Field Testing of AASHTO Pavement Overlay Design Procedures

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

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

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

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

DESCRIPTION OF FIELD TESTING PROCEDURES

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

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

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

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

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

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

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

EXAMPLES OF AC OVERLAY DESIGN FOR AC PAVEMENT

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

TABLE 1 Example AC Overlay Design for AC Pavement

REVISED CHAPTER 5 AASHTO DESIGN GUIDE OVERLAY DESIGN

NW-1 AC OVERLAY OF CONVENTIONAL AC PAVEMENT

EXISTING PAVEMENT DESIGN

AC SURFACE	4.25 inches	SUBGRADE SANDY SILT, SANDY GRAVEL
GRAN BASE	8.00	
GRAN SUBBASE	0.00	
TOTAL THICKNESS	12.25	

Future design lane ESALs = 2400000 (FLEXIBLE ESALs)

DETERMINE SNf

Vary trial SNf until computed ESALs equal future design ESALs.

SNf	MR,psi	R	Z	So	P1	P2	ESAL
3.60	5634	50	0	0.45	4.2	2.5	2417312
4.14	5634	80	0.841	0.45	4.2	2.5	2430778
4.44	5634	90	1.282	0.45	4.2	2.5	2429228
4.69	5634	95	1.645	0.45	4.2	2.5	2408097
5.19	5634	99	2.327	0.45	4.2	2.5	2403245
TRIAL			INPUT	INPUT	INPUT	INPUT	

DETERMINE SNeff BY NDT METHOD

Vary trial Ep/MR until computed D0 equals actual value.

		ACTUAL		SUBGRADE		TRIAL	COMPUTED	
LOAD,lbs	D0,mils	Dr,mils	MR,psi	C FACTOR	Ep/MR	D0,mils	Ep,psi	SNeff
9000	12.80	3.55	16901	3	8.45	12.80	142817	2.88
		r =	36 inches					
	Check r > 0.7 ae =		17.95 inches					

DETERMINE SNeff BY CONDITION SURVEY METHOD

LAYER	STR COEF	DRAIN m	SNeff
AC SURFACE	0.35	1.00	1.49
BASE	0.14	1.00	1.12
SUBBASE	0.00	1.00	0.00
		SNeff =	2.61

DETERMINE SNeff BY REMAINING LIFE METHOD

Past design lane ESALs = 400000 (FLEXIBLE ESALs)

LAYER	THICK,in	NEW ST CF	SNo
AC SURFACE	4.25	0.44	1.87
BASE	8.00	0.14	1.12
SUBBASE	0.00	0.00	0
TOTAL	12.25		2.99

SNo	MR,psi	Z	So	P1	P2	N1.5	RL, %	CF	SNeff
2.99	5634	0	0.45	4.2	1.5	1140161	65	0.93	2.78
			INPUT	INPUT	INPUT				

DETERMINE OVERLAY THICKNESS AC OL structural coefficient = 0.44

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

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

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

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

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

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

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

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

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

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

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

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

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

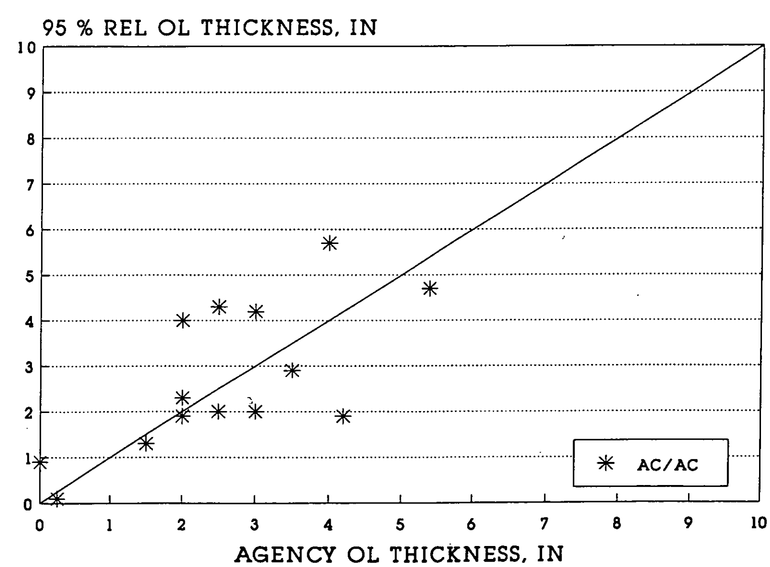

FIGURE 1 Comparison of AASHTO AC overlay thicknesses and agency AC overlay thicknesses for AC pavements (95 percent reliability).

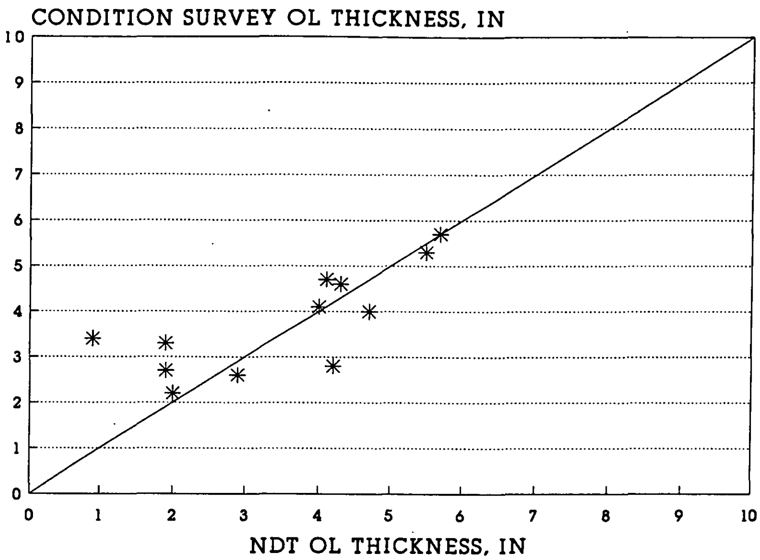

FIGURE 2 Comparison of AC overlay thickness determined by NDT and condition survey procedures for AC pavements (95 percent reliability).

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

EXAMPLES OF AC OVERLAY DESIGN FOR FRACTURED PCC PAVEMENT

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

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

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

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

EXAMPLE AC AND BONDED PCC OVERLAY DESIGN FOR JPCP AND JRCP

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

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

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

TABLE 2 Example AC Overlay Design for Fractured PCC Pavement

REVISED CHAPTER 5 AASHTO DESIGN GUIDE OVERLAY DESIGN

SW-6 AC OVERLAY OF CRACKED/SEATED JPCP (PROJ STN 353)

EXISTING PAVEMENT DESIGN

RUBBLIZED PCC	8.20 inches
C.T.BASE	3.70
SUBBASE	0.00
TOTAL THICKNESS	11.90

Future design lane ESALs = 7370000 (2/3 OF 11000000 USED AS FLEXIBLE ESALs)

DETERMINE SNf

Vary trial SNf until computed ESALs equal future design ESALs.

SNf	MR,psi	R	Z	So	P1	P2	ESAL
4.50	4350	50	0	0.49	4.5	2.5	7364787
5.15	4350	80	0.841	0.49	4.5	2.5	7516147
5.50	4350	90	1.282	0.49	4.5	2.5	7452560
5.80	4350	95	1.645	0.49	4.5	2.5	7401524
6.40	4350	99	2.327	0.49	4.5	2.5	7354079
TRIAL			INPUT	INPUT	INPUT	INPUT	

DETERMINE SUBGRADE MR BY NDT METHOD

Vary trial Ep/MR until computed D0 equals actual value.

STATION	LOAD,lbs	ACTUAL D0,mils	Dr,mils	SUBGRADE MR,psi	C FACTOR	TRIAL Ep/MR	COMPUTED D0,mils	Ep,psi
	8952	6.31	3.43	17399	4	44.00	6.32	765574

r = 36 inches

Check r > 0.7 ae = 29.70 inches

DETERMINE SNeff

LAYER	STR COEF	DRAIN m	SNeff
RUBBLIZED PCC	0.35	1.00	2.87
C.T.SUBBASE	0.15	1.00	0.56
SUBBASE	0.00	1.00	0.00
		SNeff =	3.43

DETERMINE OVERLAY THICKNESS   AC OL structural coefficient = 0.44

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

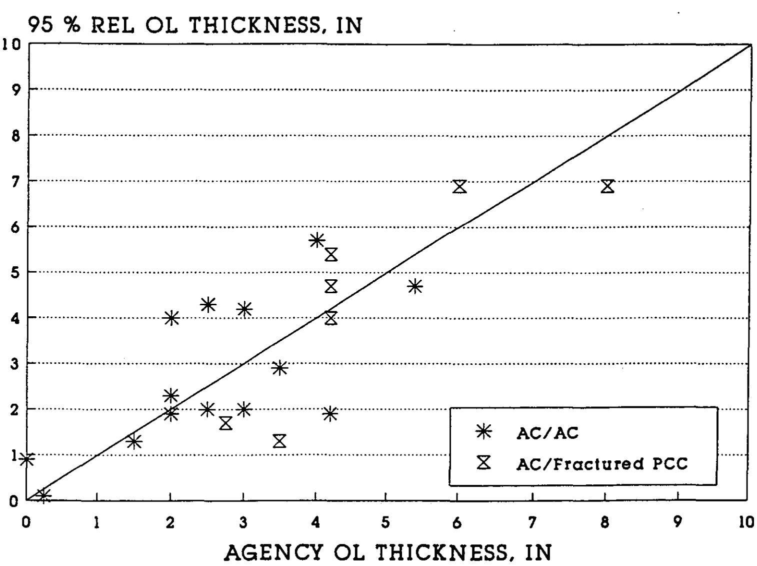

FIGURE 3 Comparison of AASHTO AC overlay thicknesses and agency AC overlay thicknesses for fractured PCC pavements (95 percent reliability).

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

REVISED CHAPTER 5 AASHTO DESIGN GUIDE OVERLAY DESIGN

MW-7 AC AND BONDED PCC OL OF EXISTING JRCP

EXISTING PAVEMENT DESIGN AND FUTURE TRAFFIC
Slab thickness 10.00 (in)
Future design lane ESALs = 10000000 (10 YEARS)

BACKCALCULATION OF Keff AND Ec

INPUT LOAD (lbs)	INPUT D0 (mils)	INPUT D12 (mils)	INPUT D24 (mils)	INPUT D36 (mils)	AREA (in)	RADIUS RELSTIFF (in)	Kdyn (pci)	SLAB Ec (psi)
11144	4.39	3.97	3.49	3.01	30.51	36.16	239	4.8E+06
10864	4.90	4.57	4.18	3.70	31.96	45.36	133	6.6E+06
10928	4.51	4.09	3.69	3.14	30.88	38.12	206	5.1E+06
10824	4.55	4.17	3.77	3.30	31.29	40.58	179	5.7E+06
							189	5.6E+06

DETERMINE Df

Vary trial Df until computed ESALs equal future design ESALs.

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

TRIAL Df (in)	R	Z	COMPUTED ESALs (millions)
8.70	50	0	9973718
9.70	80	0.84	10214587
10.30	90	1.282	10619093
10.80	95	1.645	10851107
11.60	99	2.327	10095415

DETERMINE Deff
INPUT Fjc = 0.97 (10 FAILURES/MI UNREPAIRED)
INPUT Ffat = 0.95 (50 MIDSLAB WORKING CRACKS)
INPUT Fdur = 1.00

Deff (in) = Fjc * Fdur * Ffat * Dexist = 9.22

DETERMINE OVERLAY THICKNESS

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

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

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

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

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

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

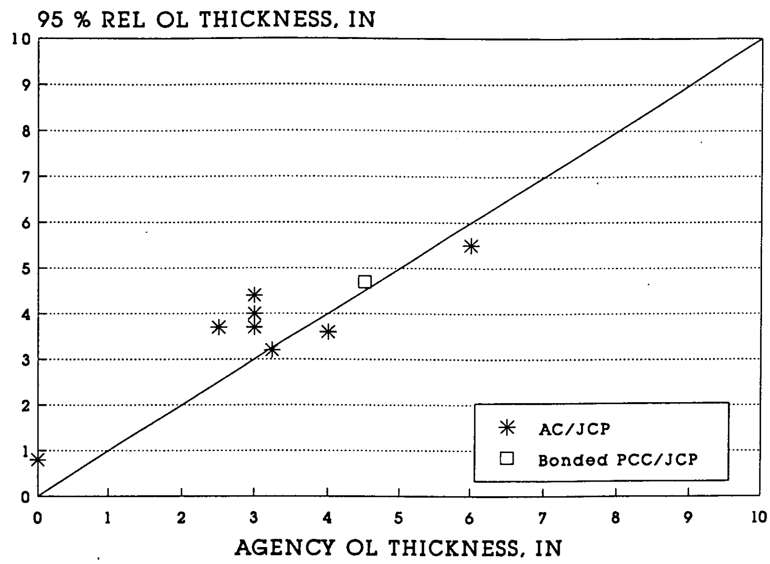

FIGURE 4 Comparison of AASHTO and agency AC overlay thicknesses and bonded PCC overlay thicknesses for JRCP and JPCP (95 percent reliability).

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

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

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

EXAMPLE AC AND BONDED PCC OVERLAY DESIGN FOR CRCP

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

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

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

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

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

EXAMPLE AC OVERLAY DESIGN FOR AC/PCC PAVEMENT

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

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

REVISED CHAPTER 5 AASHTO DESIGN GUIDE OVERLAY DESIGN

MW-9 AC AND BONDED PCC OVERLAY OF EXISTING CRCP

EXISTING PAVEMENT DESIGN AND FUTURE TRAFFIC
Slab thickness 8.00 (in)
Future design lane ESALs = 18000000 (5% ESAL GROWTH RATE, 10 YEARS)

BACKCALCULATION OF Keff AND Ec

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

DETERMINE Df
Vary trial Df until computed ESALs equal future design ESALs.

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

TRIAL Df (in)	R	Z	COMPUTED ESALs (millions)
7.40	50	0	17743150
8.40	80	0.84	18815526
8.90	90	1.282	18776535
9.30	95	1.645	18436227
10.10	99	2.327	17988918

DETERMINE Deff
INPUT Fjc = 0.96
INPUT Ffat = 0.98
INPUT Fdur = 0.85 ("D" CRACKING)
Deff (in) = Fjc * Fdur * Ffat * Dexist = 6.40

DETERMINE OVERLAY THICKNESS

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

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

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

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

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

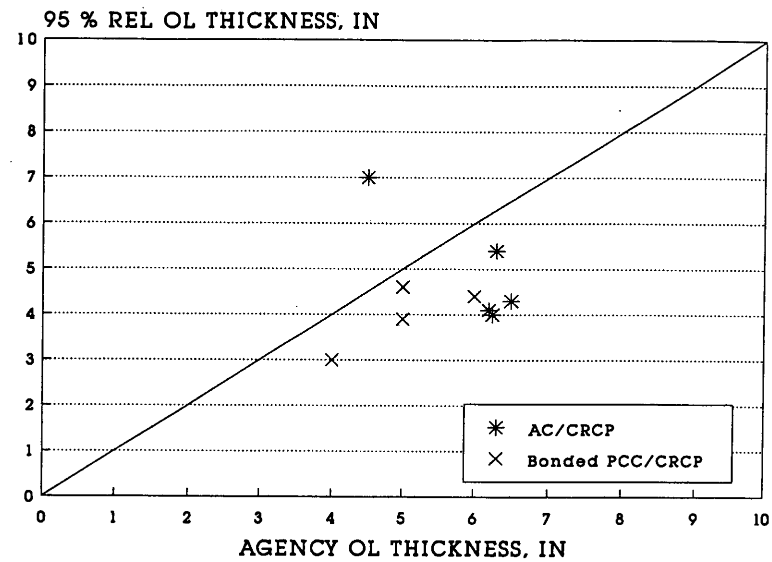

FIGURE 5 Comparison of AASHTO and agency AC overlay thickness (95 percent reliability) and bonded PCC overlay thickness (99 percent reliability).

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

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

EXAMPLE UNBONDED PCC OVERLAY DESIGN FOR PCC PAVEMENT

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

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

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

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

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

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

EXAMPLE PCC OVERLAY DESIGN FOR AC PAVEMENT

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

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

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

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

TABLE 5 Example AC Overlay Design for AC/PCC Pavement

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

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

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

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

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

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

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

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

TABLE 6 Example Unbonded PCC Overlay Design for PCC Pavement

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

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

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

   Vary trial Df until computed ESALs equal future design ESALs.

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

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

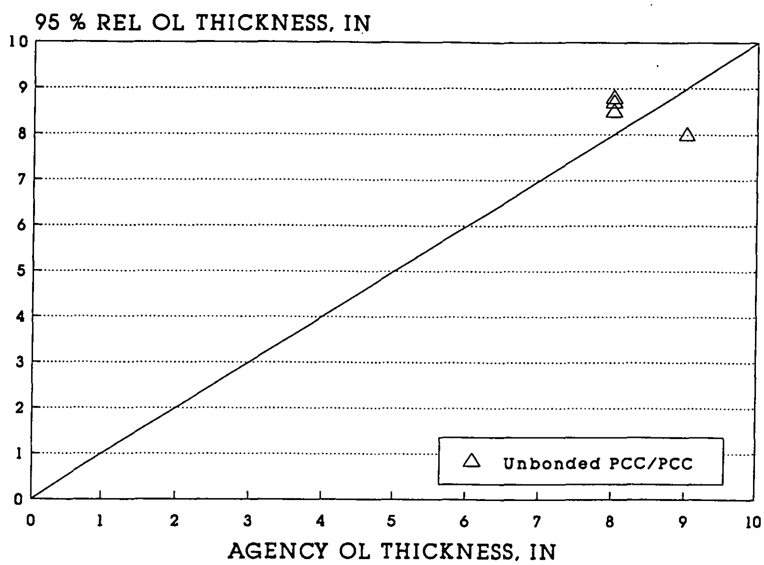

FIGURE 6 Comparison of AASHTO unbonded PCC overlay thicknesses and agency unbonded PCC overlay thicknesses for PCC pavement (95 percent reliability).

TABLE 7 Example PCC Overlay Design for AC Pavement

REVISED CHAPTER 5 AASHTO DESIGN GUIDE OVERLAY DESIGN

SE-5 JPCP OVERLAY OF AC PAVEMENT (US 1)

EXISTING PAVEMENT DESIGN AND FUTURE TRAFFIC

EXISTING PAVEMENT DESIGN

AC SURFACE	2.00	SUBGRADE:	SAND
CR STONE BASE	8.50		
SUBBASE	12.00		
TOTAL THICKNESS	22.50		

Future design lane ESALs = 1100000

DETERMINE Keff

Vary Ep/Mr until actual MR*D0/P matches computed MR*D0/p.

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

Using Figure 3.3, Part II:
Keff(dynamic) = 1200 psi/in INPUT
Keff(static) = 600 psi/in

DETERMINE Df

		INPUT	
PCC overlay modulus of rupture (psi)	=	635	(mean)
PCC overlay modulus of elasticity (psi)	=	4000000	(mean)

Vary trial Df until computed ESALs equal future design ESALs.

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

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

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

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

CONCLUSIONS

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

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

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

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

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

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

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DISCUSSION

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

BASIC OVERLAY DESIGN EQUATION

Details of the authors' revisions to 1986 AASHTO *Design Guide* (*1*) are given in a companion paper in this Record by them. An important deviation of the proposed revised procedures from the 1986 *AASHTO Guide* is the reverting to the use of traditional overlay equation $SC_{OL} = SC_y - SC_{xeff}$ and discarding the original remaining life concept that introduced a remaining life factor F_{RL} in the overlay equation. In a discussion of the companion paper, the following comments were made: (*a*) the authors' recommendation to set $F_{RL} = 1.0$, thereby reducing the 1986 AASHTO overlay equation to the traditional overlay equation, was based on an analysis that had a restrictive and weakly founded assumption on overlay performance and an incorrect procedure of computing F_{RL}, (*b*) the traditional overlay equation yields a lower bound overlay solution that underdesigns and is unconservative, and (*c*) the traditional overlay equation is conceptually unsound and inadequate because overlay thickness is derived on the basis of the overlay requirement at the time of overlay application. It does not include an analysis to examine whether the overlay provided is adequate during other stages of overlay service life.

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

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

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

USE OF FIELD TEST DATA

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

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

SUMMARY

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

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AUTHORS' CLOSURE

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

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

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

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

Fwa has defended this double penalty with the reasoning that if a deteriorated pavement with a given effective 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 or slab thickness), and the rate of deterioration is measured by loss of serviceability as predicted by these models. If the two pavements described by Fwa have the same structural capacity before overlay, and receive the same overlay, then according to the structural deficiency concept their performance after overlay will be the same. One cannot correctly apply the structural deficiency concept of overlay design and at the same time conjecture a rate of deterioration of the overlaid pavement other than the rate predicted by the performance model used to define the structural deficiency.

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

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

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

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

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