Asphalt Concrete Overlay Design Methodology for Fractured Portland Cement Concrete Pavements

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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

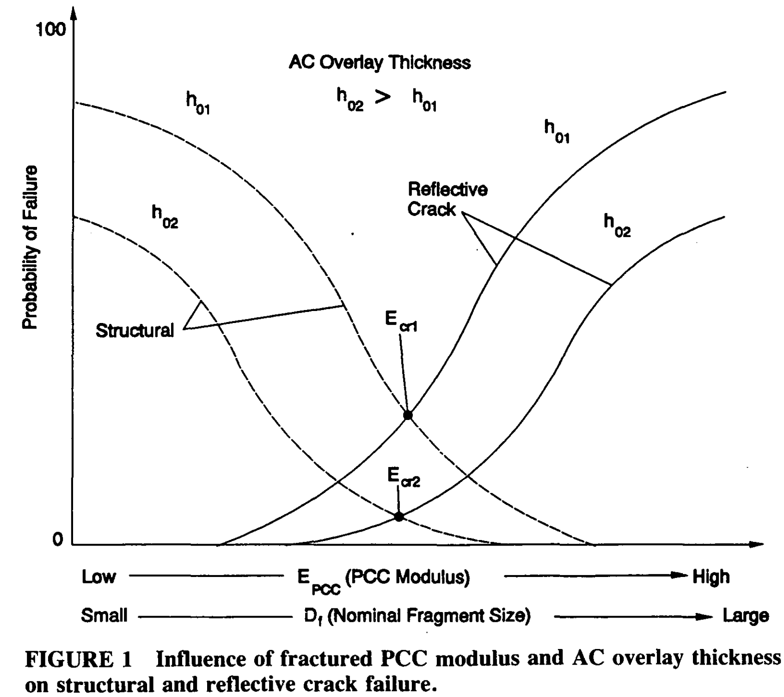


FIGURE 1 Influence of fractured PCC modulus and AC overlay thickness on structural and reflective crack failure.

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.

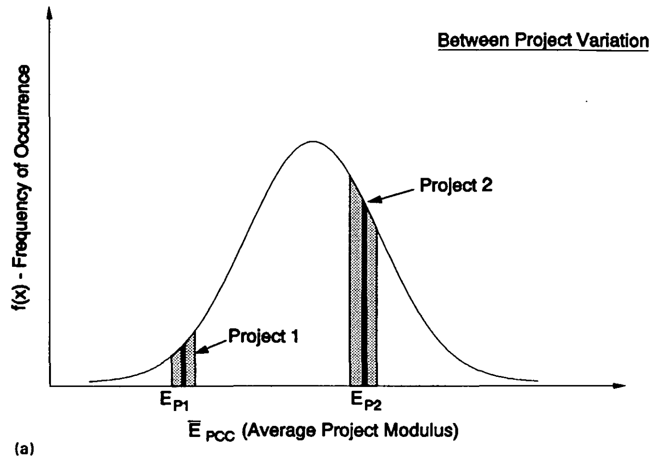
There is, however, another major consideration in the design and construction process. It should be clearly recognized that as PCC pavements are fractured, they become and act more like flexible pavement systems rather than PCC pavements designed for rigid slab action. The implication of this fact is 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



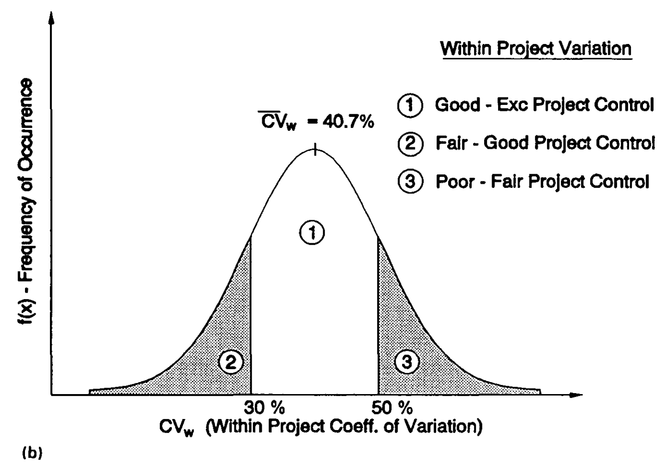


FIGURE 2 Between and within project E_{PCC} variability: (*a*) frequency distribution of average project means; (*b*) frequency distribution of within project CV value.

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:

$$a_i = a_s \sqrt[3]{\frac{E_i}{E_s}} \qquad (1)$$

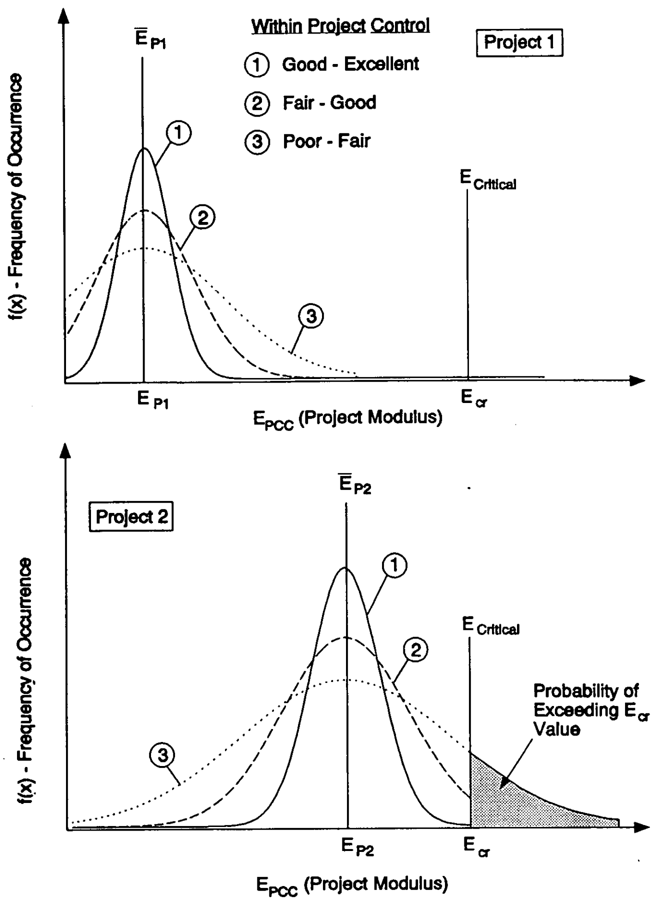


FIGURE 3 Fractured slab modulus frequency distributions reflecting between and within project variability.

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

$$a_i = 0.0045\sqrt[3]{E_i} \qquad (2)$$

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

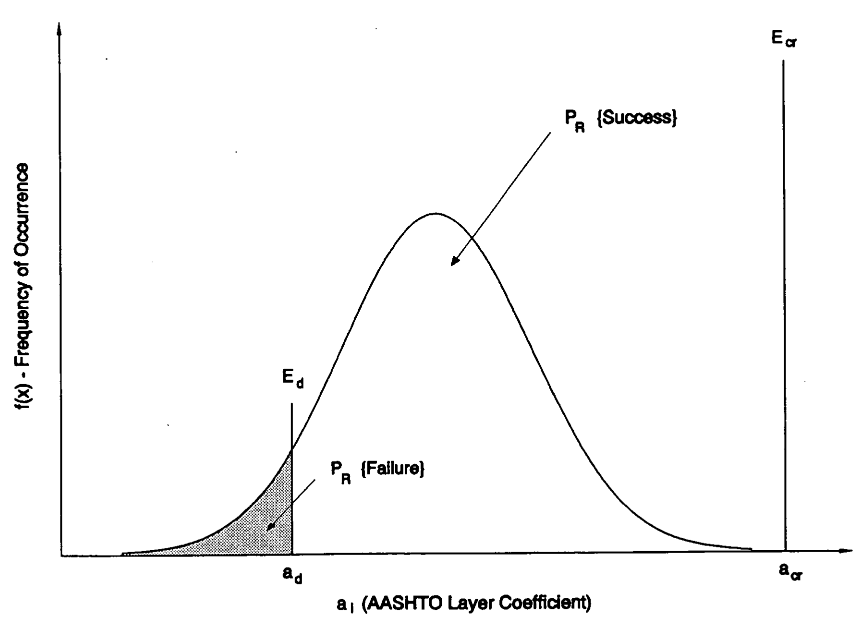


FIGURE 4 Frequency distribution of the AASHTO layer coefficient for fractured PCC slabs.

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

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

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,
- $\overline{E_{PCC}}$ = 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 24	412.5 ksi 501.8 ksi	154.4 ksi 338.9 ksi	37.4% 67.5%	Recommended (excludes 2 outliers) All data
Crack/Seat	46 64	409.0 ksi 780.6 ksi	140.7 ksi 665.6 ksi	34.4% 85.3%	Recommended (excludes all values greater than 1000 ksi i.e., Crack spacings 48" or greater) All data
Break/Seat	52	1271.5 ksi	548.7 ksi	43.2%	All data; Recommended
Combined	120 140	783.4 ksi 915.1 ksi	377.4 ksi 578.0 ksi	48.2% 63.2%	All Recommended results 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-

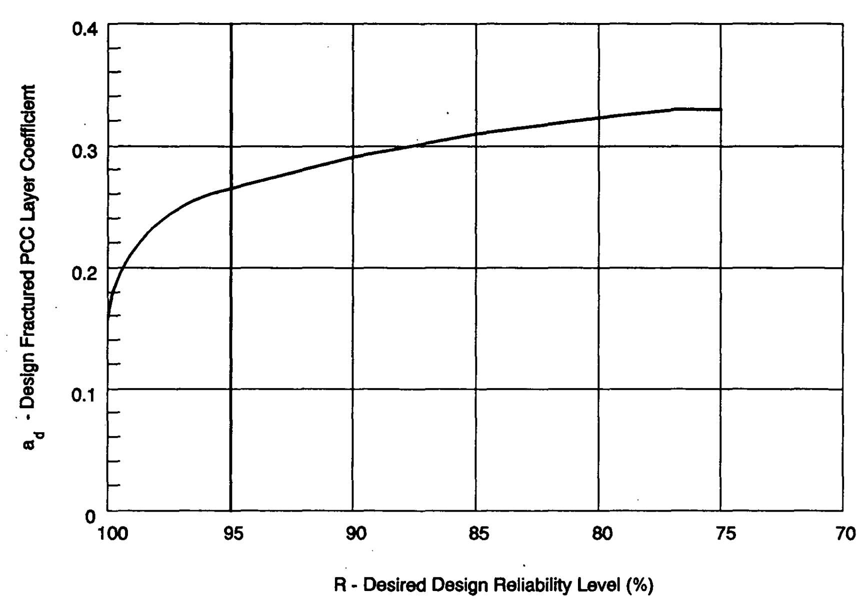


FIGURE 5 Recommended AASHTO structural layer coefficient for rubblized and crack/seat PCC layers as a function of desired reliability.

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., $K_\beta = 1.037$ 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

$$h_o = 9.73 - 20.45a_d$$

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, $\sigma_w = 385$ 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, $\sigma_w = 129$ ksi, and $CV_w = 24.5$ percent. For the criterion of $\alpha = 5$ 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

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

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.