Revision of AASHTO Pavement Overlay Design Procedures

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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 $SC_f - SC_{eff} = SC_{ol}$)

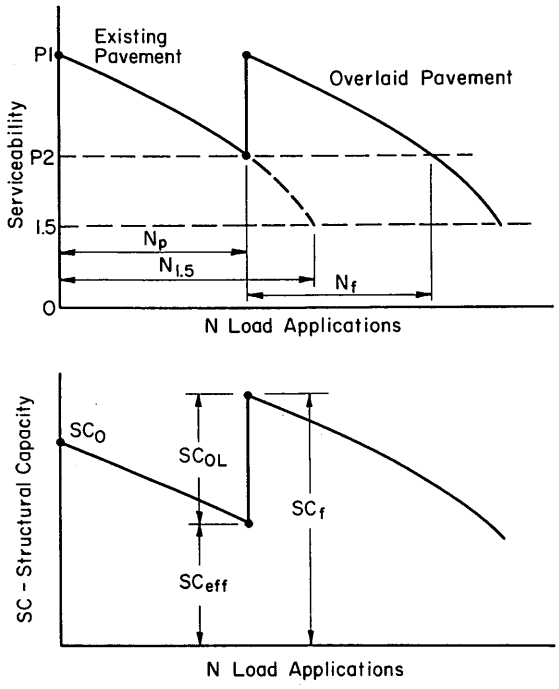

FIGURE 1 Structural capacity loss over time and with traffic.

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-

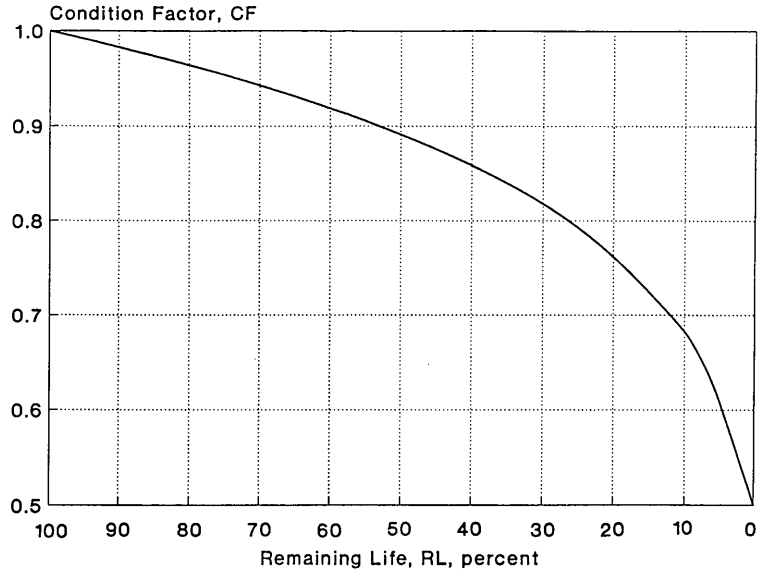

FIGURE 2 Relationship between condition factor and remaining life.

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.

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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.