlay solution that represents the real-life situation will lie somewhere between the two bounds. It is also easy to see that it is unwise to set F_{RL} = 1.0 (lower bound solution) because it would lead to an overlay solution that is underdesigned and thus unconservative.

TRADITIONAL VERSUS AASHTO OVERLAY DESIGN

The traditional overlay equation is conceptually unsound and inadequate because overlay thickness is derived on the basis of the overlay requirement at the time of overlay application. It does not include an analysis to examine whether the overlay provided is adequate during other stages of overlay service life. In terms of remaining life concept, the traditional overlay design method is equivalent to setting F_{RL} = 1.0 and assuming an old pavement will deteriorate like a new pavement after being overlaid. In contrast, as explained in Appendix CC of the *AASHTO Guide* (*1*) and demonstrated by Fwa (*3*), the 1986 AASHTO overlay design approach that incorporates remaining life consideration enables one to analyze the overlay requirements for the entire design period and select an appropriate F_{RL} value to compute from Equation 24 the overlay thickness needed. The value of F_{RL} is equal to 1.0 if the overlay requirement at the time of overlay application governs the design. In cases where overlay requirements at other stages of overlay service life are more critical, the value of F_{RL} will be less than 1.0.

SUMMARY

This discussion shows that the authors' decision to discard the 1986 AASHTO remaining life concept by ignoring the F_{RL} term and reverting to the traditional overlay equation is a move that is unwise and uncalled for. This writer hopes that the authors will make necessary amendments to their proposed revision before it is finalized.

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AUTHORS' CLOSURE

The remaining life concept has not been discarded in the proposed revisions to the AASHTO overlay design procedures. As described in the paper, three procedures are given for estimating the effective structural capacity of an existing pavement: a deflection-based approach, a condition survey approach, and a remaining life approach.

The basic concept of remaining life is that a pavement's past traffic and its total traffic-bearing capacity over its lifetime may be used together to estimate the traffic the pavement is capable of carrying for the remainder of its life. This concept did not originate with the 1986 *AASHTO Guide*, but it has been used in pavement evaluation for many years and is applicable to any pavement design procedure based on a relationship between traffic and loss of structural capacity. Indeed, this concept is intrinsic to the AASHTO design methodology.

The authors consider the basic remaining life concept to be valid. However, the application of this concept in the proposed revisions to the AASHTO overlay design procedures differs from the application presented in the 1986 guide.

In the 1986 guide's overlay design procedures, procedures were given for determining the effective structural capacity (SC_{eff}) of a pavement from deflection testing or distress observations. This effective structural capacity is expected to be less than the original structural capacity of the pavement when new (SN_o). However, the 1986 guide's overlay design procedures then applied a traffic-based remaining life factor as a multiplier to the effective structural capacity determined from deflections or distress observations. This approach is widely considered to penalize a pavement twice for the same past traffic.

Fwa has defended this double penalty with the reasoning that if a deteriorated pavement with a given effective structural capacity is overlaid, it will subsequently deteriorate at a faster rate than a newly constructed pavement of the same structural capacity that receives the same thickness of overlay. This is a considerable distortion of the structural deficiency concept of overlay design. The essence of the structural deficiency concept is that a performance prediction model may

be used to determine a required overlay, which will increase an in-service pavement's effective structural capacity to a structural capacity sufficient to carry the traffic expected over the design period. The rate of deterioration of the overlaid pavement is thus predicted by the performance model used, just as is the rate of deterioration predicted for new pavements by the same model. Within the context of the AASHTO design methodology, the flexible and rigid pavement performance models presented in Part II of the guide are used to determine required future structural capacity (structural number of slab thickness), and the rate of deterioration is measured by loss of serviceability as predicted by these models. If the two pavements described by Fwa have the same structural capacity before overlay, and receive the same overlay, then according to the structural deficiency concept their performance after overlay will be the same. One cannot correctly apply the structural deficiency concept of overlay design and at the same time conjecture a rate of deterioration of the overlaid pavement other than the rate predicted by the performance model used to define the structural deficiency.

In the proposed revisions to the overlay design procedures, a traffic-based estimate of remaining life is applied to a pavement's original structural capacity (SC_o) to estimate its current effective structural capacity but is not applied to deflection-based and condition-based estimates of the effective structural capacity. In concept, these three approaches for estimating SC_{eff} should yield similar results.

In addition to the conceptual flaw described earlier, the 1986 guide's remaining life computation was considered to be needlessly complex and poorly supported. For example, the procedure did not address the practical significance of a "negative remaining life" computed for an in-service pavement. The need to revise the application of the remaining life concept in the 1986 guide's overlay design procedures was identified by the AASHTO Joint Task Force on Pavements as one of the high-priority revisions to the overlay design procedures.

The authors have examined the work by FWA and by Easa and have concluded that although they offer modifications to the remaining life method as presented in the 1986 guide, they do not correct its major flaw. They also impose needless complexity in the application of a simple concept.

The authors have therefore recommended to the Design Subcommittee of the AASHTO Joint Task Force on Pavements that the method developed by Elliott for considering remaining life be accepted as the best solution to the problems associated with the application of this concept in the 1986 overlay design procedures. It must also be clarified that decisions concerning acceptance of this and other proposed revisions to the overlay design procedures are made not by the authors but rather by the AASHTO Joint Task Force.

The opinions expressed in this paper are those of the authors and not necessarily those of AASHTO, FHWA, NCHRP, or TRB or of the individual states participating in the National Cooperative Highway Research Program.

Field Testing of AASHTO Pavement Overlay Design Procedures

KATHLEEN T. HALL, MICHAEL I. DARTER, AND ROBERT P. ELLIOTT

The AASHTO pavement overlay design procedures were recently revised to make them easier to use, more adaptable to calibration by local agencies, and more comprehensive. The revised procedures were extensively field tested using data from many actual in-service pavements located throughout the United States. A total of 74 examples were developed for seven different categories of overlay and pavement types. State highway agency personnel provided the design, traffic, condition, and deflection data for the overlay design examples and participated in the development of the examples. The revised AASHTO overlay design procedures produce reasonable overlay thicknesses that are consistent with state highway agency recommendations. The examples illustrate the importance of selecting appropriate inputs for overlay design, the use of nondestructive testing data and condition data in overlay design, the significance of design reliability level to overlay thickness, and the importance of preoverlay repair.

Chapter 5 of Part III of the 1986 AASHTO *Guide for Design of Pavement Structures* (*1*) addresses overlay design for pavement rehabilitation. These overlay design procedures were recently revised to make them easier to use, more adaptable to calibration by local agencies, and more comprehensive. The proposed procedures are currently under consideration by AASHTO.

This paper presents the results of the extensive field testing of the revised overlay design procedures using data from many actual in-service pavements located throughout the United States. Darter et al. present the procedures in detail (*2*), document the development of the procedures (*3*), and provide complete results of the field testing (*4*).

A total of 74 examples were developed to demonstrate and validate the overlay design procedures. These results were extremely useful in verifying and improving the overlay design procedures. The example design projects may also be used by future researchers to help verify improved overlay design procedures.

DESCRIPTION OF FIELD TESTING PROCEDURES

The examples were developed for actual in-service pavements located throughout the United States. Design, traffic, condition, and deflection data were provided for these projects by 10 state highway agencies. State personnel were actively involved in developing these examples during the development of the revised overlay design procedures. The overlay design procedures were evaluated by the highway agency personnel for clarity and ease of use, and many of their comments were incorporated into the procedures.

In addition, the overlay thicknesses indicated by the procedures were evaluated with respect to state highway agencies' recommendations on the basis of their design procedures and experience with overlay performance.

Each of the example projects is identified by the region of the United States in which it is located and by number within the region. The following regional identifiers are used: NE, Northeast; SE, Southeast; MW, Midwest; NW, Northwest; and SW, Southwest.

Each of the regions is represented in the overlay design examples for each pavement and overlay type to the extent possible. Seven separate groupings of overlays designs are included:

Overlay Type	*Existing Pavement*
AC	AC pavement
AC	Fractured PCC slab
AC and Bonded PCC	JPCP and JRCP
AC and Bonded PCC	CRCP
AC	AC/PCC (composite)
Unbonded PCC	JPCP, JRCP, CRCP
JPCP and JRCP	AC pavement

Lotus 1-2-3 spreadsheets were prepared for each of the above overlap design procedures to aid in the calculations. Each example was prepared on a single-page spreadsheet showing all of the inputs used and outputs obtained. The results obtained were also summarized for each of the seven procedures.

Deflection data were used whenever available from the state agency. Typically one to five representative deflection basins were entered into a spreadsheet to keep the size of the output within reason. In some cases only a few deflection basins were provided by the agency. In other cases a few representative basins were selected for illustrative purposes from a larger deflection data set provided by the agency. The basins chosen are believed to provide overlay thicknesses close to the mean for the project. However, this does not imply that any project should be represented by this small a number of basins. On the contrary, the procedures can be programmed to handle any number of deflection basins and corresponding overlay designs very efficiently.

EXAMPLES OF AC OVERLAY DESIGN FOR AC PAVEMENT

Table 1 gives an example AC overlay design for an AC pavement (NW-1). For a range of reliability levels from 50 to 99

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TABLE 1 Example AC Overlay Design for AC Pavement

```
REVISED CHAPTER 5 AASHTO DESIGN GUIDE OVERLAY DESIGN
==================================================================================
NW-1 AC OVERLAY OF CONVENTIONAL AC PAVEMENT
==================================================================================
EXISTING PAVEMENT DESIGN
AC SURFACE              4.25 inches   SUBGRADE SANDY SILT, SANDY GRAVEL
GRAN BASE               8.00
GRAN SUBBASE            0.00
TOTAL THICKNESS        12.25
Future design lane ESALs  = 2400000 (FLEXIBLE ESALs)
==================================================================================
DETERMINE SNf
  Vary trial SNf until computed ESALs equal future design ESALs.
      SNf  MR,psi        R        Z       So       P1       P2     ESAL
     3.60    5634       50        0     0.45      4.2      2.5  2417312
     4.14    5634       80    0.841     0.45      4.2      2.5  2430778
     4.44    5634       90    1.282     0.45      4.2      2.5  2429228
     4.69    5634       95    1.645     0.45      4.2      2.5  2408097
     5.19    5634       99    2.327     0.45      4.2      2.5  2403245
    TRIAL                     INPUT    INPUT    INPUT    INPUT
==================================================================================
DETERMINE SNeff BY NDT METHOD
   Vary trial Ep/MR until computed D0 equals actual value.
                        ACTUAL            SUBGRADE              TRIAL COMPUTED
  LOAD,lbs D0,mils  Dr,mils   MR,psi C FACTOR    Ep/MR  D0,mils   Ep,psi    SNeff
     9000    12.80     3.55    16901        3     8.45    12.80   142817     2.88
                        r =       36 inches
        Check r > 0.7 ae =    17.95 inches
==================================================================================
DETERMINE SNeff BY CONDITION SURVEY METHOD
LAYER       STR COEF DRAIN m    SNeff
AC SURFACE    0.35     1.00     1.49
BASE          0.14     1.00     1.12
SUBBASE       0.00     1.00     0.00
                     SNeff =    2.61
==================================================================================
DETERMINE SNeff BY REMAINING LIFE METHOD
Past design lane ESALs =     400000 (FLEXIBLE ESALs)
LAYER              THICK,in NEW ST CF     SNo
AC SURFACE             4.25     0.44     1.87
BASE                   8.00     0.14     1.12
SUBBASE                0.00     0.00        0
TOTAL                 12.25              2.99
   SNo   MR,psi    Z     So       P1       P2     N1.5    RL, %       CF    SNeff
   2.99    5634    0   0.45      4.2      1.5  1140161       65     0.93     2.78
                      INPUT    INPUT    INPUT
==================================================================================
DETERMINE OVERLAY THICKNESS      AC OL structural coefficient =    0.44

          DESIGN                NDT           CONDITION          REM LIFE
        RELIABILITY        METHOD,in          METHOD,in          METHOD,in
             50                1.63               2.26               1.85
             80                2.86               3.48               3.08
             90                3.54               4.16               3.76
             95                4.11               4.73               4.33
             99                5.25               5.87               5.47
==================================================================================
```

percent, the required future structural capacity of the pavement, SN_f, is determined by varying trial SN_f values until the ESALs computed using the AASHTO flexible pavement design equation (from Part II of the guide) match the design ESALs for the overlay. The overall standard deviation S_o and initial and terminal pavement serviceability values P1 and P2 may also be varied. In the examples, these three inputs were set at 0.45, 4.2, and 2.5, respectively, unless other values were given by the state highway agency.

The subgrade resilient modulus M_R is backcalculated from a deflection some distance away from the center of the load plate (*2*). A check is included to ensure that the distance is greater than the minimum distance required for accurate determination of M_R. The backcalculated subgrade modulus is then divided by a factor of three to obtain the design subgrade modulus used in determining SN_f (*3*).

The existing pavement's structural capacity, SN_{eff}, may also be determined by the NDT method, by varying the ratio of pavement modulus to subgrade modulus (E_P/M_R) until the computed maximum deflection d_0 matches the deflection measured beneath the load plate. The pavement modulus (that is, the effective modulus of all pavement layers above the

subgrade) may be computed using the backcalculated subgrade modulus, and SN_{eff} is computed as a function of the pavement modulus.

SN_{eff} may also be determined by the condition survey method, in which a structural coefficient is assigned to each pavement layer above the subgrade. The layer coefficients used to determine SN_{eff} should be less than or equal to the values that would be assigned to the layer materials if new and should reflect the quantity and severity of distress present and evidence of pumping, degradation, or contamination by fines. In the examples, the layer coefficients used for the existing pavements were those provided by the state highway agencies. When layer coefficients were not provided, pavement condition information obtained from the state were used to assign reasonable layer coefficients.

The third method of determining SN_{eff} for flexible pavements is the remaining life method. This method requires the past ESALs accumulated in the design lane since construction. Layer coefficients appropriate for new pavement are assigned to each layer material in order to compute SN_0, the structural capacity of the pavement when new. The AASHTO flexible pavement design equation is then used to determine the allowable ESALs to a terminal serviceability level of 1.5 for a 50 percent reliability level. The difference between the past traffic and the allowable traffic, expressed as a percentage of the total traffic to "failure," is the remaining life. The existing pavement's structural capacity SN_{eff} may be estimated by multiplying the original structural capacity SN_0 by a condition factor, CF, which is a function of the remaining life. The past traffic data required for the remaining life method of SN_{eff} determination was typically very difficult for state highway agency personnel to obtain. As a result, the remaining life method could be used for overlay thickness design for only three of the examples submitted.

For each reliability level considered and each of the SN_{eff} methods used, the required AC overlay thickness is obtained by dividing the structural deficiency (SN_f minus SN_{eff}) by an AC layer coefficient value of 0.44.

In general, the AC overlay thicknesses for AC pavement indicated by the revised AASHTO procedure agree with state recommendations, as shown in Figure 1. Some of the differences are due to the lack of consistent data from some of the examples. For example, some projects had thicknesses that varied widely along their length, and the exact thicknesses at the locations of the deflection basins provided were unknown. Errors in assumed pavement thickness are reflected directly in errors in estimating SN_{eff} by the NDT method.

The overlay thickness designs based on NDT are generally consistent with those based on the condition survey method. Figure 2 shows a comparison between overlay thicknesses at the 95 percent reliability level determined by the NDT method and condition survey method.

The subgrade resilient modulus has a large effect on the resulting overlay thicknesses. Therefore, it is of utmost importance to obtain an appropriate modulus value to enter into the AASHTO flexible pavement design equation. Use of too high a subgrade modulus in design will result in inadequate AC overlay thickness. The reduction in backcalculated modulus by a factor of three appears to be reasonable (*3*). Some data available from one state permit a direct comparison between laboratory and backcalculated modulus values:

Project	*Lab M_R (psi)*	*Backcalculated M_R (psi)*	*Ratio*
NW-2	6,000	13,483	2.25
NW-3	6,000	19,608	3.27
NW-4	4,150	14,085	3.39
NW-5	4,500	14,286	3.17
Average	5,163	15,365	3.02

Each agency will need to evaluate this ratio, as well as other factors, to tailor the design procedure to its own conditions.

The design reliability level is very significant. The example AC pavement projects ranged from collector highways to

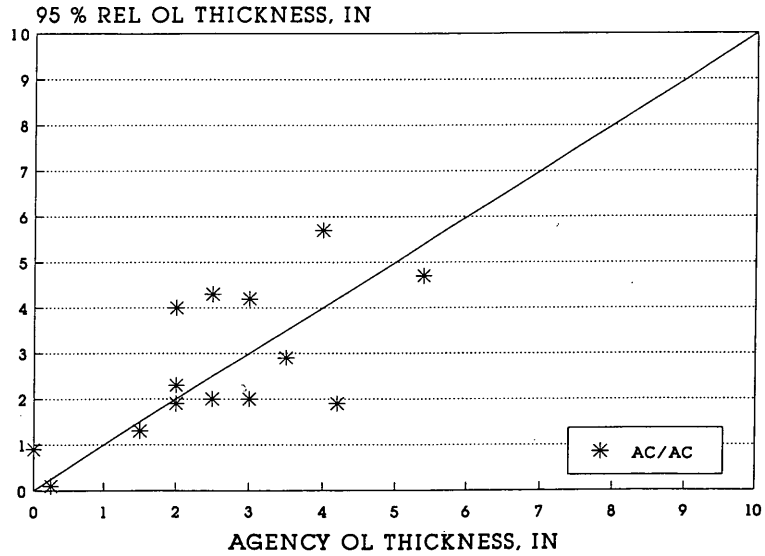

FIGURE 1 Comparison of AASHTO AC overlay thicknesses and agency AC overlay thicknesses for AC pavements (95 percent reliability).

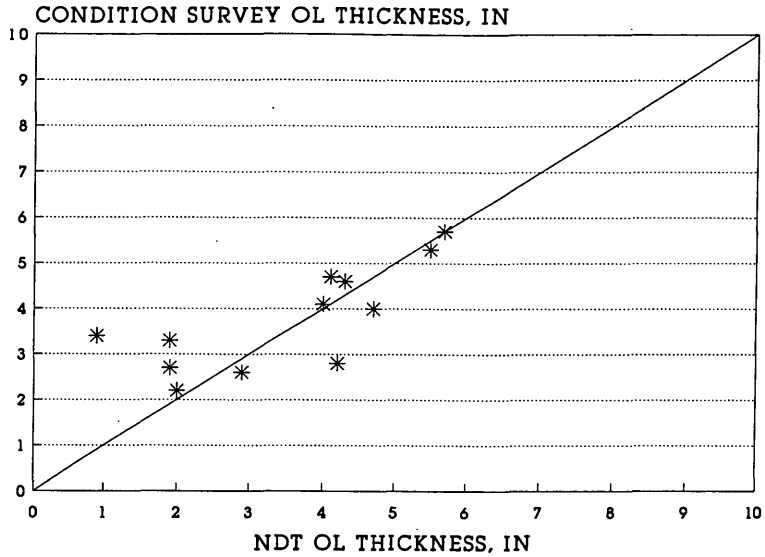

FIGURE 2 Comparison of AC overlay thickness determined by NDT and condition survey procedures for AC pavements (95 percent reliability).

heavily trafficked Interstate-type highways. A design reliability level of approximately 95 percent usually produced reasonable overlay thicknesses.

EXAMPLES OF AC OVERLAY DESIGN FOR FRACTURED PCC PAVEMENT

Table 2 gives an example AC overlay design for a fractured PCC slab pavement (SW-6). The design procedure is similar to that used for AC overlays of AC pavements, with the notable difference that the subgrade modulus backcalculated before the slab was fractured is divided by a factor of six, rather than three, to account for the increase in subgrade stress state after fracturing. The condition survey method is the only method for determining SN_{eff} for fractured PCC pavement. For this example, the state agency recommended a 4.2-in. AC overlay plus a crack relief fabric after cracking and seating the pavement.

Only seven examples could be developed for AC overlays of fractured PCC slab pavements, so it is difficult to judge the adequacy of the design procedure. The limited results show that the required AC overlay thickness of fractured slab PCC appears reasonable for most projects and generally agrees with the state recommendations. A comparison of AASHTO overlay design thicknesses at 95 percent reliability versus overlay thicknesses recommended by state agencies is given in Figure 3 along with data points from the conventional AC overlays previously shown. Three rubblized designs in the Southwest show thicker overlays than state recommendations even when the layer coefficient was at its maximum 0.35 for crack/seat, which may indicate that a thinner AC overlay is adequate in warm climates for fractured slab pavements.

The backcalculated subgrade moduli were all divided by 4 (C = 0.25), which is apparently needed to give appropriate overlay thicknesses. One section in the Northeast that had a CBR of 15 (and a corresponding estimated modulus of 12,000 psi) had very thin overlay thickness requirements. It is believed that the subgrade modulus is too high for this design.

The design reliability level is very significant. For these projects, a design reliability level of 90 to 95 percent appears to provide reasonable overlay thicknesses and in general agrees with agency recommendations.

EXAMPLE AC AND BONDED PCC OVERLAY DESIGN FOR JPCP AND JRCP

Table 3 gives an example of AC overlay and bonded PCC overlay design for a PCC pavement (MW-7). For a range of reliability levels from 50 to 99 percent, the required future structural capacity D_f is determined by varying trial D_f values until the ESALs computed using the AASHTO rigid pavement design equation (from Part II of the guide) match the design ESALs for the overlay. The overall standard deviation S_0, initial serviceability P1, and terminal serviceability P2 were set at 0.35, 4.5, and 2.5, respectively, unless other values were given by the state highway agency.

The effective dynamic k value and PCC elastic modulus were backcalculated whenever deflection data were available. The static k value used to determine D_f was obtained by dividing the dynamic k value by a factor of two (*3*). The PCC modulus of rupture was estimated from the backcalculated PCC elastic modulus unless another value was given by the state agency.

The two methods available for determining the effective structural capacity D_{eff} of the existing pavement for bare PCC pavements are the condition survey method and the remaining life method. However, past traffic data were not provided for any of the PCC pavement examples submitted, so the remaining life method could not be applied. For the condition survey method, D_{eff} is determined by multiplying the existing slab thickness by a joints and cracks condition factor F_{jc}, a fatigue factor F_{fat}, and a durability factor F_{dur}, which are se-

TABLE 2 Example AC Overlay Design for Fractured PCC Pavement

```
REVISED CHAPTER 5 AASHTO DESIGN GUIDE OVERLAY DESIGN
==========================================================================
SW-6 AC OVERLAY OF CRACKED/SEATED JPCP (PROJ STN 353)
==========================================================================
EXISTING PAVEMENT DESIGN
RUBBLIZED PCC                    8.20 inches
C.T.BASE                         3.70
SUBBASE                          0.00
TOTAL THICKNESS                 11.90
Future design lane ESALs  = 7370000 (2/3 OF 11000000 USED AS FLEXIBLE ESALs)
==========================================================================
DETERMINE SNf
  Vary trial SNf until computed ESALs equal future design ESALs.
     SNf  MR,psi        R        Z       So       P1       P2     ESAL
    4.50    4350       50        0     0.49      4.5      2.5  7364787
    5.15    4350       80    0.841     0.49      4.5      2.5  7516147
    5.50    4350       90    1.282     0.49      4.5      2.5  7452560
    5.80    4350       95    1.645     0.49      4.5      2.5  7401524
    6.40    4350       99    2.327     0.49      4.5      2.5  7354079
   TRIAL                     INPUT    INPUT    INPUT    INPUT
==========================================================================
DETERMINE SUBGRADE MR BY NDT METHOD
   Vary trial Ep/MR until computed D0 equals actual value.
                   ACTUAL          SUBGRADE              TRIAL COMPUTED
  STATION LOAD,lbs D0,mils  Dr,mils   MR,psi C FACTOR    Ep/MR  D0,mils   Ep,psi
            8952     6.31     3.43    17399        4    44.00     6.32   765574
                      r =       36 inches
        Check r > 0.7 ae =    29.70 inches
==========================================================================
DETERMINE SNeff
LAYER                       STR COEF  DRAIN m    SNeff
RUBBLIZED PCC                   0.35     1.00     2.87
C.T.SUBBASE                     0.15     1.00     0.56
SUBBASE                         0.00     1.00     0.00
                                       SNeff =     3.43
==========================================================================
DETERMINE OVERLAY THICKNESS      AC OL structural coefficient =    0.44

          DESIGN          CONDITION
        RELIABILITY       METHOD,in
             50               2.44
             80               3.92
             90               4.72
             95               5.40
             99               6.76
==========================================================================
```

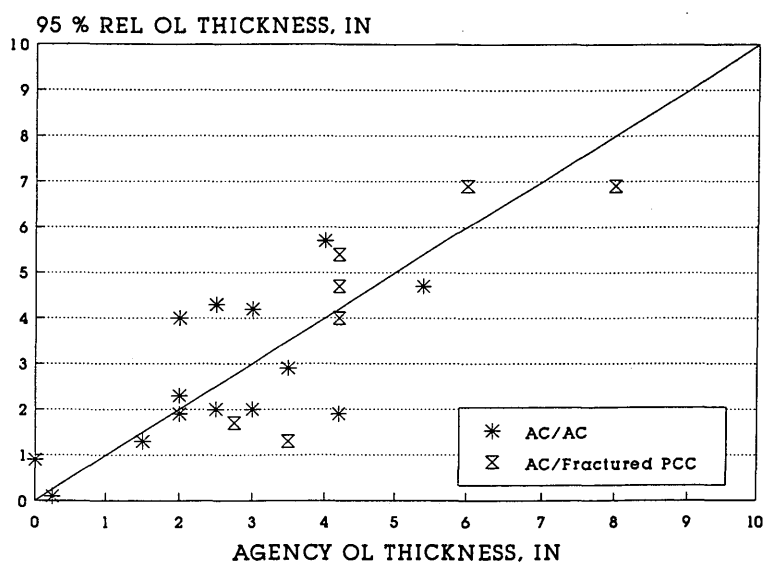

FIGURE 3 Comparison of AASHTO AC overlay thicknesses and agency AC overlay thicknesses for fractured PCC pavements (95 percent reliability).

TABLE 3 Example AC Overlay and Bonded PCC Overlay Design for JRCP and JPCP

```
REVISED CHAPTER 5 AASHTO DESIGN GUIDE OVERLAY DESIGN
==============================================================================
MW-7 AC AND BONDED PCC OL OF EXISTING JRCP
==============================================================================
EXISTING PAVEMENT DESIGN AND FUTURE TRAFFIC
Slab thickness     10.00 (in)
Future design lane ESALs =        10000000   (10 YEARS)
==============================================================================
BACKCALCULATION OF Keff AND Ec

  INPUT    INPUT    INPUT    INPUT    INPUT          RADIUS
   LOAD      D0      D12      D24      D36     AREA RELSTIFF    Kdyn SLAB Ec
  (lbs)   (mils)   (mils)   (mils)   (mils)    (in)    (in)    (pci)   (psi)

  11144     4.39     3.97     3.49     3.01   30.51   36.16      239 4.8E+06
  10864     4.90     4.57     4.18     3.70   31.96   45.36      133 6.6E+06
  10928     4.51     4.09     3.69     3.14   30.88   38.12      206 5.1E+06
  10824     4.55     4.17     3.77     3.30   31.29   40.58      179 5.7E+06
                                                                  189 5.6E+06
==============================================================================
DETERMINE Df

   Vary trial Df until computed ESALs equal future design ESALs.

          INPUT            INPUT    INPUT            INPUT    INPUT    INPUT
   Keff       J      Sc       P1       P2      Ec       So      LOS       Cd
(psi/in)          (psi)                     (psi)
     95     3.5     730      4.5      2.5 5.6E+06     0.35     0.00     1.00

  TRIAL                    COMPUTED
     Df       R       Z       ESALs
   (in)                   (millions)
   8.70      50       0     9973718
   9.70      80    0.84    10214587
  10.30      90   1.282    10619093
  10.80      95   1.645    10851107
  11.60      99   2.327    10095415
==============================================================================
DETERMINE Deff
  INPUT  Fjc =     0.97 (10 FAILURES/MI UNREPAIRED)
  INPUT  Ffat =    0.95 (50 MIDSLAB WORKING CRACKS)
  INPUT  Fdur =    1.00

Deff (in) =  Fjc * Fdur * Ffat * Dexist =    9.22
==============================================================================
DETERMINE OVERLAY THICKNESS

        RELIABILITY       PCC BOL PCC to AC  AC OL
          LEVEL             THICK   FACTOR   THICK
             50              0.00     0.00    0.00
             80              0.48     2.15    1.04
             90              1.09     2.07    2.24
             95              1.59     2.01    3.18
             99              2.39     1.91    4.56
==============================================================================
```

lected on the basis of the reported condition of the existing pavement. The ranges and recommended values for these factors are described elsewhere (*2*).

For each reliability level considered, the required bonded PCC overlay thickness is equal to the structural deficiency, obtained by subtracting the effective structural capacity D_{eff} from the required future structural capacity D_f. The required AC overlay thickness is equal to the bonded PCC overlay thickness multiplied by a factor that is a function of PCC thickness deficiency. This factor decreases as the PCC thickness deficiency increases and so is different for each reliability level considered.

For this example, the overlay design procedures yield an AC overlay thickness of 3.3 in. at the 95 percent reliability level. The state policy design is a 3.25-in. AC overlay for a 10-year design life.

The revised AASHTO overlay design procedures produce reasonable conventional AC overlay and bonded PCC overlay thicknesses for jointed PCC pavements that are consistent with state recommendations, as shown in Figure 4. The procedures are also consistent with state recommendations in identifying when no overlay is required for a pavement.

Specific difficulties in AC and bonded PCC overlay thickness design include the sensitivity of the J factor for load

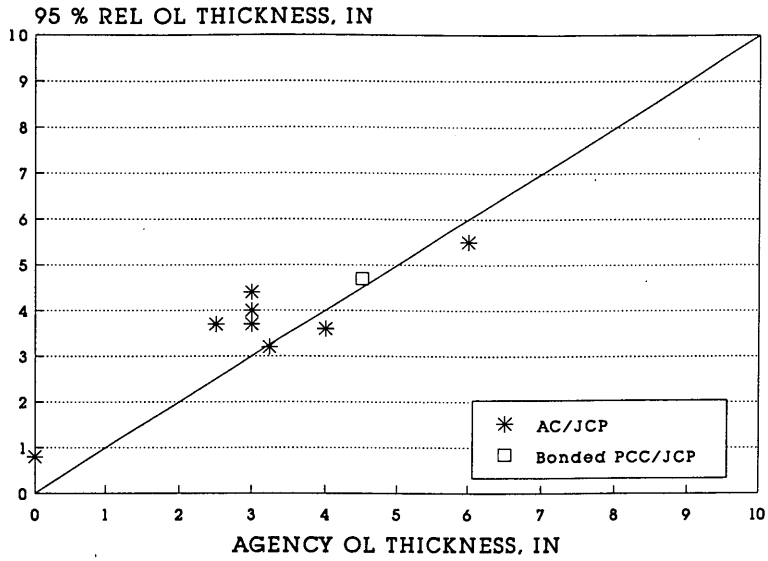

FIGURE 4 Comparison of AASHTO and agency AC overlay thicknesses and bonded PCC overlay thicknesses for JRCP and JPCP (95 percent reliability).

transfer and the necessity of imposing practical minimum and maximum values for the PCC elastic modulus, the PCC modulus of rupture, and the effective *k* value.

The design reliability level is very significant. Most of the projects were Interstate-type highways. A design reliability level of 95 percent appears to be reasonable for AC overlays of JRCP and JPCP.

A few examples yielded overlay thicknesses that appeared to be excessive. These examples were located in the Southwest region, in a state with a very mild climate, which may have a significant effect on improving overlaid pavement performance and reducing overlay thickness requirements. This could be addressed by using a lower design reliability level or by using a lower J factor to determine D_f.

EXAMPLE AC AND BONDED PCC OVERLAY DESIGN FOR CRCP

Table 4 gives an example of AC overlay and bonded PCC overlay design for CRCP (MW-9). The state agency's design procedure indicates a 6.2-in. AC overlay is needed for this pavement. However, the state's policy design is a 3.25-in. AC overlay.

The design procedure for AC and bonded PCC overlays of CRCP is the same as for JPCP and JRCP. The key difference is that a lower J (load transfer) factor is needed to produce a reasonable overlay thickness. The appropriate J factor also seems to vary from state to state, so each agency needs to determine its own value for J.

The revised AASHTO overlay design procedures produce reasonable AC overlay and bonded PCC overlay thicknesses for CRCP consistent with state recommendations, provided different reliability levels are used. For AC overlays, a reliability level of 95 percent produces overlay thicknesses comparable with state recommendations. For bonded PCC overlays, a reliability of 99 or greater is needed to match state recommendations. Figure 5 shows the comparison between design overlay thickness and agency recommendations for these levels of reliability.

The examples illustrate the importance of condition data and deflection data for overlay design. The condition factor F_{jc}, which indicates the amount of pavement deterioration left unrepaired before overlay, has a significant effect on the overlay thickness requirement. Agencies will find that much greater overlay thicknesses are required to meet desired performance lives if overlays are placed without adequate preoverlay repair. Most agencies specified thorough repair for the CRCP examples submitted.

The design reliability level is very significant. Most of the projects were Interstate-type highways. A design reliability level of 95 percent appears to be reasonable for AC overlays. Bonded PCC overlays appear to be designed at a 99 percent reliability level.

EXAMPLE AC OVERLAY DESIGN FOR AC/PCC PAVEMENT

Table 5 gives an example AC overlay design for an existing AC/PCC pavement (MW-15). The AC modulus was determined from diametral resilient modulus tests on AC cores from the pavement, adjusted to account for the difference between the laboratory testing frequency and the FWD loading frequency. The resilient modulus tests at 70°F and 90°F were used along with the deflection data to assign an appropriate AC mix temperature to each of the deflection basins. Then, using the backcalculation procedure described elsewhere (*2*), the maximum deflection d_0 and deflection basin AREA of the PCC slab were computed and used to backcalculate the effective dynamic *k* value and PCC elastic modulus.

TABLE 4 Example AC Overlay and Bonded PCC Overlay Design for CRCP

```
REVISED CHAPTER 5 AASHTO DESIGN GUIDE OVERLAY DESIGN
==============================================================================
MW-9 AC AND BONDED PCC OVERLAY OF EXISTING CRCP
==============================================================================
EXISTING PAVEMENT DESIGN AND FUTURE TRAFFIC
Slab thickness      8.00 (in)
Future design lane ESALs =        18000000 (5% ESAL GROWTH RATE, 10 YEARS)
==============================================================================
BACKCALCULATION OF Keff AND Ec

  INPUT    INPUT    INPUT     INPUT     INPUT          RADIUS
   LOAD      D0       D12       D24       D36     AREA RELSTIFF    Kdyn SLAB Ec
  (lbs)   (mils)   (mils)    (mils)    (mils)    (in)    (in)    (pci)   (psi)
   9000     4.05     3.60      3.04      2.48   29.35   31.22      280 6.1E+06
   9000     4.16     3.49       2.7      1.91   26.61   23.63      471 3.4E+06
   9000     3.49     3.04      2.59      2.14   29.04   30.12      349 6.6E+06
   9000     5.29     4.84      4.16      3.38   30.25   34.93      172 5.9E+06
                                                                    318 5.5E+06
==============================================================================
DETERMINE Df
   Vary trial Df until computed ESALs equal future design ESALs.

          INPUT              INPUT     INPUT            INPUT    INPUT    INPUT
   Keff       J        Sc       P1        P2       Ec       So      LOS       Cd
(psi/in)            (psi)                       (psi)
    159     2.2       727      4.5       2.8 5.5E+06     0.35     0.00     1.00

  TRIAL                      COMPUTED
     Df       R        Z        ESALs
   (in)                     (millions)
   7.40      50        0      17743150
   8.40      80     0.84      18815526
   8.90      90    1.282      18776535
   9.30      95    1.645      18436227
  10.10      99    2.327      17988918
==============================================================================
DETERMINE Deff
  INPUT  Fjc =      0.96
  INPUT  Ffat =     0.98
  INPUT  Fdur =     0.85 ("D" CRACKING)
Deff (in) =  Fjc * Fdur * Ffat * Dexist =     6.40
==============================================================================
DETERMINE OVERLAY THICKNESS

        RELIABILITY       PCC BOL PCC to AC  AC OL
          LEVEL             THICK   FACTOR   THICK
             50              1.00     2.08    2.08
             80              2.00     1.96    3.92
             90              2.50     1.90    4.76
             95              2.90     1.86    5.40
             99              3.70     1.79    6.63
==============================================================================
```

For PCC pavement with an existing AC overlay, the only method for determining D_{eff} is the condition survey method. The joints and cracks condition factor F_{jc}, durability factor F_{dur}, and AC quality factor F_{ac} are selected on the basis of available distress data. The ranges and recommended values for these factors are described elsewhere (*2*). In computing the effective structural capacity of the existing AC/PCC pavement, the AC surface thickness is divided by a factor of two to convert it to an equivalent thickness of PCC.

For each reliability level considered, the required AC overlay thickness is equal to the structural deficiency (obtained by subtracting the effective structural capacity D_{eff} from the required future structural capacity D_f) multiplied by a factor that is a function of PCC thickness deficiency. This factor decreases as the PCC thickness deficiency increases and so is different for each reliability level considered.

Only five examples could be developed for AC overlays of AC/PCC pavements, so it is difficult to judge the adequacy of the design procedure. The limited results show that the revised AASHTO overlay design procedure produces reasonable second AC overlay thicknesses that are consistent with state recommendations. The reliability level required to match the state recommendations is variable, however. This is not too surprising since agencies have little performance experience with second overlays.

All of the condition factors significantly affect overlay thickness, indicating that the amount of pavement deterioration left unrepaired before overlay has a significant effect on the

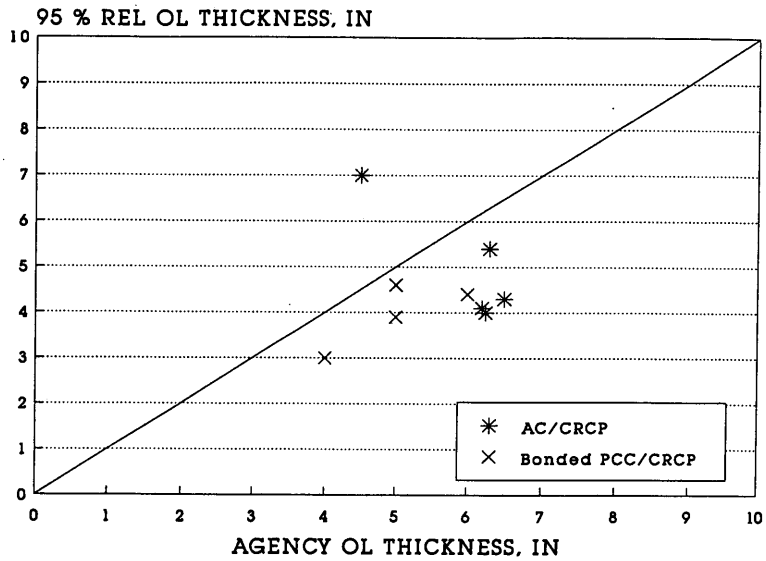

FIGURE 5 Comparison of AASHTO and agency AC overlay thickness (95 percent reliability) and bonded PCC overlay thickness (99 percent reliability).

overlay thickness requirement. Some existing AC/PCC pavements are very badly deteriorated due to PCC durability problems.

The design reliability level is very significant. A design reliability level of 90 to 95 percent appears to be reasonable for second AC overlays.

EXAMPLE UNBONDED PCC OVERLAY DESIGN FOR PCC PAVEMENT

Table 6 gives an example unbonded PCC overlay design for a PCC pavement (SW-19). The required future structural capacity D_f is determined using the PCC elastic modulus, PCC modulus of rupture, and J load transfer factor of the unbonded overlay. The design static k value used to determine D_f is the backcalculated effective dynamic k value of the existing pavement, divided by a factor of two.

The effective structural capacity D_{eff} of the existing pavement is obtained by multiplying the existing slab thickness by the joints and cracks condition factor F_{jcu}. For any given quantity of unrepaired deteriorated joints and cracks per mile, the F_{jcu} factor makes a smaller adjustment to the slab thickness than the F_{jc} factor, which is used for bonded PCC and AC overlay design, because unbonded overlays are much less sensitive to deteriorated joints and cracks in the existing slab than these other overlay types.

For each reliability level considered, the unbonded PCC overlay thickness required is the square root of the difference between the square of D_f and the square of D_{eff}. For the example pavement, the overlay design procedure yields 8.0 in. at the 90 percent reliability level and 8.7 in. at the 95 percent reliability level. The state's design procedure indicates that an 8-in. unbonded PCC overlay is needed.

Overall, it appears that the revised AASHTO overlay design procedures produce reasonable unbonded PCC overlay thicknesses that are consistent with state recommendations, as shown in Figure 6 for a reliability level of 95 percent. Only six unbonded overlay design examples could be developed from the project data submitted.

The unbonded overlay thicknesses were obtained using the original Corps of Engineers equation developed for airfields. An improved design methodology can and should be developed in the future to replace this empirical equation.

The design reliability level is very significant. Most of the projects were Interstate-type highways. A design reliability level of 95 percent appears to be reasonable.

EXAMPLE PCC OVERLAY DESIGN FOR AC PAVEMENT

Table 7 gives an example PCC overlay design for an AC pavement (SE-5). The required PCC overlay thickness is equal to the required future structural capacity D_f. The design static k value used to determine D_f is determined from the nomograph in Part II of the guide, using the total thickness of the existing pavement layers and the subgrade resilient modulus and effective pavement modulus backcalculated from deflections measured on the existing AC pavement.

For the example AC pavement, the state's design method indicated that a 6.4-in. PCC overlay was needed. The state constructed experimental sections of 6, 7, and 8 in. State recommendations were not available for the other PCC/AC examples developed.

The sensitivity of PCC overlay thickness to k value is small, as illustrated for one example project:

k value (psi/in.)	*PCC overlay thickness (in.) (R = 90 percent)*
147	9.9
147 * 2 = 294	9.5
147 * 4 = 588	9.0

TABLE 5 Example AC Overlay Design for AC/PCC Pavement

```
REVISED CHAPTER 5 AASHTO DESIGN GUIDE OVERLAY DESIGN
==================================================================================
MW-15 AC OVERLAY OF EXISTING AC/JRCP (I-74)
==================================================================================
EXISTING PAVEMENT DESIGN AND FUTURE TRAFFIC
AC layer thickness                3.00 (in)
Slab thickness                   10.00 (in)
Future design lane ESALs =          10000000 (20 years)
==================================================================================
BACKCALCULATION OF Keff AND Ec

AC temp      =                (deg F)
AC modulus   =     1,626,000 (psi) from lab tests of cores
AC/PCC       =             0 (0 for bonded, 1 for unbonded)

INPUT  INPUT   INPUT   INPUT   INPUT     AC    PCC    PCC RADIUS
 LOAD     D0     D12     D24     D36   AREA     D0   AREA RELSTIFF  Kdyn   SLAB Ec
(lbs) (mils)  (mils)  (mils)  (mils)   (in) (mils)   (in)    (in)  (pci)     (psi)
 9000   5.19    3.99    3.40    2.79  26.31   5.14  26.49   23.39    389   1.4E+06
 9000   3.82    3.20    2.85    2.38  28.74   3.77  29.02   30.06    324   3.1E+06
 9000   4.05    3.50    3.09    2.65  29.45   4.00  29.72   32.65    259   3.5E+06
 9000   3.84    3.19    2.80    2.41  28.48   3.79  28.76   29.19    341   2.9E+06

                                                                     328 2.7E+06
==================================================================================
DETERMINE Df
   Vary trial Df until computed ESALs equal future design ESALs.
          INPUT                INPUT    INPUT            INPUT   INPUT   INPUT
   Keff       J        Sc        P1       P2       Ec       So     LOS      Cd
(psi/in)              (psi)                      (psi)
    164     3.2         606       4.5      2.5 2.7E+06     0.39    0.00    1.00

  TRIAL                        COMPUTED
     Df       R         Z          ESALs
   (in)                        (millions)
   8.59      50         0        10066278
   9.73      80      0.84        10036274
  10.37      90     1.282        10036705
  10.92      95     1.645        10034620
  12.02      99     2.327        10048532
==================================================================================
DETERMINE Deff
  INPUT  Fjc  =       0.90 (50 unrepaired areas/mile)
  INPUT  Fdur =       0.90 (localized failures from "D" cracking)
  INPUT  Fac  =       0.95 (fair AC mixture)

Thickness of AC to be milled    =       0.50 (in)
Dac = Original Dac - milled Dac =       2.50 (in)

Deff = (Fjc*Fdur*Dexist) + (Fac*Dac/2.0) =              9.29 (in)
==================================================================================
DETERMINE OVERLAY THICKNESS

        RELIABILITY          PCC BOL PCC to AC  AC OL
          LEVEL                THICK   FACTOR   THICK
             50                 0.00     0.00    0.00
             80                 0.44     2.16    0.95
             90                 1.08     2.07    2.24
             95                 1.63     2.00    3.26
             99                 2.73     1.88    5.13
==================================================================================
```

TABLE 6 Example Unbonded PCC Overlay Design for PCC Pavement

```
REVISED CHAPTER 5 AASHTO DESIGN GUIDE OVERLAY DESIGN
==============================================================================
SW-19 UNBONDED JPCP OVERLAY OF JPCP
==============================================================================
EXISTING PAVEMENT DESIGN AND FUTURE TRAFFIC
Slab thickness      8.20 (in)
Future design lane ESALs =        11000000
==============================================================================
BACKCALCULATION OF Keff

  INPUT    INPUT    INPUT     INPUT     INPUT          RADIUS
   LOAD      D0      D12       D24       D36     AREA RELSTIFF     Kdyn SLAB Ec
  (lbs)   (mils)   (mils)    (mils)    (mils)    (in)     (in)    (pci)   (psi)

   9144     3.89     3.37      2.85      2.40   28.89    29.62      329 5.4E+06
   9088     3.89     3.33      2.81      2.31   28.50    28.40      355 4.9E+06
   9104     3.94     3.33      2.81      2.36   28.29    27.78      366 4.6E+06
   9128     3.94     3.42      2.85      2.40   28.75    29.17      334 5.1E+06
                                                                     346 5.0E+06
==============================================================================
DETERMINE Df
Unbonded overlay modulus of rupture (psi) =            700
Unbonded overlay modulus of elasticity (psi) =     4900000

   Vary trial Df until computed ESALs equal future design ESALs.

          INPUT              INPUT     INPUT            INPUT    INPUT    INPUT
   Keff       J       Sc       P1        P2       Ec       So      LOS       Cd
(psi/in)           (psi)                        (psi)
    173     4.0      700      4.5       2.5  4900000     0.35     0.00     1.00

  TRIAL                      COMPUTED
     Df       R       Z        ESALs
   (in)                     (millions)
   9.40      50       0      10972879
  10.50      80    0.84      11282235
  11.10      90   1.282      11337203
  11.60      95   1.645      11285326
  12.60      99   2.327      11235624
==============================================================================
DETERMINE Deff
  INPUT  Fjc =     0.94 (assume 100 deteriorated transverse cracks/mi)
  INPUT  Fdur =    1.00
Deff (in) =  Fjc * Fdur * Dexist =            7.71
==============================================================================
DETERMINE OVERLAY THICKNESS
        RELIABILITY          UBOL
          LEVEL             THICK
             50              5.38
             80              7.13
             90              7.99
             95              8.67
             99              9.97
==============================================================================
```

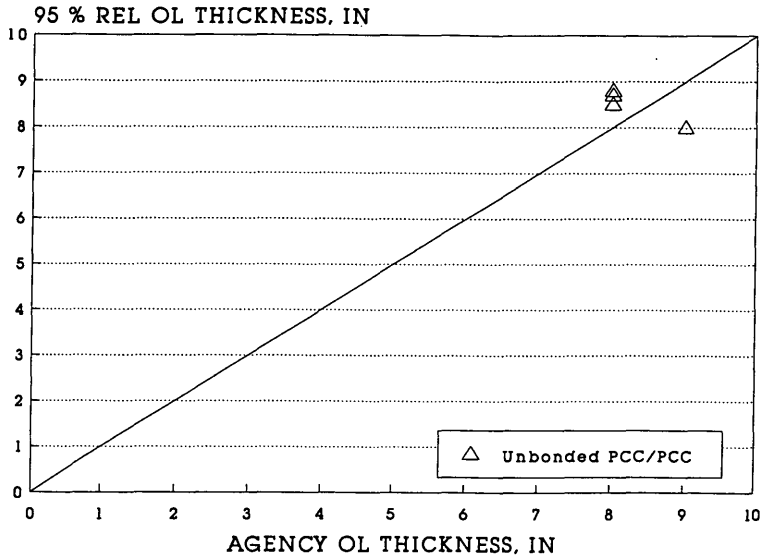

FIGURE 6 Comparison of AASHTO unbonded PCC overlay thicknesses and agency unbonded PCC overlay thicknesses for PCC pavement (95 percent reliability).

TABLE 7 Example PCC Overlay Design for AC Pavement

```
REVISED CHAPTER 5 AASHTO DESIGN GUIDE OVERLAY DESIGN
==============================================================================
SE-5 JPCP OVERLAY OF AC PAVEMENT (US 1)
==============================================================================
EXISTING PAVEMENT DESIGN AND FUTURE TRAFFIC

EXISTING PAVEMENT DESIGN
AC SURFACE          2.00          SUBGRADE:  SAND
CR STONE BASE       8.50
SUBBASE            12.00
TOTAL THICKNESS    22.50
Future design lane ESALs =        1100000
==============================================================================
DETERMINE Keff
   Vary Ep/Mr until actual MR*D0/P matches computed MR*D0/p.

                                   SUBGRADE          ACTUAL   TRIAL COMPUTED
STATION    LOAD   D0,in    Dr,in       MR          MR*D0/P   Ep/MR MR*D0/Ep     Ep
          (lbs)  (mils)   (mils)    (psi)
           9000   12.96     1.86    24604            35.43    0.80   35.63   19683
                    r =     47.2
        Check r > 0.7 ae   15.19

Using Figure 3.3, Part II:
Keff(dynamic) =     1200 psi/in   INPUT
Keff(static) =       600 psi/in
==============================================================================
DETERMINE Df
                                                 INPUT
PCC overlay modulus of rupture (psi)     =       635 (mean)
PCC overlay modulus of elasticity (psi) = 4000000 (mean)

   Vary trial Df until computed ESALs equal future design ESALs.

          INPUT              INPUT    INPUT            INPUT    INPUT    INPUT
   Keff       J       Sc       P1       P2       Ec       So      LOS       Cd
  (pci)           (psi)                      (psi)
    600     3.2      635      4.2      2.5  4000000     0.35     0.00     1.00

  TRIAL                      COMPUTED
     Df       R       Z         ESALs                Dol
   (in)                     (millions)             (in)
   3.80      50       0       1173786               3.80
   5.30      80    0.84       1127398               5.30
   5.90      90   1.282       1114201               5.90
   6.40      95   1.645       1108802               6.40
   7.40      99   2.327       1162870               7.40
==============================================================================
```

Additional work is needed to investigate effective k values for PCC overlays of AC pavements, including deflection testing on in-service PCC/AC pavements.

The design reliability level is very significant. Most of the projects were Interstate-type highways. A design reliability level of 95 percent appears to be reasonable for most projects.

CONCLUSIONS

Overall, the revised AASHTO overlay design procedures yield reasonable overlay thicknesses that are consistent with state highway agency recommendations. The following major points are made in regard to the field testing of the procedures.

Reliability level has a large effect on overlay thickness. On the basis of the examples developed, it appears that a design reliability level of approximately 95 percent gives thicknesses comparable with those recommended for most projects by the state agencies. Exceptions to this are bonded PCC overlays, which appear to be designed for a somewhat higher structural reliability. There are, of course, many situations for which it is desirable to design at a higher or lower level of reliability.

Some overlay projects were designed for huge traffic loadings (more than 25 million ESALs). Whereas thick concrete overlays should be able to handle this level of traffic, thick AC overlays may rut before their structural design life is achieved.

Designing AC overlays by the NDT method and designing AC overlays by the condition method produced similar results. However, the NDT method is believed to be the more accurate method and is highly recommended. The condition survey method, coupled with materials testing, can be developed to give adequate results.

For the very few example projects for which past traffic data were available, the remaining life method produced overlay thicknesses comparable with those produced by the NDT and condition survey methods. However, the remaining life method has some very significant limitations (*2*) and should be used with caution. Perhaps the most significant limitation of the remaining life method is that it cannot take into account the benefit of preoverlay repair.

It is apparent from the field testing results that different climatic and geographic regions require different overlay thicknesses, even if all other design inputs are exactly the same. The AASHTO guide does not provide a way to deal with this problem. Therefore, each agency will need to test the procedures on its pavements and determine their reasonableness and required adjustments. There are many ways to adjust the procedure to produce desired overlay thicknesses (e.g., reliability, resilient modulus, J factor, etc.).

ACKNOWLEDGMENT

This work was sponsored by the American Association of State Highway and Transportation Officials in cooperation with the Federal Highway Administration and was conducted in the National Cooperative Highway Research Program, which is administered by the Transportation Research Board of the National Research Council. The authors gratefully acknowledge the NCHRP staff and the NCHRP Project 20-7/Task 39 panel members for their guidance and assistance and the many state highway agency personnel who provided the project data and participated in the development of the examples.

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DISCUSSION

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This paper provides comparisons between overlay designs by a revised AASHTO approach and state highway agency recommendations. Among the conclusions drawn are (*a*) the revised AASHTO overlay design procedures yield reasonable overlay thicknesses which are consistent with state highway agency recommendations, (*b*) the reliability level has a large effect on overlay thickness (a design reliability level of 95 percent appears to be reasonable for most projects), and (*c*) because of climatic and geographic differences, each agency will need to test the procedures on its pavements and determine their reasonableness and required adjustments. This discussion highlights an important weakness in the proposed revised procedures, recommends the use of the original 1986 AASHTO overlay equation with a corrected formula for remaining life factor F_{RL}, and suggests procedures for using field tests to calibrate overlay design equations that include remaining life considerations.

BASIC OVERLAY DESIGN EQUATION

Details of the authors' revisions to 1986 AASHTO *Design Guide* (*1*) are given in a companion paper in this Record by them. An important deviation of the proposed revised procedures from the 1986 *AASHTO Guide* is the reverting to the use of traditional overlay equation $SC_{OL} = SC_y - SC_{xeff}$ and discarding the original remaining life concept that introduced a remaining life factor F_{RL} in the overlay equation. In a discussion of the companion paper, the following comments were made: (*a*) the authors' recommendation to set F_{RL} = 1.0, thereby reducing the 1986 AASHTO overlay equation to the traditional overlay equation, was based on an analysis that had a restrictive and weakly founded assumption on overlay performance and an incorrect procedure of computing F_{RL}, (*b*) the traditional overlay equation yields a lower bound overlay solution that underdesigns and is unconservative, and

(*c*) the traditional overlay equation is conceptually unsound and inadequate because overlay thickness is derived on the basis of the overlay requirement at the time of overlay application. It does not include an analysis to examine whether the overlay provided is adequate during other stages of overlay service life.

Esa (*2*) and Fwa (*3*) separately confirmed the fundamental correctness of the AASHTO overlay design approach that incorporates the concept of remaining life, and proposed different procedures to eliminate inconsistencies caused by a flaw in the F_{RL} calculation. Both identified the case with F_{RL} = 1.0 to be the lower bound overlay solution that assumes the rate of structural deterioration of an old pavement after overlay to be the same as that of a new pavement. Computationally, the traditional overlay equation is the same as this lower bound solution. The upper bound solution is one with $F_{RL} \leq 1.0$ where the old pavement after overlay is assumed to continue to deteriorate at a rate as if no overlay were applied. Easa adopted the original 1986 AASHTO design equations as the upper bound solution. He proposed using a linear combination of the lower and upper bound solutions (by means of weighting factors λ and $1 - \lambda$, $\lambda \leq 1$) to represent actual overlay performance and considered those λ values that produced consistent designs as feasible solutions.

Fwa (*3*) identified the flaw in the AASHTO formula for computing F_{RL} and derived a new F_{RL} expression in accordance with the concept of remaining life. When substituted into the 1986 AASHTO overlay equation, consistent results are obtained, and these represent the upper bound overlay solutions. Since the actual overlay requirement lies between the lower and upper bounds, a linear combination of the two solutions (by means of weighting factors α and $1 - \alpha$, $\alpha \leq 1$) was proposed. Since both the lower and upper bound solutions produce consistent overlay designs, the full range of α values give feasible overlay solutions.

The lower and upper bound analyses performed by Easa and Fwa provide a rational basis for overlay design that incorporates the fundamentally correct remaining life concept of AASHTO. Their studies also show that "the appropriate value of F_{RL} is 1.0" (see the companion paper) is not a valid claim. In the light of these findings, it appears logical for the authors to reconsider their decision to discard the 1986 AASHTO overlay equation.

USE OF FIELD TEST DATA

It is interesting to note that Easa (*2*) and Fwa (*3*) independently proposed very similar concepts of representing the deterioration of existing pavements after being overlaid, although their methods differ in the way the upper bound solutions are derived. Both methods contain a weighting parameter that requires calibration using field performance data. The field tests reported in the paper offer a good opportunity for this purpose.

As far as the three conclusions of the paper cited at the beginning of this discussion are concerned, it is believed that they would still hold because as Easa and Fwa have illustrated in their studies, the trends of variations of the feasible solutions are similar to the trend of the lower bound case (which is the solution given by the authors in the paper). However, some changes in the conclusion about the level of reliability are expected.

SUMMARY

The original 1986 AASHTO overlay equation with remaining life factor F_{RL} should be used instead of the traditional overlay equation. The AASHTO remaining life approach is fundamentally correct and technically sound, and recent studies show that it yields meaningful and consistent results. Field tests reported in the paper can be used to calibrate overlay equations for design procedures that incorporate the AASHTO concept of remaining life.

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AUTHORS' CLOSURE

The remaining life concept has not been discarded in the proposed revisions to the AASHTO overlay design procedures. As described in the paper, three procedures are given for estimating the effective structural capacity of an existing pavement: a deflection-based approach, a condition survey approach, and a remaining life approach.

The basic concept of remaining life is that a pavement's past traffic and its total traffic-bearing capacity over its lifetime may be used together to estimate the traffic the pavement is capable of carrying for the remainder of its life. This concept did not originate with the 1986 *AASHTO Guide*, but it has been used in pavement evaluation for many years and is applicable to any pavement design procedure based on a relationship between traffic and loss of structural capacity. Indeed, this concept is intrinsic to the AASHTO design methodology.

The authors consider the basic remaining life concept to be valid. However, the application of this concept in the proposed revisions to the AASHTO overlay design procedures differs from the application presented in the 1986 guide.

In the 1986 guide's overlay design procedures, procedures were given for determining the effective structural capacity (SC_{eff}) of a pavement from deflection testing or distress observations. This effective structural capacity is expected to be less than the original structural capacity of the pavement when new (SN_0). However, the 1986 guide's overlay design procedures then applied a traffic-based remaining life factor as a multiplier to the effective structural capacity determined from deflections or distress observations. This approach is widely considered to penalize a pavement twice for the same past traffic.

Fwa has defended this double penalty with the reasoning that if a deteriorated pavement with a given effective struc-

tural capacity is overlaid, it will subsequently deteriorate at a faster rate than a newly constructed pavement of the same structural capacity that receives the same thickness of overlay. This is a considerable distortion of the structural deficiency concept of overlay design. The essence of the structural deficiency concept is that a performance prediction model may be used to determine a required overlay, which will increase an in-service pavement's effective structural capacity to a structural capacity sufficient to carry the traffic expected over the design period. The rate of deterioration of the overlaid pavement is thus predicted by the performance model used, just as is the rate of deterioration predicted for new pavements by the same model. Within the context of the AASHTO design methodology, the flexible and rigid pavement performance models presented in Part II of the guide are used to determine required future structural capacity (structural number or slab thickness), and the rate of deterioration is measured by loss of serviceability as predicted by these models. If the two pavements described by Fwa have the same structural capacity before overlay, and receive the same overlay, then according to the structural deficiency concept their performance after overlay will be the same. One cannot correctly apply the structural deficiency concept of overlay design and at the same time conjecture a rate of deterioration of the overlaid pavement other than the rate predicted by the performance model used to define the structural deficiency.

In the proposed revisions to the overlay design procedures, a traffic-based estimate of remaining life is applied to a pavement's original structural capacity (SC_0) to estimate its current effective structural capacity but is not applied to deflection-based and condition-based estimates of the effective structural capacity. In concept, these three approaches for estimating SC_{eff} should yield similar results.

In addition to the conceptual flaw described earlier, the 1986 guide's remaining life computation was considered to be needlessly complex and poorly supported. For example, the procedure did not address the practical significance of a "negative remaining life" computed for an in-service pavement. The need to revise the application of the remaining life concept in the 1986 guide's overlay design procedures was identified by the AASHTO Joint Task Force on Pavements as one of the high-priority revisions to the overlay design procedures.

The authors have examined the work by Fwa and by Easa and have concluded that although they offer modifications to the remaining life method as presented in the 1986 guide, they do not correct its major flaw. They also impose needless complexity in the application of a simple concept.

The authors have therefore recommended to the Design Subcommittee of the AASHTO Joint Task Force on Pavements that the method developed by Elliott for considering remaining life be accepted as the best solution to the problems associated with the application of this concept in the 1986 overlay design procedures. It must also be clarified that decisions concerning acceptance of this and other proposed revisions to the overlay design procedures are made not by the authors but rather by the AASHTO Joint Task Force.

The opinions expressed in this paper are those of the authors and not necessarily those of AASHTO, FHWA, NCHRP, or TRB or of the individual states participating in the National Cooperative Highway Research Program.

Overlay Design Procedure for Pavement Maintenance Management Systems

ARIEH SIDESS, HAIM BONJACK, AND GABRIEL ZOLTAN

A methodology for flexible pavement rehabilitation and development of overlay thickness design curves for pavement maintenance management systems (PMMSs) is presented. The methodology is based on nondestructive testing of deflection basin measurements and on the rational approach that characterizes pavement response to major deterioration criteria, such as fatigue and rutting. Within the general framework, subgrade and pavement were classified into three categories of strength: weak, medium, and strong according to the measured deflection D_6 (at a distance of 1.80 m from the loading plate) and the surface curvature index parameter. With reference to these categories, a structural index (SI) was defined. Between the SI and overlay thickness there is a dependence that can be expressed by a design curves system related to different traffic categories and subgrade type. The present methodology can readily be incorporated as a subsystem within the general PMMS, and it enables fast solutions at the network level for economic evaluation and rehabilitation priority order determination of extensive road systems.

There are several main approaches to determining the overlay thickness of rehabilitated flexible pavements: (*a*) engineering experience and judgment, which in many cases is still being used to design overlays (*1*); (*b*) the standard overlay thickness for a given existing pavement type, traffic level, and other factors; (*c*) the empirical approach, which is based on the limited deflection criteria or correlation between the elastic deflection and pavement life (*2–6*); (*d*) the rational approach, which is based on the major deterioration criteria, such as fatigue and rutting [in this approach the pavement performance and its load response is expressed in terms of strains and stresses (*7–9*)]; and (*e*) the mechanistic approach, which is based on principles of fracture mechanics, to which the Paris law (*10*) is being applied that associates crack propagation with the stress intensity factor (*11–15*). The first two methods do not require evaluation tests of pavement materials and therefore are faster but not reliable. By contrast, the other methods demand destructive and nondestructive testing (NDT) for the evaluation of the properties and the performance of the pavement being rehabilitated.

The use of NDT is more common and widespread and nowadays is an integral part of the overlay design procedures applied in many institutions in the world (*4,6–9,16–19*). The design process carried out by these methods usually includes the following stages: (*a*) deflection basin measurements and their correction according to standards of load level, temperature, load frequency, and so forth (*1,2,16–18*); (*b*) properties derivation of the pavement layers by empirical analysis, using maximum deflection (DMD), surface curvature index (SCI), and base curvature index (BCI) (*5*) or structural analysis, based on the multilayer theory, which includes elastic moduli derivation by backcalculation programs (*7–9*); and (*c*) determining the relation between layer material properties and pavement performance in terms of pavement life by means of empirical deflection models (*2–6,16,19*) or semimechanistic models to predict pavement response and its resistance to deterioration criteria such as fatigue and rutting (*7–9*).

In pavement maintenance management systems (PMMSs) the rehabilitation solutions are subsystems within the whole system where additional indices exist, such as visual deterioration, pavement condition index (PCI), distress rating (DR), and roughness, for the purpose of economic evaluation and rehabilitation priority determination. Such a systems application for road system management of hundreds of kilometers requires fast and reliable representation of rehabilitation solutions at the network level. In this case the rational approaches, although reliable and advantageous, do not apply, because of the amount of work required by the tests to determine layer thickness, layer moduli derivation, and stresses and strains calculation to determine the rehabilitated pavement resistance to deterioration criteria.

This paper presents a methodology for the design of flexible pavement rehabilitation and development of overlay thickness design curves for PMMS. The methodology is based on NDT deflection basin measurements and on the rational approach for predicting the main deterioration criteria of fatigue and rutting. The rationale behind this methodology is that it is possible to characterize and express structural strength of the rehabilitated pavements by means of an index called structural index (SI), which is determined according to the measured deflection basins. Between this index and the required overlay thickness there is a mutual dependence, which can be expressed by curve systems for traffic categories and different subgrade types. The advantage of this method is that it can readily be incorporated as a subsystem within the overall PMMS to furnish reliable and fast solutions at the network level of extensive road systems for economic evaluation and rehabilitation priority order determination as shown elsewhere (*20,21*).

EVALUATION PARAMETERS AS INPUT DATA

The rehabilitation design methodology and overlay thickness design curves development are based on a data base of deflection basin measurements in different rehabilitated roads in Israel totaling 450 km in length and moduli derivation of

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the layers by means of backcalculation programs. The measured sections represent a wide range of pavement thicknesses and subgrade types. The deflection basins were measured at 100-m intervals by a Dynatest FWD 8002 model, which delivers an impulse load of 7 to 110 kN (1.15 to 24.2 kips). In each basin seven deflections were measured under an average load of about 75 kN (within the range of 70 to 80 kN) distributed uniformly on a circular load plate 300 mm in diameter. The deflections were measured at a distance of 0, 0.30, 0.60, 0.90, 1.20, 1.50, and 1.80 m (D_0, D_1, ..., D_6, respectively) from the center of the load plate. At the same time the asphalt layer temperature of each section was measured. Since measurements were performed at the end of the winter, temperatures ranged between 9°C and 22°C.

The moduli derivation of the pavement layers and the subgrade (the three-layer model) was carried out by a backcalculation program, DEFMOD (*20*). This program derives the moduli according to the data base approach (*22*) similar to MODULUS (*23*). The evaluation results of this software were compared with results of BISDEF (*23,24*), MODULUS (*23,24*) and ELMOD (*22*) and were found to be correct and reliable. The modulus of the asphalt layer, which was derived from the deflection basin, was corrected to fit the standard temperature of 30°C (86°F), according to the relationship proposed by Uzan et al. (*25*).

The measured deflections and layer moduli derived from them were used as input to develop a structural index for the pavement and overlay thickness design curves according to this index.

PROPOSED APPROACH FOR OVERLAY DESIGN PROCEDURE

NDT Correction to Fit Standard Conditions

The development of overlay design curves according to the proposed approach must be based on standard condition measurements. Therefore, the deflections were corrected by two factors for load level and temperature.

All the FWD measurements were performed within the load range 70 to 80 kN. Therefore, all the deflection basins were corrected according to the standard load of 75 kN in a linear manner as follows:

$$D_i^{cor} = D_i^m \times \frac{75}{P_m} \quad (1)$$

where

D_i^{cor} = corrected deflection for the standard load of 75 kN for the *i*th sensor,
D_i^m = measured deflection at P_m load for the *i*th sensor, and
P_m = load at the measurement time.

Because of the nonlinear behavior of the pavement layers, the measured deflection versus load level is also nonlinear. However, since most measurements were performed within the narrow load range and close to the reference load, the linear relation adopted in Equation 1 is precise.

The correction function of the central deflection, D_0 according to standard temperature of 30°C, was based on numerical analysis in which the modulus of the asphalt layer was determined in the temperature range 10°C to 50°C according to the relationship recommended by Uzan et al. (*25*). This function was corrected according to the temperature range 10°C to 20°C (the temperature range in which the deflection basins were measured) by means of the measured deflection basin and moduli derivation of the pavement layers. The regression function for the central deflection correction, $F_T(D_o)$, is expressed as follows:

$$F_T(D_o) = 1.694 - 3.155 \times 10^{-2} T_p + 3.286 \times 10^{-4} T_p^2 - 1.667 \times 10^{-6} T_p^3 \quad (2)$$

where T_p is the asphalt layer temperature at the time of measurement in degrees Celsius.

This correction was carried out only on the central deflection D_o (in addition to the load correction) of all the deflection basins as follows:

$$D_o(30°C) = F_T \times D_o(T_p) \quad (3)$$

The central deflection is highly sensitive to variations of temperature, whereas the other deflections are not affected by the temperature or the effect is negligible (*25*).

Subgrade Characterization and Classification

One approach to subgrade quantitative and qualitative evaluation from NDT measurements is by means of empirical indices, such as spreadability (*18*), SCI (*5*), BCI (*5*), and basin area (*8*). Figure 1 shows the relation between the modulus of the subgrade E_s derived by DEFMOD backcalculation and the measured deflection D_6, corrected to standard load of 75 kN (see Equation 1). The figure shows that irrespective of the subgrade type or pavement thickness, there is a correlation between the subgrade modulus and D_6 that allows the characterization and classification of the subgrade by that deflection. This fact is well known and is commonly used for subgrade modulus derivation (*19*). The regression that fits the relationship shown in Figure 1 is as follows:

$$D_6 = 8.588 \times 10^3 E_s^{-1.055} \quad (R^2 = 0.961) \quad (4)$$

where E_s is expressed in MPa and D_6 in μm.

On the basis of local experience, the variety of subgrade materials, and Equation 4, the subgrade was classified according to the recommended methodology into three quantitative and qualitative categories—weak, medium, and strong subgrade, as indicated in Table 1. Categories and values similar to those given in Table 1 were proposed by Scullion (*26*). Linear modification of Scullion's results for standard load of 75 kN leads to 93 μm and above for weak subgrade, 56 to 93 μm for medium subgrade, and below 56 μm for a strong subgrade.

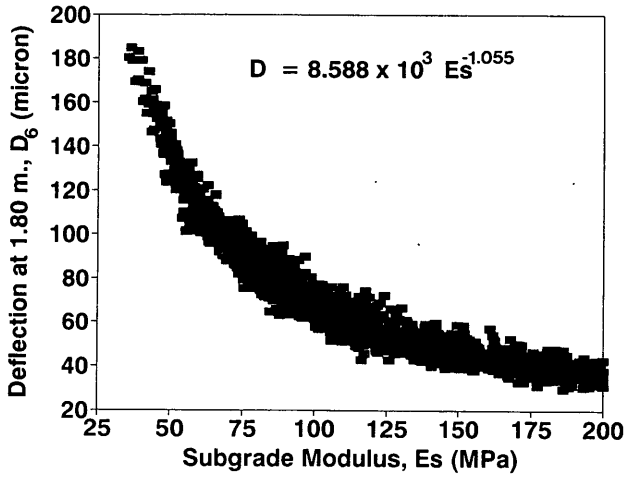

FIGURE 1 D_6 **deflection versus subgrade modulus** E_s.

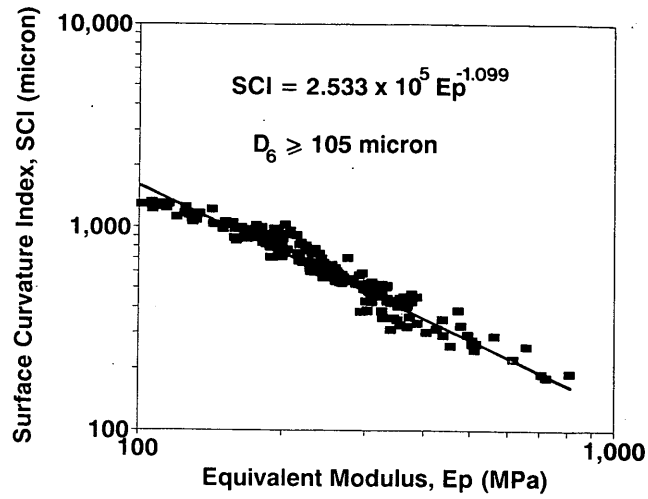

FIGURE 2 **SCI versus equivalent modulus** E_p **for weak subgrade.**

Pavement Characterization and Classification

The structural condition of the pavement is a combination of the thickness and strength of its layers. Therefore, a quantitative and qualitative characterization of the pavement according to NDT measurements requires the presentation of the above factors by means of a single index. For that purpose the SCI parameter and equivalent pavement modulus E_p expressed by the following equations were adopted:

$$SCI = D_0 - D_1 \tag{5}$$

$$E_p = \left(\frac{\sqrt[3]{E_A}\, h_A + \sqrt[3]{E_G}\, h_G}{h_A + h_G}\right)^3 \tag{6}$$

where

D_0 = central deflection corrected to standard load of 75 kN and temperature of 30°C;
D_1 = measured deflection at 0.3 m from the loading plate corrected to the standard load of 75 kN;
E_A, E_G = moduli of the asphalt (at standard temperature of 30°C) and the granular layers, respectively; and
h_A, h_G = thicknesses of the asphalt and granular layers, respectively.

Figures 2 through 4 show the relation between the equivalent modulus E_p derived from the deflection basin analysis and the SCI values for the three subgrade types, characterized respectively. The figures show that there is a correlation between E_p and SCI expressed by means of the following equations:

For a weak subgrade ($D_6 \geq 105$ μm),

$$SCI = 2.533 \times 10^5\, E_p^{-1.055} \qquad (R^2 = 0.936) \tag{7}$$

For a medium subgrade ($55 \leq D_6 < 105$ μm),

$$SCI = 3.425 \times 10^5\, E_p^{-1.152} \qquad (R^2 = 0.902) \tag{8}$$

For a strong subgrade ($D_6 < 55$ μm),

$$SCI = 1.974 \times 10^5\, E_p^{-1.052} \qquad (R^2 = 0.890) \tag{9}$$

where E_p is expressed in MPa and SCI in μm.

On the basis of local experience and a variety of materials, the pavement was classified according to the following three categories: weak pavement, $E_p \leq 200$ MPa; medium pave-

TABLE 1 **Subgrade Classification Based on NDT**

Subgrade Classification	Elastic Modulus (MPa)	C.B.R [a] (%)	D_6 [b] (micron)
Weak	≤ 65	≤ 4	≥ 105
Medium	65 - 120	4 - 8	55 - 105
Strong	> 120	> 8	< 55

[a] Based on the correlation Es=(15-16)CBR.
[b] According to Eq. 4.

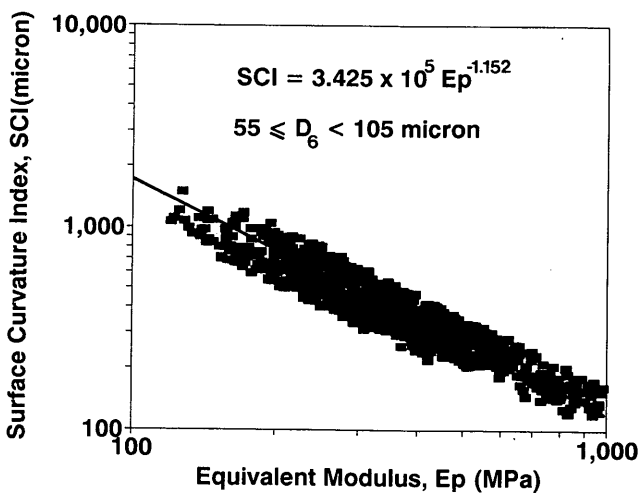

FIGURE 3 **SCI versus equivalent modulus** E_p **for medium subgrade.**

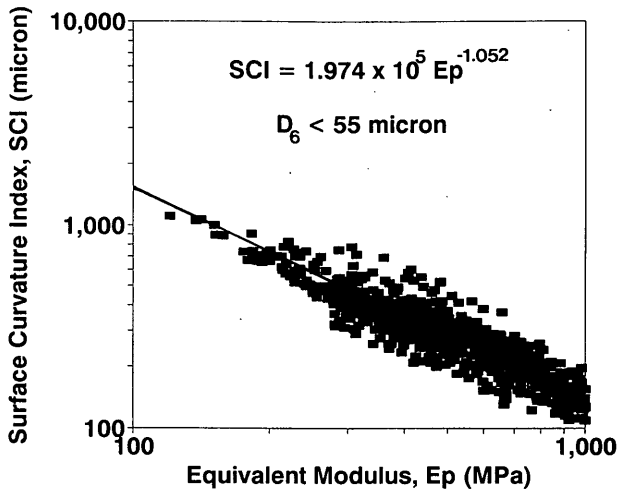

FIGURE 4 SCI versus equivalent modulus E_p for strong subgrade.

ment, 200 MPa < E_p ≤ 400 MPa; and strong pavement, E_p > 400 MPa.

Classification of the pavement according to the measured SCI determined by Equations 7 through 9 leads to identical results irrespective of the subgrade classification. Therefore, uniform criteria for pavement classification were determined as follows: weak pavement, SCI ≥ 750 μm; medium pavement, 350 μm ≤ SCI < 750 μm; and strong pavement, SCI < 350 μm.

Table 2 gives the criterion values for both subgrade and pavement classification according to the correction deflection basin measurements.

Structural Index

To express the pavement classification in a quantitative manner, SI was adopted. This is an index within the range 0 to 1.0 that expresses the performance condition of the rehabilitated pavement in terms of remaining life. SI determination for any subgrade type was based on the following assumptions:

1. For any pavement in the strong category (SCI ≤ 350 μm), SI is equal to 1.0.
2. The SI of a pavement in the other categories expresses its remaining life relative to identical pavement with an SI of 1.0. For instance, an SI of 0.3 means that the pavement's remaining life in terms of number of load application to failure, N_f according to the proposed criterion, is 30 percent relative to pavement of identical thickness with an SI of 1.0.
3. The remaining life of the actual pavement layers (the three-layer model) is determined according to the criterion of compressive strain at the top of the subgrade. For that purpose, the model of Verstraten et al. (*27*) was adopted. Since the SI refers to the measured deflection basin, the load that served for calculation was 75 kN distributed on a circular plate with a diameter of 300 mm. Accordingly, the SI is used as follows:

$$SI = \frac{N_f(SCI > 350\ \mu m)}{N_f(SCI = 350\ \mu m)} \tag{10}$$

In Figures 5 through 7 the relationship between the SCI parameter and the SI for different subgrade categories is shown. The SCI of any pavement is determined according to Equations 7 through 9. These figures show that it is possible to define the structural index depending on SCI by three distinct ranges as follows:

1. For SCI values smaller than 350 μm (strong pavement), the SI value equals 1.0. This range was determined by the assumption.

TABLE 2 Subgrade and Pavement Classification Based on NDT (micron)

Pavement Classification	Subgrade Classification: Weak	Medium	Strong
Weak	D_6 ≥105 SCI ≥750	55≤ D_6<105 SCI≥750	D_6<55 SCI ≥750
Medium	D_6 ≥105 350≤ SCI <750	55≤ D_6<105 350≤ SCI <750	D_6<55 350≤ SCI <750
Strong	D_6 ≥105 SCI <350	55≤ D_6<105 SCI <350	D_6<55 SCI <350

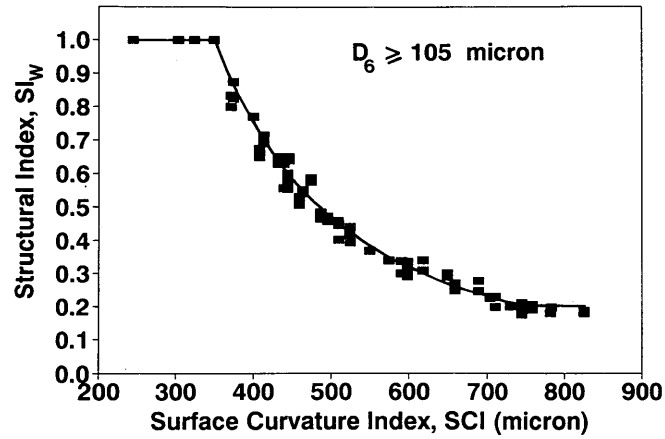

FIGURE 5 SI_W as a function of SCI for weak subgrade.

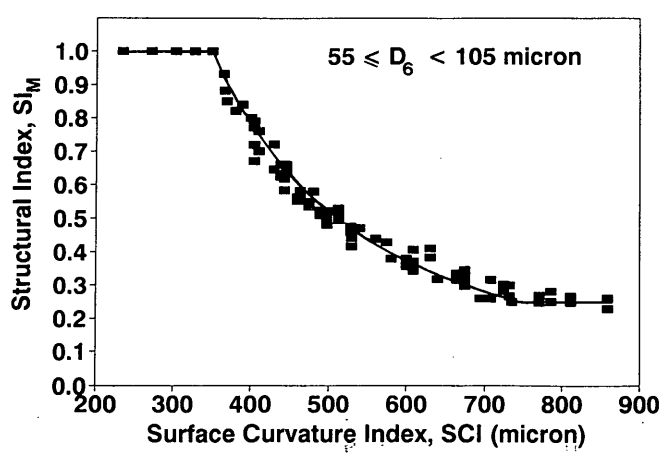

FIGURE 6 SI_M as a function of SCI for medium subgrade.

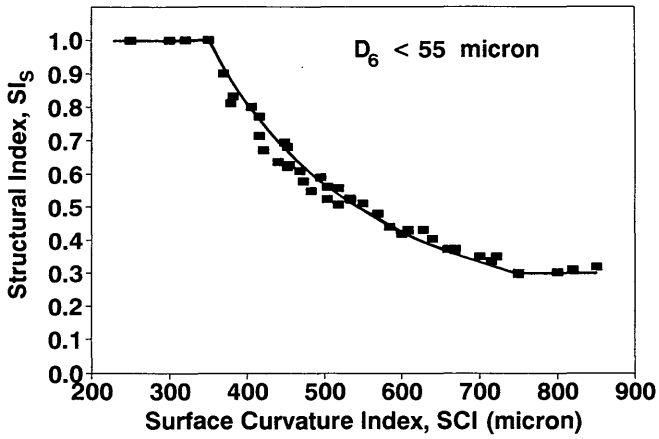

FIGURE 7 SI_S as a function of SCI for strong subgrade.

2. For SCI values between 350 and 750 μm (medium pavement), the SI values decrease in a nonlinear manner as the SCI parameter increases.
3. For SCI values greater than 750 μm (weak pavement), the SI value is constant.

The range definitions for all the subgrade types were expressed by the following regression equations.

For a weak subgrade ($D_6 \geq 105$ μm),

$$SI_W = \begin{cases} 0.2 & SCI \geq 750\,\mu m \\ 2.361 \times 10^5 SCI^{-2.112} & (R^2 = 0.987) \quad 350 \leq SCI < 750\,\mu m \\ 1.0 & SCI < 350\,\mu m \end{cases} \tag{11}$$

For a medium subgrade ($55 \leq D_6 < 105$ μm),

$$SI_M = \begin{cases} 0.25 & SCI \geq 750\,\mu m \\ 4.243 \times 10^4 SCI^{-1.819} & (R^2 = 0.975) \quad 350 \leq SCI < 750\,\mu m \\ 1.0 & SCI < 350\,\mu m \end{cases} \tag{12}$$

For a strong subgrade ($D_6 < 55$ μm),

$$SI_S = \begin{cases} 0.3 & SCI \geq 750\,\mu m \\ 1.046 \times 10^4 SCI^{-1.580} & (R^2 = 0.988) \quad 350 \leq SCI < 750\,\mu m \\ 1.0 & SCI < 350\,\mu m \end{cases} \tag{13}$$

There is no relation between the numerical value of SI in the different subgrade categories. Identical SI values in different subgrade categories are not equivalent and do not express identical strength of the pavement. The reason for this lies in the definition and determination of this index according to Equation 10.

A similar index for subgrade and pavement categories identical to those shown was developed by Scullion (*26*), who assigned each subgrade category six separate structure strength index groups for all the pavement categories.

Overlay Thickness as a Function of SI

The rational approach (*7–9*) for overlay thickness design relies on the resistance of the rehabilitated pavement to the main deterioration criteria such as fatigue and rutting. The SI indicates the strength of the pavement and provides evaluation about the structural performance of the pavement. It is therefore possible to present rehabilitation curves on the basis of the rational approach depending on the SI index. The development of overlay thickness design curves was based on the following principles:

1. Several dozen structures within the weak, medium, and strong pavement categories with a wide range of SI were adopted. The total pavement thickness was between 300 and 600 mm with asphalt thickness of 60 to 200 mm.
2. The subgrade modulus (E_S) representing the weak subgrade groups ($E_S \leq 65$ MPa) was determined as 650 MPa ($D_6 = 115$ μm according to Equation 4). The modulus representing the medium subgrade groups (65 μm $< E_S \leq 120$ μm) was determined as 90 MPa ($D_6 = 75$ μm). For the strong subgrade groups ($E_s > 120$ MPa) the representative modulus was determined as 120 MPa ($D_6 = 55$ μm).
3. For each case the pavement and subgrade were classified according to D_6, and SCI and SI were calculated according to the principles demonstrated in previous sections.
4. A constant load distribution representing the rural mixed traffic in Israel was adopted. The load range is between 20 and 180 kN for a single axle (*20*). The effect of that mixed traffic to determine overlay thickness was taken into account according to the Miner hypothesis (*28*). After having calculated the overlay thickness and to express the mixed traffic levels in the overlay design curves (Figures 8 through 10) by means of a single traffic number (for easier utilization), the various load levels were transformed to a number of equivalent applications of a 130-kN single-axle load, which is the design axle load in Israel. The transformation was performed by means of the load equivalency factor of the AASHO method (*1*).
5. Two deterioration criteria were adopted, fatigue and compressive strain at the top of the subgrade, used as rutting criteria. The fatigue criterion was adopted according to the model proposed by Finn et al. (*29*) with some modification referred to by Uzan and Gur (*30*). As a rutting criterion, the model of Verstraten et al. (*27*) was adopted. The overlay thickness for each rehabilitated pavement according to Miner's law is the thickness that fits the critical criterion between the two.

Figures 8 through 10 show overlay thickness design curves depending on SI for a weak, medium, and strong subgrade, respectively. The overlay thickness is presented for various equivalent 130-kN load applications. The required overlay thickness of 20 mm was defined as the surface treatment classification and the 10 mm as distress repair.

APPLICATION AND LIMITATION OF THE PROPOSED METHOD

The proposed methodology including all its principles and fundamental assumptions can be incorporated within PMMS

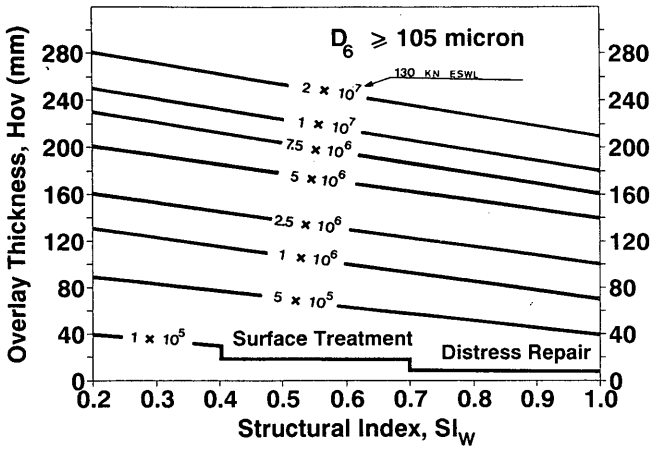

FIGURE 8 Overlay thickness versus SI_W for weak subgrade.

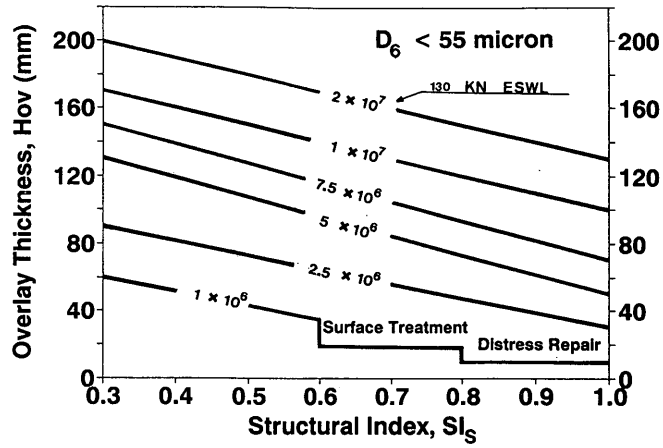

FIGURE 10 Overlay thickness versus SI_S for strong subgrade.

to provide fast rehabilitation solutions at the network level. The system was successfully applied to a road system totaling 900 km in length (*20,21*).

Figure 11 shows a general flowchart of the process for overlay thickness determination. The process stages may be summarized as follows:

1. Choose the rehabilitated section, measuring the deflection basin at suitable distances and asphalt layer temperatures.
2. Correct the central deflection D_0 for standard load of 75 kN (Equation 1) and standard temperature of 30°C (Equations 2 and 3).
3. Correct D_1 and D_6 for standard load of 75 kN.
4. Calculate the SCI parameter according to corrected deflections D_0 and D_1.
5. Determine the representative D_6 and SCI parameters of the section by statistical analysis.
6. Classify subgrade and pavement by D_6 and SCI parameters (Table 2).
7. Calculate the SI according to the subgrade and pavement classification (Equations 11 through 13).
8. Do traffic analysis for a design period.

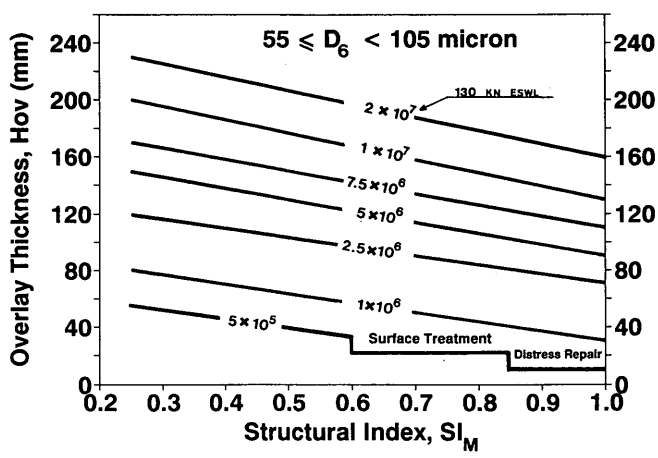

FIGURE 9 Overlay thickness versus SI_M for medium subgrade.

9. Determine the required overlay thickness according to subgrade type, SI, and traffic analysis (Figures 8 through 10).

The design curves shown in Figures 8 through 10 have two main limitations, which under certain circumstances may lead to conservative results or overdesign with reference to the rational approach. The two limitations mentioned in the fundamental assumptions may be summarized as follows:

- The curves were developed to different structure thicknesses up to 600 mm. The overlay thickness is affected considerably by the pavement thickness in the case where the critical criterion is rutting. In instances where the thickness is above 600 mm and the deterioration criterion is rutting, one must expect overdesign. These instances are characteristic of pavements based on weak subgrade.
- The subgrade modulus E_s representing the different subgrade categories was constant at 60, 90, and 120 MPa for a weak, medium, and strong subgrade, respectively (D_6 of 115, 75, and 55 μm, respectively). Therefore in cases where the subgrade modulus presented by the deflection value D_6 (see Equation 4) was radically different from the representative values, one must expect conservative results or overdesign.

These limitations are partially solved by statistical analysis for determining the representative basin properties or by calculating the overlay thickness for every single point within the section and choosing the required overlay thickness on the basis of statistically homogeneous units (*1*).

For the validation of the proposed procedure, a comparison of the overlay thickness calculated by the proposed procedure and by the rational approach was carried out. The results of the analysis showed good correlation between the solutions. The deviation of the results were in the range 0 to 20 mm, which is acceptable for solutions at network level.

The proposed method is not intended to replace the rational approach for pavement rehabilitation. To establish the approach it is necessary to develop modification factors to the said limitations and calibrate the method with extensive laboratory and field tests. However, its advantage lies in providing fast rehabilitation solutions at the network level in

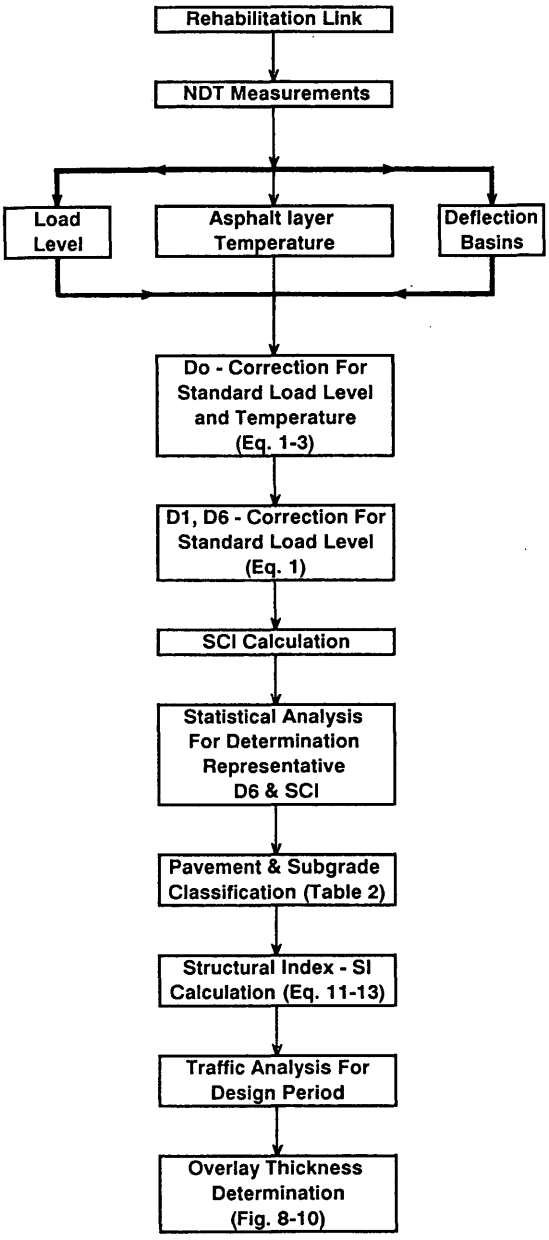

FIGURE 11 Flowchart of the overlay design procedure.

PMMS for economic evaluation and rehabilitation priority order determination of extensive road systems (*20,21*). The fast solutions at the network level save a lot of work required by the field tests in the determination of layer thickness, layer moduli derivations, and stress and strain calculations to predict pavement response.

SUMMARY

This paper presents a methodology for flexible pavement rehabilitation design and development of curves to determine the overlay thickness in flexible pavements. The methodology is based on NDT measurements and the rational approach for predicting the main deterioration criteria such as fatigue and rutting. The findings indicate that it is possible to classify the subgrade and pavement by the seventh deflection, D_6, and the SCI parameter and to express the structural pavement condition by the structural index, SI. Between this index and the overlay thickness there is a dependence, which makes itself evident in the overlay curves presented here. The advantage of the proposed procedure is in its application possibility as a subsystem within a general PMMS. This enables fast and reliable solutions at the network level of extensive road systems. Such solutions are decisive for economic evaluation and rehabilitation priority order determination.

ACKNOWLEDGMENTS

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Pavement Evaluation and Development of Maintenance and Rehabilitation Strategies for Illinois Tollway East-West Extension

ELIAS H. RMEILI, KURT D. JOHNSON, AND MICHAEL I. DARTER

The procedures used in the field evaluation and development of maintenance and rehabilitation strategies for the Illinois Tollway Authority's 70-mi, four-lane East-West Extension are presented. The extension consists of 14-in. portland cement concrete (PCC) placed directly on the silty clay subgrade with random joint spacing varying between 12 and 17 ft. Although the pavement is only 17 years old, it is experiencing rapid decrease in serviceability. The field evaluation consisted of a visual condition survey, nondestructive deflection testing including void detection and transverse joint load transfer efficiencies, coring, soil evaluation, determination of slab and soil properties, and petrographic analysis of the PCC. After the pavement evaluation was completed, several maintenance and rehabilitation strategies were developed for different life expectancies varying from 2 to 20 years. All of the strategies were evaluated using EXPEAR (EXpert system for Pavement Evaluation And Rehabilitation).

The results of an extensive pavement engineering evaluation and rehabilitation analysis of the Illinois Tollway East-West Extension on I-88 are presented. The East-West Extension begins approximately 4 mi west of the Fox River near Aurora and extends in a westerly direction for 69 mi to its terminus 1 mi east of Rock Falls. It was constructed during the 1972, 1973, and 1974 construction seasons and opened for traffic in fall 1974.

On the basis of the structural design and the traffic level, the East-West Extension was expected to provide up to 30 years of satisfactory performance with only moderate rehabilitation. However, it is experiencing a rapid decrease in the level of service, and a rehabilitation program will need to be implemented soon.

Interesting and useful information, which will help reduce future portland cement concrete (PCC) pavement design failures, was gained from this study.

OBJECTIVE

The objective of this investigation was to develop a cost-effective pavement rehabilitation program. The objective was achieved through a comprehensive engineering evaluation that answered the following critical questions about the current condition of the East-West Extension pavement:

- What are the extent and severity of the deterioration?
- What are the causes of the deterioration?
- Is the load-carrying capacity adequate for future operation or is structural improvement needed?
- What are the feasible rehabilitation alternatives?
- What are the expected life cycle costs of each feasible alternative?

BACKGROUND

The original design plans called for the typical pavement section to be built as a 10-in. concrete slab on a 4-in. cement-stabilized base on the subgrade. However, during construction, the typical section was modified to 14-in. concrete slab placed directly on the subgrade. The typical pavement sections consist of a nonreinforced concrete slab lying directly on the subgrade and randomly skewed joints with joint spacing ranging from 12 to 17 ft. The concrete shoulders are tied to the mainline slabs and decrease in thickness from 8 to 6 in. as they extend out from the traffic lanes. Portions of the East-West Extension were paved as one wide monolithic slab including the shoulders, whereas other portions were paved with the traffic lanes separate from the shoulders. The centerline joints and the longitudinal joints in the monolithic sections were formed by using a polyethylene tape embedded in the concrete.

DATA COLLECTION

The data collection efforts started with a visual survey of the pavement surface. After the survey, the cause of deterioration was not clearly defined, and it was determined that a complete PCC evaluation program must be implemented to clearly define the problem. The program included traffic information, nondestructive deflection testing (NDT) using the falling weight deflectometer (FWD), pavement coring and soil boring, and laboratory testing of the soil and PCC.

Traffic Information

The current average daily traffic (ADT) is 10,000 and the average daily truck traffic (ADTT) is 1,125 in two directions

ERES Consultants, Inc., 8 Dunlap Court, Savoy, Ill. 61874.

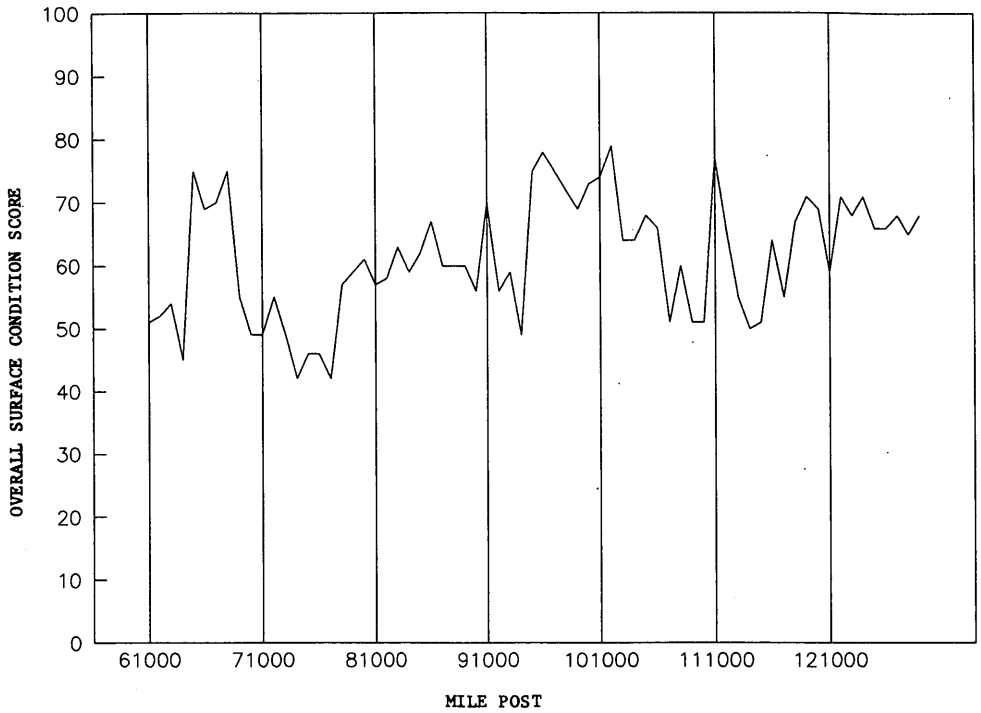

FIGURE 1 Eastbound visual rating.

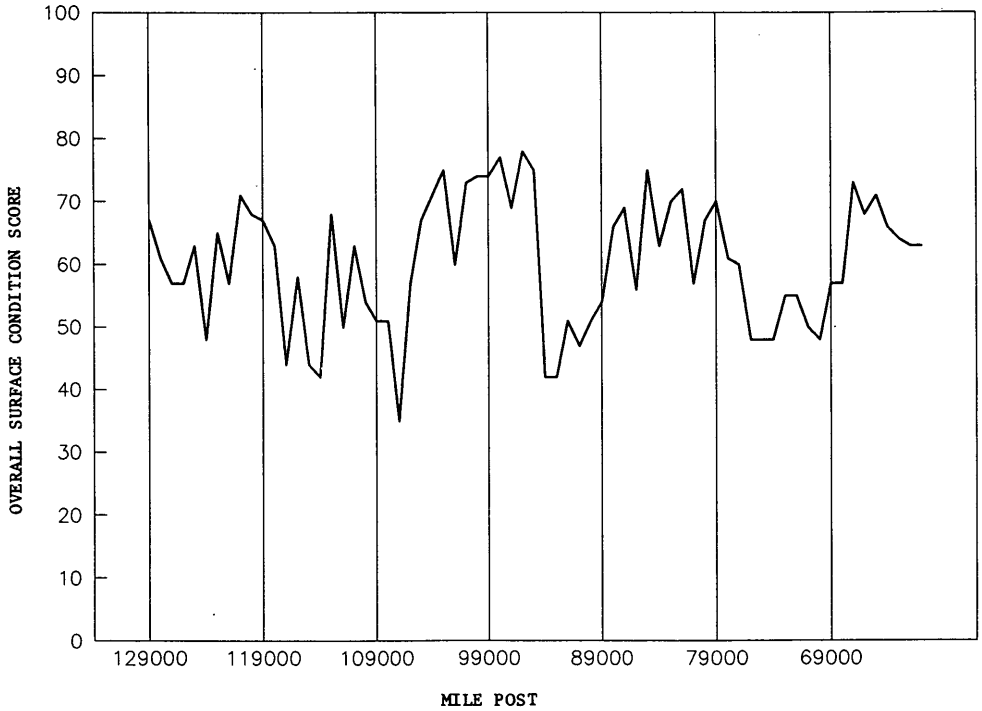

FIGURE 2 Westbound visual rating.

along most of the route (trucks include vehicles of 6 tons or greater). The total number of truck loadings on the East-West Extension was estimated using traffic data from 1988, 1989, and 1990. Assuming a yearly growth since 1974 of 7 percent, the average yearly transaction growth on the East-West Extension over the last 6 years, it is estimated that the East-West Extension has had approximately 3,895,000 truck applications or 5,452,441 equivalent single-axle loads (ESALs) in both directions (85 percent of the traffic in the driving lane and 15 percent in the passing lane).

Visual Condition Survey

The visual survey consisted of a detailed survey of the first 500 ft beyond the even mileposts in the direction of travel from Milepost 61 through Milepost 128. Information outside the 500 ft was gathered by driving at low speed on the shoulder. Conditions such as heave cracking, patched areas, broken slabs, and areas of notable in-slope erosion were visually inspected.

Several types of distress were present with the extent and severity varying significantly throughout the entire area. The predominant types of distress present were spalling of the transverse joints and spalling and cracking of the longitudinal joints in the driving lanes and on the shoulders. The overall condition scores for each direction are shown in Figures 1 and 2. A score of 100 means the pavement is in excellent condition, a score of 50 means the pavement is in fair condition, and a score of 0 means the pavement is failed.

The distress severity associated with the longitudinal joints is one of the most severe because both concrete durability spalling and longitudinal cracking exist. In several locations, the severity level of the centerline joint is so severe that it can interfere with vehicles going back and forth across the joint during lane changes.

Another major problem with the pavement was the spalling of the transverse joints. The severity varied from low to high, and in some locations the spalling was as wide as 12 in. on either side of the joint and as deep as 8 in. Figures 3 and 4 show the combined condition of the longitudinal and transverse joints throughout the project.

The entire pavement structure exhibits a nearly total failure of the transverse and longitudinal joints sealant.

The average faulting that was measured in the field was 0.123 in., and in some locations the measurement was as high as 0.27 in. A faulting of less than 0.25 in. is considered low severity.

NDT

NDT was performed using an FWD. The FWD is an impulse device that exerts a force similar in magnitude and duration to a moving vehicle tire load. By varying the weight and height from which it is dropped, the magnitude of the load can be changed. The resulting pavement deflection is measured by seven seismic deflection transducers, one of which is at the loading plate and the others at preset intervals from the loading plate.

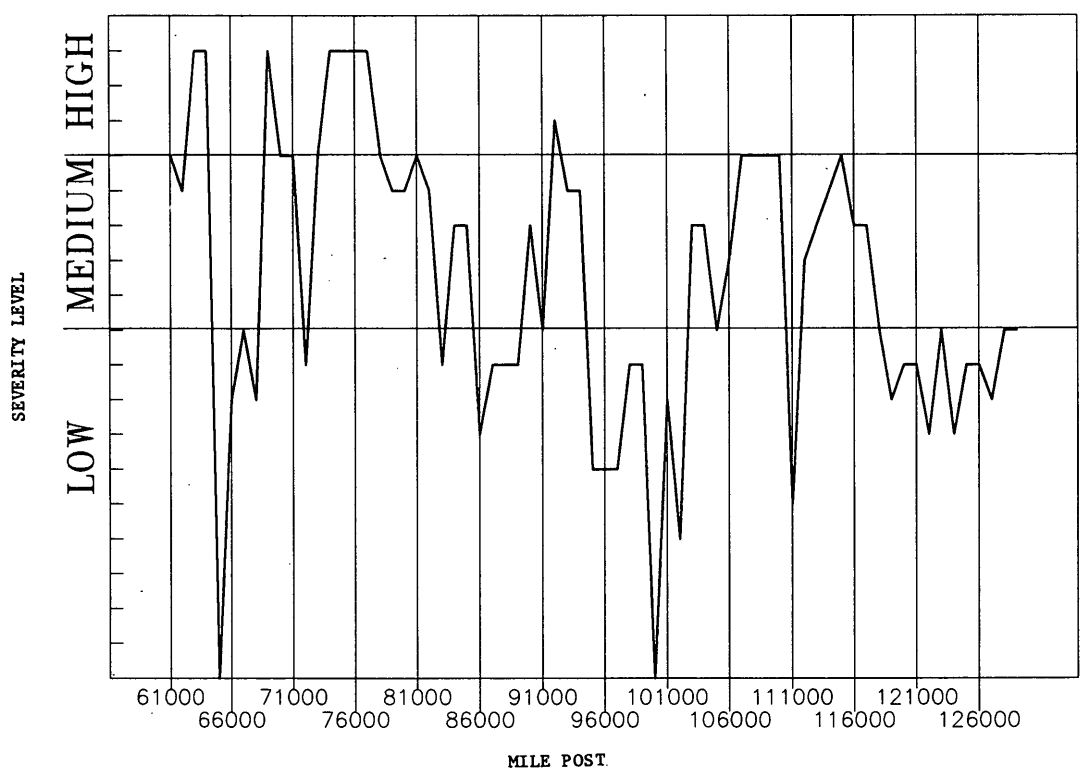

FIGURE 3 Eastbound visual rating of joints.

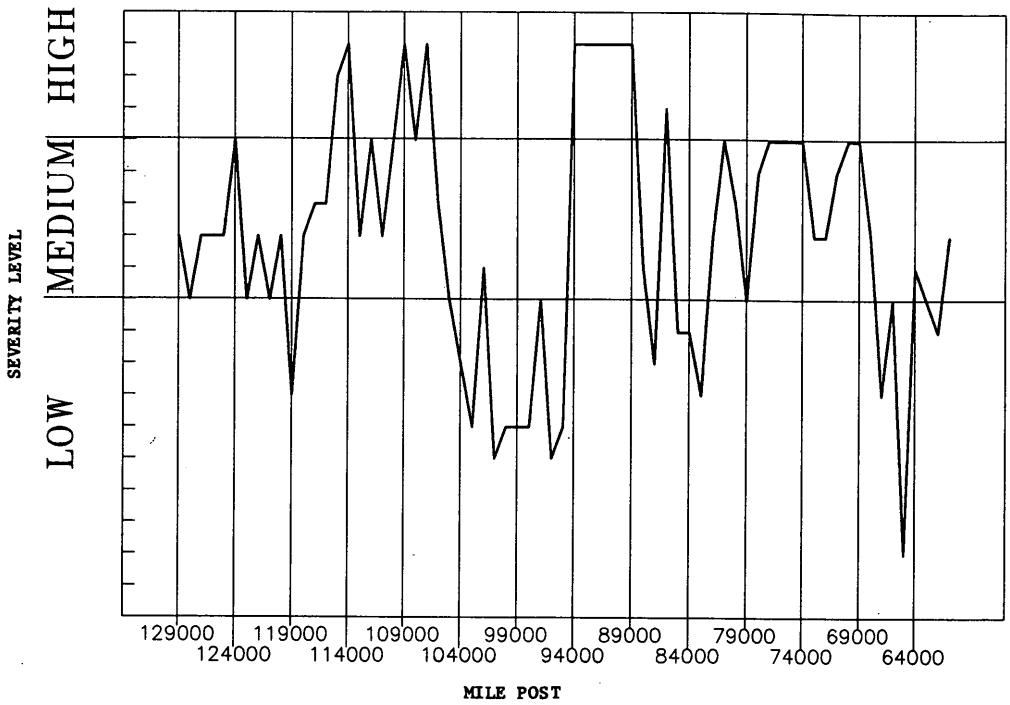

FIGURE 4 Westbound visual rating of joints.

The initial pattern called for each test site to have five center slab tests, three lead corner tests, and three lag corner tests as shown in Figure 5. However, after 3 days of testing with very little variability between test readings, the pattern was changed to four center slab, two lead corner, and two lag corner tests.

For the East-West Extension testing program, the weights were dropped from three heights to produce loads of approximately 8,000, 11,000, and 14,000 lbf. The deflections under the loading plate, 12 in. behind the plate, and at 12, 24, 36, 48, and 60 in. in front of the plate were recorded during testing.

Pavement load-deflection data were used to estimate the PCC slab modulus of elasticity, PCC modulus of rupture, transverse joint load transfer, loss of support underneath the slab, and foundation support (effective K-value beneath the PCC slab).

Coring

Concrete coring was performed to determine the extent of the joint deterioration, properties and thickness of the PCC

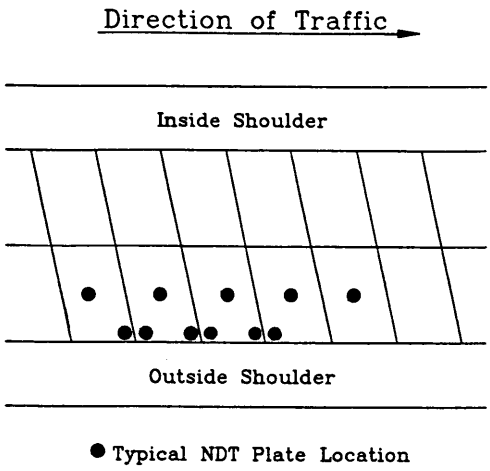

FIGURE 5 FWD testing pattern.

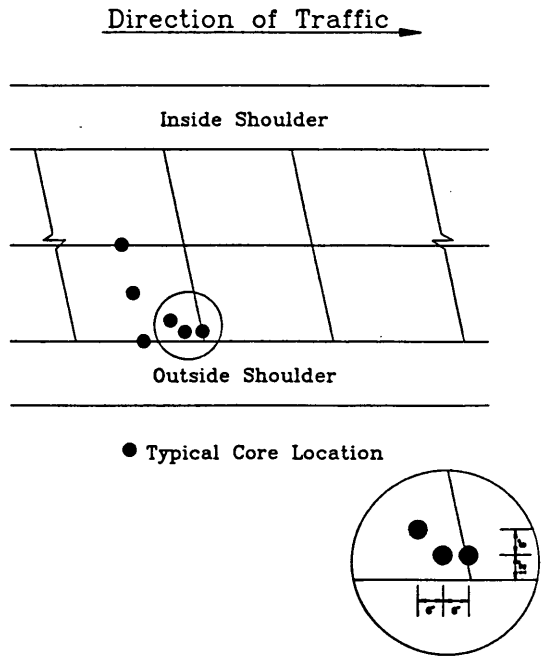

FIGURE 6 Concrete coring locations.

TABLE 1 PCC Thickness and Aggregate Type

MILE POST	THICKNESS, in.	AGGREGATE TYPE
67.5 EB	14.5	Buff and white crushed dolomite
76 EB	15.5	Buff and white crushed dolomite
92 WB	16	Gravel
98.5 WB	15	White dolomite
100 EB	14	White dolomite
112 EB	14.5	Buff and white crushed dolomite
116 WB	14	Buff and white crushed dolomite
125 WB	14	Gravel

slabs, air voids system analysis, and petrographic examinations. A total of 53 concrete cores were taken, in groups of 4 to 8, at eight different milepost locations, four eastbound and four westbound. The sites for testing were chosen to get representative samples from each soil type, pavement distress level, and construction zone on the East-West Extension.

The coring pattern called for a core to be taken at the longitudinal, transverse, and lane/shoulder joints, center slab, and corner slab. Additional cores were taken at approximately 6 in. from the longitudinal joint, on the shoulders, and in frost heave locations. Figure 6 shows the location of the cores within a site, and Table 1 gives the thickness of the PCC and the aggregate type at specific mileposts.

After coring, the drilling water remained in the holes (no drainage), showing a very low permeability of the subgrade. The water was manually removed, and the holes were patched with Set 45 concrete.

Soil Sampling

At 14 of the 53 cores taken, borings were made using a truck-mounted drill rig with the bore holes being advanced by continuous auger flight methods. Samples were taken according to the ASTM D 1586 procedure for split-spoon sampling of soils. Representative portions of the split-spoon samples were placed in glass containers with screw-type lids and taken to the laboratory for examination and testing. Laboratory work consisted of water content determinations for most of the samples, with unconfined compression strength tests being performed on representative samples. Approximate measurements of unconfined compression strengths were made for some of the samples using a calibrated pocket penetrometer. The pocket penetrometer is an indirect method for evaluating the compressive strength of a clay soil. All of the slab sections sampled rested directly on a fine-grained soil subgrade.

FWD ANALYSIS

FWD analysis was performed using finite element techniques and procedures developed by the University of Illinois and the Corps of Engineers.

Modulus of Elasticity and Modulus of Subgrade Reaction

The PCC slab modulus of elasticity (E) and the effective dynamic modulus of subgrade reaction (K-value) (at the bottom of the PCC slab) were backcalculated from deflection basin measurements. A closed-form backcalculation procedure was used, which is based on a theoretically rigorous approach using the principles of dimensional analysis as well as the concept of deflection basin area. The backcalculation method was developed and automated by Ioannides through the computer program ILLI-BACK. The approach models the pavement system as an elastic medium-thick plate resting on a dense liquid foundation.

The backcalculated mean values and ranges were as follows: E = 4,235 ksi (3,300 to 6,000 ksi) and K = 276 psi/in. (120 to 440 psi/in.).

The static K-value is estimated (*1,2*) to be approximately 276/2 = 138 psi/in., which is a typical value for this type of soil.

PCC Modulus of Rupture

The PCC modulus of rupture is most accurately determined by sawing standard-sized beams (6 by 6 by 30 in.) from several slabs and subjecting them to third-point loading tests. This is expensive and time-consuming, however. The PCC modulus of rupture can be estimated fairly well by using the indirect tensile strength of recovered 6-in.-diameter cores or even from the compressive strength of the cores.

The PCC flexural strength or modulus of rupture (MR) can also be estimated approximately from the PCC modulus of elasticity backcalculated from the FWD test results. The following relationship was developed at the University of Illinois to obtain an approximate flexural strength of pavement slabs nondestructively (*1*):

$$MR = 43.5(E/10^6) + 488.5$$

where MR is the modulus of rupture of the PCC and E is the modulus of elasticity of the PCC.

The calculated mean third-point modulus of rupture value is 673 psi.

Transverse Joint Load Transfer

The deflection load transfer across the transverse joints was measured by FWD testing. The load transfer is computed as the ratio of the unloaded slab's deflection to that of the loaded slab's deflection. A small correction is applied to this load transfer efficiency to allow for natural slab bending, which would occur even though no joint existed between the first and second sensor. The following equation was used to calculate the percent load transfer:

$$\text{Load transfer efficiency (\%)} = \frac{D_L}{D_A} \times 100$$

where D_A is the deflection in the approach slab and D_L is the deflection in the leave slab.

The FWD testing was conducted during cool temperatures when the joints were not excessively tight. The mean load transfer of transverse joints is 95.6 percent, which is very high, indicating good aggregate interlock across the joints. The high

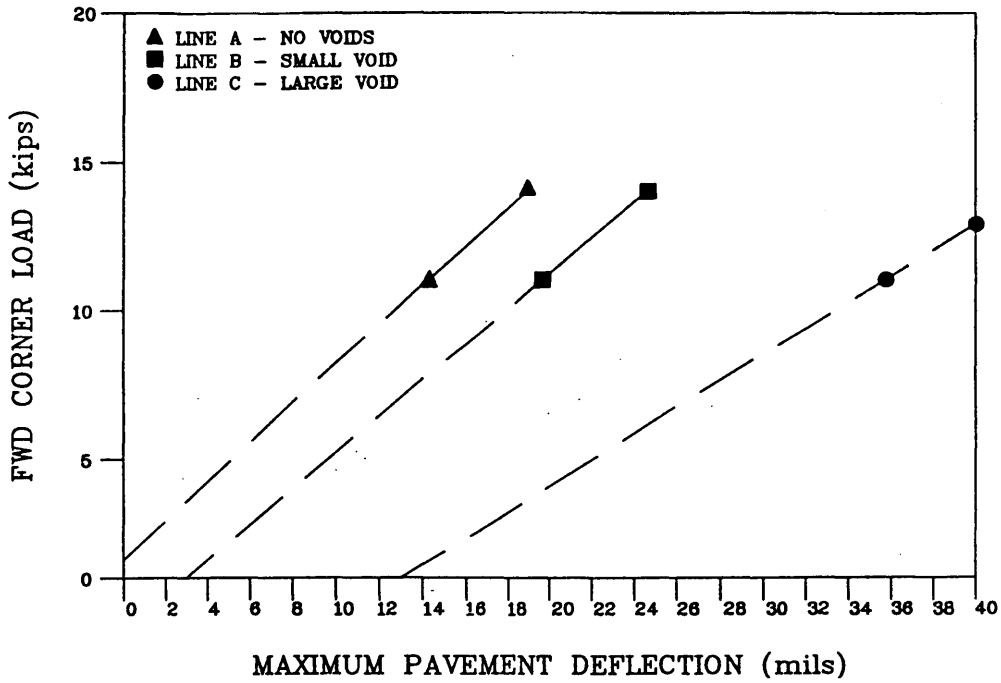

FIGURE 7 Load versus deflection void detection plot.

load transfer explains the low severity faulting of the transverse joints even though the joints are not doweled.

Corner Void Detection

A void can be described as an area of loss of support beneath the slab corners. This occurs because of heavy loads deflecting the slab corners, causing some pumping of fines with impulse water pressure. Any loss of support, even less than 0.10 in. thickness beneath the slab, will result in greatly increased slab stresses, causing corner breaks or diagonal cracks. The most common location for these voids is slab corners along the lane/shoulder joints. The loss of support can be determined using a procedure developed at the University of Illinois for the FWD (*3*). An examination of the load versus deflection at a slab corner can provide a rapid and simple indication of the existence of a void beneath the slab corner. The corner of each slab tested was loaded at three load levels and the corresponding deflections measured. Corners that have a load versus deflection plot that crosses the deflection axis near the origin of zero deflection, such as Line A in Figure 7, do not have voids beneath the slab. However, a line that passes significantly to the right of the origin, such as Lines B and C in Figure 7, is indicative of a void or loss of support beneath the slab corner. Figure 8 shows actual data from the East-West Extension.

After reviewing all of the load versus deflection plots, it was concluded that no voids or loss of support existed beneath the slabs tested except at Milepost 62.0153 eastbound. This was probably caused by the full-depth asphalt patch of the shoulder across the joint. Most likely the subgrade soil was disturbed during patching.

In addition to the NDT, an epoxy/core test procedure developed at Purdue University was used to detect voids at selected locations. The test was performed on several locations where voids were thought to exist on the basis of visual observation. No evidence of voids was detected at any of the locations tested.

LOAD-CARRYING CAPACITY

The load carrying capacity was evaluated using ILLISLAB, a computer-based finite element model, to determine the critical stresses in the PCC slab created by the truck loads. The modulus of elasticity of the PCC and the effective K-value

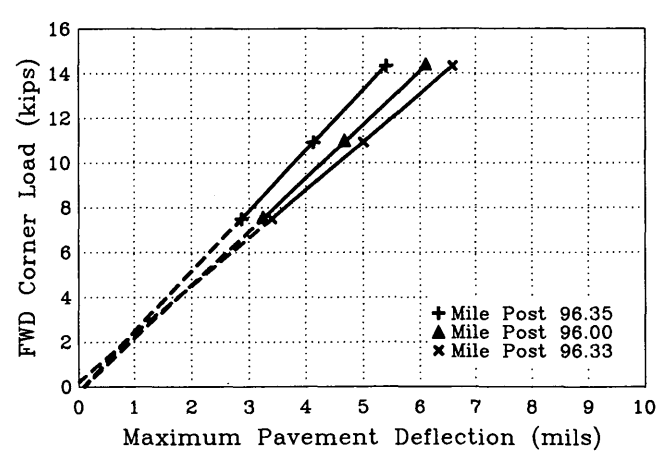

FIGURE 8 Actual load versus deflection detection plot.

were used along with the traffic characterization and weight to determine the critical stresses. The axles were positioned so that the wheel was placed directly on the slab edge along the lane/shoulder joint, the most critical location for stresses and crack development.

The number of edge loads allowable to failure (defined as the point at which 50 percent of the slabs suffer cracking) is calculated on the basis of the strength of the slabs and the critical stress produced by the traffic loading. The following equation form developed using Corps of Engineers field data was used:

$$\log_{10}(\text{axles}) = a\left(\frac{\text{MR}}{\text{stress}}\right)^b$$

where

MR = modulus of rupture of PCC slabs (psi),
stress = edge stress under traffic load (psi),
a, b = empirically derived constants developed from Corps of Engineers field data, and
axles = number of edge loading axles to failure (defined as 50 percent slab cracking).

This analysis showed that the number of edge loads to failure for an 18-kip axle was 5.45×10^{18} load applications. This indicates a very structurally sound PCC pavement, and no transverse cracking from traffic loadings is expected.

LABORATORY STUDIES

A total of 51 cores were received. Several of those cores were used for petrographic examinations and air void parameter determinations. The petrographic studies were done according to ASTM C 856, and the air content studies were done according to ASTM C 457. The modified point count method was used.

Detailed petrographic studies were conducted on 1 core from each section of roadway, plus 2 additional cores from selected area, making a total of 10 cores studied. Air void studies were also done on one core from each section plus two from selected areas. Cursory petrographic examinations were conducted on the remainder of the cores.

Three distinct coarse aggregate types were used in the concretes. They consist of a buff and white crushed dolomite, a white limestone/dolomite, and a gravel consisting of dolomite, granite, chert, and basalt. All cores contained natural sand, which did not contribute to the deterioration.

The deterioration observed in the cores fell generally into two distinct categories: (*a*) freeze-thaw damage to the portland cement binder and (*b*) D-cracking of the coarse aggregates.

Freeze-thaw damage occurs in concrete that is sufficiently saturated and does not contain an adequate air void system. D-cracking results from a combination of a specific aggregate type, the presence of moisture, and freeze-thaw cycling.

Soil Results

All of the soils were fine-grained (sand, silt, and clay) ranging in AASHTO classification from A-2-6 to A-7-6; most were A-6. The liquid limits (LLs) ranged from 20 to 51 and the plasticity indices (PIs) ranged from 10 to 30. The cohesive soils varied from hard to very tough, whereas all of the cohesionless soils were firm. A thin layer of organic topsoil was occasionally encountered, with characteristically high moisture contents and low dry densities. Occasional lenses of fill directly beneath the pavement were well compacted, with moderate moisture contents.

Analysis of Air Void Systems

Properly air-entrained PCC is protected from the increase of hydraulic pressure during freeze-thaw cycles. The air void bubbles would act as pressure relief valves or "safety valves" that allow the excess water to escape to them and freeze without damaging the PCC (*4*).

To evaluate the air void parameters of the project, 10 cores were selected for testing. The results are given in Table 2. ACI Committee 345 has recommended that the number of voids per lineal inch be significantly greater than the percentage of air, that the specific surface (**a**) be greater than 600 in.^{2}/in.^{3} and that the spacing factor (L) be less than 0.008 in. For the ¾-to 1-in. maximum aggregate size in the I-88 cores, these requirements should be met with an air content of about 4½ to 7½ percent. However, although essentially all 10 of the cores tested had total air contents within this range, only one (125-4) met the requirements listed. The causes for this are not determinable. But the undesirable air void parameters measured are certainly sufficient to explain the cyclic freezing damage observed in most of the cores subjected to petrographic examination.

In general, the entrained air voids of the entire project are not well distributed. They tend to form in clusters and around coarse aggregate particles; thus, the air void systems are not well developed. In addition, the air in many of the cores occurs as large entrapped voids, which furnish a minimum protection from cyclic freezing damage. This explains the premature failures of the transverse and longitudinal joints.

SUMMARY OF EVALUATION RESULTS

This section presents the findings and engineering analyses on which maintenance and rehabilitation strategies were developed. The results of the evaluation study are as follows:

1. The concrete slab is 14 in. thick or more. Most pavement cores were greater than 14 in., and those that were not were within ¼ in.

TABLE 2 Results of Air Void Analysis

Mile Post	Total % Air	Voids/in	Specific Surface (a) in^2/in^3	Spacing Factor (L) in
67.5 EB	7.0	3.9	224	0.014
76 EB	6.2	3.3	210	0.017
92 WB	5.0	3.8	303	0.012
98.5 WB	5.5	4.3	313	0.015
98.5 WB	4.4	4.1	367	0.014
100 EB	5.6	4.8	342	0.009
112 EB	5.3	3.6	271	0.014
116 WB	6.7	4.4	262	0.016
125 WB	5.1	7.8	621	0.006
125 WB	4.9	3.8	308	0.014

2. Estimated total air contents range from 4.4 percent at Milepost 98.5 WB to 7.0 percent at Milepost 67.5 EB, and much of this was entrapped air. In general, the entrained air voids are not well distributed. The voids tend to form in clusters and around coarse aggregate particles; thus, the air void systems are not well developed. In addition, the air in many of the cores and in spalled pieces of PCC observed in the field occurs as large entrapped voids, which furnish a minimum of protection from cyclic freezing damage. This is believed to be the major cause of the spalling.

3. Some of the aggregate is susceptible to D-cracking damage. This has resulted in some of the spalling in limited specific sections along I-88.

4. The average dynamic modulus of subgrade reaction is 276 psi/in. (static value of about 276/2, which is typical of this type of soil).

5. The average modulus of elasticity of the PCC interim slabs is 4,200,000 psi, which is considered sound concrete.

6. The average modulus of rupture of the PCC estimated by NDT testing was 673 psi, which is considered good.

7. The average load transfer across the transverse joints is 95 percent. Any value 70 percent or more is considered good.

8. The average transverse joint faulting is 0.123 in. In several locations, the faulting was as high as 0.27 in.

9. The PCC slab is structurally sound and can carry a large number of repetitions before load fatigue cracking. Existing cracking is caused from other causes such as frost heaving or inadequate longitudinal joint construction.

10. There were no indications of any loss of support beneath any of the slabs tested.

11. The soils found on the extension were generally AASHTO Classfication A-6.

12. All of the joints throughout the entire project are experiencing deterioration caused by damage from cyclic freezing, D-cracking of the coarse aggregate particles, or a combination of the two.

13. The pavement has become very rough from the severe joint spalling.

MAINTENANCE AND REHABILITATION STRATEGIES

Several maintenance and rehabilitation strategies were considered for the East-West Extension project. All of the strategies were evaluated using EXPEAR (EXpert system for Pavement Evaluation And Rehabilitation) (5–7). EXPEAR is a practical and comprehensive computerized system to evaluate concrete highway pavements, develop rehabilitation alternatives, and predict the performance and cost-effectiveness of the alternatives. EXPEAR was originally developed for FHWA by the University of Illinois and it was recently modified by ERES Consultants, Inc.

On the basis of the recommendation of the Illinois Tollway Authority, strategies with different life expectancies were developed. These strategies were grouped into three different groups (short-term, intermediate-term, and long-term performance) so that the Illinois Tollway Authority can base its choice of strategy on budget constraints and performance period. The strategies were evaluated by life cycle cost analysis, which makes it possible to compare alternatives with different initial construction costs on the basis of equivalent annual cost. The unit costs information was obtained from several local areas and is included in Table 3. Table 4 gives all of the strategies, life expectancy, total initial costs, and equivalent annual costs.

The overlay thicknesses in Strategies 4, 5, 6, 7, and 9 are only approximate and must be verified through an engineering design analysis.

1. Spray patch (1): This strategy includes removing loose materials from the center longitudinal and transverse joints and spray patching the joints with emulsion and aggregate.

2. Spray patch (2): This strategy includes cleaning all of the joints except the shoulder joints and spray patching them with emulsion and aggregate.

3. Repair spalls with PCC: This strategy includes milling out only the center longitudinal and transverse joints 2 ft wide by 6 in. deep, repairing them with PCC, and then sawing and sealing the joints.

4. AC overlay (1): This strategy includes repairing the joints with AC and applying a 5-in. AC overlay.

5. AC overlay (2): This strategy includes repairing the joints with PCC and applying a 5-in.-minimum AC overlay.

6. Crack and seat: This strategy includes breaking all of the PCC slabs of Lanes 1 and 2 into small pieces (1 to 2.5 ft), seating the pavement, and applying a 5-in. AC overlay.

7. Rubblizing: This strategy includes breaking all of the PCC slabs into rubble (less than 12 in. in size), compacting the rubble, and applying a 7-in. AC overlay.

TABLE 3 Unit Costs Information

DESCRIPTION	UNIT COST(dollars)	UNIT
Unbonded PCC Overlay	3.05 per inch	SY
AC Overlay	33.22	TON
Crack and Seat	1.00	SY
Rubblizing	1.65	SY
AC Partial Patch (Spray Patch)	1.31	LINEAR FT
	11.79	SY
PCC Partial Patch	15.00	LINEAR FT
	68.00	SY
PCC Recycling + Base	39.06	SY
Longitudinal Subdrains	2.46	LINEAR FT
Reconstruct Heaves	50.00	SY

TABLE 4 Summary of the Rehabilitation Strategies

Performance Period	Strategy	Life Expectancy (Years)	Total Initial Cost (U.S. Dollars)	Total Annual Cost (U.S. Dollars)
Short-Term	1. Spray Patching (1)	2	4,115,160	2,087,940
	2. Spray Patching (2)	2	6,024,252	3,056,700
	3. Patching with PCC	5	29,158,986	6,181,572
Intermediate-Term	4. AC Overlay (1)	7	33,938,064	5,288,574
	5. AC Overlay (2)	10	58,933,728	6,707,628
	6. Crack & Seat/ AC OL	10	34,919,658	3,974,400
	7. Rubblizing/ AC OL	10	52,246,662	5,946,558
Long-Term	8. Reconstruct Outer Lane & Restore Inner Lane	15	45,315,198	3,685,290
	9. Unbonded PCC Overlay	20	84,640,230	5,523,450
	10. Reconstruction	20	107,454,390	6,985,284

8. Recycling and restoring: This strategy includes recycling Lane 1 into a 4-in. stabilized base with a 10-in. PCC pavement and the joint spacings matching those of Lane 2. Also, all of the joints in Lane 2 would be repaired with PCC.

9. Unbonded PCC overlay: This strategy includes applying a 1-in. AC separation layer over Lanes 1 and 2 and then placing a 9-in. jointed plain PCC overlay with 15-ft joint spacing (*8*). The shoulders will be overlayed with 9 in. AC.

10. Reconstruction: This strategy includes removing the PCC from the mainlines and shoulders and rebuilding the pavement with 4-in. base, 12-in. PCC surface with 15-ft joint spacing, longitudinal edge drain, and 7.5-in. PCC shoulders.

CONCLUSIONS AND RECOMMENDATIONS

The engineering analysis conducted to determine an effective rehabilitation plan for the East-West Extension had many associated complexities. A major effort was made to determine the primary cause of the current surface distress affecting the user's satisfaction and riding comfort. After the pavement engineering investigations and analysis were completed, it was concluded that the main problem was the spalling of the transverse and longitudinal joints. The spalling of the joints was caused primarily by damage from cyclic freezing of the cement paste and D-cracking of the coarse aggregate particles. Even though the air content in the concrete was within specifications, the size and distribution of the voids were not within specifications to protect the concrete from freeze-thaw cycles, which caused the premature failure of the joints.

The pavement itself is structurally sound and has a remaining life left in it as would be typically expected. The primary concern is rehabilitation of the transverse and longitudinal joints and restructuring the pavement's resistance to infiltration of surface moisture. By accomplishing this, any chances of further damage from freeze-thaw will be reduced, and the service life of the pavement will substantially increase.

The preferred rehabilitation for the East-West Extension is primarily composed of three components. These are recommended on the basis of effectiveness, performance, and user satisfaction. Annual budget constraints have also been recognized along with the cost-efficiency of interim maintenance operations.

Low-Severity Sections

The low-severity areas should be addressed as soon as possible to prevent further disintegration of the pavement surface. The recommendation is to clean and reseal all the longitudinal and transverse joints. There will also be a need to do limited slab replacements and some joint repairs in isolated areas. There are locations within the low-severity areas with few broken slabs due to frost action in the soils. Those could be replaced and the soil removed and replaced. However, this recommended treatment will not address the profile of the pavement.

Medium- and High-Severity Sections

Before recommendations are made for repair in these areas, it must be recognized that everything cannot be done immediately from budget, project development, and overall tollway system planning aspects. Interim alternatives to address the current effort required to maintain the surface profile and provide user comfort should be performed. It was recommended to the Tollway Authority that the transverse and longitudinal joints be repaired temporarily using the spray patch procedure until a permanent rehabilitation strategy is implemented.

The overall rehabilitation of the project will be based on the service life the Tollway Authority wishes to obtain, budget constraints, and construction time and lane closures. The patching alternatives are not favorable due to insufficient life. The other alternatives pose a trade-off between longer service life and lower initial construction cost and increased future rehabilitation costs. For example, if the Tollway Authority favors a long-term (20 or more years) strategy, an unbonded PCC overlay is recommended. If the Tollway Authority favors a 10-year strategy, an AC overlay after rubblization or crack and seat is recommended.

ACKNOWLEDGMENTS

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PARES—An Expert System for Preliminary Flexible Pavement Rehabilitation Design

TIMOTHY ROSS, STEPHEN VERZI, SCOTT SHULER, GORDON MCKEEN, AND VERNON SCHAEFER

The development of a knowledged-based expert system to assist the New Mexico State Highway and Transportation Department in the evaluation and design of rehabilitation schemes for flexible pavements is described. The system uses information provided by users to establish preliminary rehabilitation schemes that would be reasonable and cost-effective. A cost-estimate module for ranking the rehabilitation schemes according to relative cost is integrated into the system. The need for such a system in New Mexico and the knowledge base used to construct the IF-THEN-ELSE type rules in the expert system are described, and the distress conditions addressed and the rehabilitation strategies considered are discussed. The system is rich in the sense that it also distinguishes among distress situations requiring routine maintenance as opposed to rehabilitation requiring more extensive construction efforts. An example session using the expert system is provided.

The selection of pavement rehabilitation alternatives depends on distress type present in the pavement, ride quality, traffic volume, structural section, maintenance history, and other factors. Although the manual process of determining rehabilitation schemes has been effective, a computerized knowledge-based expert system would allow a more detailed preliminary estimation of rehabilitation needs such that costs could be better ascertained. This would contribute to a more accurate identification of the number and extent of projects to be scheduled for rehabilitation.

Such an expert system could also be used to reduce the time required for new personnel to develop an adequate level of on-the-job experience. More experienced personnel may use the expert systems to make their own designs a more expeditious and economical process. The expert system for preliminary rehabilitation design will immediately benefit the New Mexico State Highway and Transportation Department (NMSHTD) by providing assistance to personnel responsible for estimating initial costs of rehabilitation projects with expert guidance regarding the most cost-effective alternatives using available information. The expert system could be queried by users for details on the construction or rehabilitation problem of concern, with the output used to identify potential problems and offer alternative solutions for obtaining the best pavement rehabilitation scheme for a given situation.

This paper summarizes a recent New Mexico study (*1*) to develop a PAvement Rehabilitation Expert System (PARES) for the preliminary rehabilitation design of New Mexico flexible pavements. Good reviews of expert system technology in transportation and other civil engineering disciplines are available in the literature (*2*). The paper presents some relevant previous efforts on the application of expert systems technology in highway pavement management, discusses the current practices in the state of New Mexico, addresses the particular features and utility of the PARES code for use in flexible pavement rehabilitation, and concludes with an example application of the system.

RELATED EXPERT SYSTEMS FOR HIGHWAY PAVEMENTS

An expert system originally developed for the Washington State Highway system by Ritchie et al. involves the area of flexible pavement rehabilitation using a code called SCEPTRE (*3*). The SCEPTRE code is used to provide a user with several rehabilitation strategies based on the existing condition of a roadway and the user-specified service life of the desired rehabilitated pavement. SCEPTRE is based on "IF-THEN" type rules and uses a backward-chaining inference method (reasoning goes back from known facts to a hypothesis).

Haas and his colleagues (*4–6*) have developed expert systems for flexible pavement management, pavement distress analysis, and pavement condition data inventory. One of these systems, PRESERVER (*4*), assists field engineers and supervisors in analyzing pavement distress data and proposes routine maintenance strategies. This system is similar to SCEPTRE, except that it proposes maintenance rather than rehabilitation strategies.

In other developments, Hall et al. (*7*) have developed an expert system, called EXPEAR, to assist the design engineer in the evaluation and preliminary rehabilitation design for jointed reinforced, jointed plain, and continuously reinforced concrete pavement (JRCP, JPCP, and CRCP). EXPEAR uses information provided by pavement engineers to determine the type and cause of distress so that an appropriate rehabilitation strategy can be selected.

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Aougab et al. (*8*) have developed an expert system, PAMEX, for maintenance management of flexible pavements. Ritchie (*9*) has developed an expert system, termed OVERDRIVE, to assist local engineers in designing the structural thickness of asphalt concrete overlays. Haas and Shen (*4,6*) have developed PRESERVER, an expert system for the Canadian province of Ontario to help field engineers and supervisors analyze pavement distress information to propose routine maintenance strategies. Hajek et al. (*10*) have developed ROSE, an expert system for recommending routing and sealing of asphalt concrete pavements in cold areas of Canada. Tandon and Sinha (*11*) have developed an expert system to estimate highway pavement routine maintenance needs and expected costs at the subdistrict level. And finally, to underscore the growing emphasis and importance of expert systems in pavement management, Barnett et al. (*12*) have published a Federal Highway Administration report that provides guidelines to the states for the development and distribution of highway-related expert systems.

NEW MEXICO PAVEMENT REHABILITATION SYSTEM

Performance of each of 3,000 evaluation sections in the New Mexico pavement network is documented periodically through visual condition surveys and roughness measurements. Rehabilitation procedures for flexible pavements in New Mexico are intended to provide 10 years service with routine maintenance; however, the routine maintenance required during this interval will vary depending on the rehabilitation method selected. The repair strategies vary in effectiveness, cost, and intended purpose.

The current New Mexico pavement management system consists of a very detailed description of the roadway to be evaluated. Information collected from the field is transferred to a computer system to present the user with seven types of inquiries regarding the roadway segment: (*a*) pavement data, (*b*) condition data, (*c*) planned projects, (*d*) project history, (*e*) traffic data, (*f*) road safety data, and (*g*) distress detail. In New Mexico distress is quantified on the basis of American Public Works Association (APWA) guidelines (*13*).

A priority ranking system based on field condition surveys and traffic volume has been developed by NMSHTD to assess which of the sections should be rehabilitated or reconstructed. This system has been developed such that a priority assignment indicates that rehabilitation is necessary. Therefore, pavements requiring routine maintenance theoretically would not receive a priority value and therefore would not be considered for rehabilitation. An exception to this might include pavements with escalating maintenance costs, which a particular highway district judges as requiring more than routine treatment.

After the priority assignments are made an initial estimate of cost for rehabilitation is made. The preliminary cost estimate is used to determine the number and extent of projects to be considered for rehabilitation depending on the funds available. After the projects to be rehabilitated are identified, a more comprehensive preliminary design is initiated. This design is based on a visual survey by the design engineers, results of the condition survey and roughness data, construction history, and other data, if available. Rehabilitation alternatives are compared on the basis of initial and long-term cost-effectiveness for a design period of 10 years.

Although the manual system in New Mexico was effective in determining appropriate rehabilitation alternatives, an expert system will be advantageous in the assessment of initial rehabilitation costs for at least two reasons. First, the initial cost estimate for prioritized projects would be significantly more accurate. Second, much of the iterative process involved with comparing the preliminary design with initial estimates made by planning personnel would be reduced because the initial estimate involves the same reasoning that is included in the preliminary design procedure.

THE EXPERT SYSTEM—PARES

PARES was implemented in EXSYS Professional (*14*), a commercially available expert system shell. EXSYS has been used extensively in other applications (*15*). EXSYS allows for both backward-chaining (goal driven) and forward-chaining (data driven) inferencing. The IF-THEN-ELSE structure is the general form of the rules of an EXSYS knowledge base. This structure is used for the rule base irrespective of whether the rule is chained in a forward or backward manner. All portions of the IF clause must be satisfied before the conclusion (THEN clause) of a rule is activated. If a single portion of the IF clause is disproved, the ELSE portion is activated.

THE INFERENCING SYSTEM—EXSYS

Each of the conditions in the IF portion of a rule is specified by a Boolean-valued formula that will evaluate to either true or false. The formula can be composed of mathematical variables in a logical relation (<, >, =, etc.) or propositional variables in a predicate calculus relation. When all IF conditions have a truth assignment, the rule can be invoked, and either the actions of the THEN portion or the ELSE portion are carried out. Actions in the THEN or ELSE portions of the rule can perform many different functions, such as execution of external programs (e.g., the PARES cost module), manipulation of mathematical variables (e.g., calculation of mill depths and overlay thickness), setting of conditions for the IF portions of other rules, and selection of final rehabilitation strategies. In EXSYS the ELSE portion of the rule is optional.

In a forward-chaining inference, an existing knowledge base is used to invoke as many rules as possible, where the actions from these rules are used as the conditions for new rules and the invocation proceeds forward until no more rules can be invoked. Backward chaining proceeds by selecting a rule in which it is desired to have one or more of the actions executed in the THEN portion of the rule (goal). For the action to take place, all of the conditions in the IF portion of the rule must be satisfied (i.e., evaluated to be true). Backward chaining proceeds in a "depth-first" manner through the rule base, searching the rule base for rules whose actions will enable the firing of rules that have already been considered in the chain of rules.

DEVELOPMENT OF THE KNOWLEDGE BASE

The PARES knowledge base was developed from procedures documented in New Mexico state highway agency manuals, some AASHTO procedures, and a group of NMSHTD pavement rehabilitation experts. The state highway experts provided the heuristic rules used to formulate the knowledge base. The experts were particularly important to this work, but the input from different individuals invariably resulted in some conflicts of opinion. The resolution of these differences was addressed by the research team in selecting among the available alternatives.

The first step in the development of the knowledge base was to construct a list of the data to be entered by the users into the expert system. Rehabilitation of roadway surfaces is necessitated by the existence of certain types and levels of distress. The development of the PARES code used standard distress types as documented in the APWA *Pavement Condition Index for Asphalt Pavements* (*13*). The expert system PARES considers 23 types of distress and 3 levels of distress severity for each distress type. The distresses and levels are given in Table 1.

The expert system also considers the extent and the severity of each distress type. For most distresses, the extent is entered in terms of the percentage of the road covered by the particular distress severity. For longitudinal cracking and transverse cracking the extent is entered as the number of lineal feet of cracks per project. For depressions at bridges and railroad crossings, the extent is the number of these distresses of a particular severity present in the project. These two are inherently localized distresses and, as such, can be treated separately from the rest of the distresses.

The second step in representing the asphalt pavement rehabilitation knowledge was the creation of hierarchical structures (a logic tree). The logic tree forms the shell in which knowledge-based rules stating the declarative and procedural knowledge are inferenced.

Interviews were conducted with five expert New Mexico pavement designers. From these interviews, a multilevel logic tree was conceived to capture the generality of possible distress conditions. This logic tree attempts to structure the knowledge of the experts that typically is not reducible to algorithmic form. Both the distresses and plausible rehabilitation strategies were classed into special categories that would be useful in relating the distresses to the rehabilitation strategies. The research team acted as additional experts to develop categorizations of distresses and rehabilitation strategies as well as the logic to tie those together into a multilevel logic tree. The experts decided that 21 rehabilitation strategies (given in Table 2) captured the experience in the past of rehabilitations to New Mexico road surfaces and, to a lesser extent, plausible strategies not frequently used in New Mexico. The multilevel logic tree relating the distress situations to the potential rehabilitation strategies is shown in Figure 1, where the encircled numbers represent the strategies given in Table 2.

To categorize the pavement distresses, the research team classified the possible distresses into five distress type sets. Some of these sets may be empty for a particular design situation, indicating that no distresses in that category have been seen on the existing pavement surface. These five distress categories are general maintenance distresses, localized maintenance distresses, surface-mix distresses (due to the asphaltic material), surface cracking distresses, and subgrade (subsurface) distresses. When all known distresses have been specified by the user, PARES inferences on the rule base to provide possible rehabilitation strategies to the user.

SPECIFICS OF THE RULE BASE IN PARES

There are 278 rules in the PARES rule base. These rules, in conjunction with the geometric information about the roadway surface provided by the user, compose the knowledge base for a particular application of the PARES code. Roadway length and width (without shoulders) are the only geometrics used in PARES. Rules 1 to 73 in the PARES rule base embody the logic used to segregate all the input data and the distress conditions provided by the user into the five general distress type sets. Each time a distress type set is confirmed in a rule a counter is incremented so that PARES can use the number of distresses in a particular distress type set in logic deeper in the logic tree. Rules 1–73 are inferenced

TABLE 1 Distress Types and Severity Levels for PARES

Distress Types	
Alligator (fatigue) Cracking	Patch Deterioration
Bleeding (flushing)	Polished Aggregate
Block Cracking	Potholes
Bumps and Sags	Pumping and Bleeding
Corrugation	Railroad Crossings
Depressions	Raveling and Weathering
Depressions at Bridges	Rutting
Edge/Center-line Cracking	Shoving
Joint Reflection Cracking	Slippage Cracking
Lane/Shoulder Drop-off	Swelling
Lane/Shoulder Separation	Transverse Cracking
Longitudinal Cracking	

Distress Severity Levels
Low Severity
Moderate Severity
High Severity

TABLE 2 Rehabilitation Strategies

1. Do nothing
2. Crack seal
3. Chip seal
4. Asphalt Overlay
5. Crack seal + overlay
6. Asphalt rubber interlayer + overlay
7. Geotextile fabric strip + overlay
8. Geotextile fabric sheet + overlay
9. Cold in situ recycling + chip seal
10. Cold in situ recycling + overlay
11. Hot recycling + chip seal
12. Hot recycling + overlay
13. Heater scarification + overlay
14. Cold milling + overlay
15. Cold milling + interlayer + overlay
16. Cold milling + cold in situ recycling + overlay
17. Pulverization + overlay
18. Portland Cement Concrete
19. Reconstruction
20. Dig out and patch
21. Patch surface

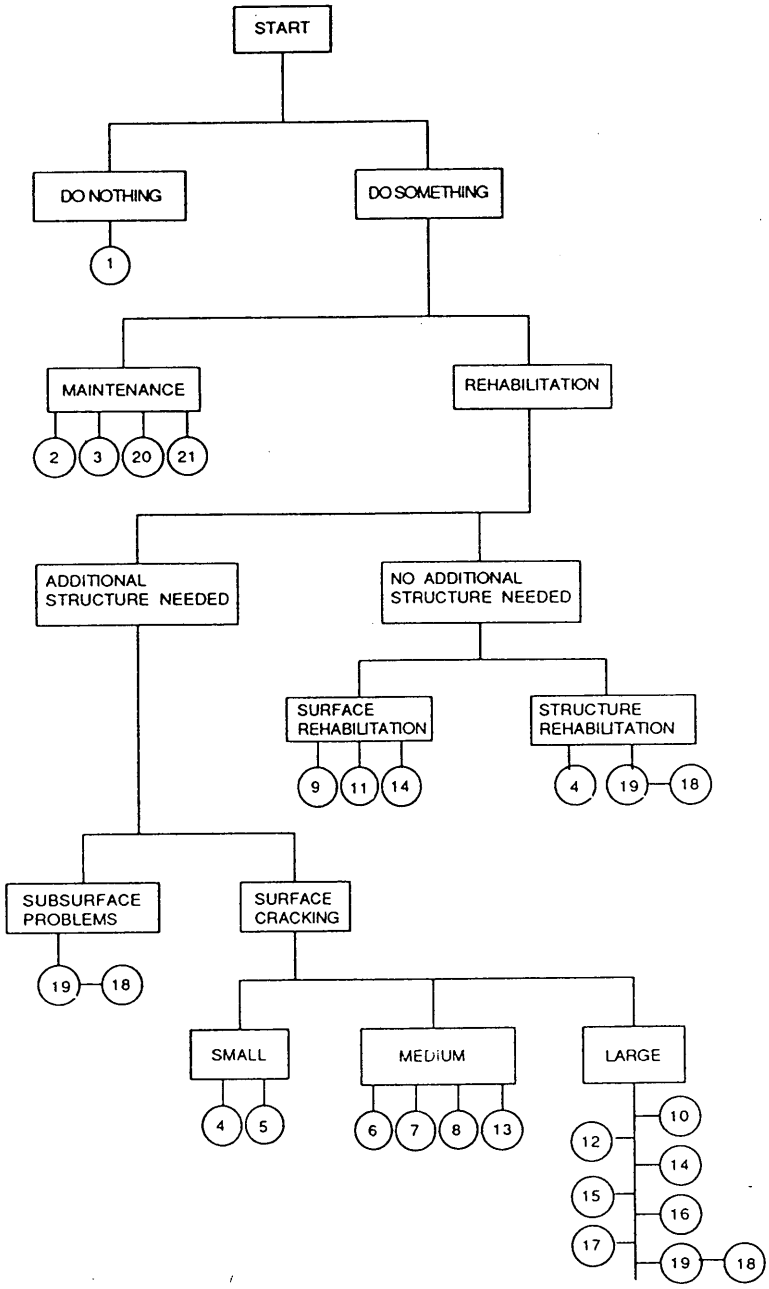

FIGURE 1 Multilevel logic tree for PARES.

in a forward-chaining manner since classification is inherently a data-driven task. In PARES, the main body of rules (Rules 74–249) are inferenced in a backward-chaining mode, where the output consists of rehabilitation strategies for roadway repair for the current input situation. Finally, Rules 250–278 are inferenced using a forward-chaining mode, since these rules calculate specific values for mill depth and overlay thickness for the rehabilitation strategies chosen in Rules 74–249. Examples of the first 73 rules follow:

RULE NUMBER: 1
IF distress type set contains alligator cracking
AND the alligator cracking is type C.
THEN localized distress set contains alligator cracking
AND increment localized distress set counter by 1.
(Note: as described later, PARES will query the user about the "type" of alligator cracking present.)

RULE NUMBER: 28
IF distress type set contains lane/shoulder drop-off
AND lane/shoulder drop-off severity is low
AND the extent of lane/shoulder drop-off is greater than 10 percent,
THEN subsurface distress set contains lane/shoulder drop-off
AND increment subsurface distress set counter by 1.

RULE NUMBER: 72
IF distress type set contains alligator cracking
AND the alligator cracking is Type A OR Type B
AND distress type set contains rutting
AND rutting severity is moderate OR high
THEN subsurface distress set contains alligator cracking AND rutting
AND the rutting is Type B (surface mix material failure)
AND increment subsurface distress set counter by 1.

The categorization of rehabilitation strategies was accomplished at different levels within the logic tree. At each major branch point in the logic tree in Figure 1, decisions have to be made on the basis of available evidence provided by the user. It is instructive to list here some of the rules that affect some of these decisions. For example, there is the strategy of do nothing, which is an "escape route" strategy for the expert system when no distresses are entered by the user. If PARES receives no distress types, it must assume that no rehabilitation is required. The rule governing this escape is

RULE NUMBER: 74
IF all distress type sets contain nothing,
THEN do nothing.

To decide whether a distress is maintenance only or rehabilitation, a typical decision rule is

RULE NUMBER: 82
IF subsurface distress set contains nothing
AND surface cracking set contains nothing
AND surface mix distress set contains nothing
AND average daily traffic is less than 5,000
AND total extent of maintenance distresses present is less than 50 percent,
THEN the type of strategy needed is maintenance only.

The difference between a maintenance strategy and a rehabilitation strategy is not clear in many repair situations. Although it is easy to determine that railroad crossings and bridge depressions can almost always be addressed with maintenance strategies, other distress types that are typically nonlocal phenomena may not be easily categorized as maintenance problems. Generally local distress types such as bumps and sags, corrugation, depressions, patch deterioration, and potholes are usually found in low extent, and so can be handled using maintenance strategies. However, if these same types of distress are apparent over a significant extent of the project surface and the average daily traffic is high enough, the pavement would tend to require rehabilitation. For simplicity, it was decided that when a generally nonlocal distress type occurs with generally local distress types, if the local distresses occurred in low severity, they could be handled by maintenance strategies. But, if the extent of local distresses becomes too high and if the average daily traffic is too high, rehabilitation would be required rather than maintenance. The numerical thresholds for this transition are shown in Figure 2.

In PARES there are four maintenance strategies (Rehabilitation Strategies 2, 3, 20, and 21 in Table 2) that can be recommended if the distress conditions are not sufficient to warrant new construction, or these same four maintenance strategies may be recommended by PARES as "additional construction" (see typical example later) if they are associated with recommended rehabilitation strategies as explained above. These maintenance strategies are also considered escape routes from PARES, since the primary purpose of PARES is rehabilitation. Typical maintenance rules follow:

RULE NUMBER: 84
IF the type of strategy needed is maintenance only
OR maintenance with rehabilitation
AND maintenance distress set contains edge/centerline cracking
THEN crack seal.

RULE NUMBER: 91
IF the type of strategy needed is maintenance only
OR maintenance with rehabilitation
AND localized distress set contains alligator cracking
THEN dig out and patch.

Whether additional pavement strength is needed is determined by the difference between the required new pavement structural number (SN_{new}) and the existing structural number (SN_{old}). If SN_{new} minus SN_{old} is greater than 0.30, an overlay, or recycled pavement, is recommended because additional strength is necessary. In PARES the new SN_{new} is calculated using the 1972 AASHTO guide (*16*) and SN_{old} is provided by the user. The choice between strategies that rehabilitate the

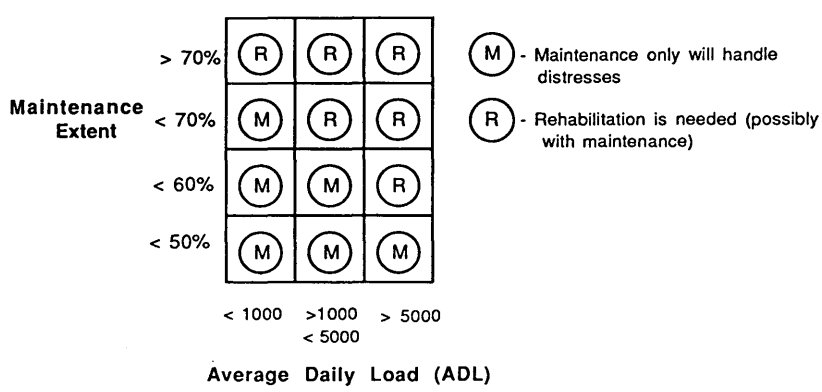

FIGURE 2 Transition zones between maintenance and rehabilitation.

pavement surface only and strategies that rehabilitate both the subgrade and the surface is made on the basis of whether a subsurface distress condition is present.

If a subsurface distress condition is present (e.g., swelling, Type B alligator cracking, or Type A rutting) in significant extent, then a rehabilitation strategy that treats the subgrade would be recommended. There are exceptions (such as if the moisture condition of the pavement is stable) to such a recommendation, but the general rule is that a rehabilitation strategy resulting from a subsurface distress will override rehabilitation strategies resulting from other types of distress conditions, giving rise to rules of the following form:

RULE NUMBER: 110
IF the type of strategy needed is rehabilitation
AND structural strength of the existing road is not adequate for future design
AND subsurface distress set contains swelling,
THEN remove and replace with asphalt (reconstruction).

Some rehabilitation strategies are used when no additional strength is needed (i.e., either to improve the surface course or to improve the subsurface as well as the surface). A rule to determine whether an increase in the structure of the road is necessary is

RULE NUMBER: 102
IF $(SN_{new} - SN_{old}) > 0.3$
THEN structural strength of the existing road is not adequate for future design
ELSE structural strength of the existing road is adequate for future design.

If no additional strength is required, PARES determines whether improvements to the asphalt concrete are required through either recycling material or reconstruction. An example of a rule in this situation is (note: rehabilitation strategies are conclusions at this point)

RULE NUMBER: 238
IF the type of strategy needed is rehabilitation
AND structural strength of the existing road is adequate for future use
AND surface mix distress set contains bleeding
THEN hot recycle + chip seal
OR cold mill + overlay.

If additional strength on the roadway surface is required, PARES determines whether the rehabilitation should be based on subsurface problems or surface cracking problems. The decision whether additional strength rehabilitation is due to subsurface or cracking problems is given in the following rule:

RULE NUMBER: 111
IF the type of strategy needed is rehabilitation
AND structural strength of the existing road is not adequate for future design
AND subsurface distress set contains nothing,
THEN the surface cracking matrices should be used,
ELSE the surface cracking matrices should not be used.

The surface cracking matrices mentioned in this rule embody the knowledge from experts used to assess issues associated with surface cracking problems. The surface cracking problems are addressed by a variety of rehabilitation strategies, depending on the severity of the cracking and the number of cracking-related distresses.

In PARES, the surface cracking rehabilitation strategies are divided into three categories (see Figure 1): small (cracking is not a problem), medium (cracking could be handled with an interlayer), and large (cracking should be eliminated), which are the output from the second matrix shown in Figure 3 (S, M, or L). A choice among the three categories is made through the use of the surface cracking matrices in Figure 3. There are two surface cracking matrices. The first matrix shown in Figure 3 is designed to be used with each nonmaintenance, nonsubsurface distress type, where a repair level (L, M, S) is determined on the basis of severity (low, medium, high) and extent (in percentage, 0–100) of the distress type in question. In other words, the first matrix addresses the different types of cracking that can happen (e.g., alligator cracking, transverse cracking, etc.), and it also addresses those distresses that can appear along with the cracking (e.g., raveling, rutting, etc.). The second matrix in Figure 3 is designed to take the repair level outputs from the first matrix (S, M, L) and produce an overall repair level (S, M, or L corresponding to small, medium, and large), which is then used in the rule base to choose among the three categories of strategies. The interface code used at the beginning of the expert system to get pertinent information from the user uses arrays for keeping track of the distresses and calculates the necessary overlay depth. Typical rules for surface cracking repair strategies follow:

RULE NUMBER: 112
IF the surface cracking matrices should be used
AND distress type set contains alligator cracking
AND alligator cracking (fatigue Type A) severity is low
AND extent of alligator cracking is less than 10 percent,
THEN repair level is small.
(Note: in this rule PARES would query the user about the type of alligator cracking.)

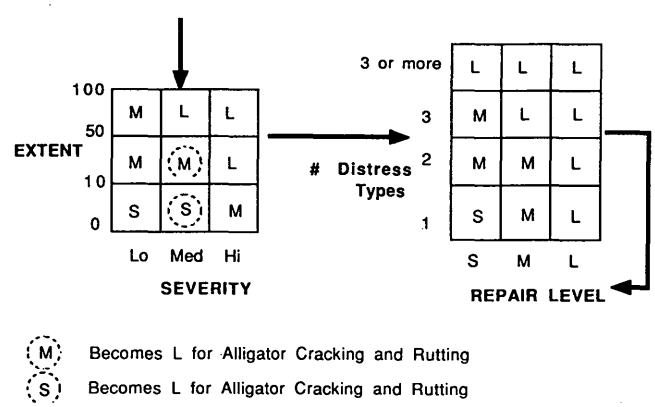

FIGURE 3 Surface cracking matrices developed in the knowledge base.

RULE NUMBER: 139
IF the surface cracking matrices should be used
AND distress type set contains edge/centerline cracking
AND edge/centerline cracking severity is moderate
AND extent of edge/centerline cracking is between 10 and 50 percent,
THEN repair level is medium.

Finally, when PARES is at the level of abstraction below "surface cracking" in the logic tree (at the numbered circles in Figure 1), its rule base recommends rehabilitation strategies. Typical recommendation rules follow:

RULE NUMBER: 210
IF existing cracking should be addressed with an interlayer
AND surface cracking set contains alligator cracking,
THEN asphalt rubber interlayer + overlay
OR geotextile fabric sheet + overlay
OR heater scarification + overlay.

RULE NUMBER: 224
IF existing cracking should be destroyed
AND surface cracking set contains alligator cracking,
THEN cold in situ recycle + overlay
OR hot recycle + overlay
OR cold mill + overlay
OR pulverization + overlay
OR remove and replace with asphalt (reconstruction).

RULE NUMBER: 237
IF existing cracking should be destroyed
AND distress type set contains swelling
AND the moisture condition of the pavement is stable,
THEN cold mill + overlay.
(Note: if this rule is invoked, PARES will query the user about the moisture condition of the road.)

Another escape route designed into PARES is the situation requiring a rigid pavement rehabilitation scheme. Since PARES is an expert system for flexible pavements, it treats a situation requiring a concrete pavement as a special situation. The following rule governs in this situation:

RULE NUMBER: 254
IF reconstruction is a recommended rehabilitation strategy
AND the average daily traffic exceeds 30,000
AND the expected design life desired is greater than 10 years
AND the time since the last repair on this road is less than 10 years,
THEN go concrete.
(Note: if this rule is invoked, PARES will query the user about the time since last repair.)

PARES even has logic built into it to avoid overlapping repair strategies. An example of this logic is the following rule:

RULE NUMBER: 252
IF crack seal
AND overlay
AND overlay + crack seal, all are recommended strategies,
THEN crack seal + overlay will cover all the situations.

ADDITIONAL FEATURES OF THE KNOWLEDGE BASE

If the user specifies that alligator cracking is one or more of the distresses, PARES will ask the user to specify the type of alligator cracking present and to determine whether the distress is primarily related to surface or subgrade problems. PARES asks the user this question whenever the first rule involving alligator cracking is addressed in the inferencing process. A typical question would be the following:

The alligator cracking is (choose one or more of the following)

1. Type A [alligator cracking in the surface only (i.e., due to loading fatigue)]
2. Type B (alligator cracking as a result of a subgrade problem)
3. Type C [localized alligator cracking (i.e., low extent in a small portion of the roadway surface)].

PARES can also query the user as to the type of rutting, if rutting is a distress indicated by the user. More information on rutting is available from Pavlovich et al. (*17*). A typical question is

The rutting is (choose one or more of the following)

1. Type A (indicates a subgrade problem)
2. Type B (indicates a surface mix material problem).

Both of the preceding user questions involve visual inspection of the road surface, and the user is assisted in PARES with a schematic illustrating Type A and B rutting.

To assist in determining milling depth, if milling is necessary, or to determine the amount of crack seal to apply, if crack seal is necessary, PARES will query the user about the depth of surface cracking. A typical question is

The depth of cracking is known to be ___ (inches) (user fills in the blank).

An additional feature in PARES is that it has the capability to calculate mill depths and overlay depths for specific rehabilitation strategies used in New Mexico; hence it is a design tool. If the user has provided PARES with surface cracking depths, these values are used for the milling depths. If the user does not know these values, PARES estimates the depths on the basis of the level of severity of the cracking (e.g., low severity implies mill depth = 1 in., medium severity implies mill depth = 2.5 in., etc.). The layer (structural) coefficient (SC) for the old pavement is input by the user (see example later), and this value, along with the difference between SN_{new} and SN_{old}, is used to determine the new overlay thickness, if

required. In PARES, new or recycled asphalt material carries an SC of 0.4, and existing pavement carries a value for SC between 0.1 and 0.4 depending on user judgment.

OPERATION OF THE PARES CODE

Complete details of the physical operation of the PARES code are addressed elsewhere (*1*). A single screen data entry interface (see Figure 4a) minimizes the user's interaction with EXSYS. Built-in error checking is accomplished on the information entered by the user. The interface also contains an explanation window, which is designed to be helpful to the user in terms of additional on-line help to explain what is to be entered in each field (or section) for the infrequent user. In PARES a cost module was implemented to rank rehabilitation strategies. The costs estimated are for construction of the roadway surface only. The cost module was designed to be interactive with the user and to have considerable flexibility. The user can use a default unit cost file, call his own unit cost file, or change unit costs during run time in a PARES session.

TYPICAL EXAMPLE OF A PARES SESSION

A typical session of the PARES code is shown in Figure 4. This session was compared with an actual rehabilitation test job done in New Mexico in early 1988 near Cimarron. Figure 4a shows the input for a pavement segment 0.2 mi in length, with a layer coefficient (SC) of 0.2 for the existing surface and other parameters as shown. The user specifies the input that is known (PARES can run with incomplete input data). Such quantities as the equivalent single-axle load (ESAL) and the required new structural number, SN_{new}, are calculated from information provided by the user (see Figure 4a). The rules used within a typical PARES session are a function of the distress information provided by the user. Recommended rehabilitation strategies ranked according to cost of the constructed surface (exclusive of shoulders) are shown in Figure 4b.

In this example, six strategies are recommended with the top two being very competitive in cost according to New Mexico practice. To illustrate how PARES selects various rehabilitation strategies, the inferencing involved in the first strategy, cold in situ recycling 2.5 in. + overlay 1.75 in., will be described. Only a few of the rules listed in this example session of PARES appear in this paper.

Since in this example alligator cracking was provided twice by the user as two of the distress conditions, PARES queries the user as to the types of alligator cracking; the user answers "Types A and C." PARES uses this information to invoke rules 2, 34, and 69 to classify the distress into surface cracking, and it counts three surface cracking distress conditions. Then it invokes rules 76 and 77 to infer that rehabilitation is needed, not maintenance alone (see Figure 1). PARES invokes Rule 102 to determine that the existing strength of the pavement is inadequate and that the pavement needs more strength. Rule 111 then determines that the surface cracking matrices (Figure 3) should be used to estimate the needed repair level. Rules 113, 148, and 190 are then invoked for alligator cracking, longitudinal cracking, and transverse cracking, respectively, to determine that a medium repair level is needed. Rule 198 uses the results of the recommended repair level and the fact that three surface cracking conditions exist to determine that destruction of the existing cracks is necessary before any new overlay. Finally, Rules 224, 228, and 229 use these results and conduct simple calculations to recommend that the rehabilitation strategy should first cold in situ recycle 2.5 in. of the old pavement structure (this repairs the existing cracks), then add 1.75 in. of new overlay (this reinforces the pavement structure to the recommended strength).

(a)

Number of Lanes: 2	Roadway Length: 0.20 miles
Roadway width: 24 feet	Regional Factor: 1.5
Design Life: 10 years	ESAL: 2920000
SC for old AC: 0.2	Serviceability Index: 2.0
Estimated R-Value: 45	Existing SN: 1.8

DISTRESSES

Type	Severity	Extent
Transverse Cracking	Low	1000 feet
Transverse Cracking	Medium	300 feet
Alligator Cracking	Low	15%
Alligator Cracking	Medium	5%
Longitudinal Cracking	Medium	300 feet

Calculated future SN: 3.068 Exit (Y/N)?:

EXPLANATIONS

.....this window provides the user with explanations and instructions for any of the fields in which the computer cursor is awaiting data entry.....

(b)

Rehabilitation Strategies	Cost ($)
Cold insitu recycling 2.5 inches + overlay 1.75 inches	13,636
Hot recycling 2.5 inches + overlay 1.75 inches	13,693
Cold mill 2.5 inches + overlay 4.25 inches	26,660
Cold mill 2.5 inches + interlayer + overlay 4.25 inches	33,700
*Pulverization + overlay	59,523
*Reconstruction - remove and replace asphalt	69,661

Additional Construction	Cost ($)
Dig out + patch (These are supplemental maintenance strategies)	5,333

* This strategy includes an estimated overlay pavement depth

FIGURE 4 (*a*) Typical input screen and (*b*) typical results screen with cost ranking in PARES.

CONCLUSIONS

An expert system for preliminary pavement rehabilitation design for flexible pavements in New Mexico has been described. Its implementation when compared with an actual New Mexico rehabilitation project is illustrated. The rehabilitation strategy used on the actual project was one of the recommended strategies developed by PARES. The system currently is in use in New Mexico and has been shown to be both a rapid initial estimator of rehabilitation job costs and a tool for new engineers to understand and learn current procedures used by expert designers.

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Abridgment

Interlayers on Flexible Pavements

HONG-JER CHEN AND DOUGLAS A. FREDERICK

A study was initiated to evaluate the effectiveness of stress-relieving interlayers in reducing reflective cracking on asphalt overlays over existing flexible pavements. Six lane-wide interlayers were installed on three construction projects under New York's two standard overlay thicknesses (1 and 2½ in.). Strip interlayers 1 ft wide were also placed on two additional construction projects to cover individual transverse cracks. The strip applications failed within 1 year and were considered inappropriate for future use. Performance of full-lane sections was monitored for 7 years. From statistical analysis it is concluded that overlays with interlayers have lower average crack returns than those without them. Coring showed that half the interlayers at cracked areas did not remain intact. Results of a simplified life cycle cost analysis indicated that interlayer treatments were not cost-effective compared with normal overlays. However, a 1-in. overlay with interlayer was shown to be more economical than a 2½-in. overlay without one. Interlayer products should continue to be considered as experimental features.

The purpose of an overlay is to extend the service life of an existing pavement by restoring its riding quality and correcting its structural deficiencies. Reflective cracking caused by propagation of existing cracks or joints in the original pavement up through the new surface, however, is a problem that has long troubled highway engineers. New York conducted a study (*1*) to address reflective cracking in asphalt overlays on rigid pavements. Methods investigated included bond breakers, membrane reinforcement, sawing and sealing, breaking and seating, and thicker overlays. The sawing-and-sealing method [sawing joints in the new surface directly over those in the original pavement and sealing them, expecting cracks to reflect through the sawed joints (*2*, p. 47)] was found to be the most effective. For severely deteriorated pavements where slabs were not intact, breaking-and-seating or "rubblizing" methods were recommended.

The reflective cracking problem, however, is not unique to rigid pavements, but occurs over flexible pavements as well. Reflected cracks lead to premature failure of overlays by allowing water to enter the subbase and cause loss of support.

In the early 1970s manufacturers promoted use of geotextiles as stress-relieving interlayers. Proprietary stress-relief systems using rubberized asphalt made from waste tires were also developed. These various materials received extensive national attention, and it was decided to initiate a study to determine whether they could be cost-effective in reducing reflective cracking in overlaid asphalt pavements in New York State.

The benefits claimed for interlayers were (*a*) increased overlay service life and (*b*) cost savings because thinner overlays could be used. This paper summarizes 7 years of evaluating performance of interlayers between asphalt pavements and overlays. Construction details were reported in the study's interim report (*3*).

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INVESTIGATION

Materials

Materials selected for this study may be classified into two general categories: applied full-lane width and applied in strips over single cracks (Table 1). They were designed to provide stress-relieving overlay reinforcement and an impervious membrane to prevent water intrusion. Strip materials to cover individual transverse cracks were supplied in 1-ft-wide rolls. Full-width materials to cover an entire lane were used over more extensively cracked pavements.

Test Sites and Construction

Three sites were selected for full-lane applications and two for strips. The two strip applications were on Routes 156 and 9, both near Albany. Route 156 and most of Route 9 were conventional flexible pavements, but portions of Route 9 were composite (i.e., asphalt over concrete). Full-lane interlayers were installed on I-481 in Syracuse, Route 10 in Schoharie County, and Route 12 in Jefferson County. All were flexible pavements. Figure 1 shows a Route 12 cross section and its structural components. New York's two standard overlay thicknesses (1 and 2½ in.) were both placed on Routes 10 and 12; only the 1-in. overlay was placed on I-481. Figure 2 shows layouts of the test sites. In all, there were 28 control sections and 22 treated sections. I-481 had both temperature and load-associated cracking. On Route 10, wheelpath alligator cracking was the predominant distress, plus some areas of edge cracking. Block cracking was extensive on Route 12. All test sites were overlaid in 1980 and 1981. Several problems were encountered, including improper application rate, wrinkling during placement, and insufficient overlay thickness. I-481 was given another 1-in. overlay course in 1982, when the 1980 overlay was found to be only ⅜ to ½ in. thick. In some areas on Route 10, severe rutting and edge cracking resulted in truing-and-leveling courses of up to 5 in. being placed before overlay.

Performance Evaluation

Cracks were sketched on survey sheets by surveyors walking along shoulders. Individual cracks were measured in linear

TABLE 1 Summary of Materials Tested

Brand Name	Manufacturer	Description
FULL-WIDTH APPLICATION		
Petromat	Phillips Fibers Corp.	Non-woven polypropylene fabric
Mirafi	Celanese Fibers Marketing Co.	Heat-stable polyester and polypropylene woven fabric[a]
Typar/ReePav[b]	E.I. duPont de Nemours & Co.	Spun-bonded polyester fabric[a]
Bidim[c]	Monsanto Textile Co.	Non-woven polyester fabric (Style C-22)
Propex	Amoco Fabrics Co.	Non-woven polypropylene (Style CEF4545)
Arm-R-Shield	Arizona Refining Co.	Blend of reclaimed rubber and modified asphalt cement applied as a binder coat with a subsequent layer of stone chips
STRIP APPLICATION		
Bituthene	W.R. Grace and Co.	Polypropylene woven mesh laminated to a layer of self-adhesive rubberized-asphalt
Roadglas	Owens-Corning Fiberglas Corp.	Composite system of high-strength fiber glass woven roving and a proprietary hot asphalt binder (Roadbond)
Polyguard	Polyguard Products Inc.	Rubberized asphalt waterproofing element with a polypropylene mesh laminated to the outer surface

[a] 1980 and 1981 fabric weights differed.
[b] Introduced experimentally in 1980 as Style T-323 under the brand name "Typar"; reintroduced at a different weight in 1981 as Style T-376 under the brand name "ReePav."
[c] Manufacturer discontinued production of this engineering fabric in 1981.

feet and alligator cracks in square feet. They were summed for each test section and reported in linear feet. All sections were surveyed before overlay and annually through 1987. A measure of performance called crack return percentage was obtained by dividing the amount of cracking in 1987 by the amount existing before overlay. Crack density was also calculated, defined as linear feet of cracking per 100 lane-ft, providing the extent of cracking in each section regardless of its length.

Benkelman beam deflections were measured to examine structural adequacy and uniformity among sections at each site. Several cores were taken in 1987 from Route 12 to check the manufacturer's claim of the fabric's ability to remain intact and keep water from entering the subbase.

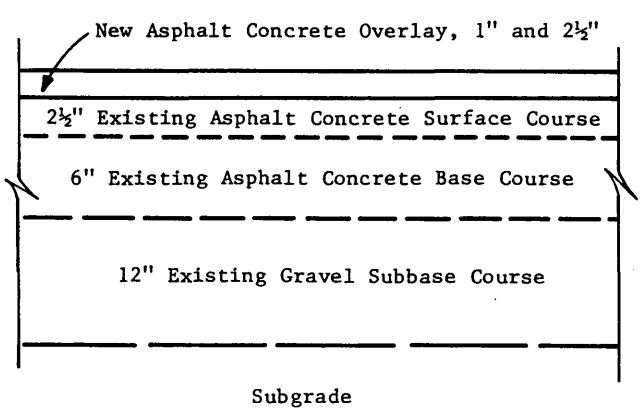

FIGURE 1 Pavement cross section on Route 12.

RESULTS AND DISCUSSION

Strip Applications

Cracks in all treated and control sections reflected through the overlay in the first winter. Cracks over the fabrics required more maintenance than those without fabrics. Overlays with interlayers raveled and delaminated, thus requiring patching. Cracks in control sections only needed sealing.

Full-Lane Applications

After 7 years average crack return was about 20 to 30 percent. Crack returns were generally lower on interlayered sections

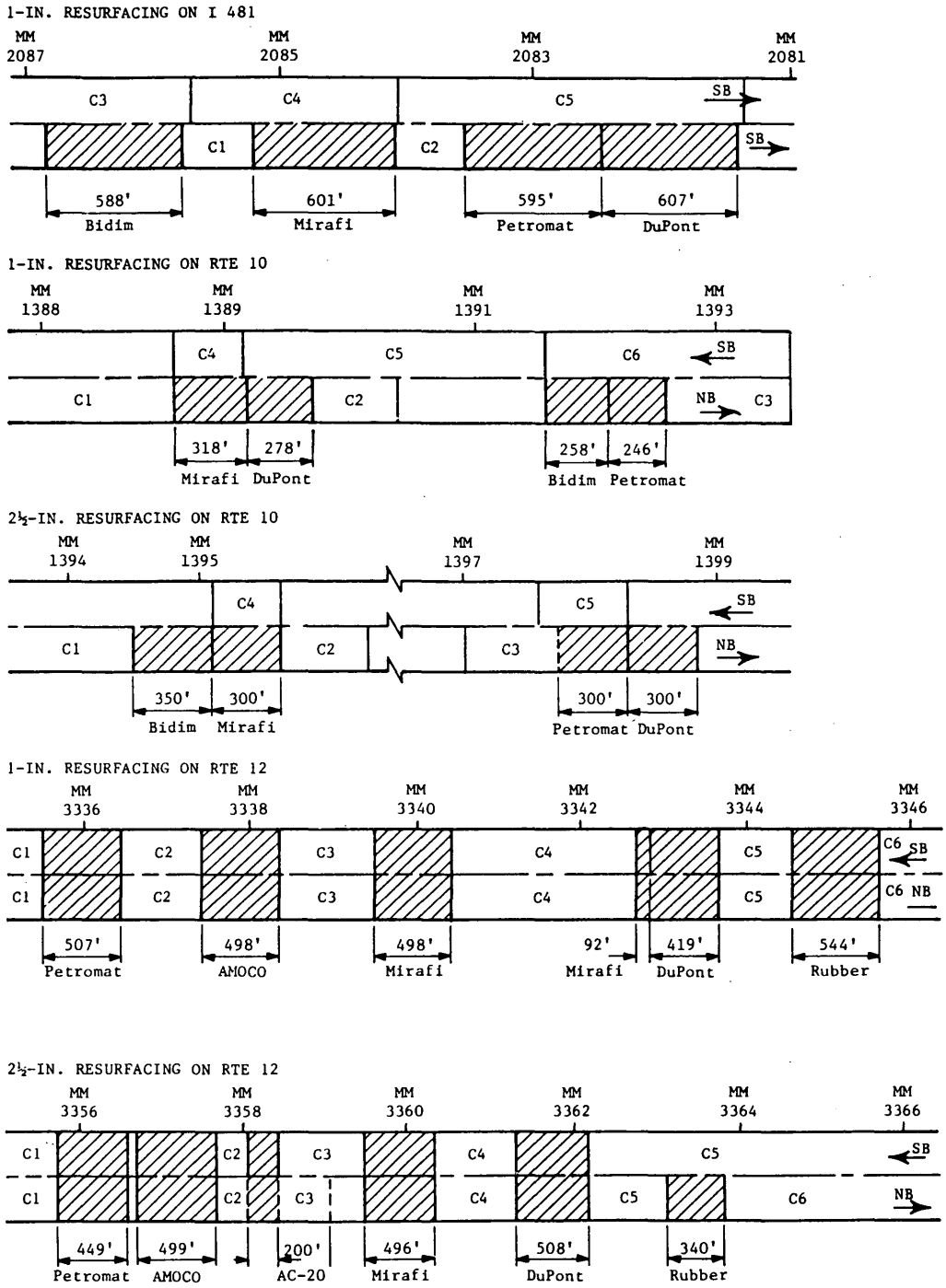

FIGURE 2 Test section layouts.

than on the controls. Crack-return ratios of interlayered to control sections, after eliminating some extreme cases, ranged from 40 to 70 percent. Route 10 had a nonuniform condition due to the previously mentioned localized distress and edge failures. Deflection measurements also showed this nonuniformity among sections. Route 10 data thus were discarded from the analysis. Condition on Route 12 was relatively uniform and hence offered consistent results. Because of the added 1-in overlay on I-481, the overlay (with an actual thickness of about 1½ in.) had a performance between that of 1- and 2½-in. overlays. Average crack returns of control sections on Route 12 for 1- and 2½-in. overlays were 52 and 14 percent, respectively. The average for I-481, after eliminating one extreme section, was 26 percent.

A *t*-test was used to assess effectiveness of interlayers in reducing reflective cracking. The test sections are assumed to represent typical conditions and are random samples. Testing was performed separately for 1- and 2½-in. overlay sections. Detailed testing [given in this study's final report (*4*)] shows that for 1-in. overlay sections on I-481 and Route 12, the null

hypothesis of mean percentage of crack return on treated sections equaling or exceeding that on control sections is rejected at the 0.025 significance level. On sections with 2½-in. overlays on Route 12, the null hypothesis can be rejected at a significance level of 0.05 and the alternative hypothesis favored. This supports the conclusion that interlayers are effective in reducing crack return. Using the same testing procedure on Route 12 for percentage of crack return on sections with 2½-in. versus 1-in. overlay, crack return for 2½-in. cover is significantly less than with 1-in. cover, justifying the conventional approach of using thicker overlays.

A comparison of average crack return between this study and two previous New York State studies (*5,6*) shows that crack return on 1-in. overlays in this study is similar to those of the two other studies, but those on 2½-in. overlays have lower return percentages.

Besides overlay thickness and interlayer treatment, a third factor—crack density of the pavement before overlay—was found to affect return of cracking. An "increase-decrease" relationship was found between these two variables for control sections with 1-in. overlays. This could probably explain the lower crack return percentages in 2½-in. sections just discussed because both Routes 10 and 12 had very high initial crack densities—202 and 484, respectively. Possible explanations for this phenomenon were found and are discussed in the final report (*4*).

Results of coring on Route 12 pavements showed that about half the interlayers did not remain intact when cracks reflected through them. The benefit of keeping water from entering the original pavement is under discussion. If water enters cracks in the overlay and is retained by an interlayer, it may cause pumping, stripping, or huge pressure buildups. Whether this is an advantage or disadvantage was not examined.

Economic Analysis

Route 12 was chosen for economic analysis to represent the whole study. Overlay lives of the four alternatives were determined by defining a failure criterion and extrapolating the reflective crack progressing trend (*4*). The resulting lives are as follows:

Sections	*Years in Service*
1-in. control	8
1-in. treated	11
2½-in. control	12
2½-in. treated	15

On the basis of these overlay lives, 1987–1988 bid prices on interlayers and asphalt concrete, and a discount rate of 4 percent assumed by New York State Department of Transportation, simplified life cycle costs were analyzed to see whether interlayers were cost-effective alternatives (*4*). Three analyses were conducted: (*a*) 1-in. overlay with and without interlayer, (*b*) 2½-in. overlay with and without interlayer, and (*c*) 1-in. overlay with interlayer versus 2½-in overlay. Results of the first two analyses showed that interlayers offered no cost benefits for either 1- or 2½-in. overlays, given the relative life of each alternative. The third analysis examined substitution of 2½-in. overlay for 1-in. interlayered overlay. Costs of overlaying shoulders were included. Results indicated that the 1-in. interlayer option is cheaper.

This simplified analysis could be viewed as only an approximate assessment of relative benefits among treatments. For interlayer treatments on pavements receiving both overlay thicknesses, the 3-year extended life does not warrant the expense. Interlayers are more cost-effective for 2½-in. overlays than for 1-in. overlays. Because the shoulder is involved in the cost, the 1-in. interlayer alternative is more economical than the conventional 2½-in. overlay. The complexity and difficulty involved in determining the lives preclude general conclusions as to benefits of interlayers. Engineers should continue to consider this option on an experimental basis.

Other Considerations

Factors other than performance and economy should be considered for overlay projects. If distresses other than cracking are present—for example, rutting, edge cracking, or local depressions—they may call for a truing-and-leveling course before overlay. This additional asphalt thickness would also reduce reflective cracking. Other construction procedures are also available for cracked pavements, such as milling before overlay and cold in-place recycling. These procedures may be more cost-effective than interlayers.

CONCLUSIONS

1. Test sections treated with strips all failed. These applications consequently should not be considered for further use.
2. Statistical analysis indicated that overlays with full-lane interlayers had lower average crack return percentages than those without them. Testing also confirmed that 2½-in. overlays had significantly less cracks reflected than 1-in. overlays.
3. An increase-decrease relationship was found between crack densities on overlaid pavements and crack return percentages on overlays. Possible explanations were also found.
4. Coring results indicated that some fabrics did not remain intact. The benefit of keeping water from infiltrating into the subbase is questionable.
5. Simplified life cycle cost analyses performed for the four alternatives on Route 12 showed that interlayers were not cost-effective compared with normal overlays for both overlay thicknesses. The 1-in. interlayer option was cheaper than the normal 2½-in. option, but this analysis was limited in scope and based on many assumptions that may be subject to discussion.

In summary, there is no question regarding abilities of stress-relieving interlayers, if installed properly, to reduce or delay reflective cracking on overlays over flexible pavements. Their cost-effectiveness, however, depends on how long they can delay cracks from occurring. For heavily cracked Route 12, interlayers were more effective for 2½-in. than for 1-in. overlays. Because mixed results were obtained, engineers should continue to consider using various interlayer products primarily on an experimental basis. Other techniques should also

be considered when a flexible pavement overlay is being designed.

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