

MECHANISTIC OVERLAY DESIGN PROCEDURES FOR RIGID PAVEMENTS

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The review of currently available overlay design methods indicates that these may be grouped in three categories: those based on a visual condition rating, those using NDT deflections and empirically derived relationships between allowable deflection and overlay needs, and semi-rational procedures using stress analysis. The more promising of these procedures were subjected to detailed analysis and compared using data from a variety of projects. The result of this analysis shows that none of the existing models is very satisfactory and that a new overlay design procedure is needed. This paper presents the formulation of a method that attempts to overcome the difficulties and limitations of current methods. The proposed method utilizes both NDT and visual condition evaluation as well as limited destructive and laboratory testing to evaluate existing pavement condition. A coupled FEM-multilayer elastic subgrade program is used in stress analysis, and the amount of overlay is determined based on limiting formation of load-induced fatigue cracking. This program will be used in a re-examination of the AASHTO Road Test data as well as other available performance data to identify and correlate available field data with a performance distress function. Reflection cracking analysis incorporating load-induced shear as well as temperature induced stresses (both horizontal and curling stresses) will be used to minimize the probability of reflective crack formation.

Over the past twenty-odd years many miles of pavements have been built both in association with the Interstate Highway System and individual states upgrading their primary and secondary road systems. Much of this mileage has been built with concrete pavement structures; many of these pavements are experiencing serious distress and are in need of structural rehabilitation.

Rehabilitation of rigid pavements usually includes application of an overlay. Both rigid and flexible overlays have been utilized to correct deficiencies and to extend the existing pavements serviceability. Rigid overlays include both jointed and unjointed continuously reinforced pavements and are normally grouped into three types depending upon the design

bond condition (fully bonded, partially bonded, and unbonded) between the existing pavement and the overlay. Asphaltic concrete overlaying of rigid pavements is the most common rehabilitation method, primarily due to low initial cost and minimal traffic disruption associated with the quick-setting properties of the material.

Design of overlaid rigid pavements must address the usual distress phenomena such as fatigue cracking, reflective cracking and probably to a lesser extent, rutting. Reflective cracking is the cracking of an overlay above underlying cracks or joints. This problem occurs in both rigid and flexible overlays, but is most serious in asphalt concrete resurfacing over jointed concrete pavements. An effective overlay method must consider reflective mechanisms and should include either treatments for the existing pavement which minimize reflective cracking conditions or measures which will enhance the capability of the overlay to perform satisfactorily with the magnified stresses.

The increasing expense of pavement rehabilitation and the magnitude of the undertaking makes it essential to have a fast and reliable method to determine the existing pavement worth and the type and thickness of overlay needed to bring the structure to required strength. Many methods have been developed over the past twenty years for this task; all of them, however, have limitations and drawbacks which make them unsuitable, either because they are cumbersome to use or because they are based on limited regional experience and lack universal applicability. The method presented in this paper is an attempt to overcome most of these limitations.

Review of Existing Methods

The review of literature indicates that present rigid pavement overlay designs are based on subjective rating systems and empirical or semiempirical guidelines developed by various user agencies and do not, in general, consider the stress-strain distribution within the pavement layer structure, interaction of load and geometry, and rational and mechanistic methodology for remaining life analysis. The overlay design methodologies may be divided into three categories: Those based on visual condition survey, such as the procedures developed by the Army Corps.

of Engineers (1) CRSI, (2) PCA (3) ACI (4) AASHTO (5) Asphalt Institute (6) Illinois (7), etc; those based on deflection measurements, such as Louisiana (8), Mississippi (9), etc; and rational or semi-rational designs such as Wes (10) ARE (11), Texas (12) and OSU (13).

The Corps of Engineers, CRSI, PCA, ACI and AASHTO methods are all very similar in that they assign a "C" value to the existing pavement based on visual condition evaluation and determine the overlay thickness from the "C" value and the required thickness of new pavement. Two major difficulties exist with these methods: The assignment of the "C" value is rather subjective, and the determination of required new pavement thickness is determined from empirical relationships based on rather narrow and limited experience. The CRSI design overcomes some of the subjectivity of the "C" value in that more definite guidelines are given for the determination of this value; however, the evaluation is still subjective. These methods determine the required thicknesses for flexible overlays as well as for bonded and unbonded rigid overlays; however, as will be seen later, there are severe inconsistencies in the required overlay.

The Asphalt Institute method also uses a visual inspection of the existing pavement and is based on component analysis. Each layer is assigned conversion factors for converting existing pavement component layer thicknesses to an effective thickness of equivalent asphalt layer T_e . A table is presented giving condition descriptions and assigned conversion factors for various pavement layers. The method also requires estimation of subgrade support either CBR or R value. The total thickness of asphalt required for particular traffic and soil support condition is determined from a nomograph which has been derived empirically. The amount of overlay required is the difference between this total and the effective thickness T_e determined above. Like the previous methods described above, the assignment of effective thickness is subjective; also only flexible overlay thicknesses can be determined.

Unlike the previous methods, which may be used to determine the required overlay thickness for pavements in various stages of deterioration, the Illinois procedure assumes that the existing pavements are properly repaired and have served their useful life. The procedure does require the soil support value, but this is usually obtained from original soil surveys or from construction or design records. Once traffic factor and soil support value have been determined, a nomograph is used to determine the resurfacing structural number SN_r . The overlay thickness is then calculated from an empirically derived equation relating SN_r to existing pavement (PCC) thickness. Again, only flexible overlays are considered.

The deflection based methods are all very similar to the Mississippi procedure in that dynamic deflection measurements (generally with a Dynaflect) taken at midslab positions are used to evaluate the present pavement condition. A design deflection is derived statistically from the measured deflection and used in a nomograph to determine the required thickness of flexible overlay for a particular design traffic. While these methods remove the subjectivity from pavement condition evaluation, the determination of required overlay thickness is empirical and is based on limited experience over narrow geographic and climatic conditions and therefore lack universal applicability.

The overlay design methods developed by WES, ARE, and OSU are similar in that they use NDT deflection measurements to determine subgrade support values and use these along with either laboratory determined or assigned moduli for concrete and subbase layers in elastic layer theory to compute critical stresses in the concrete layer. The overlay thickness is determined by using layer theory to limit critical stresses

to allowable values, i.e., the critical stresses in the concrete are computed from layer theory and these stresses are used in a fatigue distress function to compute the allowable number of load repetitions to failure. The WES procedure is only concerned with midslab stresses; such factors as stress concentration at edges or corners, effects of voids and partial contact, variable load transfer at joints, etc., are not considered, and reflection cracking is not addressed.

The procedure developed at OSU is somewhat more sophisticated in that midslab stresses are modified to take into account load position (edge or corner load) and load transfer effects, and the reflection cracking model incorporates both long-term temperature-induced horizontal stresses as well as curling and warping stresses. However, the effect of the above variables have been developed using Westergaard Theory and FEM plate on a Winkler foundation; both theories are simplistic and unrealistic representations of a pavement structure. Also, the reflection cracking model does not incorporate the effects of load-induced shear stresses due to differential movement across joints or cracks.

Perhaps the most sophisticated currently available overlay design procedure was developed for FHWA by ARE. The primary design criteria in the ARE method are the prevention of fatigue cracking and the prevention or minimizing of reflection cracking. The procedure is automated using four different computer programs. The RPOD1 program is used to determine the required overlay thickness to prevent fatigue cracking and RFLCR1 is then used to check for reflection cracking. The other two programs are used for statistical analysis of deflection data. It is possible to eliminate the use of any of these programs if the function of that particular program is not required for a specific design problem or if the designer chooses to do that particular operation in another way. For instance, the reflection cracking program, RFLCR1, may be used on its own or may be omitted if not applicable to the program. The reflection cracking model is intended for use with A.C. overlays, but could be modified to include rigid overlays.

There are three basic steps in this procedure:

1. evaluation of the existing pavement,
2. determination of design inputs, and
3. overlay thickness analysis

Evaluation of the existing pavement is done by means of a deflection survey and a condition survey. The deflection survey is used to divide the roadway under consideration into design sections that will behave differently from each other under load and to select design deflections for each section. Statistical methods are applied in selection of design sections as well as the determination of design deflection. Condition survey information is used to classify the pavements into three categories: pavements with a potential of having remaining life, pavements so severely cracked that they would not be considered to have remaining life, and pavements that will be mechanically broken up before overlay.

The condition survey is also used to assign moduli values to the concrete layer: pavements with remaining life are assigned moduli equal to new concrete, pavements with no remaining life are assigned a modulus of 3447 MPa ($.5 \times 10^6$ psi), and pavements that are very severely cracked or will be mechanically broken up are assigned a value of 484 MPa (70,000 psi).

If the reflection cracking analysis is applicable to a particular design, additional condition survey information is needed, such as differential vertical movement at cracks or joints and the amount of joint movement with change in temperature.

Determination of design inputs includes both the past and projected future traffic (in terms of 80 KN (18 kip) equivalent single axle loads), environmental considerations, material properties, and dimensions of layers. Laboratory testing is required to determine elastic properties of the various pavement layers. Deflection information, as well as laboratory determination of resilient modulus at different deviatoric stress levels, is used in characterizing the subgrade material. For the reflection cracking analysis, additional input data, such as the creep modulus of asphaltic concrete, material thermal coefficients, and temperature information, are required. In all, a great deal of detailed field and laboratory measurements are required with this method. Up to 23 variables need to be specified for fatigue analysis, and an additional 20 variables are used in the reflection cracking analysis, making this method rather time-consuming and cumbersome to use. Also, many user agencies lack the sophisticated laboratory facilities required for the various tests.

The overlay thickness analysis considers the criteria of fatigue cracking as well as reflection cracking. However, the reflection cracking analysis is only required for those conditions where reflection cracking is expected to be a problem. In general, the RFLCRL program is intended for use with asphaltic concrete overlays.

In the fatigue cracking analysis, linear elastic layered theory is used to characterize the subgrade material and to compute stresses, strains and deflections. The in-situ subgrade support condition is determined from mid-slab or mid-crack deflection measurements. When deflection testing is done with loads that are different from the design load, the procedure recognized that subgrade modulus is stress-dependent and an iterative procedure is used to adjust the subgrade modulus from test load to design load using the laboratory-determined relationship.

$$M_r = K_1 \sigma_o^{k_2} \quad (1)$$

(where M_r is the resilient modulus, σ_o is the deviatoric stress, and K_1 & K_2 are constants that depend on material properties). However, the stress effects in granular base/subbase layers are ignored. A fatigue distress function developed from the AASHTO Road Test data relating critical stress to allowable load repetitions is used in the overlay determination, along with Miner's linear damage hypothesis for the determination of remaining life of the original pavement.

Governing stresses used in the fatigue life computations are assumed to be horizontal tensile stresses due to the applied wheel loads. The position of the governing stress is assumed to be at the bottom of the overlay for pavements without remaining life and at the bottom of the existing pavement for pavement with remaining life.

Stresses computed by the linear elastic layer program are taken to be interior stresses, and stress factors have been derived by means of the discrete element theory program, SLAB49, as well as by Westergaard and Pickett theory. Stresses predicated by ELSYM5 are increased by these factors or give the maximum stress at the critical point for a given combination of pavement and overlay type. Continuously reinforced pavements are designed for edge loading and jointed pavements for corner loading.

Void factors have also been determined using slab theory and are used in this program to account for increased stresses due to voids under pavement slabs; however, no guidelines are given for estimating or measuring the presence and size of voids.

The computer program, RPOD1, is used for the fatigue cracking analysis and can handle both asphaltic and portland cement concrete overlays on various types of portland cement concrete existing pavements. The output of this program is the required overlay thickness for a specified design life.

The reflection cracking model includes both temperature-induced horizontal stresses and load associated shear stresses but ignores curling and warping stresses.

Aside from the applicability of elastic layer theory to stress analysis of concrete pavements, two major assumptions are inherent in this overlay determination procedure:

1. The subgrade modulus can be determined from dynamic deflection measurements and adjusted for differences between test load and design load.
2. The ARE fatigue distress function can be used to determine the formation of class 3 and 4 cracks in concrete pavements.

As will be seen from the data presented below, both of these assumptions lead to some difficulty. The researchers at ARE attempted to correlate the degree of pumping with the formation of class 3 and 4 cracks in developing the fatigue distress function; however, elastic layer theory was used in the analysis and the efforts resulted in failure.

Determination of Subgrade Moduli from Deflection

In an attempt to verify that subgrade moduli could be computed from measured dynamic deflections, deflection data taken with various testing devices and various loads were compiled and analyzed. The data for the two projects presented below come from the following sources:

1. Thumper Data - unpublished data from R. May FHWA
2. O'Hare Airport - Treybig, H.J., vonQuintus, H.L. and McCullough, F.B., "Data Collection and Analysis, Runway 4R-22L, O'Hare International Airport", DOT Report 5-76-11.

The following conditions were used in the analysis:

1. Concrete, base, subbase moduli were taken from the information supplied, and these layers were assumed to be stress and frequency independent.
2. Dynamic deviator stress was compiled from

$$\sigma_d = \sigma_z - 1/2(\sigma_x + \sigma_y)$$

at the top of the subgrade layer and at the point of deflection measurement.

3. Total deviator stress includes both static (when present) load and gravity induced stress in addition to dynamic stress.

4. Subgrade modulus was varied until measured and computed deflections matched.

The Thumper data was taken on a concrete pavement in Florida with the FHWA Thumper at 8 H and variable dynamic load (8-32 KN). The two sections were tested: Section 2 consists of 229mm (9 in.) plain concrete and 152 mm (6 in.) of cement treated subgrade on cohesive subgrade, and Section 4B consisting of 76mm (3 in.) plain concrete, 229mm (9 in.) of Econocrete and 152mm (6 in.) of cement treated subgrade on cohesive subgrade. The moduli values and Poisson's ratio used in the analysis are shown in Table 1.

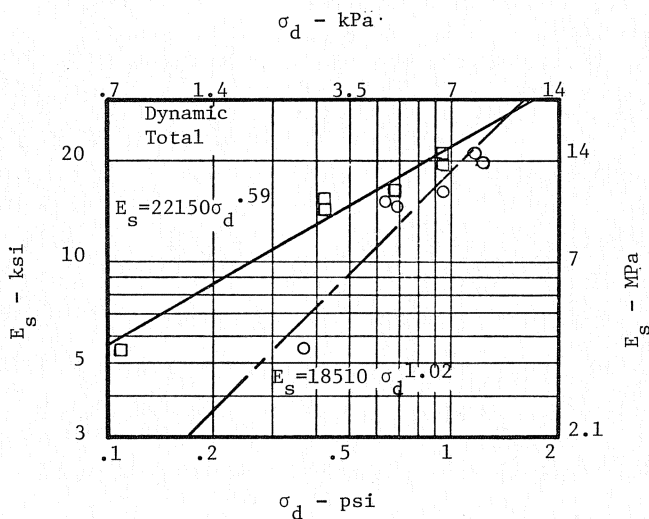
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Table 1. Moduli and Poisson's ratios of Florida project.

Material	Modulus-MPa	Poisson's ratio
Concrete	25000	.15
Econocrete	14000	.15
CTS	280	.30
Subgrade	Var.	.45

Note: 1 MPa = 145 psi

The results of this analysis are presented in Figure 1; each point on this figure represents the result of one test. As may be seen, the subgrade modulus increases with increasing load and σ_d , both for dynamic and total deviator stress. No lab data for this soil is available; however, based on lab data on cohesive soils, the modulus would be expected to decrease with increasing σ .

Figure 1. E_s vs σ_d for FHWA Thumper Florida data

The runway pavement consists of 356mm (14 in.) of concrete, 203mm (8 in.) of cement aggregate base, 203mm (8 in.) of gravel subbase, 2286-3086mm (90-120 in.) of variable subgrade (this layer is used in subgrade modulus determination) and an undisturbed subgrade below. Table 2 shows the layer properties for this pavement, supplied in the O'Hare report, (from lab data).

According to lab tests, the subbase modulus is independent of stress, and the subgrade has a negative 0.68 slope.

Deflection data was taken with the Dynaflect, the WES Device at 15 Hz, 4.4-6.7 KN (10 & 15 Kips) a plate simulating a 727, and a 727 and 747 plane. The plate, 727 and 747 deflections were measured with an LVDT.

The deflection data has been analyzed by averaging the deflection readings and using the average deflections in computing subgrade modulus and deviator stress. The results of this analysis are presented in Figures 2 and 3. As may be seen from this data, both negative and positive, as well as zero slopes are obtained, depending on comparison, and the only comparison consistent with lab data is 4.4 & 6.7 KN (10 & 15 Kip (at Lane 4 shown by line 2 in Figure 3) using total deviator stress (static, dynamic & gravity).

Table 2. Layer properties of O'Hare runway

Layer Type	Modulus (MPa)	Poisson's Ratio	Thickness* (mm)
Concrete	21000	.15	356,365
CAM Base	8100	.20	208,216
Subbase	480	.37	203
Subgrade	Var.	.45	2286,3048
Undisturbed Subgrade	1030	.45	Semi-inf.

Note: 1 MPa = 145 psi. 1mm = .039 in.

* Depending on location.

Excluding the 727 measurement (this represents one deflection only), the range of moduli is from 45 to 99 MPa (6500-14,400 psi) so a reasonable argument could be made that the subgrade is stress independent.

It should be emphasized that the above results are fairly typical of all the data analyzed and show that certain difficulties exist in determining subgrade moduli under a concrete pavement from deflection measurements. NDT deflections have been successfully used by many researchers to determine subgrade moduli under flexible pavements; it is therefore possible that the erratic and inconsistent relationships found in this study are the result of unknown temperature effects (such as curling and warping) in the concrete slab. If the slab had been warped at the time of deflection measurements (center of slab out of contact), part of the measured deflection would represent the deflection of a semi-supported slab; thus the measured deflections would increase more slowly with increasing load than would be expected if the slab had been in contact with the underlying layers. While the Thumper data was taken during a short time span, the O'Hare Airport data was taken over the span of several days under quite variable and unknown temperature conditions.

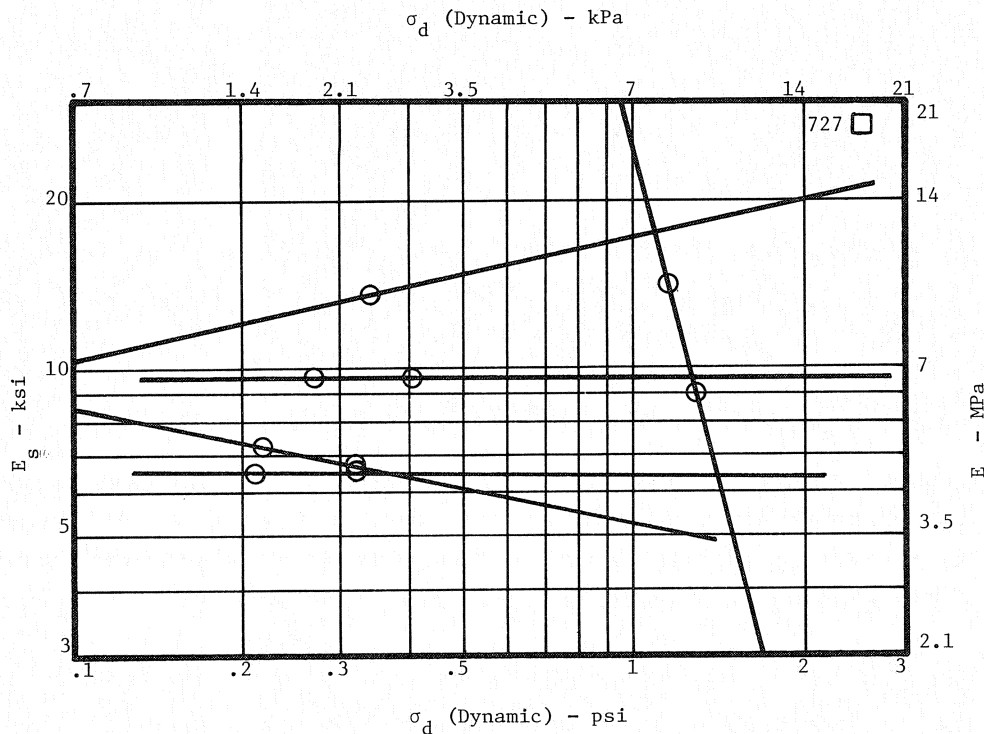
The above analysis is not intended to discredit the concept of determining subgrade moduli from NDT measurements; it does, however, point out that the results may be unreliable if the warping and partial contact conditions are disregarded in the analysis. The inclusion of these conditions makes the analysis considerably more complicated, since not only is it necessary to have vertical temperature gradient information, but also finite element plate on an elastic foundation theory instead of elastic layer theory is needed to interpret the results.

Comparison of Overlay Methods

Since very little data exists for the purpose of verifying overlay design methodologies, several different overlay design methods were compared. Presented below are comparisons of ARE, OSU-RII, AASHTO, Louisiana and Georgia methods applied to these projects. Tables 3, 4, and 5 give the pertinent information about each project, and Tables 6, 7 and 8 summarize the results, which are fairly typical of the data investigated.

Several interesting points are immediately apparent from this comparison:

1. As may be seen from Tables 6 and 7, the overlay thicknesses for flexible, bonded and unbonded overlays as determined from the AASHTO equations are inconsistent within themselves; that is from Table 6, 25.9mm (1.02 in.) of AC is as effective as 28.4mm

Figure 2. E_s vs σ_d (dynamic) for O'Hare Airport

Legend:

1. DYNAFLECT & WES - 10 ^k	Sept. 72	$E_s = 17,000 \sigma_d^{0.22}$	CENTERLINE	Avg. of 15 pts.
2. WES - 10 & 15 ^k	May 75	$E_s = 5,500 \sigma_d^{-0.18}$	LANE 4	Avg. of 30 pts.
3. WES - 10 & 15 ^k	May 75	$E_s = 9,600$	LANE 3	Avg. of 58 pts.
4. WES - 10 & 15 ^k	May 75	$E_s = 6,500$	SITE 1	Avg. of 7 pts.
5. Plate & 747	June 73	$E_s = 25,150 \sigma_d^{-4.00}$	SITE 1	Avg. of 3 pts.

Table 3. Mississippi I-59, Jones County.

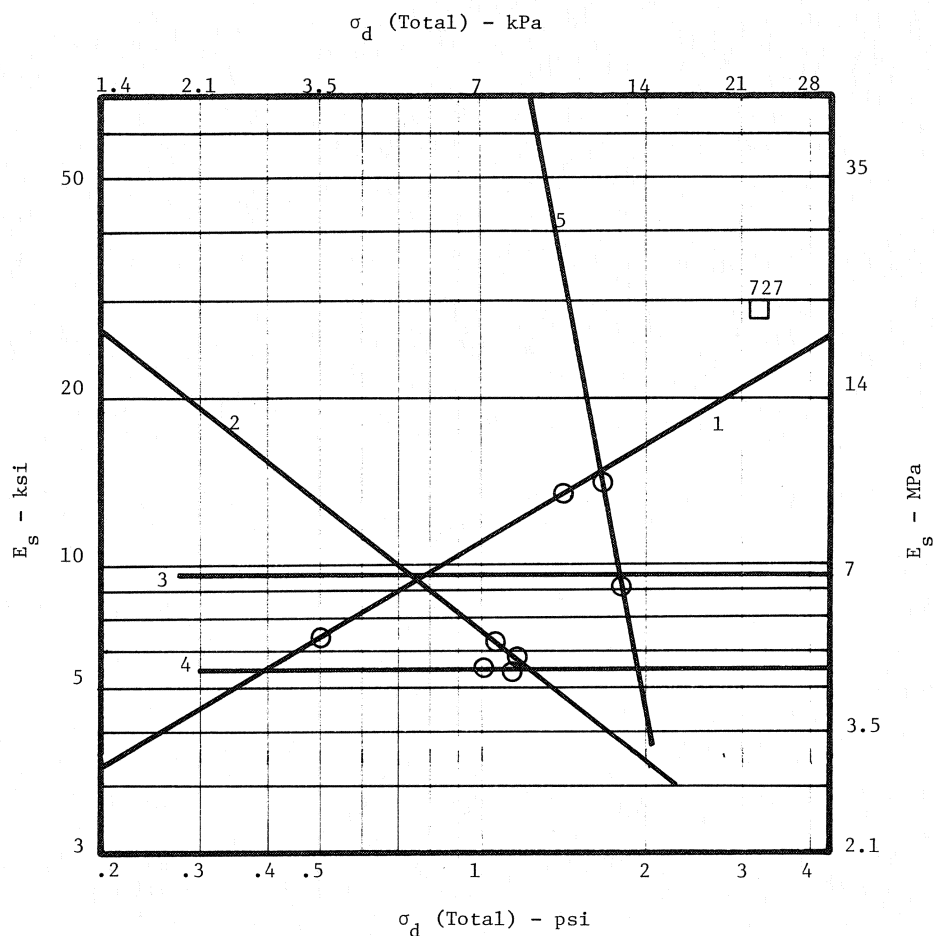
Pavement Type:	203 mm (8 in.) CRCP
Crack Spacing:	1 1.5 m (3-5 ft.)
Pavement Condition:	Class 3 cracking at location where smooth wire mesh had inadequate overlays. No faulting or pumping, but pumping may be starting to develop. Rideability is good. Some spalling at cracks.
Condition Rating C:	0.85 estimated from condition information.
Concrete Modulus:	44800 MPa (6.5×10^6 psi) measured from cores.
Flexural Strength:	5.1 MPa (750 psi) - measured from cylinders
Base Type:	Sand-clay soil cement with 7% cement by Vol. unconfined compressive strength over 4.1 MPa (600 psi).
Base Modulus:	6890 MPa (1×10^6 psi)
Base Thickness:	152 mm (6 in.)
Subgrade Type:	A2 (sand-clay) with occasional pockets of A6 (clay). Average CBR=10%, Design CBR=6.2% based on A6 soil.
Measured Deflection:	W1 .0163 mm (.000643 in.), std. dev. = .0058 mm (.00023 in.). $S_p = 76.83\%$, std. dev. = 3.72%
No. of Tests:	98
Comments:	The Dynaflect measurements were taken on 4-10-79, corresponding to a time of the year when the subgrade is close to its weakest condition, therefore the need for a seasonal factor is not anticipated.
Previous Traffic:	3.5×10^6 EAL
Design Life Traffic:	9.2×10^6 EAL

(1.12 in.) of bonded concrete, whereas 73.4 mm (2.89 in.) of concrete is required when a 50.8 mm (2 in.) AC bond break is used. Similarly from Table 5, no AC overlay is needed, but when a 50.8 mm (2 in.) AC bond breaker is used, the method requires an additional 60.7 mm (2.39 in.) of concrete. One reason for this discrepancy

is that the AASHTO method has no way to account for the thickness or strength contribution of the bond breaker layer; however, the discrepancy between AC and bonded overlay still exists.

2. There is reasonable agreement between the Mississippi and Louisiana methods, even though they

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Figure 3. E_s vs σ_d (total) for O'Hare Airport.

Legend:

1. DYNAFLECT & WES - 10 ^k	Sept. 72 $E_s=11,200 \sigma_D^{0.62}$	CENTERLINE	Avg. of 15 points
2. WES - 10 & 15 ^k	May 75 $E_s=7,350 \sigma_D^{-0.87}$	LANE 4	Avg. of 30 points
3. WES - 10 & 15 ^k	May 75 $E_s=9,600$	LANE 3	Avg. of 58 points
4. WES - 10 & 15 ^k	May 75 $E_s=6,500$	SITE 1	Avg. of 7 points
5. PLATE & 747	June 73 $E_s=170,700 \sigma_D^{-5.41}$	SITE 1	Avg. of 3 points

Table 4. Ohio I-70, Muskingum 780-66

Pavement Type:	206 mm (8.1 in.) CRCP
Crack Spacing:	7-9 mm (8-10 ft.)
Pavement Condition:	Deflection data was taken approximately 1 year after opening to traffic. No faulting, pumping or spalling. Rideability is good.
Condition Rating C:	0.95
Concrete Modulus:	31000 MPa (4.5×10^6 psi) from cores
Flexural Strength:	4.8 MPa (700 psi) from cylinder tests
Base Type:	301 plant-mix asphalt base with 51 mm (2 in.) maximum aggregate.
Base Modulus:	2400 MPa ($.35 \times 10^6$ psi)
Base Thickness:	76 mm (3 in.)
Subgrade Type:	A6B silty clay soil. L.L = 40, PI = 20, 78% passing #200 sieve Design CBR=5.3%
Measured Deflection:	W1 = .0093 mm (.000366 in.), std. dev. = .0021 mm (.000082 in.). Sp = 72.44% std. dev. = 6.35%
No. of Tests:	61
Comments:	Dynaflect measurements were taken on 10-15-70 when the subgrade is reasonably dry. However, a seasonal factor was not used in the analysis.
Previous Traffic:	0.5×10^6 EAL
Design Life Traffic:	10×10^6 EAL

Table 5. Ohio I-270, Franklin 510-69.

Pavement Type:	208 mm (8.2 in.) CRCP
Crack Spacing:	7-9 m (8-10 ft.)
Pavement Condition:	Deflection data taken before road was opened to traffic, pavement condition excellent.
Condition Rating C:	0.95
Concrete Modulus:	32750 MPa (4.75×10^6 psi) from cores
Flexural Strength:	5.0 MPa (720 psi) from cylinders
Base Type:	301 plant-mix asphalt base with 51 mm (2 in.) maximum aggregate.
Base Modulus:	2400 MPa ($.035 \times 10^6$ psi)
Base Thickness:	71 mm (2.8 in.)
Subgrade Type:	A6B silty clay soil. L.L. = 40, PI = 20, 78% passing #200 sieve Design CBR=5.3%, Measured Field CBR=10% (June '70)
Measured Deflection:	$\overline{W}_1 = .015$ mm (.000415 in.), std. dev. = .0028 mm (.000109 in.) $S_p = 78.18\%$, std. dev. = 3.71%
No. of Tests:	96
Comments:	Dynaflect measurements were taken on 8-9-71 when subgrade is reasonably dry. However, a seasonal factor was not used in the analysis.
Previous Traffic:	0
Design Life Traffic:	7×10^6 EAL

Table 6. Comparison of overlay thickness, remaining life and subgrade modulus by different methods for Mississippi I-59, Jones County.

Design Method	Overlay Type	50 Percentile			85 Percentile			98 Percentile		
		Hov (mm)	RL (10^6 EAL)	Es (MPa)	Hov (mm)	RL (10^6 EAL)	Es (MPa)	Hov (mm)	RL (10^6 EAL)	Es (MPa)
ARE ^a	AC	77.0	5.71	34	144.8	4.87	21	191.8	4.22	16
	CRC bond ^b	36.8	"	"	63.5	"	"	89.6	"	"
	CRC unbd	16.5	"	"	44.5	"	"	64.8	"	"
RII-OSU	AC	0	10.0	127	100.6	2.50	107	157.0	0.82	97
	CRC bond ^b	0	"	"	31.8	"	"	57.9	"	"
	CRC unbd	0	"	"	23.6	"	"	50.3	"	"
AASHTO ^c	AC	d	d	d	25.9	6.21	64	d	d	d
	CRC bond ^b	d	d	d	28.4	"	"	d	d	d
	CRC unbd	d	d	d	73.4	"	"	d	d	d
Mississippi		6.6	7.69	d	53.3	2.94	d	107.2	1.41	d
Louisiana		21.6	d	d	70.1	d	d	116.3	d	d

Note: 1mm = .039 in. 1 MPa = 145 psi 1 MN/m³ = 9.82 pci.

a Slope of subgrade stress relationship

b 51 mm of A.C. bond breaker

c $C = 0.85$ K = 42 MN/m³ F = .91 Es = 10*CBR (MPa)

d Not available from method

have been developed independently.

3. The ARE method considerably underestimates the existing pavement worth. This is especially apparent for the Ohio data (Table 7 and 8), which were tested when the pavements were new. I-70 has been overlaid after it had experienced some 8-9 million equivalent axle loads, and I-270 is approaching the need for overlay after approximately 6 million EAL. Aside from the problems encountered in determining subgrade moduli from deflections (discussed in the previous section), the ARE distress function may not be applicable to pavements that do not have significant pumping problems since it was developed from performance data of pavements that experienced pumping at various stages before failure. Thus the worth of existing pavements without pumping problems would be considerably under-

estimated.

4. Based on the performance of the Ohio pavements, it appears that the RII-OSU method overestimates the worth of new pavements and may underestimate the worth of pavements in need of repair.

5. The subgrade moduli computed by the ARE and the RII-OSU methods are significantly different and both differ from the value expected from $E_s = 10$ CBR (E_s in MPa).

Based on the above comparison, it is apparent that none of the existing overlay design methodologies is very satisfactory and that a new overlay design method is needed. Although the Mississippi and Louisiana procedures seem to be relatively successful, they are based on limited regional experience and therefore lack universal applicability;

Table 7. Comparison of overlay thickness, remaining life and subgrade modulus by different methods for Ohio I-70, Muskingum 780-66

Design Method	Overlay Type	50 Percentile			85 Percentile			98 Percentile		
		Hov (mm)	RL (10 ⁶ EAL)	Es (MPa)	Hov (mm)	RL (10 ⁶ EAL)	Es (MPa)	Hov (mm)	RL (10 ⁶ EAL)	Es (MPa)
ARE ^a	AC	108.2	3.49	114	131.1	2.82	86	153.2	2.35	68
	CRC bond ^b	57.7	"	"	69.3	"	"	81.0	"	"
	CRC unbd	36.8	"	"	49.8	"	"	62.5	"	"
RII-OSU	AC	0	65.8	267	0	13.9	267	45.7	3.56	267
	CRC bond ^b	0	"	"	0	"	"	26.4	"	"
	CRC unbd	0	"	"	0	"	"	0	"	"
AASHTO ^c	AC	d	d	d	0	8.5	55	d	d	d
	CRC bond ^b	d	d	d	14.0	"	"	d	d	d
	CRC unbd	d	d	d	60.7	"	"	d	d	d
Mississippi		0	45.2	d	0	24.0	d	0	14.1	d
Louisiana		0	d	d	0	d	d	0	d	d

Note: 1mm = .039 in. 1 MPa = 145 psi 1 MN/m³ = 9.82 pci.

a Slope of subgrade stress relationship

b 51 mm of A.C. bond breaker

c C = .95 K = 38 MN/m³ F = .93 Es = 10*CBR (MPa)

d Not available from method

Table 8. Comparison of overlay thickness, remaining life and subgrade modulus by different methods for Ohio I-270, Franklin 510-69

Design Method	Overlay Type	50 Percentile			85 Percentile			98 Percentile		
		Hov (mm)	RL (10 ⁶ EAL)	Es (MPa)	Hov (mm)	RL (10 ⁶ EAL)	Es (MPa)	Hov (mm)	RL (10 ⁶ EAL)	Es (MPa)
ARE ^a	AC	-	-	-	80.0	3.36	67	99.3	2.86	51
	CRC bond ^b	-	-	-	41.7	"	"	51.6	"	"
	CRC unbd	-	-	-	18.0	"	"	28.7	"	"
RII-OSU	AC	0	96.9	187	0	28.5	170	0	11.8	161
	CRC bond ^b	0	"	"	0	"	"	0	"	"
	CRC unbd	0	"	"	0	"	"	0	"	"
AASHTO ^c	AC	d	d	d	0	9.4	64	d	d	d
	CRC bond ^b	d	d	d	0	"	"	d	d	d
	CRC unbd	d	d	d	0	"	"	d	d	d
Mississippi		0	30.5	d	0	14.6	d	0	8.1	d
Louisiana		0	d	d	0	d	d	0	d	d

Note: 1mm = .039 in. 1 MPa = 145 psi 1 MN/m³ = 9.82 pci.

a Slope of subgrade stress relationship

b 51 mm of A.C. bond breaker

c C = .95 K = 38 MN/m³ F = .93 Es = 10*CBR (MPa)

d Not available from method

also only flexible overlays can be determined.

Features of a New Overlay Design Method

At the present time two theories exist for the determination of stresses and strains in a rigid pavement system.

1. Elastic (visco-elastic) layer theory
2. Finite Element plate on an elastic solid foundation

There are, of course, plates-on-Winkler-foundation theories; however, the Winkler foundation is a very unrealistic representation of subgrade support. The elastic layer theory is unable to account for discontinuities such as joints, cracks, edges, voids and partial contact, and the FEM-elastic solid theory is too expensive to use as a routine analysis method. It is, however, feasible to use the FEM program to develop influence functions to modify midslab stresses (determined from layer theory) as a function of the various discontinuities listed above. Thus elastic layer theory modified by influence functions becomes a rational and economical analysis method for stress-strain analysis. The use of layer theory requires the knowledge of pertinent layer properties (thicknesses, Poisson's ratios, moduli), requiring at least some laboratory and destructive field testing. In order to keep required testing to a minimum, the following method is proposed:

1. A simplified visual pavement condition rating system shall be used to:
 - a. Determine if structural overlay or surface treatment is needed.
 - b. Evaluate the extent and type of pavement distress, including an estimation of voids and partial contact conditions.
 - c. Determine amount and type of additional testing needed.
2. Pavement layer thicknesses to be obtained from construction records, or from small-diameter field cores when construction records are no longer available.
3. Concrete modulus and flexural strength to be obtained from construction records, from lab tests or field cores, or assigned by design engineer based on experience and guidelines.
4. Base (subbase) modulus to be obtained from material type and AASHTO guidelines, tempered by experience.
5. Subgrade modulus to be obtained either from lab tests of undisturbed samples (triaxial tests at confining pressures and deviator stress expected in the pavement), or determined from soil classification and CBR values, modified by the results of NDT measurements. If the latter method is used, the following procedure is recommended:
 - a. NDT deflection measurements to be taken (with any testing devices) at midslab or mid-track location at at least 40 locations per overlay project. Measurements to be taken during early morning hours or on cloudy days when the vertical temperature gradient in the slab is small.
 - b. Subgrade support moduli to be determined from layer theory by matching measured and computed deflections.
 - c. The mean support modulus from NDT results to be equated to assigned modulus based on soil classification and CBR values.

d. Statistical analysis to be conducted on NDT moduli to determine standard deviation and coefficient of variation of modulus.

e. The results of statistical analysis to be applied to assigned modulus to determine design modulus.

6. NDT deflection measurements across joints and cracks at at least 40 locations per overlay project to determine load transfer efficiency. Since LTE is dependent on temperature, measurements should be made during times when slab temperature is near annual air temperature.

7. Joint/crack width measurements. Since the width is a function of temperature, it is also necessary to estimate the average concrete temperature at time of measurement.

With the above information, critical stresses and strains in the pavement structure can be determined; however, a distress function relating critical stresses to allowable axle loads is still needed. It was shown in the previous section that the distress function developed by ARE may be applicable only to pavements that have pumping problems. Also, the distress function was developed using semi-infinite layer theory and did not realistically consider such effects as load transfer, joint spacing, etc. If any overlay design method is to be successful, it is imperative that the distress functions be developed based on the same theory that is used in overlay analysis. It is therefore proposed that not only the AASHTO Road Test data, but also performance data from the Maryland Test Road (MD-1), Zero Maintenance Analysis Test results, as well as other available field data be analyzed with the proposed stress analysis model in an attempt to identify and correlate the available field data with distress functions.

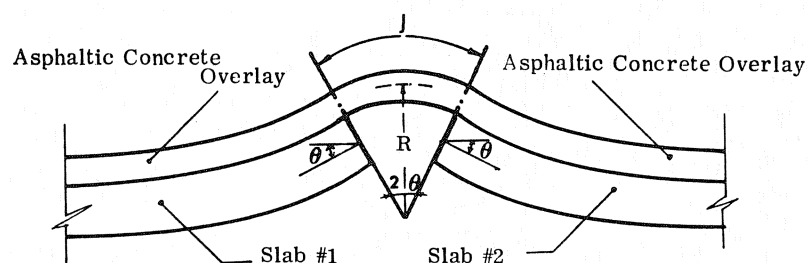
The model outlined above is intended to determine the thicknesses of various types of overlay in order to prevent load-associated damage to the overlaid pavements, but does not consider reflection cracking problems. The proposed reflection cracking analysis model is a combination and modification of the ARE and OSI models and will consider stress contributions from the following sources:

1. Horizontal tensile stress in the overlay as a result of thermal contraction of the slab.
2. Load associated vertical shear stresses resulting from differential deflections across joints or cracks.
3. Tensile stresses induced at the top of the overlay due to slab curling, as shown in Figure 4.

In the stress computation due to thermal contraction, the coefficient of friction between the slab and underlying layer is assigned a value based on material type and the friction force diagram is assumed to be triangular, making it possible to compute the horizontal movement with temperature instead of having to be measured. Also, laboratory measurements of the bonding strength (shear) of an A.C. overlay of concrete have shown that relative movements greater than 0.51 mm (.02 in.) cause loss of bond of the overlay so that horizontal stresses can no longer be transmitted. Thus, while joint/crack width measurements are still necessary, measurements of joint movement with temperature are no longer needed. Also, theoretical investigations have shown that the resulting stresses are only slightly dependent on joint width, so that default values can be used with negligible loss of accuracy.

Investigations of reflection cracking in Ohio and elsewhere have shown that reflection cracking

Figure 4. Bending of overlay by joint vertical movement.



begins at the top of the overlay and develops downward, i.e., the top fiber stress is greater than the bottom fiber stress in an overlay. To explain this phenomenon, a curling model was developed. This model, shown schematically in Figure 4, assumes that a slab will curl due to differential vertical temperature distribution in the concrete. The resulting stresses can be computed using the Plate program, or a finite element program. These curling stresses are a function of differential temperature, slab length, joint width, overlay modulus and thickness, and concrete thickness and modulus in the order of importance. The only field measurement needed is the joint width and, of course, the joint and/or crack spacing.

The stress developed in the overlay as a result of curling is generally much greater than that due to horizontal movement, as may be seen from the following example for a 229 mm (9 in.) JCP with 18.3 m (60 ft.) joint spacing and joint width of 19 mm (.75 in.). For a 11.1°C (20°F) temperature change, the horizontal movement is greater than 0.51 mm (.02 in.), consequently the top fiber stress due to a 0.51 mm (.02 in.) (limiting value) movement is 276 kPa (40 psi). For a 11.1°C (20°F) differential vertical temperature, the top fiber stress is 1.4 MPa (210 psi).

The above model incorporates all distress mechanisms contributing to reflection cracking but has the disadvantage that it requires the incorporation of a finite element program into the reflection cracking package. However, since this program would be seen only once for each project, the additional cost will not be prohibitive.

Summary and Conclusions

The analysis and discussion in the previous sections have shown that although a number of overlay design schemes exist, each method has limitations and inconsistencies. Perhaps the most widely used method is the AASHTO method (some states use a variation of this); based on the results in Table 6 through 8, the amount of flexible overlay predicted by this model appears to be reasonable; however, the predicted thicknesses of bonded and unbonded rigid overlay are totally inconsistent with the predicted flexible overlay thickness as well as with experience. The empirical methods (Mississippi and Louisiana) are in reasonable agreement with experience, but since they have been developed from local experience, extrapolation to other geographic or climatic regions is risky. Also, only flexible overlay thicknesses can be determined. Aside from the large amount of laboratory and field measurements required by the ARE model, its major drawbacks are the method of determining subgrade moduli and the use of the fatigue distress function which was developed using stress analysis methods that are somewhat different from those used in overlay analysis.

The proposed overlay design model attempts to overcome these difficulties by:

1. Using a stress analysis model capable of representing the features and geometry of rigid pavements realistically.
2. Using the same theory in developing a performance distress function as is used in the overlay thickness computation.
3. Using projects from different geographic and climatic regions in deriving a performance distress function.
4. Minimizing the amount of laboratory and destructive field testing required.

However, the model is still under development; therefore the degree of success is not known at the present time.

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