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Evaluation of Airfield Pavement Condition and Determination of Rehabilitation Needs

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This paper presents a comprehensive procedure for the evaluation of the condition of an airfield pavement and the determination of its maintenance and rehabilitation needs. The overall procedure consists of three steps: The first is the determination of the airfield-pavement-condition rating based on a pavement-condition index. This index is a score between 0 and 100 that describes the structural integrity of the pavement and its surface operational condition and is based on measured types, severities, and amounts of distress. The index, and hence the pavement-condition rating (i.e., excellent, very good, good, fair, poor, very poor, or failed), agrees closely with the collective judgment of experienced pavement engineers and is strongly correlated to the need of the pavement for maintenance and rehabilitation. The second step is the evaluation of the pavement through a stepwise procedure. The purpose of the evaluation is to provide the necessary background for a rational determination of feasible maintenance and rehabilitation alternatives. The stepwise evaluation procedure depends largely on the pavement-condition-index and distress data but other direct measurements, such as profile roughness, hydroplaning potential, and load-carrying capacity, are also included. The third step is the determination of the optimum maintenance and rehabilitation alternative. Feasible alternatives are determined through the use of guidelines that are based on the results of the stepwise evaluation and include recommended methods for the localized repair of different types of distress at different levels of severity. After the feasible alternatives are identified, an economic analysis is performed. The optimum alternative is selected based on the results of the economic analysis, the mission of the pavement, and the policies of the airfield management. The procedure is illustrated by an example.

The first step in determining the maintenance and rehabilitation (M&R) needs of and alternatives for a given pavement feature is an accurate and comprehensive evaluation of its existing condition.

The condition of an airfield pavement can be evaluated in terms of factors called condition indicators; comprehensive pavement-condition evaluation requires the measurement of these condition indicators, which include at least the following:

1. Operational surface indicators: (a) roughness (both localized and profile roughness), (b) skid resistance and hydroplaning potential, and (c) potential for foreign object damage (FOD) (to jet engines);
2. Structural indicators: (a) structural integrity (cracking, distortion, and disintegration) and (b) load-carrying capacity; and
3. Other indicators: (a) rate of deterioration and (b) amount of previous M&R applied.

Many of these condition indicators are interrelated; for example, surface distortion and disintegration are related to surface roughness. A complete condition evaluation requires the consideration of each condition indicator individually and of all the indicators collectively.

Most of these pavement condition indicators are related to observable pavement distress, as shown in Figure 1 (for asphalt-surfaced pavement); there is a similar correlation for rigid pavements (1). In most cases, the observable pavement distress gives a good indication of pavement condition: FOD potential, structural integrity, roughness (short wave lengths only), and rate of deterioration can be determined in this manner.

In this paper, the development is described of a composite index that relates airfield pavement cracking, distortion, and disintegration. The index, which is known as the pavement-condition index (PCI) has been officially adopted and is being used extensively by the U.S. Air Force. The PCI is a score between 0 and 100 that agrees closely with the average rating (collective judgment) of experienced pavement engineers and is strongly related to the need for M&R.

The stepwise evaluation procedure presented in this paper is based on the use of the PCI, distress data, rate of deterioration, and other direct measurements (such as skid potential and profile roughness). The PCI and distress data were selected as the basis for the evaluation because they showed strong correlations with M&R needs. Guidelines for identifying feasible M&R alternatives based on the results of the evaluation are presented. The selection of the optimum M&R alternative should be based on economics, pavement mission, and management policies.

The application of these procedures is illustrated for a plain-jointed concrete runway in Illinois.

PAVEMENT-CONDITION INDEX

Description

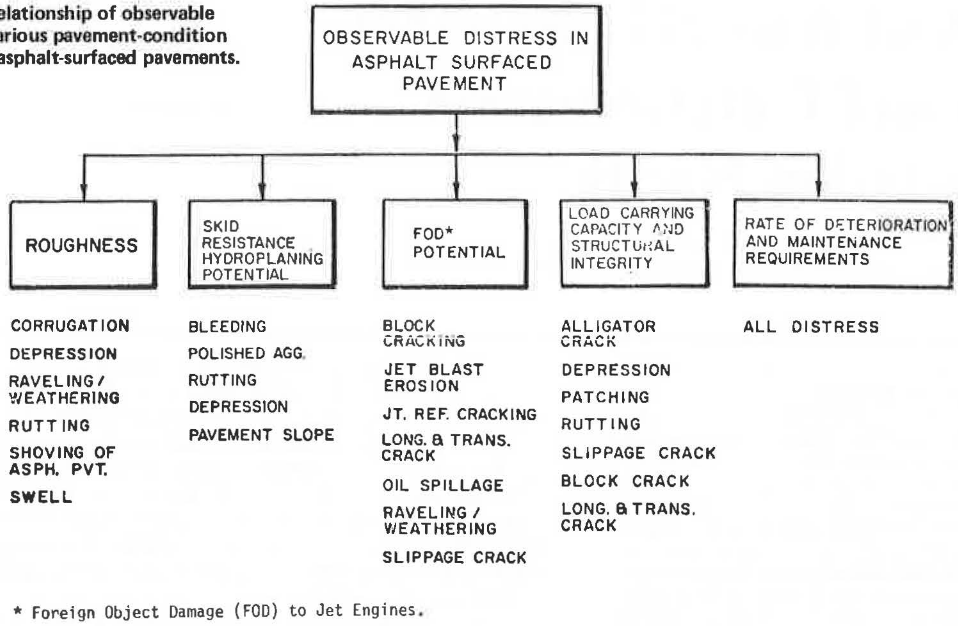
The PCI is a numerical indicator of pavement condition that is directly related to the structural integrity of the pavement (its ability to resist fracture, distortion, and disintegration) and its surface operational condition. The PCI is a function of (a) type of distress (T_i); (b) severity of distress (S_j), such as width and degree of spalling of cracks or depth of ruts; and (c) density of distress (D_{ij}), which is the percentage of the area of the pavement that is distressed. The development of a meaningful condition index requires the inclusion of all three of these distress characteristics. The PCI is expressed mathematically as follows:

$$\mathrm{PCI} = C - \left[\sum_{i=1}^{p}\sum_{j=1}^{m_i} a(T_i,S_j,D_{ij})\right] \times F(t,q) \tag{1}$$

where

C = constant that depends on desired maximum scale value;
$a(\ )$ = deduct weighting value that depends on T_i, S_j, and D_{ij};
i = counter for types of distress;
j = counter for levels of severity;
p = total number of types of distress for pavement type under consideration;
m_i = number of severity levels on the ith type of distress; and
$F(t, q)$ = adjustment function for multiple distresses that

Figure 1. Relationship of observable distress to various pavement-condition indicators: asphalt-surfaced pavements.



varies with total summed deduct value (t) and number of deducts (q).

The development of the PCI consisted of defining the types of distress and the levels of its severity (2) and of developing the individual distress deduct curves and an adjustment function for multiple-distress correction (3).

Determination of Pavement-Condition Index of a Pavement Feature

A pavement feature is defined as a portion of pavement that (a) has consistent structural thickness and materials, (b) was constructed at one time, and (c) is subjected to the same type and approximately the same number of traffic repetitions.

The PCI of a given pavement feature is determined by carrying out the following steps (see Figure 2):

1. The pavement feature is first divided into sample units. A sample unit for concrete pavement is approximately 20 slabs; a sample unit for asphalt is an area of approximately 465 m^2 (5000 ft^2).
2. The sample units are inspected and the types of distress and their severity levels and densities are recorded. The criteria given by Shahin and others (4) should be used in identifying and recording the types of distress.
3. For each type, density, and severity level of distress within a sample unit, a deduct value is determined from an appropriate curve (4) (see step 3 in Figure 2 for an example of such a curve).
4. The total deduct value (TDV) is determined by adding all of the deduct values for each distress condition observed for each sample unit inspected.
5. A corrected deduct value (CDV) is determined from the appropriate curve (4); the CDV is based on the TDV and the number of distress conditions observed that have individual deduct values higher than five points (see step 5 in Figure 2).
6. The PCI for each sample unit inspected is calculated as follows:

$$PCI = 100 - CDV \qquad (2)$$

7. The PCI of the entire feature is computed by averaging the PCIs of all the sample units inspected.
8. The pavement condition rating of the feature is determined by using step 8 of Figure 2, which presents verbal descriptions of pavement condition as a function of PCI value.

An inspection by sampling procedure has been developed to expedite inspection without loss of accuracy and has been widely accepted and used by the Air Force engineers. A computer program has also been developed to expedite the PCI calculations (4).

EVALUATION OF PAVEMENT FEATURE FOR SELECTION OF MAINTENANCE AND REHABILITATION ALTERNATIVES

In this section, the steps are presented for the evaluation of the condition of a pavement feature. The major emphasis is on the use of the PCI and distress data for the determination of condition because these items have been found to be strongly correlated with maintenance and repair needs, but the use of other direct measurements to supplement and verify evaluations in critical situations is also described.

Figure 3 is a summary of the steps in the pavement-condition evaluation. Following is a brief description of each:

1. Overall condition: The mean PCI of a pavement feature is an estimation of the overall condition of the pavement and represents the consensus of opinion of a group of experienced pavement engineers.
2. Variation of PCI within feature: Because of variations of materials, construction, subgrade, and traffic loadings, certain portions of a given pavement feature may show a significantly different condition than the average for the overall feature. Areas that have a poorer condition are of major concern. Variation within a feature occurs on both a localized random basis (i.e., from material and variability) and a systematic basis (i.e., from traffic patterns).

Figure 4 has been developed from field data to provide guidelines for determining whether localized random variation exists. For example, if the mean PCI of a

Figure 2. Steps in determining pavement-condition indicator of a pavement feature.

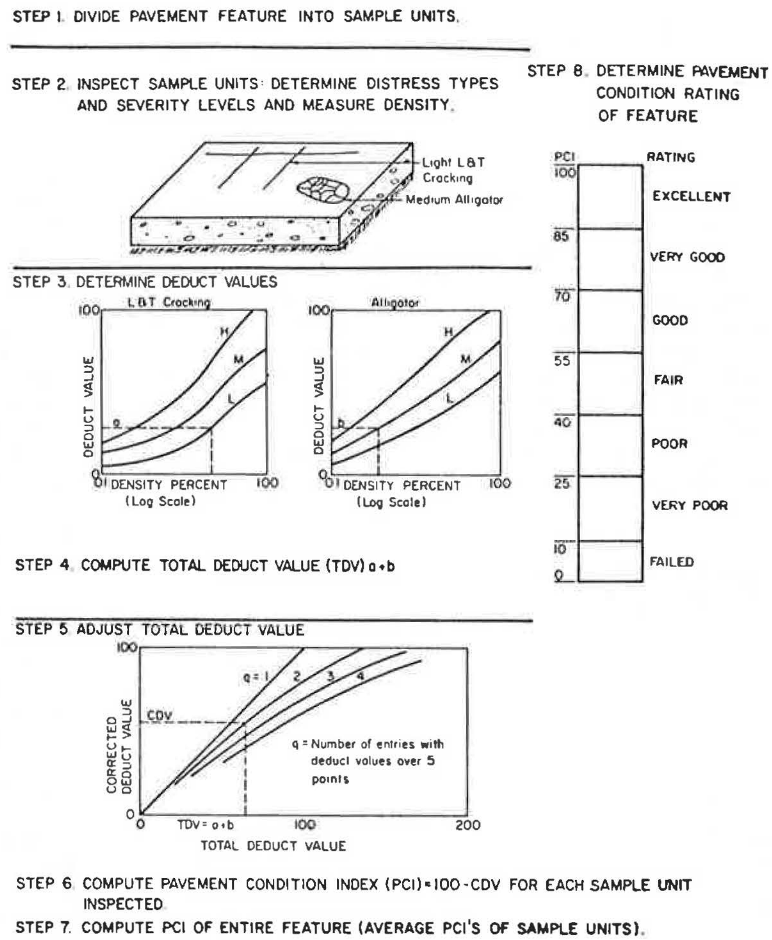


Figure 3. Form for evaluation of airfield pavement condition.

1. Overall Condition Rating - PCI
   Excellent, Very Good, Good, Fair, Poor, Very Poor, Failed.
2. Variation of Condition Within Feature - PCI
   a. Localized Random Variation — Yes, No
   b. Systematic Variation — Yes, No
3. Rate of Deterioration of Condition - PCI
   a. Long-term period (since construction) — Low, Normal, High
   b. Short-term period (1 year) — Low, Normal, High
4. Distress Evaluation
   a. Cause
      Load Associated Distress — 58 percent deduct values
      Climate/Durability Associated — 36 percent deduct values
      Other (Fault) Associated Distress — 6 percent deduct values
   b. Moisture Accelerated Distress — Minor, Moderate, Major
5. Load Carrying Capacity Deficiency — No, Yes
6. Surface Roughness — Minor, Moderate, Major
7. Skid Resistance/Hydroplaning (runways only)
   a. Mu-Meter — No hydroplaning problems are expected; Transitional; Potential for hydroplaning; Very high probability
   b. Stopping Distance Ratio — No hydroplaning anticipated; Potential not well defined; Potential for hydroplaning; Very high hydroplaning potential
   c. Transverse Slope — Poor, Fair, Good, Excellent
8. Previous Maintenance — Low, Normal, High

Note: Circled items relate to field-case study described in this paper.

feature is 59, any sample unit having a PCI of less than 42 should be identified as a localized bad area. This variation or localized bad area should be considered in determining M&R needs.

Systematic variation occurs whenever a large concentrated area of the feature has a condition that is significantly different from the rest. For example, on a wide runway or a large apron where traffic is channeled to a certain portion, that portion may show much more distress (or poorer condition) than the rest of the area. Whenever there is a significant degree of systematic variability within a feature, strong consideration should be given to dividing the feature into two or more features.

3. Rate of deterioration: For jointed concrete-surfaced pavements, the relative rate of long-term deterioration from initial construction can be determined from Figure 5, which was obtained by plotting all available data on the features surveyed [a similar figure was also developed for asphalt-surfaced pavements (1)]. In Figure 5 (and the similar one for asphalt-surfaced pavements), each point represents one pavement feature, the solid central line represents the overall average rate of deterioration of the pavement features, and the dashed lines are intended to envelop a majority of the data to represent normal rates of deterioration. These data points represent a great variety of pavement designs, traffic, climates, soils, and other factors, and thus the overall average expected loss of condition. A pavement feature that is above the upper dashed line is considered to have a low rate of deterioration, and a feature that is below the lower dashed line is considered to have a high rate of deterioration. Thus, a concrete pavement feature that has a PCI of 40 after 20 years is considered to have a high long-term rate of deterioration, but one that has a PCI of 60 is considered to have a normal long-term rate of deterioration.

The rate of deterioration of a pavement can also be evaluated in terms of the short-term or annual decrease in the PCI. A decrease in the mean PCI of a feature (assuming only normal routine M&R has been applied) of seven or more PCI points indicates a high short-term rate of deterioration; a decrease of four to six points indicates a normal or average short-term rate of deterioration.

4. Pavement distress: The PCI is a composite index of existing airfield-pavement distress. However, examination of the specific types, severities, quantities, and causes of individual distresses also provides a valuable aid in determining condition and eventually selecting M&R needs. Distress occurs as a result of traffic loads, climatic conditions, material durability, and other factors. The types of distress have been divided into three main groups: (a) those caused primarily by traffic loadings, (b) those caused primarily by material durability and climate, and (c) those caused by other factors. Conditions at each pavement will dictate the specific distresses that belong in each group.

The following steps constitute a procedure for determining the primary cause or causes of the deterioration of the pavement condition of a given feature:

First, the total deduct values attributable to load, climate and durability, and other distresses are separately determined. For example, the following distresses were measured on an asphalt feature and the deduct values were determined:

Figure 4. Procedure for determination of minimum sample unit PCI based on mean PCI of feature.

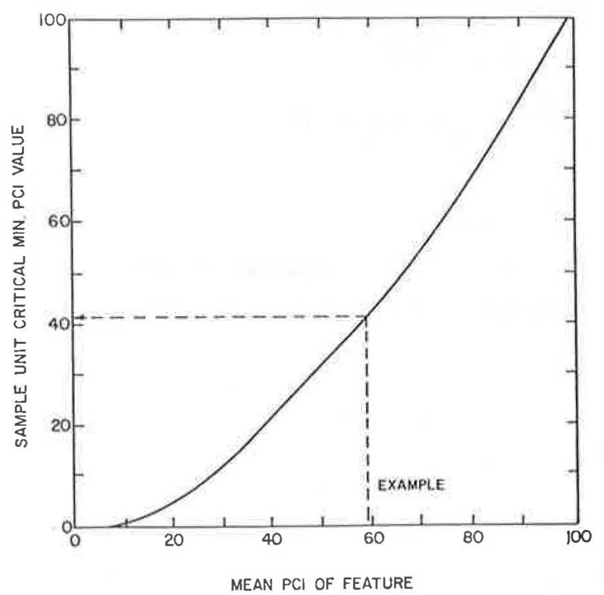
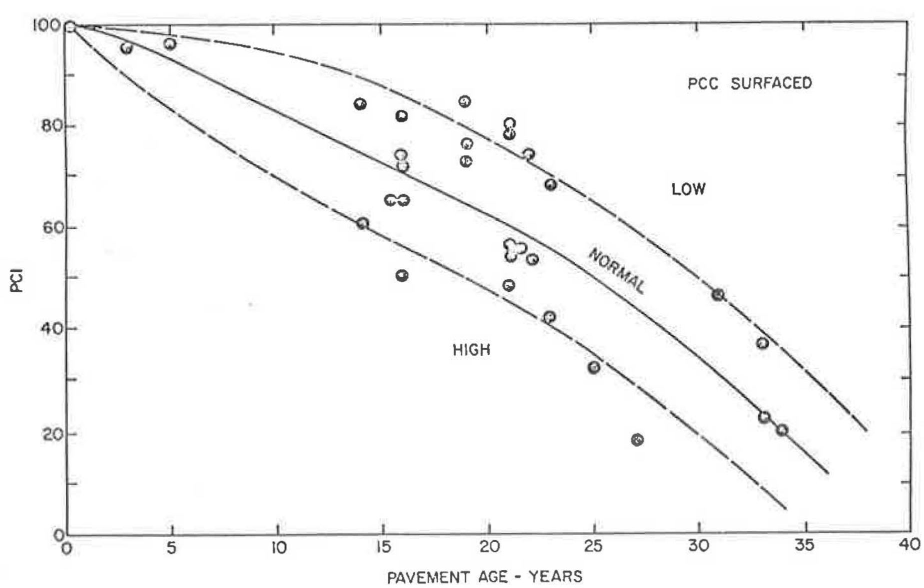


Figure 5. Relationship between PCI of pavement feature and time since construction: concrete-surfaced pavement.



Type of Distress	Deduct Value	Cause
Alligator cracking	50	Load
Transverse cracking	8	Climate and durability
Rutting	20	Load

Thus, the total deduct value attributable to load is 70 and that attributable to climate and durability is 8.

Second, the percentage deduct values attributable to load, climate and durability, and other causes are computed. For the above example feature, the calculation is as follows:

Load = $(70/78) \times 100 = 90$ percent
Climate and durability = $(8/78) \times 100 = 10$ percent
Total = 100 percent.

Third, the percentage deduct values attributed to each cause form the basis for the determination of the primary cause(s) of pavement deterioration. In this example, distresses caused primarily by load have caused 90 percent of the total deduct value, whereas all other causes amount to only 10 percent. Thus, traffic load is by far the major cause of deterioration of this pavement feature.

5. Evaluation of load-carrying capacity: The load-carrying capacity of an airfield pavement is defined in terms of three factors: (a) the aircraft gross weight, (b) the type of aircraft, and (c) the number of aircraft passes over the pavement until a "failed" condition is predicted. If these three factors are held constant, the load-carrying capacity depends on the pavement structure and material properties and the subgrade soil properties. For years, the U.S. Air Force has determined the load-carrying capacity of airfield pavements by using procedures developed by the U.S. Army Corps of Engineers (5). Research efforts are under way to develop nondestructive testing methods and criteria for evaluating the load-carrying capacity of airfield pavements. The results of this development could be used to replace the older procedure.

6. Surface roughness: There are currently three methods for estimating surface roughness. First, pilot complaints are subjective but highly reliable sources of qualitative roughness information. The pilot reports reflect aircraft ride quality as well as surface roughness; the additional factor of aircraft vibration is therefore included.

Second, certain types of distress contained in the PCI can be correlated with localized roughness as shown in Figure 1. However, it is difficult or impossible to see the longer wave-length roughness that affects aircraft ride quality when inspecting a runway surface.

Third, the roughness can be quantitatively evaluated, on a relative basis, by analyzing measured profile-elevation data. [The development of this approach formed a large part of a joint Federal Aviation Administration and U.S. Air Force research program and is discussed in more detail elsewhere (6).] This method has required the development of rapid elevation-measuring instruments and suitable data-processing techniques involving filtering and statistical analysis of random data as well as the use of computer programming for the estimation of aircraft vibration response.

7. Skid resistance and hydroplaning potential: The Air Force (7) reports pavement skid resistance in terms of the coefficient of friction as measured by a Mu-Meter (8) and the wet-to-dry stopping distance ratio as measured by the diagonally braked vehicle (9). Transverse slope can also be measured by survey techniques.

8. Previous M&R applied: A pavement feature can be kept in operating condition almost indefinitely if extensive M&R is continually applied. There are major disadvantages to this maintenance strategy, however, such as overall cost, downtime of the feature, the increase in roughness caused by excessive patching, limitations of personnel and equipment, and airfield mission requirements. The amount and types of previous M&R applied to a pavement feature are important factors in determining currently needed M&R. A pavement where a large portion has been patched or replaced must have had many previous distress problems that are likely to continue in the future.

Permanent patching and slab replacement may be used as a criterion for evaluating previous maintenance. Patching and slab replacement of 1.5-3.5 percent is considered normal, more than 3.5 percent is considered high, and less than 1.5 percent is considered low. Some pavement features may also have received an excessive amount of M&R other than patching. If, in the judgment of the engineer, this should be evaluated as high previous maintenance, then this evaluation should take precedence over the evaluation based only on patching and slab replacement.

MAINTENANCE AND REHABILITATION NEEDS

The selection of the optimum M&R category (i.e., routine, major, or overall) for a given pavement feature is a major decision that requires many years of experience in pavement maintenance and repair. In many cases, a group of experienced pavement engineers will agree on a recommended M&R category. In many other cases, however, disagreement will occur and a thorough examination of the pavement condition evaluation and a comprehensive economic analysis will be required for the selection of the correct M&R category and the optimum M&R alternative.

Maintenance and Rehabilitation Categories

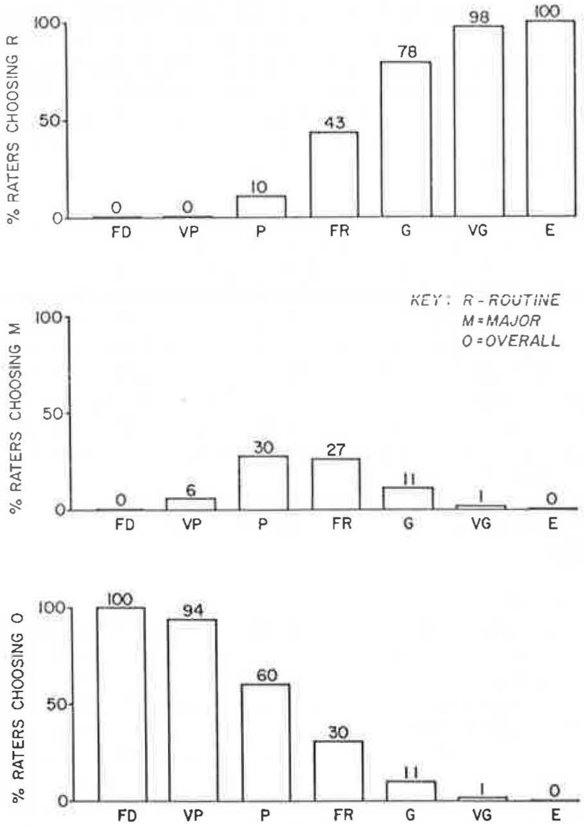
M&R methods (such as crack filling, patching, slab replacement, and overlay) are grouped into three general categories for convenience of analysis and discussion.

1. Routine M&R: Routine M&R consists of performing preventive or localized M&R. Preventive M&R includes methods that preserve the condition of the pavement and retard its deterioration. These methods include crack sealing, joint sealing, and application of fog seals and rejuvenators. Application of aggregate seals, however, is considered to be major localized M&R. Localized M&R methods are those that restore pavement condition. Some repair methods are considered localized if they are applied to only a small area of the pavement feature; for example, skin patching, applying heat and rolling sand, placing small patches [$0.46\ m^2$ ($5\ ft^2$)], and patching joint and corner spalls are considered localized regardless of amount. On the other hand, partial-depth or full-depth patching, slab replacement, slab undersealing, slab jacking, and slab grinding are considered localized only if applied to a small area of the pavement feature (usually less than 3.5 percent).

2. Major localized M&R: Major localized M&R is an extensive form of localized M&R. It includes partial-depth or full-depth patching, slab replacement, slab undersealing, and slab grinding when applied to a considerable area or portion of the pavement feature (usually more than 3.5 percent). Other M&R methods included in the major localized category are the application of an aggregate seal over the entire feature and the reconstruction of many joints in a concrete pavement.

3. Overall M&R: Overall M&R includes procedures that cover the entire pavement feature and usually im-

Figure 6. Relationship between percentage of engineers recommending M&R category and PCI.



proves its load-carrying capacity. Overall M&R includes overlays with asphalt or concrete, reprocessing or recycling of existing pavements, and total reconstruction.

Maintenance and Rehabilitation Guidelines

Excellent correlation was found between the PCI and the M&R categories. The correlation was based on results obtained from 37 airfield pavement features that included runways, taxiways, and aprons and represent a wide variety of climates, traffic, ages, and structures. Eighteen of the features were asphalt- or tar-surfaced pavements, and 19 were jointed concrete. During the field surveys of the features, all existing distress was measured, 35-mm color slides were taken, the pavement structure and age were determined, and the primary aircraft were identified. This information was given to 10 experienced engineers to aid them in making M&R decisions (the PCIs for the features were not available to the engineers when recommending M&R requirements).

Figure 6 shows the percentage of engineers recommending routine, major, or overall M&R within the next two years of the life of the pavement for each condition-rating zone. These results show that the higher the PCI, the greater the percentage of engineers recommending only routine M&R and the lower the PCI, the greater the percentage recommending overall M&R. In the middle of the PCI scale (40 to 70), there is a lack of consensus as to which to recommend.

From these results, four M&R zones were established to provide guidelines for the selection of M&R. The four zones conveniently fit the condition-rating zones used with the PCI, as shown in the table below.

M&R Zone	PCI	Rating
Routine	100	Excellent
	85	Very good
Routine, major, overall	70	Good
	55	Fair
Major, overall	40	Poor
Overall	25	Very poor
	10-0	Failed

1. Routine M&R zone (R-zone). In this zone, nearly all engineers recommended only routine M&R for the next two years. The specific routine M&R methods are determined based on types and severities of distress, as shown in Tables 1 and 2. Major or overall M&R would be recommended only in exceptional cases such as those where the pavement-condition evaluation (Figure 3) indicates that one or more of the following items exists: (a) load-associated distress accounts for a major portion of the distress deduct value, (b) load-carrying capacity is deficient as indicated by a "Yes" rating, (c) rate of pavement deterioration is rated high, (d) previous M&R applied is rated high, (e) surface roughness is rated major, (f) skid resistance and hydroplaning potential is rated very high, and (g) a change in mission requires greater load-carrying capacity. Thus, the pavement engineer should concentrate on applying routine M&R to pavement features within this zone. Timely and effective routine M&R will reduce the rate of deterioration of the pavement.

2. Routine-major-overall zone (R-M-O zone): This zone includes all pavement features that have PCIs between 41 and 70 or a condition rating of fair or good. As shown in Figure 6, there is no general agreement among engineers as to which type of M&R should be applied. Generally, however, the higher the PCI in this zone, the higher the percentage of engineers recommending routine M&R. It is therefore recommended that either routine or major M&R generally be applied to pavement features in this zone (particularly those that are rated good). The specific routine or major M&R alternative that should be selected depends on the types and severities of distress as presented in Tables 1 and 2. Overall M&R should be considered only if the condition evaluation indicates that one or more of items a through g listed above exist.

3. Major-overall zone (M-O zone): This zone includes all pavement features having PCIs between 26 and 40 or a condition rating of poor. As shown in Figure 6, there is a consensus of opinion that pavement features in this condition should receive either major or overall M&R within the next two years. For example, for one feature that had a PCI of 35, overall M&R was recommended by 80 percent of the engineers and major M&R was recommended by 20 percent (none recommended routine). Some engineers apparently feel that a pavement in this condition needs significant M&R to prevent it from exceeding the point of economic repair, but many others feel that it has already exceeded that point. The decision to select major or overall M&R should be based primarily on an economic analysis of the alternatives. However, if the condition evaluation indicates that one or more of items a through g exist, overall M&R should be strongly considered.

4. Overall zone (O-zone): This zone includes all pavement features that have PCIs between 0 and 25 or a condition rating of very poor or failed. As shown in Figure 6, there is a consensus of opinion that pavement features in this condition should receive overall M&R within the next two years. The experienced engineers apparently feel that a pavement feature in this condition is beyond the point of economical repair and that only overall M&R will provide adequate results. The decision as

Table 1. Recommended preventive and localized M&R: jointed concrete-surfaced airfield pavements.

Type of Distress	Doing Nothing	Crack Sealing	Joint Sealing	Partial-Depth Patching (bonded)	Full-Depth Patching	Slab Replacement	Slab Undersealing	Slab Grinding	Slab Jacking Grouting
Blowup				L or M[a]	H[a]	H[a]			
Cornerbreak	L	L, M, or H			M or H				
Longitudinal, transverse, or diagonal cracking	L	L, M, or H		H[b]	H	H			
D-cracking	L	L[c]	L[c]	M or H	M or H	H			
Joint-seal damage	L		M or H[d]						
Small patches (<0.46 m²)	L	M		M or H[e]	H[e]				
Large patches (>0.46 m²)	L	M		M or H[e]	H[e]	H			
Pop-outs	A								
Pumping		A	A				A		
Crazing and sealing	L			M or H		H			
Settlement and faulting	L					H		M or H	M or H
Divided slab		L, M, or H				M or H			
Shrinkage cracking	A								
Joint spalling	L		L or M	L, M, or H	M or H[g]	M or H[f]			
Corner spalling	L		L or M	M or H					

Notes: 1 m² = 10.8 ft².
A = type of distress that has only one severity level; L = low-severity distress; M = medium-severity distress; and H = high-severity distress.
[a]Must provide expansion joint.
[b]Allow crack to continue through patch except when using asphalt concrete.
[c]Seal all joints and cracks.
[d]Joint seal local areas.
[e]Replace patch.
[f]Only when surface is unacceptable.
[g]If caused by keyway failure, provide load transfer.

Table 2. Recommended preventive and localized M&R: asphalt- or tar-surfaced airfield pavements.

Type of Distress	Doing Nothing	Crack Sealing	Partial-Depth Patching	Full-Depth Patching	Skin Patching	Heating and Sand Rolling	Fog Sealing[a] (emulsion)	Application of Rejuvenator	Application of Aggregate Sealing Coat
Alligator cracking			M or H	M or H				L or M	
Bleeding	A					A			
Block cracking	L	L, M, or H						L	L or M
Corrugation	L	M or H	M or H						
Depression	L	M or H	M or H	M or H					
Jet blast	A		A		A		A		A
Joint reflection cracking	L	L, M, or H	H						
Longitudinal and transverse cracking	L	L, M, or H	H					L	L or M
Oil spillage	A		A	A					
Patching	L	M	M[b]	H[b]					
Polished aggregates	A								A
Raveling and weathering	L	H					L or M	L	M or H
Rutting	L		M or H	M or H	M or H				
Shoving	L	M or H							
Slippage cracking	A		A						
Swelling	L			M or H					

Note: A = type of distress that has only one severity level; L = low-severity distress; M = medium-severity distress; and H = high-severity distress.
[a]Requires prior approval by command pavement engineer.
[b]Replace patch.

to which overall M&R alternative to select should be based on an economic analysis of the feasible alternatives.

Economic Analysis of Maintenance and Rehabilitation Alternatives

Based on the results of pavement-condition evaluation and the guidelines for M&R selection, the engineer may need to consider more than one M&R alternative for restoring the structural integrity and operational condition of the pavement. The selection of the best alternative often requires performing an economic analysis that compares the costs of all feasible alternatives. This section presents an economic analysis procedure that compares M&R alternatives based on total present worth.

1. Select an economic analysis period (in years). The period generally used in pavement analysis is in the range of 5-30 years, depending on the future use of the feature (e.g., abandonment or change of mission). When

the present-worth method of economic analysis is used, the alternatives must be compared over the same number of years. Thus, all alternatives must have equal life.

2. Select the interest rate (r_t) and the inflation rate (r_f) to be used for calculating the present cost.

3. Estimate the annual M&R cost for each M&R alternative for every year for which work is planned during the analysis period. The cost estimates should be based on current prices.

4. Determine the salvage value of the M&R alternatives. These are the values or worths of the pavement at the end of the analysis period and can be determined by subtracting the cost of rehabilitating or reconstructing the existing pavement structure from the cost of constructing a new pavement structure over the subgrade (assuming that no pavement exists). This difference in costs, then, is the value of the existing pavement (which may be a negative value if the pavement is badly deteriorated).

5. Calculate the total present worth for each M&R alternative as follows:

$$\text{Total present worth} = \left[\sum_{i=0}^{n} C_i \times f_i\right] - (S_v \times f_n) \qquad (3)$$

where

n = number of years in analysis period,
C_i = M&R cost for year i based on current costs,
S_v = salvage value based on current costs,
f_i = present-worth factor for ith year that is a function of r_t and r_f; i.e., $f_i = [(1 + r_f)/(1 + r_t)]^i$.

After completion of these basic steps, comparison of the present worth for all M&R alternatives will assist the pavement engineer in selecting the most economic M&R alternative.

It should be emphasized that many predictions and assumptions must be made to perform the analysis. The engineer must therefore exercise judgment in selecting the best inputs and use the results of the analysis as an aid in decision making.

APPLICATION OF MAINTENANCE AND REHABILITATION GUIDELINES AND ECONOMIC ANALYSIS: EXAMPLE

This section describes an example of the procedure for determining the optimum M&R alternative for a pavement feature. The steps included are data collection, condition evaluation, selection of feasible M&R alternatives, economic analysis, and selection of the optimum M&R alternative.

The pavement used in this example is a portion of a runway constructed in 1947 of plain jointed concrete. The pavement is 46 m (150 ft) wide and 841 m (2760 ft) long. The individual slab size is 3.8×6.1 m (12.5×20 ft). Figure 7 shows the slab layout for the runway.

The critical aircraft using the runway for the past eight years has been the DC-9 (before that time only light-load aircraft operated on the runway). The pavement is exhibiting distress that began after the DC-9 started operation on the runway. The pavement engineer is concerned about the current pavement deterioration and the amount of maintenance required.

A pavement-condition survey was performed on the feature in 1977. Before the actual survey, it was observed that most of the distress occurred within the central 15 m (50 ft) (i.e., in slab rows 5, 6, 7, and 8) and that all but a few of the tire rubber marks were contained within the central 22.7 m (75 ft) (slab rows 4, 5, 6, 7, 8, and 9). The condition of rows 1 to 3 and 10 to 12 was very similar, in that they exhibited only minor distress. Therefore, the six center rows of slabs were grouped as the pavement feature to be surveyed and analyzed.

All 828 slabs in the central six slab rows of the feature were surveyed by inspecting 46 sample units of 18 slabs each (6 slabs wide by 3 slabs long). The entire feature was surveyed, because it was desired to have extensive information for this example. A few random samples from the outer three rows of slabs on both sides of the runway were also surveyed. A plot of the PCI along the runway is shown in Figure 8. A summary of the types of distress found in the central six slab rows and the calculated percentage deduct values due to load and climate are shown in Table 3. (The overall evaluation summary of this pavement is shown in Figure 3.)

Selection of Feasible Maintenance and Rehabilitation Alternatives

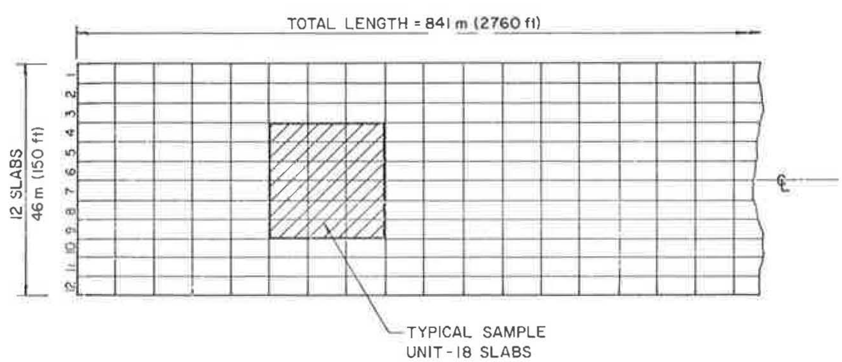
The PCI of the center six rows of slabs is 65. Thus, the feature is placed in the R-M-O zone. The outer three rows on each side of the center six rows of the runway have a PCI of 79, which places these slabs in the R-zone. Again, this supports the consideration of the center six rows as a single feature.

The M&R guidelines for the R-M-O zone state that routine or major M&R should generally be applied to pavement features in this zone unless the evaluation shows that one or more of the condition indicators is rated in a high or major category or that load-associated distresses account for a majority of the deduct values.

For this feature, the evaluation summary sheet (Figure 3) shows the following:

1. Load-associated distresses account for a majority of the deduct values,
2. Load-carrying capacity is deficient, and

Figure 7. Layout of runway feature.



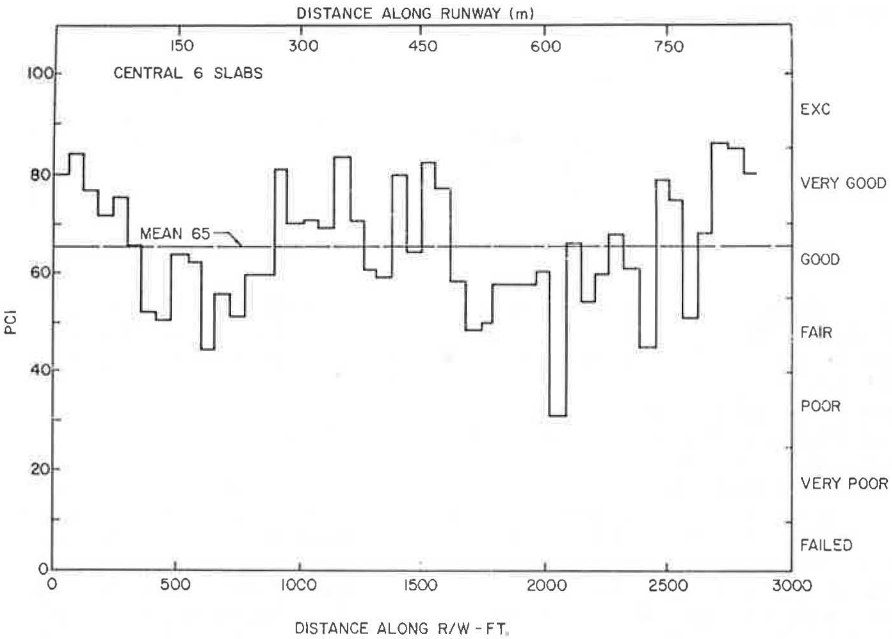


Figure 8. PCI profile along runway feature.

Table 3. Summary of distresses occurring in center six rows of slabs of runway feature.

Cause	Type of Distress	Deduct Value	Percentage Deduct Value
Load	Corner break	4	
	Longitudinal and transverse cracking	22	
	Patching (>0.46 m^2)	5	
	Shattered slab	6	
Total		37	58
Climate and durability	D-cracking	2	
	Joint-seal damage	12	
	Patching (<0.46 m^2)	1	
	Shrinkage cracking	1	
	Joint spalling	4	
	Corner spalling	3	
Total		23	36
Other	Faulting and settlement	4	
Total		4	6
Total			100

Note: 1 m^2 = 10.7 ft^2.

Table 4. Economic analysis of alternatives.

Year	M&R Work Description	Cost ($)	f	Present Worth ($)
1977	Initial construction	547 772	1.0	547 772
1982	Joint sealing	25 530	0.9108	23 353
1984	Routine	1 000	0.8774	877
1985	Routine	1 000	0.8661	866
1986	Routine	1 000	0.8452	845
1988	Routine and joint sealing	26 530	0.8141	21 598
1990	Routine	2 000	0.7843	1 569
1993	Routine and joint sealing	29 110	0.7415	21 585
1995	Routine	2 000	0.7143	1 429
1998	Routine and joint sealing	31 000	0.6753	20 934
Total				640 828

Note: r_t = 8 percent and r_i = 6 percent.

3. Previous maintenance is excessive and rated as high.

Therefore, as the guidelines indicate, overall M&R was strongly considered and, based on these considerations, the following alternatives were selected for consideration:

1. Application of major M&R to specific distresses based on recommendations in Table 1 and field conditions,
2. Replacement of the center six rows of slabs by slabs of adequate design (keel replacement),
3. Overlaying of the entire width of the runway with concrete,
4. Overlaying of the entire width of the runway with asphalt, and
5. Performance of major M&R for a few years and then performance of either items 2, 3, or 4 above.

Each of these was considered a feasible M&R alternative. Overlaying the total width was considered only because 46 m is the minimum allowable runway width. For the same reason, if the central six rows of slabs are replaced, the outer slabs must also be maintained so as to provide an acceptable operational condition.

Each alternative has its own associated costs, downtime, and personnel and equipment needs. Thus, a comprehensive economic analysis is needed to aid in selection of the best alternative.

Economic Analysis

An economic analysis period of 25 years was selected. Table 4 shows the calculation of the total present worth of one of the alternatives, the application of a 4-cm (10-in) thick, partially bonded, no. 3 concrete overlay on the entire runway in 1977. The material costs used in the analysis were obtained from local contractors. The thickness designs of the overlays and reconstruction were determined by using Corps of Engineers design methods. Given the salvage value of the pavement as \$352 560 × 0.6267 = \$220 949, its present value is then \$419 774.

Table 5 shows a summary comparison of all four M&R alternatives analyzed. Based on the total present worth, replacing the central six rows of slabs in 1977 and continuing routine M&R on the outside slabs (alternative 2)

Table 5. Summary comparison of M&R alternatives.

Alternative No.	Alternative Description	Total Present Worth Exclusive of Salvage Value ($)	Salvage Value ($)	Total Present Worth ($)	Ratio of Cost of Alternative to Cost of Alternative 1
1	Major M&R	864 232	117 962	746 270	1.00
2	Replace keel (1977)	531 131	186 620	344 510	0.46
3	Overlay with portland cement concrete (1977)	640 723	220 949	419 774	0.56
4	Overlay with asphalt cement (1977)	649 836	184 848	464 988	0.62

is the least expensive alternative. On the other hand, performing major M&R on the central six rows of slabs and routine M&R on the outer slabs (alternative 1) is the most expensive alternative.

It should be recognized that the economic analysis was based on several assumptions. Thus, the numbers shown in Table 5 cannot be exact. The main uncertainty lies in future prediction of performance. However, the analysis does provide a reasonable relative comparison among the alternatives and makes clear that a strategy of major M&R is not the best alternative.

The results of the economic analysis should not be used as a rigid rule for the selection of the best M&R alternative; rather, they should be used as an aid to the engineer in making the selection. For example, the engineer may decide on an alternative other than replacing the keel (alternative 2) because of factors not considered in the analysis, such as available funding, runway downtime during construction, and elimination of the need for routine M&R for the outer slabs required if the keel is replaced.

SUMMARY

This paper has introduced a procedure for evaluating the condition of an airfield pavement that has been officially adopted and is being used extensively by the U.S. Air Force. The procedure is based on a PCI that is a score between 0 and 100 and based on measured types, severities, and amounts of distress. The PCI agrees closely with the average rating (collective judgment) of experienced pavement engineers and correlates well with the need of the pavement for M&R.

The paper has also introduced a stepwise procedure for evaluation of a pavement feature and guidelines for the selection of the optimum M&R alternative. The procedure is largely dependent on the PCI and distress data. The guidelines are based on the results of the stepwise evaluation procedure and economic analysis and were developed based on the evaluation by many field-experienced maintenance engineers of 37 airfield pavements.

The use of the above procedures and guidelines is illustrated for a plain-jointed-concrete runway in Illinois.

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Pavement-Condition Ratings and Rehabilitation Needs

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Pavement-condition surveys are used to determine the order of priority of rehabilitation needs, to identify problems and thus promote the use of more effective short-term remedial or longer-term rehabilitation alternatives, to allow maintenance staffs to match as closely as possible practical corrective and preventive treatments and ideal solutions, and to increase the effectiveness of maintenance and rehabilitation procedures through timely and cost-effective strategies determined from pavement-management optimization procedures. The practice of obtaining pavement-condition ratings through individual raters or panels allows the effects of human bias and judgment to affect these ratings and introduces inconsistencies in priority rehabilitation lists. Two states have adopted objective measurement procedures for determining pavement-condition ratings within the last decade. But despite the effects of human bias, there are advantages to retaining subjective assessments of distress, at least for maintenance and the design of rehabilitation projects. In Ontario, a system of uniform word descriptions of extent and severity of distress is used that should lead to retention of its subjective rating system for design and maintenance; the pavement-condition rating values calculated from these word descriptions should provide consistent ratings. To develop weighting values for the various types of distress and their extent and severity and the ride-rating scaling factors that apply to each of the five regions of the province of Ontario, the results of subjective pavement-condition ratings of about 6000 km (3720 miles) of highway were subjected to iterative best-fit analysis. Although there is still room for improvement, the resulting equation is remarkably similar to that used in the state of Washington.

Pavement performance has been defined as the serviceability history of the pavement surface. That is, it is the measure over time of how well the pavement has served its function, which is to provide safe and comfortable passage to persons and goods (1).

As a pavement ages, the effects of traffic and environment decrease its initial high level of serviceability. At some future time, then, the serviceability of the pavement falls from an acceptable level to an unacceptable level. This failure to continue to provide acceptable service may stem from structural inadequacies, heavy overloads, problem pavement materials, climatic and/or environmental effects on materials, or from combinations of these. At this stage, the engineer must decide whether to do nothing and accept the consequent lower level of serviceability, to prolong the life of the pavement by a higher level of maintenance activities, to rehabilitate it by resurfacing or some similar treatment, or to upgrade its structural (and traffic) capacity by reconstruction or thick overlays. To ensure that the best decision is made, it is essential to record the condition of the pavement at defined time intervals. Then, if one possesses sufficient knowledge of probable future behavior, timely action may be taken to proceed with an appropriate treatment within the funding available.

One of the aims of a pavement management system is the analysis of the most cost-effective rehabilitation treatment and when it should be applied to optimize the use of available funds (2). Pavement-condition ratings are an essential part of the process.

In the past, assessing the condition of a pavement has been a task assigned to experienced engineers and rating panels. In more recent years, systems have been developed that minimize the effects of human judgment and bias in condition ratings because these effects lead to inconsistencies in the priority lists that are used in fund allocation.

This paper describes the efforts in Ontario to reduce the effects of human bias through the use of uniform word descriptions for pavement distresses and the application of weighted values for different distresses in determining pavement-condition ratings. When there has been sufficient experience in using the method, it is expected that the consistency of the ratings will improve and that the use of additional resources to monitor pavement conditions by the actual measurement of ride quality and distresses can thus be avoided.

EXAMPLES OF CONDITION-RATING SYSTEMS THAT USE MEASUREMENTS

Objective pavement-condition-rating systems are exemplified by the slope-variance method developed at the American Association of State Highway Officials (AASHO) Road Test at Ottawa, Illinois. Here, the present serviceability index (PSI) is a function of slope variance, rutting, cracking, and patching (1).

Because the CHLOE profilometer (which was used in the AASHO Road Test) is slow and requires a relatively large crew, in Florida the Mays ride meter, which correlates well with the CHLOE, is used. The ride meter gives the Mays meter reading (MMR) from which the ride rating (RR) can be calculated by using equations that derive from correlations with the CHLOE PSIs (slope variance only). For example, at 48 km/h (30 mph), for a certain vehicle,

$$RR = 95.1459 - 0.1792MMR \qquad (1)$$

The defect reading (DR) can be calculated by using Equation 2;

$$DR = 100 - \text{sum of deduct points} \qquad (2)$$

where deduct points are amounts for rutting, cracking, and patching that are agreed on by engineers from construction, maintenance, design, and research and applied to measurements of short representative sections; the final pavement rating (PR) is then given by

$$PR = (RR \times DR)^{1/2} \qquad (3)$$

[This method has been fully described by Smith (3).]

In the state of Washington, a modified Portland Cement Association road meter is used to measure pavement ride quality. This gives a reading in terms of counts per mile (CPM), which is used to calculate the ride score (Rs);

$$Rs = [(CPM)^{1/2}/c^{1/2}] - 1 \qquad (4)$$

in which different values of c are used for three different types of pavement to try to equalize the inherent roughness characteristics of each type and Rs = 0 represents a glass-smooth ride and Rs = 9 represents a very rough ride. The structural rating (SR) is then calculated by using Equation 5;

Figure 1. Form for evaluation of flexible pavement condition.

RIDING COMFORT RATING (AT 80 km/h)	EXCELLENT	GOOD	FAIR	POOR	VERY POOR

PAVEMENT DISTRESS MANIFESTATIONS			SEVERITY OF PAVEMENT DISTRESS					DENSITY OF PAVEMENT DISTRESS (EXTENT OF OCCURRENCE)					CHARACTERISTICS OF PAVEMENT DISTRESSES					
			1 VERY SLIGHT	2 SLIGHT	3 MODERATE	4 SEVERE	5 VERY SEVERE	6 FEW	7 INTERMITTENT	8 FREQUENT	9 EXTENSIVE	10 THROUGHOUT	REFLECTION CRACKING	PAVEMENT EDGE CRACK: PROGRESSIVE 12" from edge 0.3 m	PAVEMENT EDGE CRACK: NON PROGRESSIVE 12" from edge 0.3 m	TRANSVERSE CRACK SPACING	ALLIGATOR CRACKING: ALLIGATOR BLOCK SIZE	ALLIGATOR CRACKING: DISTORTION OF ALLIGATOR AREAS
								< 10%	10 - 20 %	20 - 50 %	50 - 80 %	80 - 100 %				m	mm	mm
SURFACE DEFECTS		1 COARSE AGGREGATE LOSS																
		2 RAVELLING																
		3 FLUSHING																
SURFACE DEFORMATION		4 RIPPLING																
		5 SHOVING																
		6 WHEEL TRACK RUTTING																
		7 DISTORTION																
CRACKING	LONGITUDINAL WHEEL TRACK	8 SINGLE																
		9 MULTIPLE																
		10 ALLIGATOR																
	MIDLANE	11 SINGLE																
		12 MULTIPLE																
	CENTER LINE	13 SINGLE																
		14 MULTIPLE																
		15 ALLIGATOR																
	MEANDER	16 SINGLE																
		17 MULTIPLE																
	PAVEMENT EDGE	18 SINGLE																
		19 MULTIPLE																
		20 ALLIGATOR																
	TRANSVERSE	21 PARTIAL																
		22 HALF																
		23 FULL																
		24 MULTIPLE																
		25 ALLIGATOR																
		26 RANDOM																
		27 SLIPPAGE																
		28																
		29																
MAINTENANCE PATCHING		SPRAY																
		SKIN																
		HOT MIX																
ADDITIONAL REMARKS																		

Revised October 1977

$$SR = 100 - \text{sum of defect values} \quad (5)$$

where defect values are amounts assigned to various states of different distresses for both flexible and rigid pavements (4), and the final pavement condition rating (PCR) is given by

$$PCR = SR[1 - (Rs/10)]^{1/2} \quad (6)$$

SUBJECTIVE RATINGS

Rationale

There have been extensive attempts to determine how well ride quality can be assessed subjectively by single raters or rating panels when compared with mechanically measured roughnesses (5-7). Hutchinson (8) has pointed out rating-scale problems. However, the advantage of using mechanical roughness-measuring devices over using raters is at present only theoretical because of problems with equipment that include poor reliability, low speed of operation, the need for frequent calibration, speed and temperature variables, and the additional costs of acquisition and operation. In Ontario, it is still found attractive to continue the assessment of ride quality by raters, not only because the rater must visit the highway and examine it for pavement distresses and other deficiencies and assess what remedial measures are needed, but also because the visits can be scheduled at convenient times and the need for coordination with roughometer schedules does not arise.

The need for "standard nomenclature and definitions for pavement components and deficiencies" was partly answered by the Highway Research Board special report issued in 1970 (9). However, this report does not provide sufficient detail to enable an observer (or a recipient of a condition report) to accurately describe (or visualize from the description) all of the defects or deficiencies of the pavement. Thus, in Ontario, a formalized procedure that has uniformly worded descriptions of distress manifestations has been provided by the preparation of two manuals (10, 11) for use by raters. These manuals contain illustrative photographs of the various types of distresses and provide guidelines for the use of descriptors of the extent of occurrence and the severity of the distress. As shown in Figures 1 and 2, the evaluation work sheets for flexible and rigid pavements, the distresses are first listed under main headings and these are then subdivided; i.e., surface distress includes factors such as raveling and flushing, surface distortion or deformation includes such items as shoving and rutting, joint deficiencies are categorized, and cracking is di-

Figure 2. Form for evaluation of rigid pavement condition.

RIDING COMFORT RATING (AT 80 km/h)	EXCELLENT	GOOD	FAIR	POOR	VERY POOR

	PAVEMENT DISTRESS MANIFESTATIONS	SEVERITY OF PAVEMENT DISTRESS					DENSITY OF PAVEMENT DISTRESS (% OF OCCURENCE)					CHARACTERISTICS OF PAVEMENT DISTRESS			
		VERY SLIGHT	SLIGHT	MODERATE	SEVERE	VERY SEVERE	FEW 10%	INTERMITTENT 10-20 %	FREQUENT 20-50 %	EXTENSIVE 50-80 %	THROUGHOUT 80-100 %	REFLECTION CRACK	TRANSVERSE CRACK SPACING (m)	JOINT OR CRACK WITH PUMPING	JOINT OR CRACK WITH DEBRIS
SURFACE DEFECTS	POLISHING														
	LOSS OF COARSE AGGREGATES														
	POT HOLE														
	SCALING														
	RAVELLING														
SURFACE DEFORMATION	FAULTING (STEPPING)														
	SETTLEMENT (SAGGING)														
JOINT DEFICIENCIES	JOINT CREEPING														
	JOINT SEALANT LOSS														
	JOINT SPALLING														
	JOINT FAILURES (BLOW UP, ETC)														
CRACKING	LONGITUDINAL														
	MEANDERING														
	CORNER														
	D														
	TRANSVERSE — SINGLE														
	TRANSVERSE — MULTIPLE														
	DIAGONAL														
	EDGE CRESCENT														
	MISCELLANEOUS CRACKS														
MISCELLANEOUS DISTRESSES	LANE SEPARATION														
	SLAB WARPING														
	WHEEL TRACK WEAR														
	OTHERS														
MAINTENANCE	FULL WIDTH JOINT REPAIR														
	FULL DEPTH RELIEF JOINT														
	PRECAST SLAB														
	COLD MIX PATCHING														
	FULL WIDTH HL PATCH														
ADDITIONAL REMARKS															

vided into types such as pavement edge and traverse (cracks are further described by characteristics such as size of alligator block or the spacing of transverse cracks). The descriptors of severity and density are based on a scale of zero to five. For the density of the extent of occurrence, the standard words used are few, intermittent, frequent, extensive, and throughout. For the severity of the distresses, the standard words are very slight, slight, moderate, severe, and very severe. Thus, for example, a distress may be described by the phrase "moderate multiple centerline crack occurs frequently over the section." This description indicates that the cracks are about 1.3-1.9 cm (0.5-0.75 in) wide and that a multiple centerline crack occurs over 20-50 percent of the length of the section. Descriptions of condition are entered on the form shown in Figure 3.

Ontario Pavement-Condition-Rating Procedure

The ride quality of the pavement is rated subjectively on a riding comfort rating (RCR) scale of 0 to 10, where 10 represents a perfectly smooth surface and 0 represents a very rough (almost impassable) road.

The condition of the pavement surface is described by using uniform word sets after a visual inspection of the density of occurrence and the severity of the various distresses.

The pavement-condition rating (PCR) is determined on a scale of 0 to 100. For flexible pavements, eight stages in the life of typical pavement have been identified by word descriptions of ride quality, distortion, and distress, and the range of rating numbers appropriate to each stage has been assigned (see Table 1). The rater(s) compare their evaluations of the RCR, the distortion, and the distress with the standard descriptions of the eight stages and then decide which stage most closely fits the pavement being rated and whether the pavement is closer to the top or the bottom of the range for the stage. The rater next assigns a PCR value to the rated pavement. Because the rater also does the pavement design work, this rating is influenced by his or her perception of the

Figure 3. Pavement-condition report.

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

REGIONAL MATERIALS & TESTING
PAVEMENT CONDITION REPORT

CONDITION RATING ______

DISTRICT No. ______ HWY. No. ______ W.P. No. ______ LENGTH ______

LOCATION: ______

Reference No. From ______ To ______ Last Contract No. ______

Offset Mileage ______ ______ R.C.R. ______

PAVEMENT: Type ______ Width ______ SHOULDER: Width ______

TRAFFIC: Year ______ A.A.D.T. ______ S.A.D.T. ______ % Trucks ______

SOILS DATA:

PAVEMENT STRUCTURE DATA:

MAINTENANCE HISTORY:

PERFORMANCE AND CONDITION:

REMARKS:

PROPOSED REMEDIAL MEASURES:

PROGRAM YEAR: Present ______ Suggested ______

DATE OF SURVEY: ______ PREPARED BY: ______

need for maintenance and rehabilitation. For rigid pavements, a similar system is used, except that this system has only six stages (see Table 2).

The PCR surveys are carried out in each of the five regions of the province of Ontario (see Figure 4) by two or three raters, generally in late spring and early summer. One region assesses all of its pavements on an annual basis. The other regions make rating surveys on a three-year cycle. The data are kept in a central computerized file and can be retrieved by the use of a remote terminal (12).

The procedure that is followed in examining the pavement is to first drive at a normal highway speed over the pavement and determine the ride quality and to then drive at a speed that does not exceed 48 km/h along the shoulder and observe the cracks and other distresses. The rater may stop occasionally to examine and measure particular distresses. The rater summarizes his or her impressions of any uniform section within a contract area by placing check marks in the appropriate boxes of the checklist of distresses (Figures 1 and 2). The rater then compares the condition of the pavement as just described against a number of descriptions of typical pavement conditions that represent various stages in the life of a pavement (i.e., column 1 of Tables 1 and 2). This comparison allows the rater to evaluate the particular pavement being examined and, from column 2 of the table, assign an appropriate condition rating number (whole numbers are sufficiently accurate). The rater also sees from column 3 of the table what rehabilitation may be needed and when it should be applied. The rater is thus alerted, where necessary, to the need for closer examination in order to make recommendations for remedial measures. (The rehabilitation alternatives listed are not all inclusive, thus leaving the way open for an examination of the whole range of possible rehabilitation strategies.)

ALTERNATIVE PROCEDURE FOR DETERMINING CONDITION RATING

The elements of a condition rating are ride quality and pavement distress. Ride quality can be measured and converted into a value on some convenient scale. For example, in Florida, a 0 to 100 ride-rating scale is used and, in the state of Washington, a 0 to 9 scale is used. In Ontario the 0 to 10 scale called the RCR is used. Pavement distresses are quantified in Florida by deducting points for rutting, cracking, and patching from a total of 100 and in Washington by a similar procedure that, however, includes a wider variety of defects and is also computerized. To determine the condition rating, in Florida, ride quality and pavement distress are combined by taking the square root of their product and, in Washington, the defect score is multiplied by the square root of a function of the ride rating. The development in Ontario of an alternative method that combines pavement distress and ride quality in a more systematic way is described below.

Currently, a method is being developed for determining a numerical defect value. This is being done by using various suitable weighting values for various types, densities, and severities of distress in a trial-and-error

Table 1. Guide for estimation of pavement-condition rating and rehabilitation priority: flexible pavements.

Pavement Condition	PCR	Rehabilitation Indicated
Poor to very poor—extensive severe cracking, alligatoring, and dishing; poor ridability, very rough and uneven surface	0-20	Reconstruct within 2 years
Poor—moderate alligatoring and extensive severe cracking and dishing, poor ridability, very rough and uneven surface	20-30	Reconstruct in 2-3 years
Poor to fair—frequent moderate alligatoring and extensive moderate cracking and dishing, poor to fair ridability, moderately rough and uneven surface	30-40	Reconstruct in 3-4 years
Poor to fair—frequent moderate cracking and dishing and intermittent moderate alligatoring, poor to fair ridability, moderately rough and uneven surface	40-50	Reconstruct in 4-5 years or resurface within 2 years with extensive padding
Fair—intermittent moderate and frequent slight cracking and intermittent slight or moderate alligatoring and dishing, fair ridability, slightly rough and uneven surface	50-65	Resurface within 3 years
Fairly good—frequent slight cracking, slight or very slight dishing, and a few areas of slight alligatoring; fairly good ridability; intermittent rough and uneven sections	65-75	Resurface in 3-5 years
Good—frequent very slight or slight cracking, good ridability, a few slightly rough and uneven sections	75-90	Normal maintenance only
Excellent—only a few cracks, excellent ridability, a few areas of slight distortion	90-100	No maintenance required

Table 2. Guide for estimation of pavement-condition rating and rehabilitation priority: rigid pavements.

Pavement Condition	PCR	Rehabilitation Indicated
Very poor—severe cracking and stepping, frequent badly broken and tilted slabs, very poor ridability, extremely rough and uneven surface throughout	0-20	Reconstruct within 2 years
Poor—severe cracking and stepping, intermittent badly broken or tilted slabs, poor ridability, very rough and uneven surface throughout	20-40	Reconstruct in 2-3 years
Fair to poor—moderate to severe stepping at cracks and joints, fair to poor ridability, moderately rough and uneven surface throughout, occasional blow ups, surface moderately polished by traffic	40-50	Cut relief joints if necessary, resurface within 2 years
Fair—moderate stepping at cracks and joints, fair ridability, slightly to moderately rough and uneven surface throughout, occasional blow ups, surface moderately polished by traffic	50-75	Cut relief joints if necessary, resurface in 2-5 years
Fair to good—slight stepping at cracks and joints, fair to good ridability, intermittent slightly rough sections; surface slightly polished by traffic	75-90	Groove or resurface to restore skid resistance if necessary, otherwise normal maintenance only
Good—little cracking between joints, intermittent slight stepping at joints, good ridability, satisfactory skid resistance	90-100	Normal maintenance only, repair joint seals as necessary

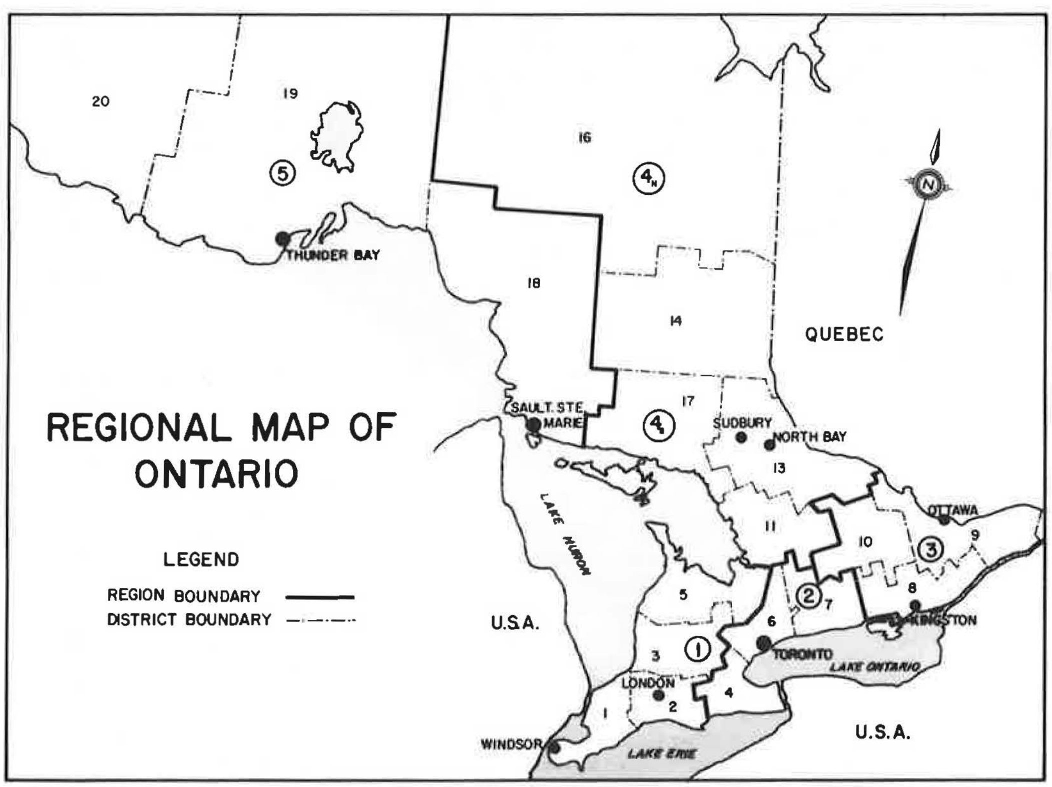


Figure 4. Regional areas for alternative pavement-condition-rating procedure.

procedure that combines the distress and ride fractions in accordance with the following equation for the distress index (DI):

$$DI = 100 \times [a \times (RCR/10)^{b}] \times [(320 - DM)/320]^{c} \quad (7)$$

where DM = sum of defects (obtained by summing the products of the sum of the density and severity weights multiplied by the weight for the type of distress) and has a probable maximum value of 320. The DI calculated by using Equation 7 is then compared with the PCR assigned by the rater. The distress weighting values and the values of a, b, and c in Equation 7 are changed appropriately to minimize the differences between the DI and the PCR (3).

By using the PCRs assigned in the five regions of Ontario (see Figure 3) during 1977, for 450 construction contracts that included more than 6000 km (3750 miles) of highway, it has been found that b can be conveniently assigned a value of $\frac{1}{2}$. This value is identical with that used in the state of Washington.

The value of a, which is believed to represent a scaling factor, was found through best-fit iterations and deviates substantially from 1.0 in three regions as shown below.

Region	Districts	Value of a
1	1, 2, 3, and 5	1.2
2	4, 6, and 7	0.8
3	8, 9, and 10	0.95
4	11 and 13	1.0
	14, 16, and 17	0.95
5	18 and 19	0.85

This scaling factor represents a contraction or expansion of the ride scale by raters who are accustomed to riding on either generally smooth pavements or on a population of pavements that have a much wider range of roughness.

c can be assigned a value of 1.0 for all regions, but the weighting values for density and severity will change from region to region. This is not unexpected, because the climatic extremes in the southwestern part of the province are not as great as those in the northern and eastern parts, although there is more freeze-thaw cycling. Furthermore, the traffic in the less densely populated areas in the north is lighter than that in the central and southwestern areas where the majority of the population is located. It is logical to expect traffic and weather to affect the significance or weight that is placed on the severity and density of various distresses.

In summary, the trial-and-error procedure has led to the following results:

1. $DI = 100(a \times RCR/10)^{1/2} \times (320 - DM)/320 \quad$ (7a)
2. The best values of a for each region are those given in the table above.
3. The best weighting values for density and severity for each region are those given in Table 3.
4. The best weighting values for the types of distress are those given in Table 4.

The correlations between the DIs calculated by using the values of a given above and the weighting values given in Tables 2 and 3 and the subjectively assigned PCRs are shown in Figures 5-10 (14). In all regions, the majority of the correlations fall within five points of the 45° line of equality. However, a significant number are between 5 and 10 points away from the line, although very few are more than 10 away. But even these few serious disagreements are undesirable and, thus, further investigations are needed to refine the weighting values or add other parameters that would reduce the disagreements to values of less than 10 points.

PAVEMENT-CONDITION RATINGS IN REHABILITATION PROGRAMMING

The PCRs derived subjectively according to the scheme outlined above reflect not only the present condition of the pavement but also a forecast of the time when remedial action will become necessary. The DI calculated directly from a description of the pavement condition will represent only the present condition. The correct interpretation of the DI depends on having a DI history for the particular section of highway that shows the rate of deterioration of the pavement and thus enables accurate forecasting of future conditions. This forecasting ability should increase our ability to examine rehabilitation alternatives and their effectiveness at different times of application and result in use of more optimum strategies.

A low PCR is a signal that a pavement section should be included on a preliminary program listing for further consideration. The list is divided into projects that should be done in the next year, those for the next two years, and those for the next five years. At the time the lists are first compiled, cost estimates based on average cost figures from past experience are also prepared so as to outline and limit the size of the programs to within predicted funding levels. Then, as a project is moved in priority from the five-year program to the two-year program and then to the final program, the rehabilitation designs are reexamined and recosted. The choice of the design to be used is determined by the availability of funds and by factors such as regional equity, regional development policy, and general public acceptability. A certain amount of fitting is necessary to make the final program conform to all of the constraints and include as many of the more deficient projects as possible.

The pavement-condition survey provides a sound initial basis for the rehabilitation design. The structural deficiencies can be identified from the description of the distresses. For example, extensive wheel-track cracking is an indication of load-induced failures that may have been caused by fatigue or heavy axle loads. If the past records show that the defect has progressed rapidly, it may be necessary to upgrade the structural capacity. The presence of alligator failures of any magnitude also indicates the need to upgrade the structural capacity. Severe rutting not accompanied by cracking of the asphalt surface may have been caused by instability of the underlying layers (perhaps because of excess moisture) or by instability of the asphalt mix itself, although rutting is generally greater on weak pavements. Areas where such deficiencies are found should have overlay thicknesses designed after nondestructive testing with a Benkelman beam or a Dynaflect (9).

Where the defects described in the rating procedure do not indicate the type of structural inadequacy and borings along the edge of pavement or cores through the pavement show an adequate pavement depth, the descriptions can be used to indicate the type of rehabilitation treatment that should be used. For example, in the colder parts of the province, a possible rehabilitation treatment for a pavement that had severe transverse cracking throughout would be to pulverize the old asphalt and use the pulverized material as the base for a new asphalt surfacing. Another possibility would be hot-mix recycling of the total depth of pavement. Severe pavement-edge cracking would suggest to the designer that the overlay should be extended beyond the normal lane width to shift the area of softening of the base during late winter and early spring out of the range of the effects of wheel loads in the normal wheel path. Exten-

Table 3. Density and severity weighting factors.

		Severity					Density				
Region	Districts	Very Slight	Slight	Moderate	Severe	Very Severe	Few	Inter-mittent	Frequent	Exten-sive	Through-out
1	1, 2, 3, and 5	1.0	2.0	5.0	5.0	5.0	1.0	2.0	5.0	5.0	5.0
2	4, 6, and 7	0.0	0.0	0.5	2.0	5.0	0.0	0.0	0.5	2.0	5.0
3	8, 9, and 10	0.0	0.0	2.0	4.0	5.0	0.0	0.0	2.0	4.0	5.0
4	11 and 13	0.0	1.0	2.5	4.5	5.0	0.0	1.0	2.5	4.5	5.0
	14, 16, and 17	0.0	0.0	0.5	3.5	5.0	0.0	0.0	0.5	3.3	5.0
5	18 and 19	0.0	0.0	0.5	2.0	5.0	0.0	0.0	0.5	2.0	5.0

Table 4. Distress weighting factors.

Pavement Distress Manifestation	Weight	Pavement Distress Manifestation	Weight
Surface defects		Cracking (continued)	
Loss of coarse aggregate	0.5	Centerline	
Raveling	0.5	Single	0.5
Flushing	0.5	Multiple	1.0
Surface deformation		Alligator	2.0
Rippling	0.5	Meander	
Shoving	0.5	Single	0.5
Wheel-track rutting	3.0	Multiple	1.0
Distortion	3.0	Pavement edge	
Cracking		Single	0.5
Longitudinal wheel track		Multiple	1.0
Single	1.0	Alligator	1.5
Multiple	1.5	Transverse	
Alligator	3.0	Partial	0.5
Midlane		Half	0.5
Single	0.5	Full	1.5
Multiple	1.0	Multiple	2.0
		Alligator	3.0
		Random	0.5
		Slippage	0.5

Figure 6. Relationship between distress index and pavement-condition rating: region 2.

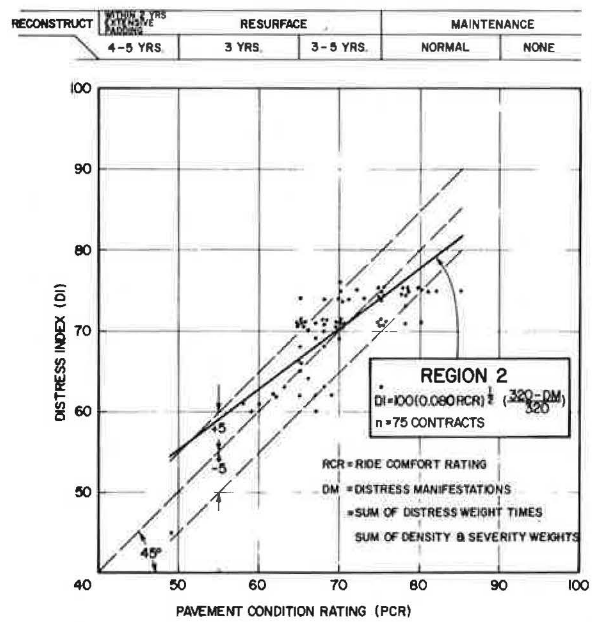


Figure 5. Relationship between distress index and pavement-condition rating: region 1.

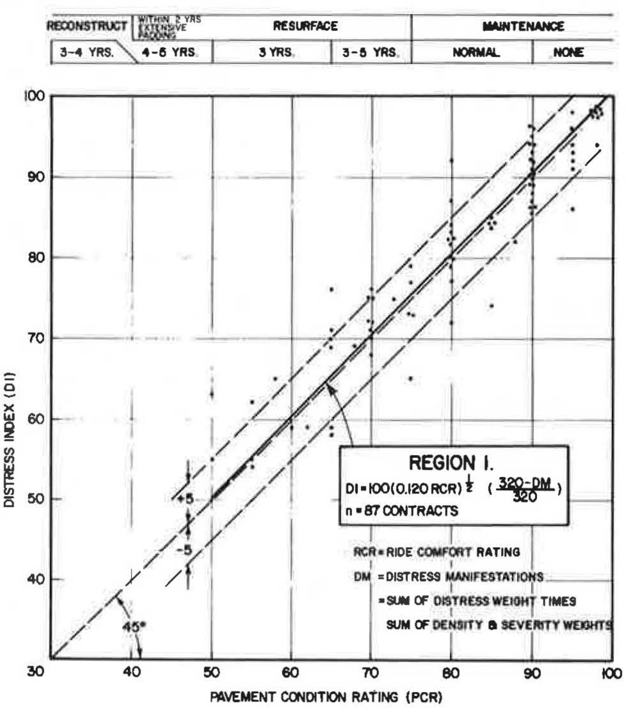


Figure 7. Relationship between distress index and pavement-condition rating: region 3.

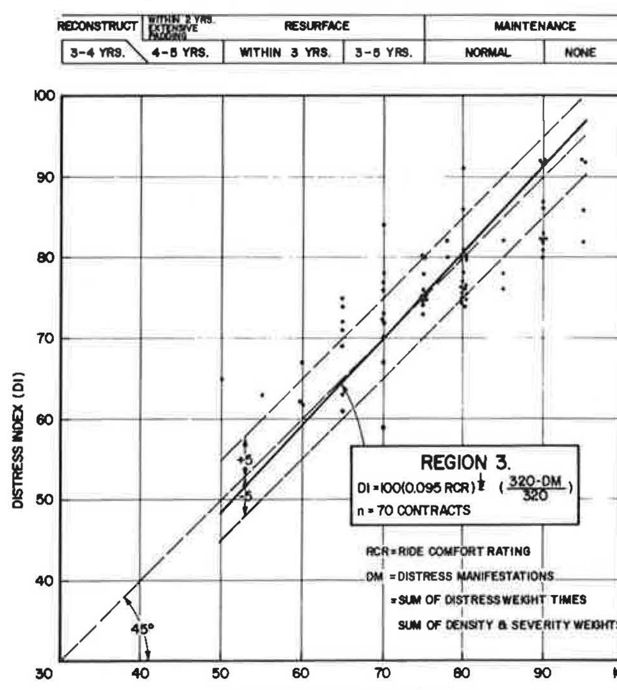


Figure 8. Relationship between distress index and pavement-condition rating: region 4 (districts 11 and 13).

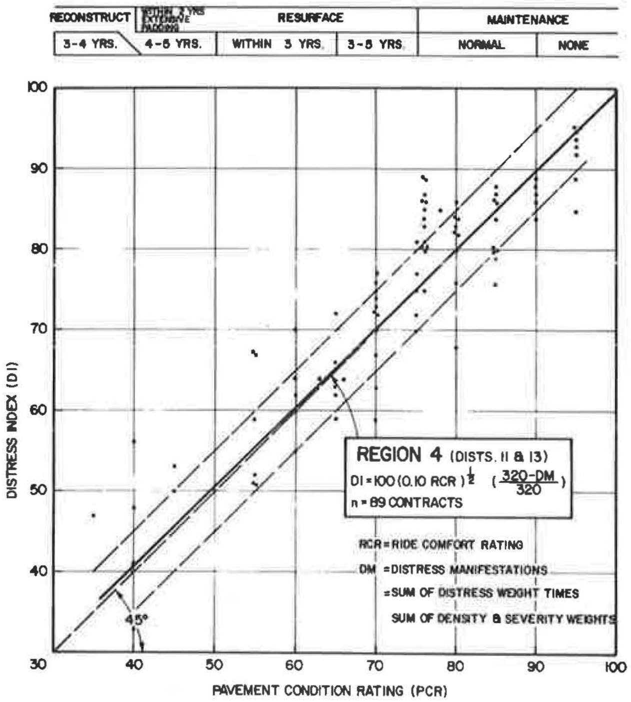
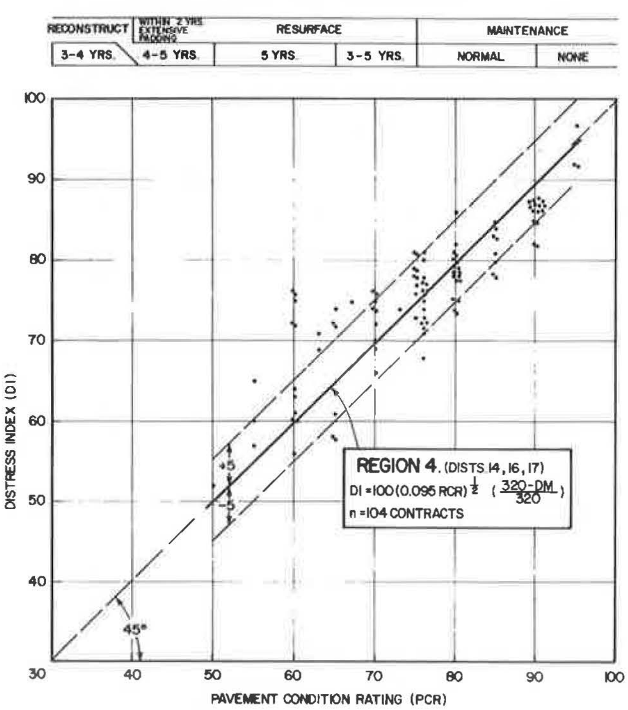


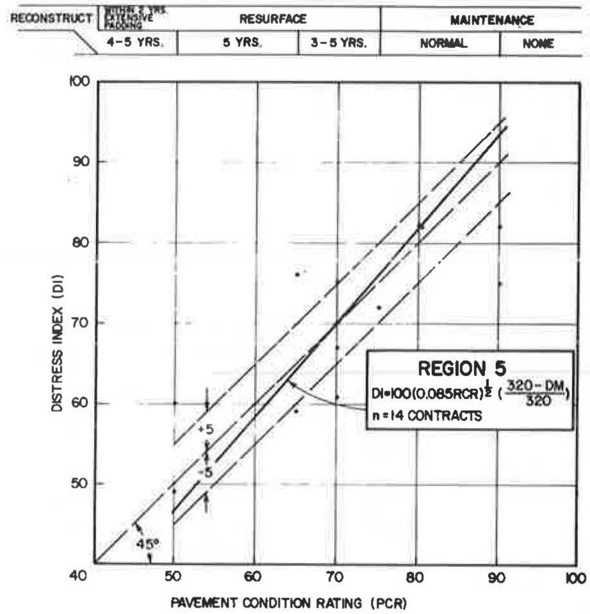
Figure 9. Relationship between distress index and pavement-condition rating: region 4 (districts 14, 16, and 17).



sive joint spalling and blowups in concrete pavements would alert the designer to the need to create pressure relief joints and to the probable need to install subdrains before overlaying to minimize future blowups from moisture expansion.

The severity and density of deformations other than rutting might point out the need for frost-heave treatments or drainage improvements and indicate the amount

Figure 10. Relationship between distress index and pavement-condition rating: region 5.



of preparatory padding or leveling that might be required before overlaying.

When the condition surveys from previous years are available, an examination of the development and rate of progression of various types of distress is useful in assessing the effects of postponing rehabilitation treatment.

CONDITION SURVEYS IN MAINTENANCE

The use of condition surveys in maintenance activities is primarily to ensure safe passage of vehicles over the highway; the preservation of the pavement and shoulders must play a secondary role. The preservation and prolongation of pavement life will, however, probably increase in importance in future years as a result of the effects of inflation on costs and of static or reduced construction budgets. Condition surveys for maintenance purposes are thus needed more to direct immediate and short-term corrective measures than to handle preventive or medium- and longer-term maintenance. In this context, Bartell (15) has indicated that the alphanumeric defect-rating system used in California, which reflects the extent and severity of a distress, is more relevant to the near-term type of maintenance action than was the previously used simple defect number.

The alternatives that may be considered in maintenance work range from filling cracks and potholes to full-width hot-mix patching, from work that can be done by regular maintenance patrol crews to work that requires specialized equipment and trained personnel and even to full-scale contract work.

It is obvious that, despite the many maintenance alternatives available, the selection that is suitable for use in correcting any particular distress condition is more limited. It is also clearly impractical to try to match the ideal corrective action and the specific distress condition in all cases. Nevertheless, the effectiveness of maintenance should be improved if better matching is accomplished. A uniform approach to descriptions of pavement distress is a first step toward systematically improving the overall effectiveness of maintenance (16).

The manuals of pavement-distress manifestations (10, 11) that contain complete descriptions of the total range of distress conditions appear to be too detailed for maintenance purposes. The maintenance rater is not attempting to trace the performance history, he or she is trying to determine immediate and short-term maintenance needs, i.e., only those distress conditions that he or she must do something about. Therefore, a simplified manual of distress manifestations is currently in preparation, designed specifically for condition surveys for maintenance needs.

In the maintenance manual, the descriptions of any type of distress are restricted to the words for severity conditions, i.e., slight, moderate, and severe. The guidelines for use of these terms in maintenance-need surveys are similar to the guidelines for condition rating for rehabilitation purposes. It is not intended that the maintenance guidelines be used in general periodic inspections—these guidelines will be used only as required to plan immediate and short-term maintenance activities. However, the main purpose of these guidelines is to ensure as far as is practicable that effective maintenance treatments are used as a rule rather than by chance or good judgment.

Condition-rating surveys for maintenance purposes may use the same elements as condition-rating surveys for rehabilitation purposes. However, because of the different purposes to which they are put, it is essential to clarify, through the use of adequate manuals, the system applicable in each case. In condition rating for rehabilitation purposes, it appears sufficient to assess defects in terms of weighted numbers when the purpose is to determine priorities, but details of distress are needed when the purpose is the design of a rehabilitation treatment. In condition surveys for maintenance purposes, it is important that details of the extent and severity of any specific distress be separated to facilitate the choice of the most effective maintenance treatment.

The advantages of using word descriptions rather than arbitrary defect values in rating surveys are that

1. Word descriptions are on value scales associated with the language itself, a language that has been learned early in life and is used daily by raters;
2. The rater assesses conditions in familiar terms; and
3. Word descriptions can be translated into weightings for condition ratings and may also serve for choosing appropriate maintenance treatments.

The system used in Washington for condition rating has undergone several stages. The alternative system used in Ontario, started a few years ago, has benefited from previous reports from Washington. The Ontario system resembles the Washington system when the equations that combine ride quality and distress manifestations to derive a condition rating are compared. The similarities in the two systems then become obvious.

Condition rating systems are a fundamental part of evaluating pavement performance. They will probably remain subjective until distresses can be measured by mechanized systems, a task that has not yet been successfully or effectively addressed. Meanwhile, subjective condition ratings remain a convenient and relatively satisfactory procedure for use in assessing rehabilitation needs.

It is hoped that the use of a dual condition-rating system in Ontario will serve both as a method of determining the order of priority of rehabilitation needs and as an aid in the selection of suitable treatments.

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Pavement Evaluation and Overlay Design: A Method That Combines Layered-Elastic Theory and Vibratory Nondestructive Testing

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A procedure has been developed for the determination of the load-carrying capacity and required overlay thickness of airport pavements. The procedure combines a layered-elastic theoretical approach and vibratory nondestructive testing to determine the value of the Young's modulus of the subgrade. A computer program SUBE is used to determine the value of the Young's modulus of the subgrade from the measured dynamic response of a pavement. A computer program PAVEVAL is used to calculate the load-carrying capacity and required overlay thickness in terms of the structure of the pavement and subgrade and in terms of limiting strain and stress conditions. The procedure was evaluated by calculating the load-carrying capacity and overlay thicknesses for single-wheel and multiple-wheel loadings on rigid and flexible pavements.

The increasing cost of pavement construction and rehabilitation makes it essential to have a fast and reliable method of accurately determining the load-carrying capacity of a pavement and of predicting the overlay thickness that will be required to upgrade it. A method for the rapid evaluation of airport pavements has been developed at the U.S. Army Engineer Waterways Experiment Station (WES) for the Federal Aviation Administration. This method of pavement evaluation and overlay design is based on vibratory nondestructive testing combined with a layered-elastic theoretical formalism (1-6).

This method of pavement evaluation and overlay design consists of the determination of the Young's modulus of the subgrade from the dynamic response of the pavement as measured by vibratory nondestructive tests, followed by the use of layered-elastic theory and the predicted value of the Young's modulus of the subgrade for the calculatation of the allowable load-carrying capacity and the required overlay thickness of the pavement.

The method of layered-elastic theory and vibratory nondestructive testing is compared with the conventional method for evaluating asphalt concrete (AC) pavements that uses the California bearing ratio (CBR) and with the Westergaard method for evaluating portland cement concrete (PCC) pavements (7). It is also compared with the method of pavement evaluation that uses a correlation between the strength of a pavement and the dynamic stiffness modulus (DSM) that can be obtained from vibratory nondestructive testing (1).

The CBR and the Westergaard methods require destructive tests that measure the CBR and the coefficient of subgrade reaction, respectively. To circumvent these destructive tests, a vibratory, nondestructive testing method for evaluating AC and PCC pavements was developed at WES that directly correlates the load-carrying capacity and the required overlay thickness to a mechanical impedance that is measured at the pavement surface (the DSM).

The DSM is calculated from data that are obtained by using a hydraulic vibrator developed at WES that can generate dynamic loads up to 71 kN [16 000 lbf(16 kips)]a constant 71-kN static load and a constant frequency of 15 Hz (4). The data obtained consist of curves of the dynamic load versus the deflection that is measured at the pavement surface. These dynamic-load-deflection curves are generally nonlinear; the DSM is the slope of the curve at a dynamic load of about 62-67 kN [14 000 to 15 000 lbf (14-15 kips)]. The measured DSM is corrected to that at a common pavement temperature of 21°C (70°F), and the corrected value of the DSM is correlated with the load-carrying capacity and required overlay thickness of the pavement (1, 6). The DSM method is empirical and does not include the effects of the layered elastic structure of the pavement or of the interface conditions between the pavement layers.

This method of directly correlating pavement performance with vibratory, nondestructive testing data can be improved by combining the layered-elastic theory of pavement behavior with the pavement-impedance values measured by the vibratory testing. In this way, the effect of the pavement structure can be considered. The layered-elastic model of pavement behavior requires that the Young's moduli and Poisson's ratios of the subgrade and pavement layers be known. The elastic moduli of the pavement layers were estimated by various means; the Young's modulus of the subgrade was obtained by vibratory nondestructive tests.

IMPEDANCE METHOD FOR DETERMINATION OF YOUNG'S MODULUS OF SUBGRADE

Measurement of Pavement Impedance

The WES 71-kN vibrator applies to the pavement surface a static load of 71 kN and a dynamic load of up to 67 kN at a frequency of 5-100 Hz. Both loads are applied to the pavement surface through a circular 46-cm (18-in) diameter baseplate. The vibrator can perform two types of nondestructive impedance tests:

1. Tests that determine the dynamic deflection of the pavement surface as a function of the applied load at a fixed frequency (i.e., tests that produce dynamic-load-deflection curves) and
2. Tests that determine the dynamic deflection as a function of the frequency at a fixed dynamic load (i.e., tests that produce frequency-response-spectrum curves).

Only method 1 above is used in this paper to determine the Young's modulus of the subgrade. In general, these dynamic-load-deflection curves are nonlinear and a nonlinear dynamic theory is required to extract the value of the subgrade Young's modulus from them by removing the extraneous effects of the static and dynamic loads developed by the vibrator on the predicted values of the subgrade Young's modulus (3, 4). The computer program SUBE was developed from the nonlinear theory of pavement response to dynamic loads and used to determine the Young's modulus of the subgrade from the measured dynamic-load-deflection curves.

A typical dynamic-load-deflection curve measured at 15 Hz is shown in Figure 1.

Nonlinear Dynamic Theory of Pavement Response

In the nonlinear dynamic theory of pavement response that was developed to describe the dynamic-load-deflection curves and to predict the value of the Young's modulus of the subgrade, the dynamic response of the pavement surface to forced vibrations is modeled as a nonlinear harmonic oscillator whose equation of motion is

$$m\ddot{x} + C\dot{x} + k_{00}x + bx^3 + ex^5 = F_V = F_S + F_D \qquad (1)$$

where

- m = effective mass of pavement,
- $\ddot{x}$ = acceleration of pavement surface,
- C = damping constant,
- $\dot{x}$ = velocity of pavement surface,
- k_{00} = linear spring constant,
- x = total displacement of pavement surface,
- b = third-order nonlinear coefficient,
- e = fifth-order nonlinear coefficient,
- F_V = total force applied by vibrator,
- F_S = static force applied by vibrator, and
- F_D = dynamic force applied by vibrator.

From Equation 1, the static force is

$$F_S = k_{00}x_e + bx_e^3 + ex_e^5 \qquad (2)$$

where x_e = static elastic displacement of the pavement surface.

The solution of the equation of motion of the pavement surface is

$$x = x_e + \xi \qquad (3)$$

where ξ = dynamic displacement of the pavement surface. Thus, the solution of Equation 1 is given by considering a sum of harmonic terms, $\cos \omega t$, $\cos 3\omega t$, and $\cos 5\omega t$ to obtain the equivalent linear spring constant:

$$k = k_0 + (3/4)b\theta\xi^2 + (5/8)e\eta\xi^4 \qquad (4)$$

where

$$k_0 = k_{00} + 3b\epsilon_2 x_e^2 + 5e\epsilon_4 x_e^4 \qquad (5)$$

and

- k = dynamic spring constant,
- k_0 = static elastic spring constant, and
- θ, η, ϵ_2, and ϵ_4 = dimensionless parameters.

When these substitutions are made, the solution to the equation of motion is

$$\xi = (F_D/S_0)(1 + \alpha_1\psi + \alpha_2\psi^2) \qquad (6)$$

$$S = S_0(1 + \beta_1\psi + \beta_2\psi^2) \qquad (7)$$

$$DSM = S_0(1 + \delta_1\psi + \delta_2\psi^2) \qquad (8)$$

where

- S_0 = impedance at zero dynamic load,
- α_1 and α_2 = frequency-dependent coefficients related to the nonlinear elastic parameters b and e,
- ψ = F_D^2/S_0^4,
- S = secant dynamic modulus (impedance),
- DSM = tangent dynamic modulus (impedance),

and the β_j and δ_j coefficients are given by

$$\beta_1 = -3\alpha_1 \qquad (9)$$

$$\beta_2 = \alpha_1^2 - \alpha_2 \qquad (10)$$

$$\delta_1 = -3\alpha_1 \qquad (11)$$

and

$$\delta_2 = 9\alpha_1^2 - 5\alpha_2 \qquad (12)$$

The terms $\alpha_1\psi$ and $\alpha_2\psi^2$ in Equations 6-8 describe the departure from linear of the dynamic-load-deflection curve. The quantities α_1 and α_2 are measured directly from these curves.

The dynamic quantities S_0, α_1, and α_2 can be related to the static elastic parameters k_{00}, b, and e, which in turn are related to the Young's moduli of the pavement layers and the subgrade. Therefore, the shape of the dynamic-load-deflection curve depends on the layered-elastic structure of the pavement. For example,

$$\alpha_1 = -(3/4)b\theta(k_0 - m\omega^2) \qquad (13)$$

The values of k_{00}, b, and e can be expressed in terms of the elastic moduli of the pavement layers and the subgrade and in terms of the finite depth of influence of the stress and strain field that is produced in the pavement and the subgrade by the static load of the vibrator.

The general expressions for k_{00}, b, and e in terms of the elastic structure of a pavement are rather complex but, for the case of a homogeneous half space, they simplify as follows:

$$l = l_0 + l_2 x_e^2 + l_4 x_e^4 \qquad (14)$$

$$k_{00} = 2\pi a^2 \Psi(1 - \nu_S)G_S/l_0(1 - 2\nu_S) \qquad (15)$$

$$b = -4\pi a^2 \Psi l_2(1 - \nu_S)G_S/l_0^2(1 - 2\nu_S) \qquad (16)$$

and

$$e = 6\pi a^2 \Psi\delta(1 - \nu_S)G_S/l_0(1 - 2\nu_S) \qquad (17)$$

where

$$\delta = (l_2/l_0)^2 - l_4/l_0 \qquad (18)$$

- l = finite depth of influence of the static stress and strain field on the pavement and subgrade,
- l_0, l_2, and l_4 = coefficients of series expansion of finite depth of influence,
- a = radius of vibrator baseplate,
- ψ = volume factor for the frustum of the cone of stress and strain in the pavement,
- ν_S = Poisson's ratio of subgrade, and
- G_S = shear modulus of subgrade.

[More general expressions for a layered system are given by Weiss (3).] Therefore, it is possible to relate the

Figure 1. Typical dynamic-load-deflection curve for AC pavement.

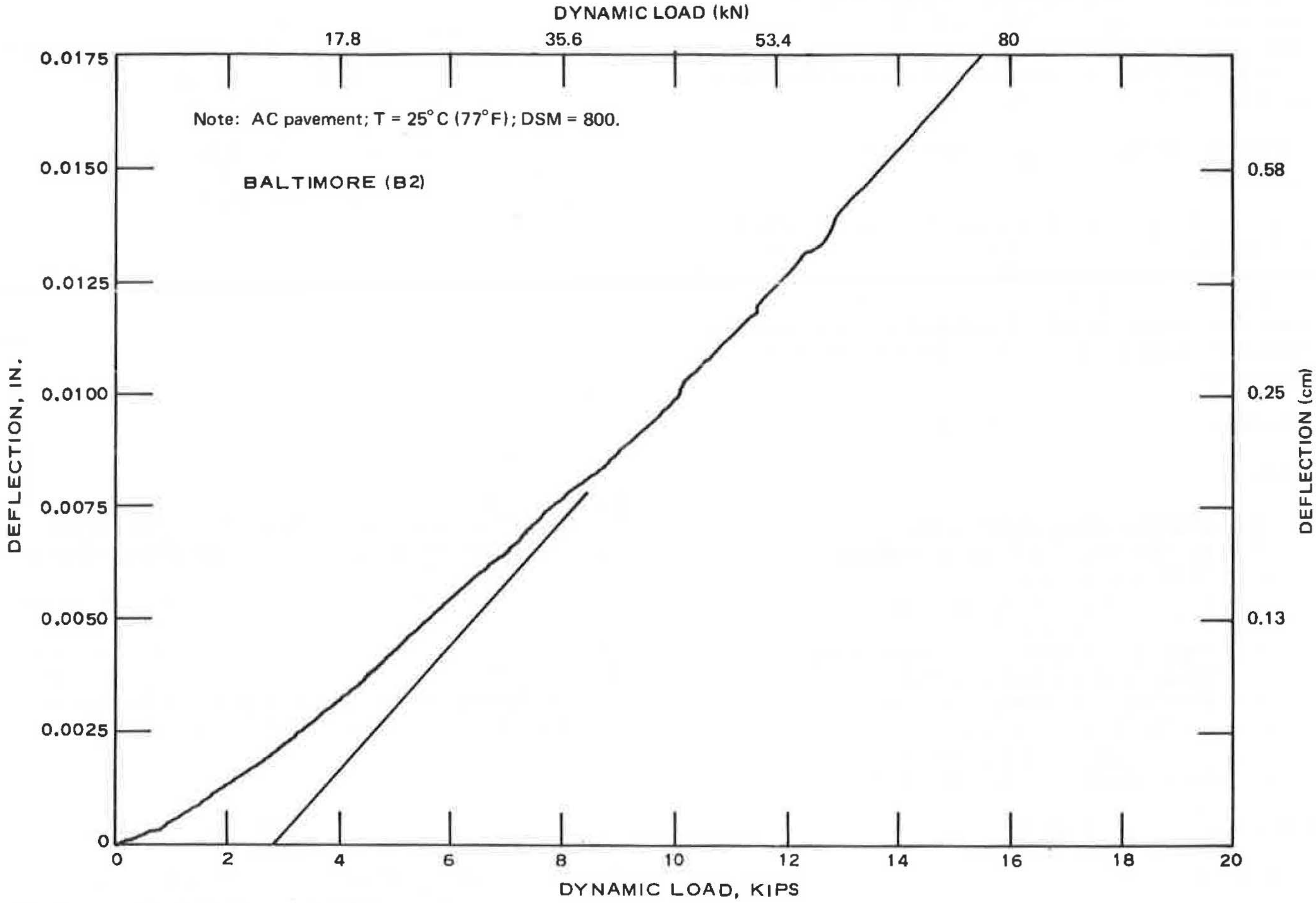
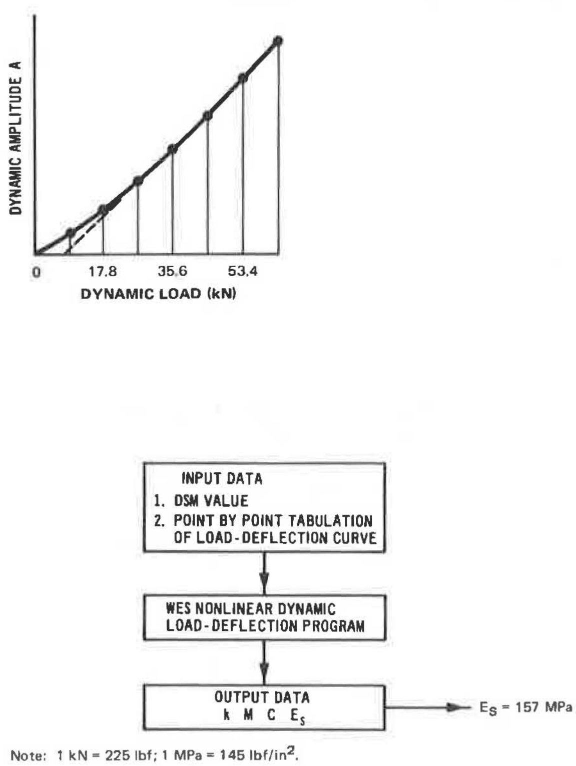


Figure 2. Determination of modulus of subgrade from a measured load-deflection curve.



Note: 1 kN = 225 lbf; 1 MPa = 145 lbf/in^2.

elastic structure of a pavement and its subgrade to a dynamic-load-deflection curve as described by the parameters S_0, α_1, and α_2.

For a specific choice of the elastic moduli of the pavement layers (and the choice $\nu = 0.35$ for the subgrade), the shape of the theoretically predicted dynamic-load-deflection curve depends only on the value of the Young's modulus of the subgrade. This value is obtained by requiring that the theoretically predicted dynamic-load-deflection curve agree with the measured dynamic-load-deflection curve.

Dynamic Pavement-Response Computer Program

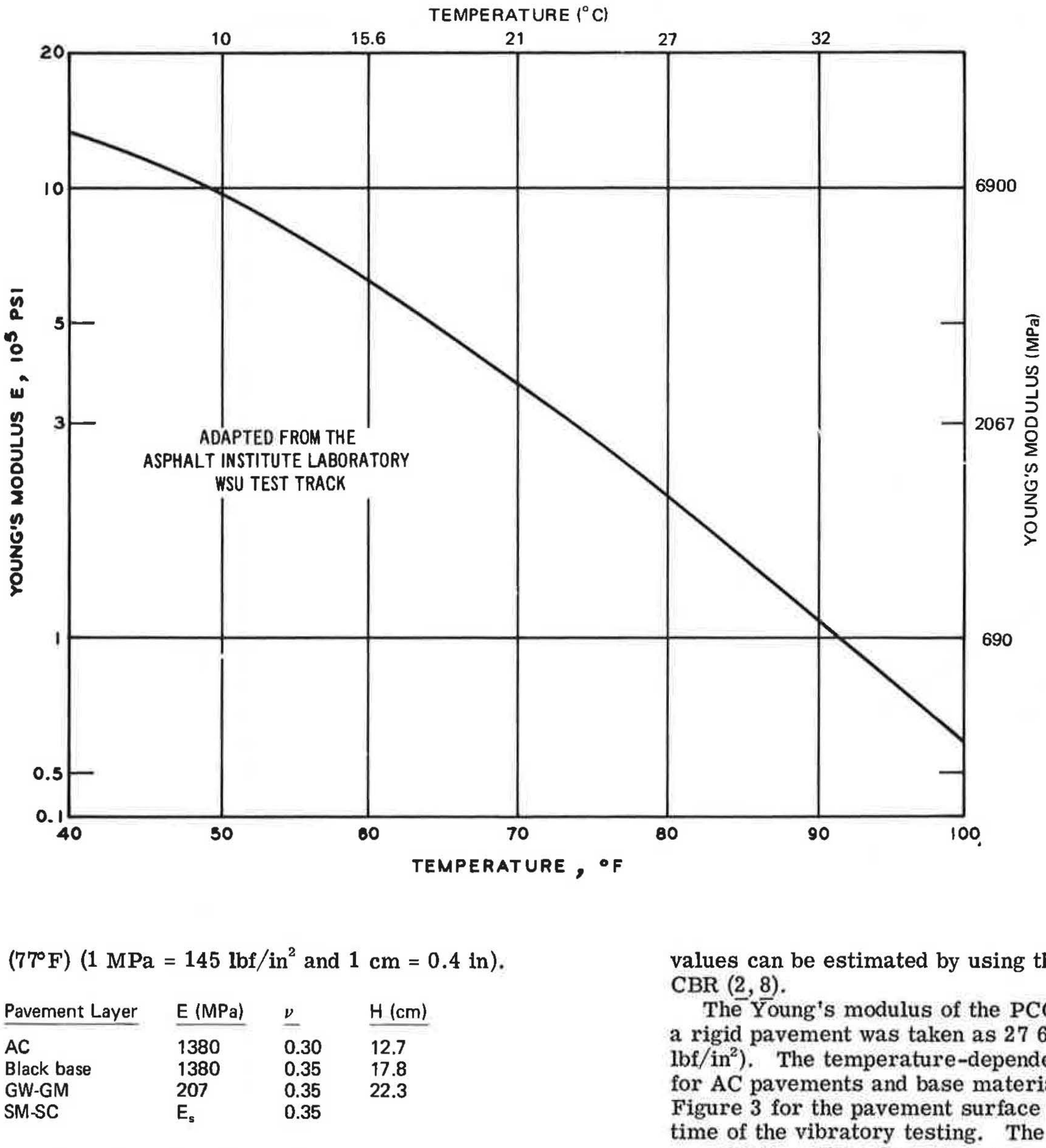
The computer program SUBE is used to calculate the value of the Young's modulus of the subgrade from input data taken from the measured dynamic-load-deflection curves (4). The pavement input parameters for the program include the Young's modulus, Poisson's ratio, and thickness of each pavement layer and the Poisson's ratio of the subgrade. The input that is taken from the vibratory, nondestructive testing data is the DSM value and a point-by-point description of the measured dynamic-load-deflection curve. The program iterates the value of the Young's modulus of the subgrade and determines the value of it that makes the theoretically predicted DSM value agree with the measured DSM value so that the theoretically predicted dynamic-load-deflection curve will agree with the measured dynamic-load-deflection curve. The procedure is outlined in Figure 2 for the pavement described below at 25°C

Table 1. Young's moduli and Poisson's ratios of base and subbase materials.

Material	Description	Assigned Value of Young's Modulus (MPa)	Assigned Value of Poisson's Ratio
Crushed limestone	Crushed limestone	551	0.35
GW	Well-graded gravel	413	0.35
GW-GM	GW and silty gravel	345	0.35
GP	Poorly graded gravel	276	0.35
GP-GC	GP and clayey gravel	241	0.35
SP-SM	Poorly graded sand and silty sand	207	0.35
Black base	Mineral aggregate and bituminous material	Temperature dependent	0.30

Note: 1 MPa = 145 lbf/in².

Figure 3. Assumed temperature dependence of Young's modulus of AC pavements and AC base materials.



(77°F) (1 MPa = 145 lbf/in^2 and 1 cm = 0.4 in).

Pavement Layer	E (MPa)	ν	H (cm)
AC	1380	0.30	12.7
Black base	1380	0.35	17.8
GW-GM	207	0.35	22.3
SM-SC	E_s	0.35	

The Poisson's ratios of the wearing surfaces, base courses, and subbase courses were chosen as $\nu = 0.2$ for PCC pavements, $\nu = 0.3$ for AC pavements and base materials, and $\nu = 0.35$ for all other base and subbase materials. The Poisson's ratio for all subgrade soils was taken to be $\nu = 0.35$. Reasonable estimates of the values of the Young's moduli of base and subbase materials are given in Table 1. When the CBRs of the base and subbase materials are known, the Young's modulus values can be estimated by using the equation E = 1500 CBR (2, 8).

The Young's modulus of the PCC wearing surface of a rigid pavement was taken as 27 600 MPa (4 000 000 lbf/in^2). The temperature-dependent Young's modulus for AC pavements and base materials was obtained from Figure 3 for the pavement surface temperature at the time of the vibratory testing. The value of the temperature-dependent Young's modulus is entered into the SUBE computer program to determine the Young's modulus of the subgrade.

Laboratory Resilient-Modulus Tests

The values of the Young's modulus of the subgrade predicted from the vibratory nondestructive field tests by using the SUBE computer program were correlated with the values of the Young's modulus of the subgrade determined by laboratory resilient-modulus (M_R) tests. The laboratory resilient modulus is expressed in terms of the applied dynamic deviator stress and the static confining pressure (9, 10). Some examples of resilient-modulus test data (obtained from undisturbed subgrade soil samples taken at three selected airport pavement sites) are shown in Figures 4-6.

The results of the laboratory resilient-modulus test can also be described in terms of a nonlinear harmonic oscillator. An analysis (similar to that used to describe the dynamic-load-deflection curves obtained in the field) of the laboratory test data gave the following theoretical expressions for the resilient modulus (4):

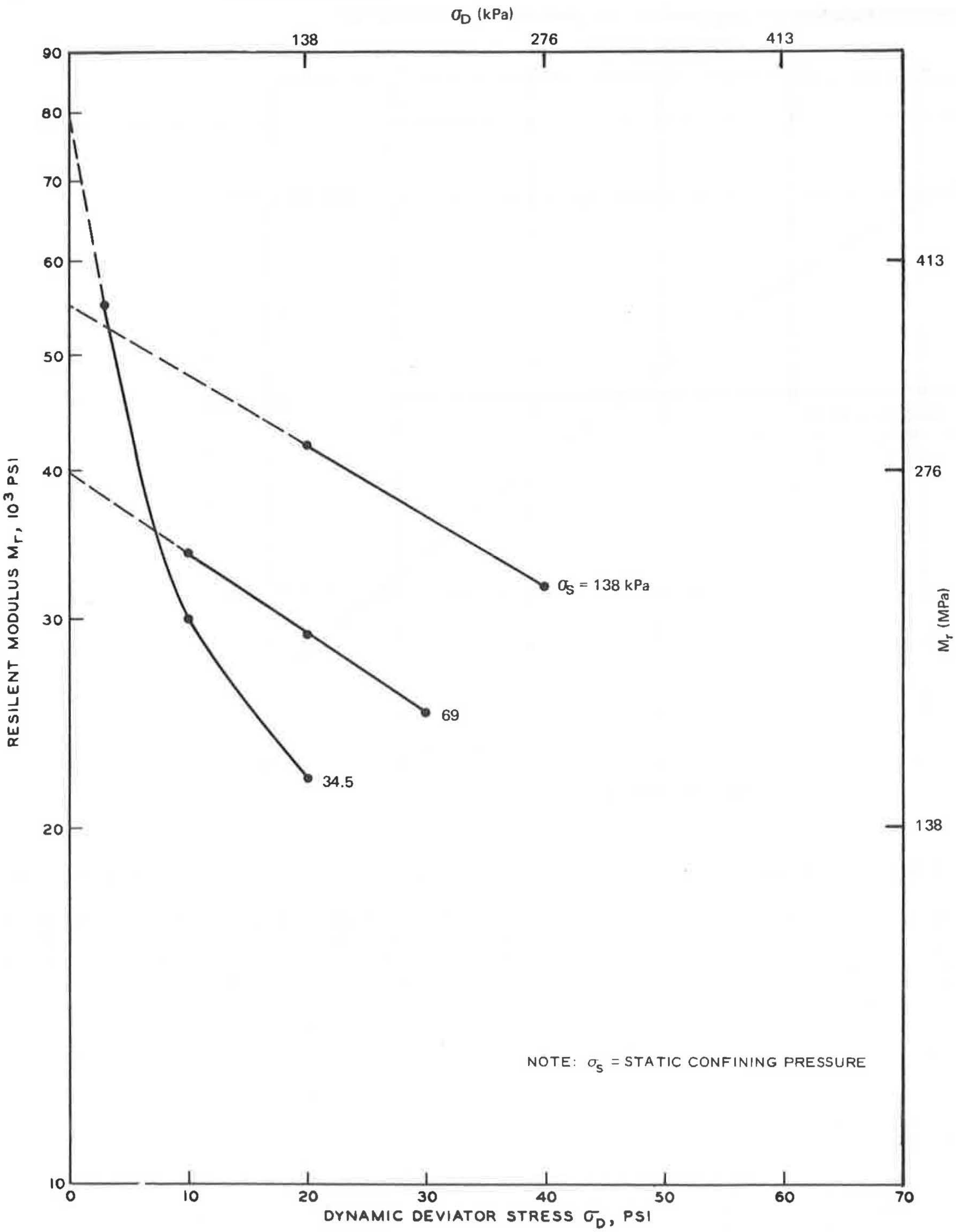
$$M_{rs} = M_{r0}[1 + \beta_1' \psi' + \beta_2' (\psi')^2] \quad (19)$$

$$M_{rt} = M_{r0}[1 + \delta_1' \psi' + \delta_2' (\psi')^2] \quad (20)$$

where

M_{rs} = secant resilient modulus,

Figure 4. Laboratory resilient modulus: Albuquerque site 17.

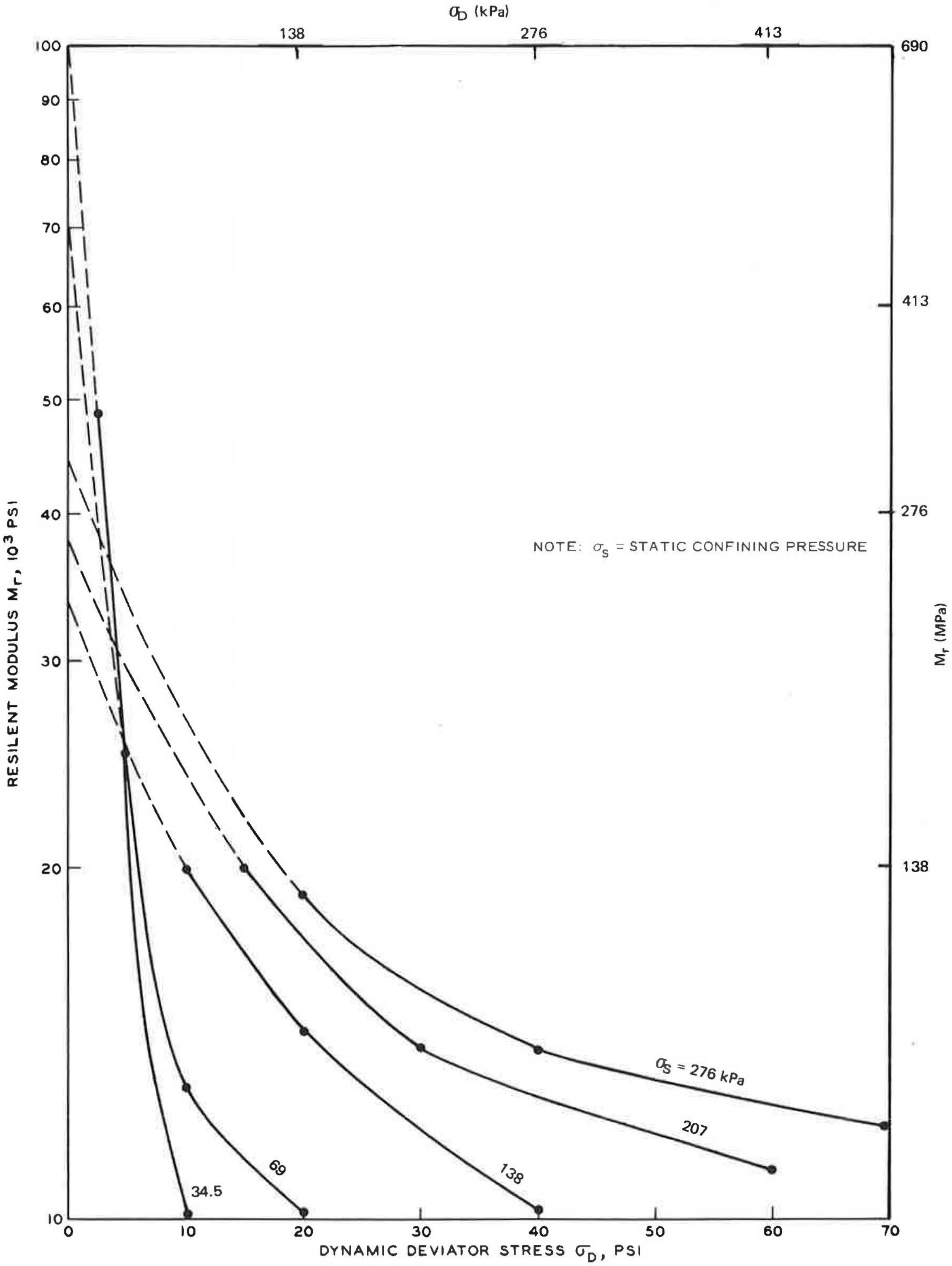


M_{rt} = tangent resilient modulus,
M_{r0} = resilient modulus at zero dynamic deviator stress,
ψ' = $A^2\sigma_D^2/S_0'^4$,
A = area of circular base of cylindrical soil specimen,
σ_D = dynamic deviator stress, and
S_0', β_j', and δ_j' = coefficients that depend on the value of the static confining pressure.

The static confining pressure can be described in terms of the static displacement of the cylindrical soil sample as follows:

$$F_S = \sigma_S A = k_{00}' x_e + b'(x_e')^3 + e'(x_e')^5 \tag{21}$$

Figure 5. Laboratory resilient modulus: Rockland site 1.

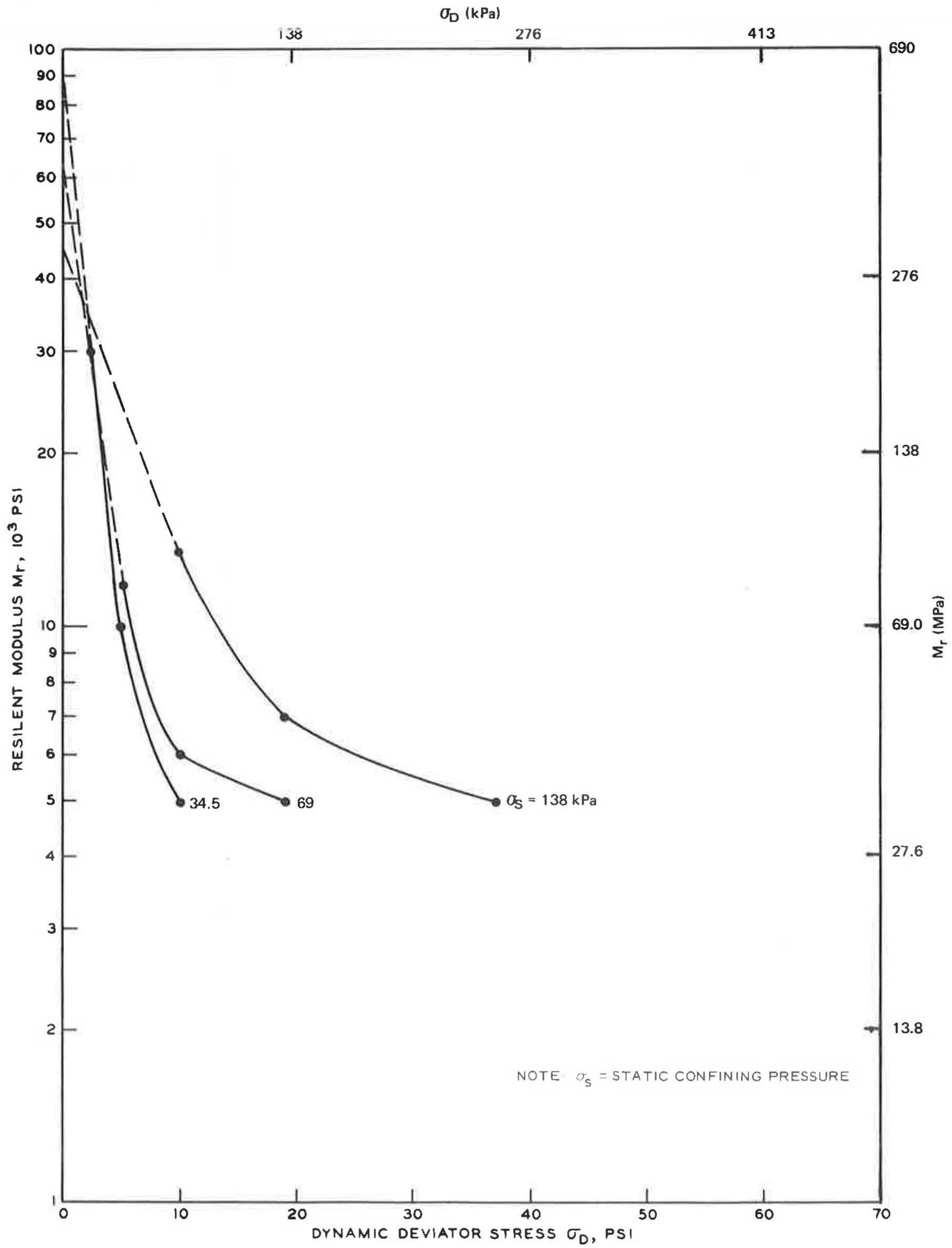


where

σ_s = static confining pressure,
k'_{00} = linear elastic spring constant,
b' = third-order nonlinear elastic coefficient of soil sample,
e' = fifth-order nonlinear elastic coefficient of soil sample, and
x'_e = axial static elastic displacement of soil sample.

The coefficients k'_{00}, b', and e' are related to the coefficients S'_0, β'_1, and β'_2 that appear in the expression for the resilient modulus given by Equation 19 (4). Therefore, the measurement of the resilient modulus allows the determination of the static elastic coefficients k'_{00}, b', and e'.

Figure 6. Laboratory resilient modulus: Rockland site 7.



The value of the Young's modulus of the subgrade depends on the static confining pressure and has been found (11) to be

$$E = L\sigma_S/x'_e = E'_0 + E'_2(x'_e)^2 + E'_4(x'_e)^4 \quad (22)$$

where

$$E'_0 = Lk'_{00}/A \quad (23)$$

$$E'_2 = Lb'/A \quad (24)$$

$$E'_4 = Le'/A \quad (25)$$

and L = length of cylindrical soil sample. (This procedure for determining the Young's modulus of the subgrade from the measured resilient-modulus test data is still under study; only preliminary numerical results are thus far available for comparison with the results obtained from field test data.)

Numerical Values of Predicted Young's Modulus of Subgrade

Laboratory soil-gradation tests were done on samples taken from the base, subbase, and subgrade at the three airport pavement sites investigated. Field measurements of the thicknesses and CBRs were also made on the base, subbase, and subgrade materials (11). The mean pave-

Figure 7. Comparison of values of Young's moduli of subgrades: computer program SUBE versus Shell formula—various airport sites.

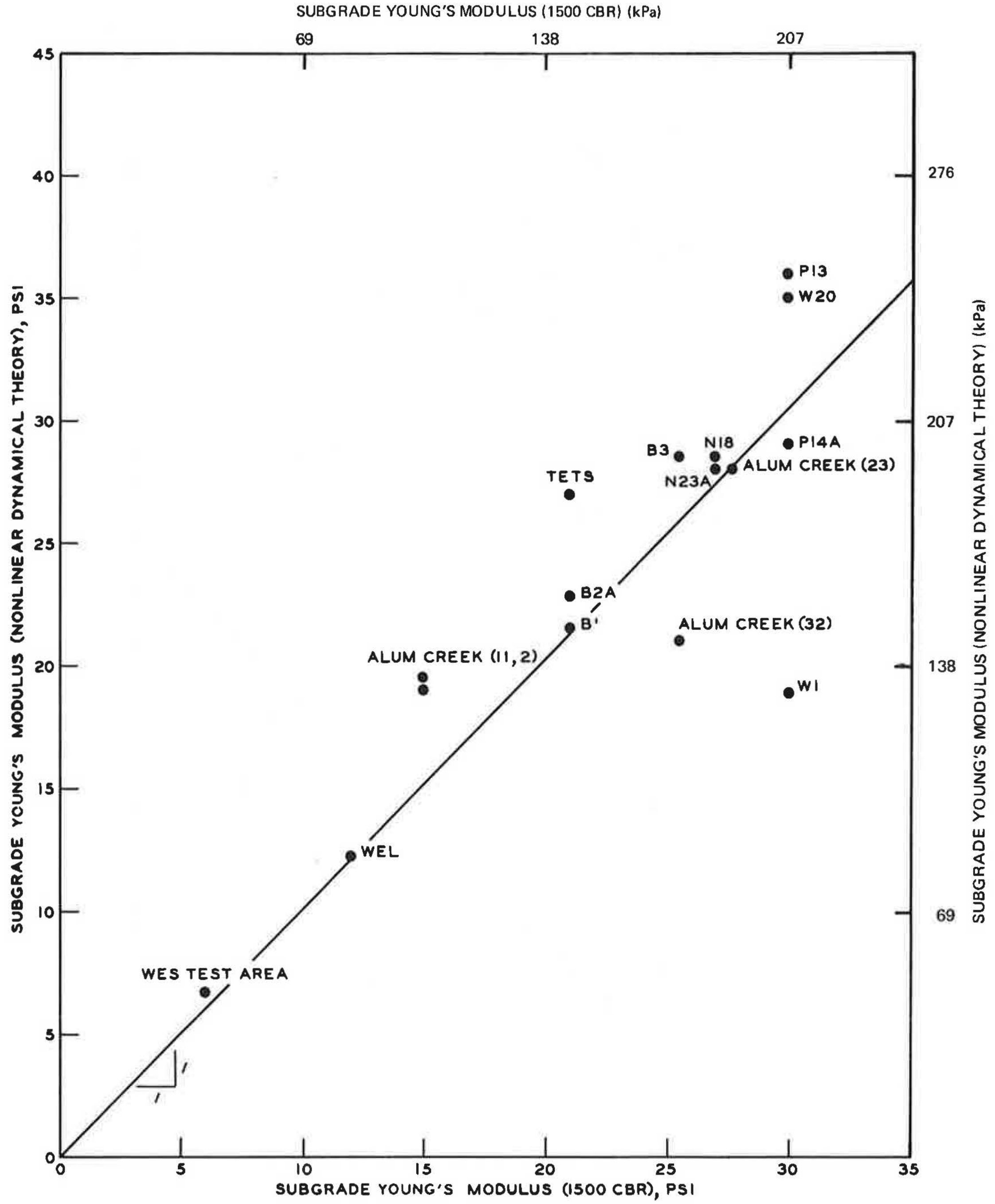


Figure 8. Comparison of values of Young's moduli of subgrades: computer program SUBE versus Shell formula—Minneapolis-St. Paul International Airport.

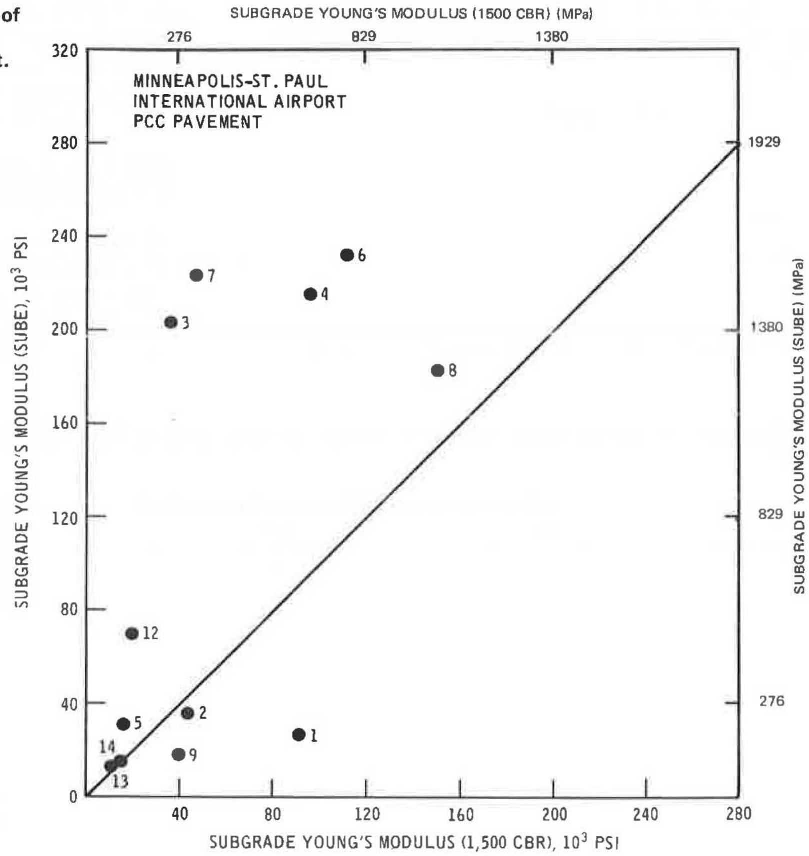


Figure 9. Comparison of values of Young's moduli of subgrades: computer program SUBE versus Shell formula—Knox County Airport, Rockland, Maine.

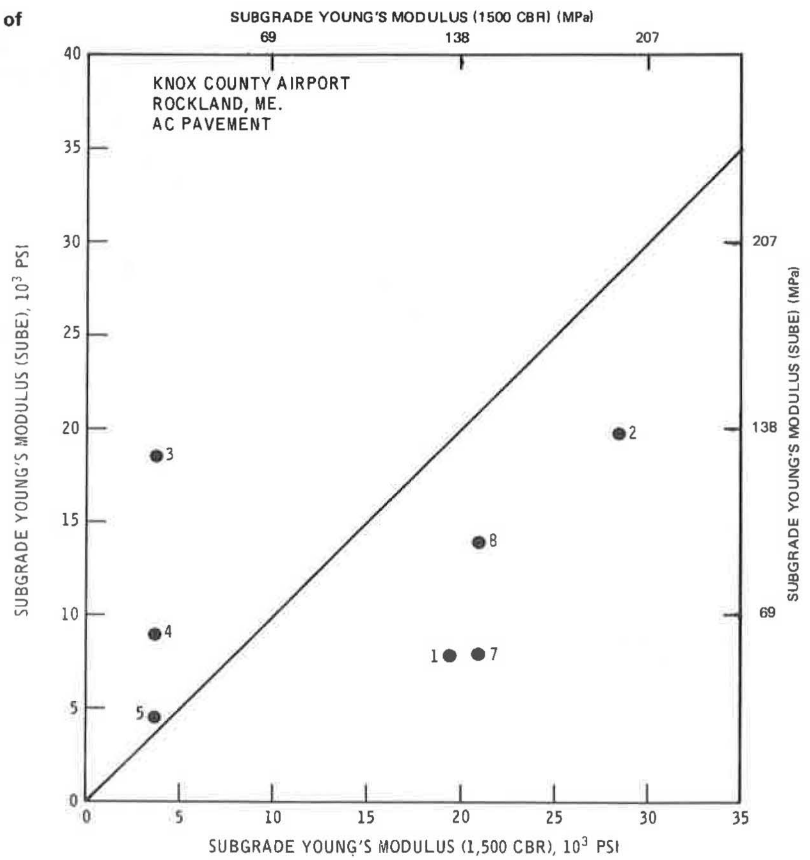
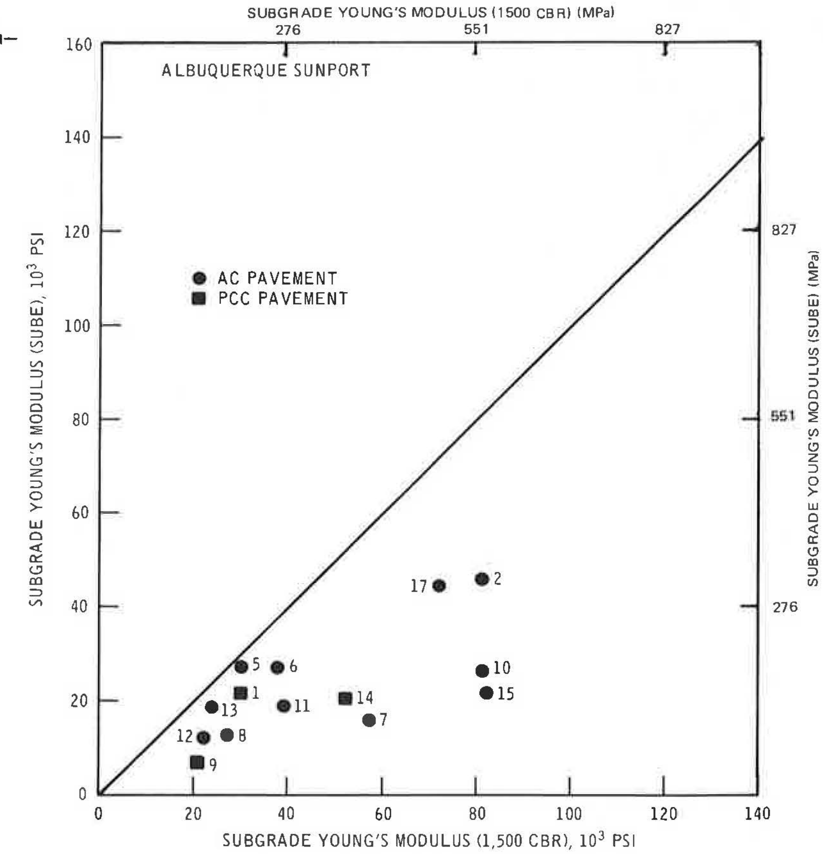


Figure 10. Comparison of values of Young's moduli of subgrades: computer program SUBE versus Shell formula—Albuquerque, New Mexico, Sunport.



ment temperatures of the AC wearing surface were measured at the time the vibratory nondestructive tests were conducted. From these data, the elastic constants of the pavement layers could be estimated.

A simple relationship between the Young's modulus of the subgrade and the CBR has been derived by wave propagation techniques; this is given by the Shell formula—E_s = 1500 CBR, where E_s = Young's modulus of subgrade (8). The nonlinear dynamic theory of pavement response and the associated computer program SUBE were developed to predict values of the Young's modulus of the subgrade that are in reasonable agreement with the predictions of the Shell formula (4).

Figures 7-10 show comparisons of the values of the Young's moduli of the subgrades predicted by using the nonlinear dynamic-response theory and the computer program SUBE and the values predicted by using the Shell formula. Figures 11 and 12 show comparisons between the values of the Young's moduli of the subgrades determined from the laboratory resilient-modulus tests and predicted by the SUBE and 1500-CBR methods, respectively.

LOAD-CARRYING CAPACITY AND REQUIRED OVERLAY THICKNESS OF PAVEMENTS

PAVEVAL Computer Program

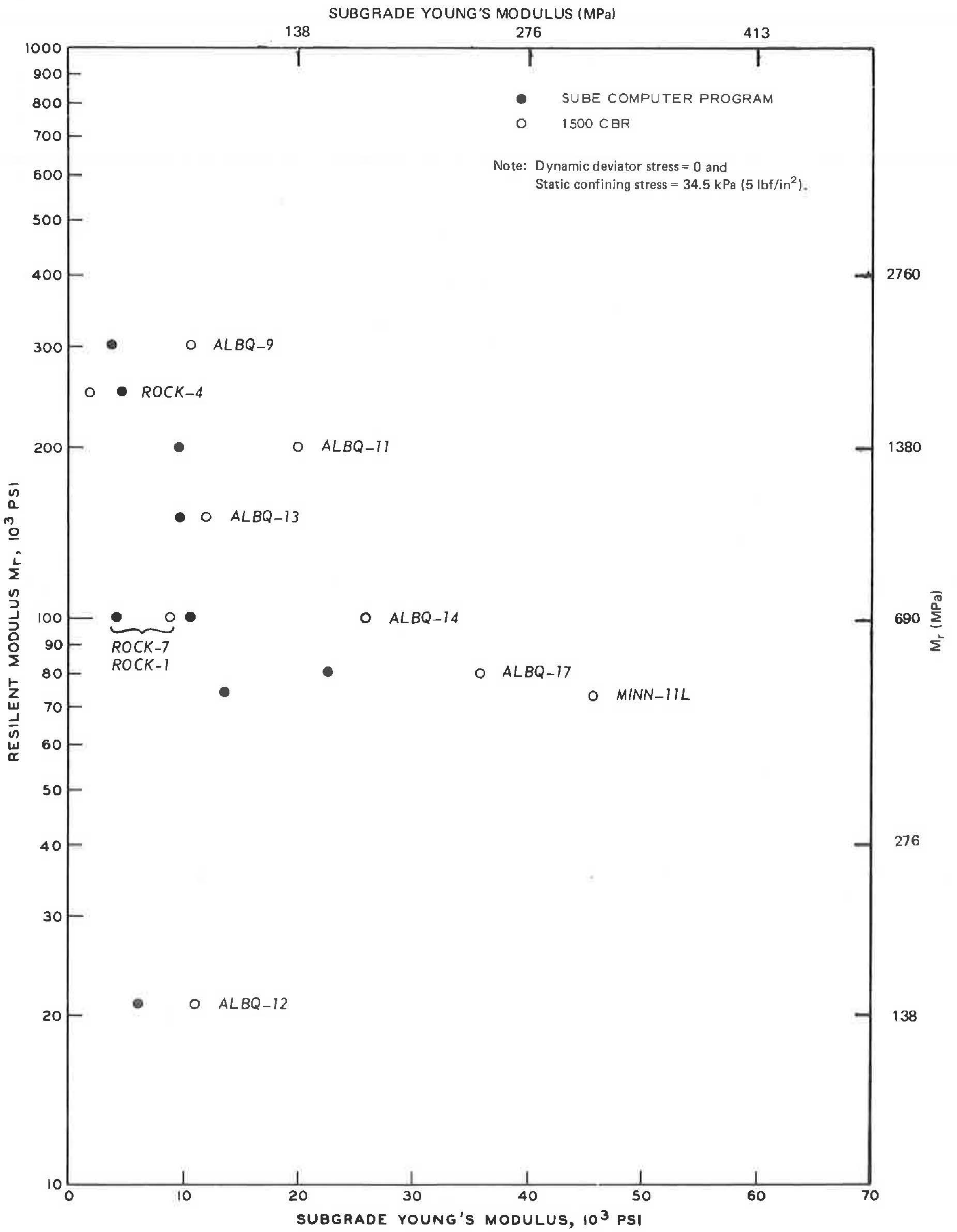
In the context of layered-elastic theory, a pavement is represented as a stack of elastic layers, the subgrade being of infinite extent. This layered-elastic model of a pavement structure can be used to calculate the elastic stress and strain at any point in the pavement or subgrade. Each pavement layer is characterized by a Poisson's ratio (ν), a Young's modulus (E), and a layer thickness (h). The Shell BISAR computer program, which is based on layered-elastic theory, relates the stress and strain in each pavement layer to the static load applied to the surface of a pavement.

The condition of failure in an AC pavement can be described by a limiting elastic (resilient) vertical strain in the top of the subgrade and a limiting tensile strain at the bottom of the AC pavement layer, and the condition of failure in a rigid pavement can be described by a limiting tensile stress at the bottom of the PCC layer (12, 13). For a given load at the pavement surface, the values of the stress and strain in the pavement and subgrade depend on the Young's moduli and the Poisson's ratios of the subgrade and each pavement layer. Therefore, if the elastic moduli of the pavement layers are known, it is the Young's modulus of the subgrade that is the unknown parameter that determines the stress and strain in the pavement and subgrade; thus, this is the parameter that must be determined by vibratory nondestructive testing and the computer program SUBE. The procedure is outlined in Figure 13.

The basic BISAR computer program was modified to include a procedure for the iteration of the surface load and the overlay thickness until, for AC pavements, the vertical strain at the top of the subgrade equals the limiting value of the vertical strain or the tensile strain at the bottom of the AC layer equals the limiting value of the tensile strain or, for PCC pavements, the tensile stress at the bottom of the PCC layer equals the limiting value of tensile stress. The resulting computer program is called PAVEVAL and was used to calculate the load-carrying capacity and required overlay thickness of a pavement (14). The aircraft characteristics required for the PAVEVAL computer program include tire contact area, load on one wheel, wheel spacings, and total number of main-gear wheels.

The values of the elastic moduli of the pavement layers that are entered into the PAVEVAL computer program were the same as those used in the computer program SUBE with the exception that the Young's modulus of AC pavements and base materials was chosen to have the value E = 31 000 MPa (450 000 lbf/in²) in the PAVEVAL program for the numerical calculations described in this paper. This value of Young's modulus was obtained from Figure 3 and corresponds to an assumed average annual pavement temperature of 21°C, a value of temperature that was chosen so that the results obtained by using the PAVEVAL program could be compared with the results obtained by using the DSM evaluation procedure. However, the PAVEVAL computer program has a greater capability for pavement-evaluation purposes because it can be used to study the seasonal variation of the pavement load-carrying capacity by using Figure 3 to select the proper seasonal value of

Figure 11. Comparison of values of Young's moduli of subgrades: resilient-modulus measurements versus computer program SUBE—various airport sites.



the Young's modulus of the AC pavement layers. For this purpose, the seasonal variation of the Young's moduli of the base, subbase, and subgrade must also be considered, such as during frost-thaw conditions. The seasonal variation of these Young's moduli values may be determined either by conducting vibratory nondestructive tests during the different seasons or by extrapolating laboratory-measured Young's moduli according to seasonal temperature and moisture changes.

Limiting Stress and Strain Conditions

The load-carrying capacity of an AC pavement and the overlay thickness required for its upgrading are related to the limiting tensile strain at the bottom of the AC layer and the limiting vertical strain at the top of

Figure 12. Comparison of values of Young's moduli of subgrades: resilient-modulus measurements versus Shell formula—various airport sites.

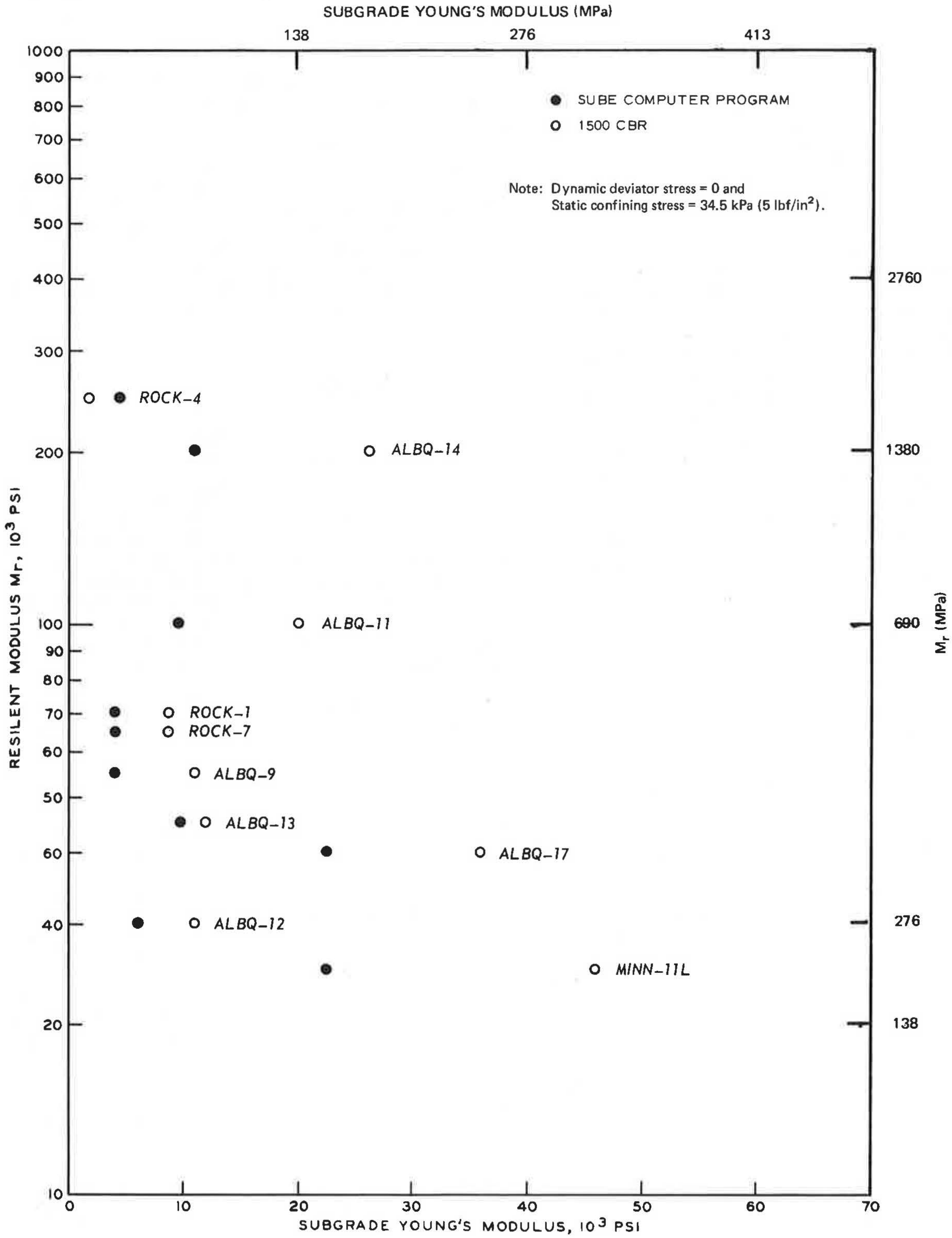
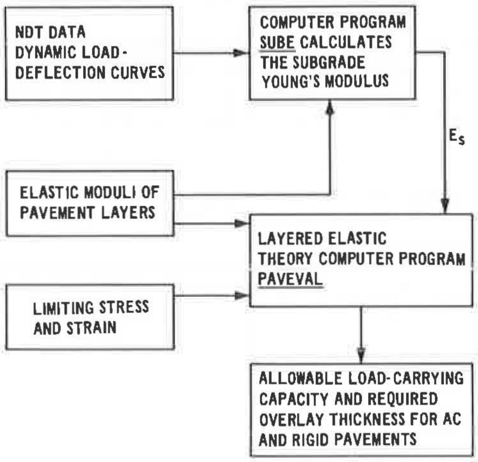


Figure 13. Pavement evaluation and overlay design by method that combines layered-elastic theory and vibratory nondestructive testing.



the subgrade; those of a rigid pavement are related to the limiting tensile stress at the bottom of the PCC layer (11, 12). The limiting value of the vertical strain at the top of the subgrade depends on the number of strain repetitions and on the Young's modulus of the soil in the subgrade (13). The curves in Figure 4 are assumed to be valid for all types of subgrade soils and for single- and multiple-wheel loadings. The limiting value of the tensile strain at the bottom of the AC layer is given by Barker and Brabston (13).

When a load is applied to the surface of a rigid pavement, the maximum tensile stress in the PCC layer occurs at the bottom of this layer and cracking is expected to first occur at this location. The limiting tensile stress is expressed in terms of the number of load (stress) repetitions and of the flexural strength of the PCC layer as follows (15):

$$\sigma_{RL} = R/[A + B \log(COV)] \tag{26}$$

where

R = flexural strength (lbf/in^2),
COV = number of coverages,
A = 0.58901,
B = 0.35486, and
σ_{RL} = limiting value of tensile stress (lbf/in^2).

This expression is assumed to be valid whether the stress in the PCC layer is produced by single-wheel or by multiple-wheel loading. The lateral distribution of traffic was handled by using the pass-to-coverage ratios for individual aircraft given by Brown and Thompson (16). For the four types of gear configurations treated in this paper, the pass-to-coverage ratios for PCC pavement are (a) single wheel = 5.18, (b) Boeing 727 (dual wheel) = 3.48, (c) DC-8-63F (dual tandem wheels) = 3.14, and (d) DC-10-10 (dual tandem wheels) = 3.64. Mixed traffic was not considered in this study, but it can easily be incorporated into the PAVEVAL computer program provided that the frequency distribution of the operating aircraft is known.

Single- and Multiple-Wheel Loadings

To determine the load-carrying capacity and the required overlay thickness for a single wheel loading on a pavement surface, the stress and strain due to the single load is compared with the limiting stress and strain values in the pavement and subgrade. The load on one wheel is entered into the computer program PAVEVAL. The maximum values of the stress and strain in the pavement and subgrade occur directly beneath the single-wheel load. The allowable load and the overlay thickness required are determined by requiring that the stress and strain in the pavement directly under one wheel be equal to the limiting stress and strain values.

Actual aircraft loadings on a pavement occur through two or more wheels in close proximity. Dual-wheel (two-wheel) and dual-wheel, tandem-wheel (four-wheel) configurations are commonly used. For the case of multiple wheels, the total strain or stress in the pavement beneath one wheel is affected by the presence of the other wheels. The maximum values of the stress and strain at some depth in the pavement occur at some point between the wheels of the gear configuration but are, to a good approximation, equal to the values of the stress and strain in the pavement beneath one of the wheels of a multiple-wheel configuration. The multiple-wheel calculations are done within this approximation. The PAVEVAL computer program (and the BISAR program on which it is based) calculates the stress and strain at any point in the pavement or subgrade due to a multiple-wheel loading and then compares them with their corresponding limiting values.

Numerical Values of Load-Carrying Capacity and Required Overlay Thickness

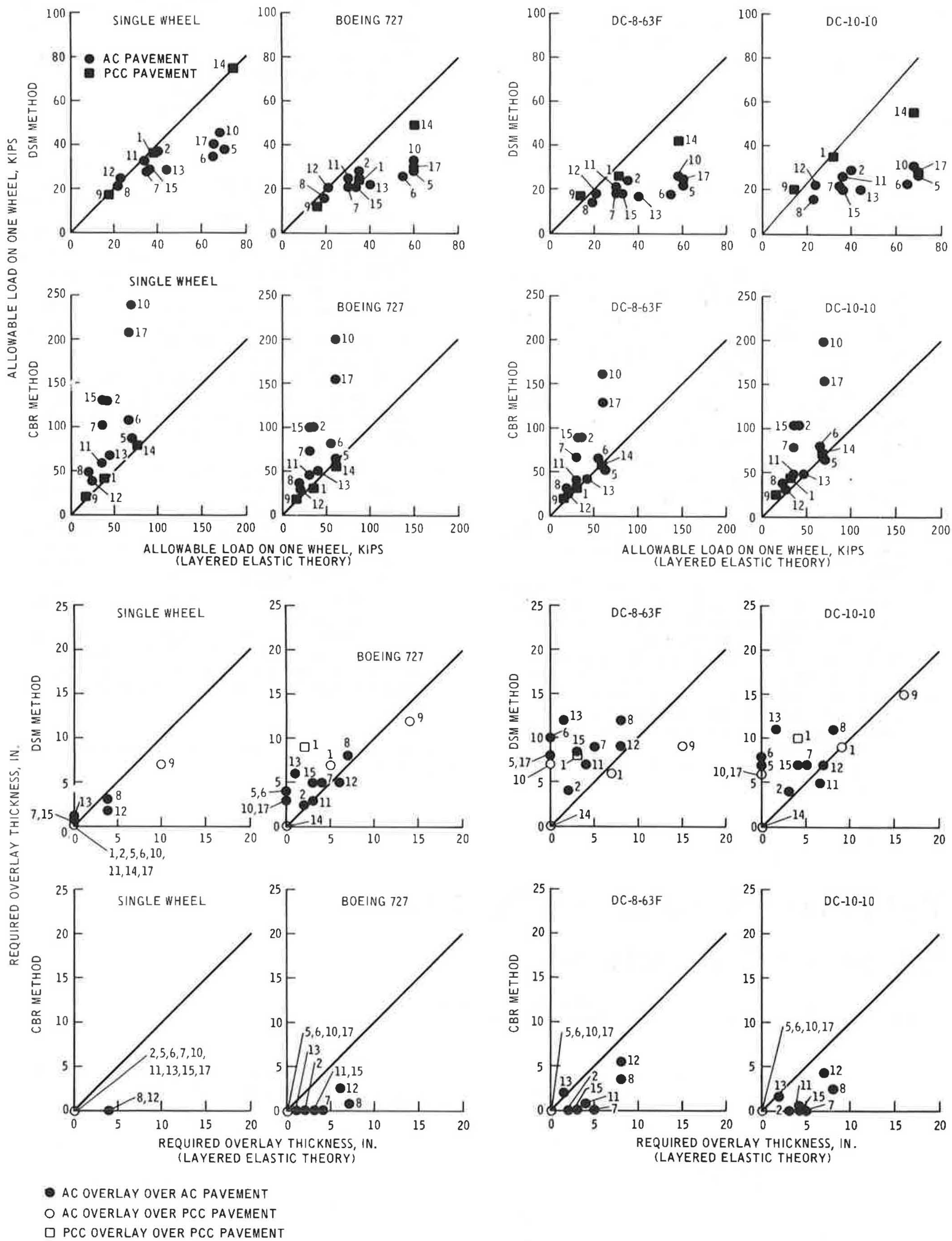
To validate the procedures outlined in this paper, a number of rigid and AC pavement structures were evaluated for single- and multiple-wheel loadings, and the load-carrying capacity and required overlay thickness were calculated. Then, the load-carrying capacity and required overlay thickness were also calculated by the conventional CBR and DSM methods for AC pavements and by the Westergaard and DSM methods for rigid pavements. For the calculation of required overlay thickness, the load on one wheel was taken to be (a) single wheel = 158.53 kN (35 625 lbf), (b) Boeing-727 = 182.85 kN (41 090 lbf), (c) DC-8-63F = 189.17 kN (42 510 lbf), and (d) DC-10-10 = 288.82 kN (51 420 lbf). The results are shown in Figure 14. (The coefficients in Equation 26 were derived for U.S. customary units; therefore, values in Figure 14 are not given in SI units.)

SUMMARY AND CONCLUSIONS

The ability to determine the load-carrying capacity of a pavement and the overlay thickness required to upgrade it is of much importance to pavement engineers. A simple method of pavement evaluation that combines vibratory nondestructive field tests and a theoretical layered-elastic formalism was developed in an attempt to satisfy the needs of the pavement engineer. The layered-elastic-theory approach to the calculation of the required overlay thickness and load-carrying capacity of a pavement requires the value of the Young's modulus of the subgrade, and this value is determined by vibratory nondestructive testing.

For the airfield sites considered, there was only fair agreement between the values of the Young's modulus of the subgrade predicted by the computer program SUBE and those predicted by the E = 1500 CBR method or those determined from laboratory resilient-modulus tests. The exceptionally high values of the Young's modulus predicted by the SUBE program for the Minneapolis-St. Paul test area may be due to the pres-

Figure 14. Comparisons of values of load-carrying capacity and required overlay thickness predicted by computer program PAVEVAL and by CBR and DSM methods: Albuquerque Airport.



ence of bedrock near the surface of the subgrade.

The values of the load-carrying capacity and required overlay thickness obtained by using the PAVEVAL computer program for AC pavements fall in between the values predicted by the DSM and CBR methods. Both the DSM method and the layered-elastic theory method (PAVEVAL) predict load-bearing capacities for AC pavements that are somewhat lower than the values predicted by the CBR method. There is reasonable agreement among the three pavement evaluation methods for PCC pavements. Further study on more airfield pavement sites will be required before more definite comparisons among these three methods of pavement evaluation can be made.

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Pavement Evaluation by Using Dynamic Deflections

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Dynamic test deflections were duplicated by elastic theory by using the Chevron N-layered computer program. Dynamic surface deflections obtained by using the road rater were used in conjunction with elastic theory to analyze pavement behavior. A procedure was developed to use field-measured road rater deflections for the estimation of the elastic modulus of the foundation material and the determination of the equivalent thicknesses of new material that approximate the behavior of the structure. The estimated moduli and the equivalent thicknesses can be used as inputs to design overlay thicknesses. An analysis of the deflections of the first three sensors of the road rater also makes it possible to distinguish weaknesses in asphalt concrete layers from weaknesses in the supporting foundation.

The stiffness of the foundation (subgrade) is one of the factors that affect the behavior of a pavement structure. Variations in subgrade support occur mainly as a result of variations in moisture content or of soil type. A significant decrease in subgrade stiffness (modulus of elasticity) will result in a decrease in ability to support the pavement structure and lead to increased distress in the layers of the structure. Signs of distress are rutting, increased roughness, and cracking (1).

Nondestructive tests have been empirically correlated with field-strength tests. There has been considerable use of elastic theory and dynamic testing for the estimation of layer moduli (2-7). The equipment used includes the Dynaflect, the California traveling deflectometer, the Benkelman beam, and the road rater. Since 1971, road rater deflections have been under study in Kentucky, as indicators of the characteristics of individual layer components of the pavement structure.

An estimate of subgrade strength is necessary for the evaluation of the overall condition of a pavement. A design condition exists when there has been no loss of effective thickness in any of the layers. A knowledge of the as-built thicknesses of the layers is necessary before an evaluation of the pavement structure can be made. Those thicknesses should be available from construction or maintenance records. Generally, the pavement condition involves deterioration in the layers of the structure. This means that the individual layers are behaving in a way similar to a different combination of layer thicknesses of new-quality materials; i.e., the structure is behaving as an effective structure. In this case, it becomes necessary to estimate the layer thicknesses of the deteriorated or effective structure.

The analysis of deflections involves the shape of the deflection bowl (2, 6). When the logarithms of road rater deflections are plotted against the distance from the load, a secant line can be drawn through two points on the deflection bowl. The combination of the slope of this line and the magnitudes of the deflections is indicative of the types of problems in the pavement structure.

SIMULATION OF ROAD RATER BY ELASTIC THEORY

Characteristics of Road Rater

The testing head on the Kentucky road rater consists of a vibrating mass that weighs 72.6 kg (160 lb) that impulses the pavement; the forced motion of the pavement is measured by velocity sensors normally located at 0, 305, 610, and 914 mm (0, 1, 2, and 3 ft) from the center of the test head. Frequency of the vibrator can be chosen from preselected frequencies of 10, 20, 25, 30, and 40 Hz. The vibrating mass is lowered to the pavement by a hydraulic system. At a hydraulic pressure of 4.82 MPa (700 lbf/in^2), the static load is 7.43 kN (1670 lbf).

The response to the vibrating mass of the road rater was determined for several full-depth asphalt concrete (AC) pavements and conventional three-layer pavements. Resonant frequencies of the total pavement structure were usually multiples of approximately 7 Hz. The thickness of the AC layer appeared to cause the resonant frequency to shift 1 or 2 Hz at the 21 and/or 28 Hz normal resonant frequencies. Resonance at these frequencies was indicated by oscillations of the needle of the meter as opposed to its normally rock-steady behavior. In all cases, the meter response remained steady at 25 Hz, which thus was chosen as the reference frequency.

At a frequency of 25 Hz and an amplitude of vibration of 1.52 mm (0.06 in), the road rater has a peak-to-peak dynamic force of 2.67 kN (600 lbf). Once the dynamic force is set for a given frequency and amplitude, the other preset frequencies will vary the amplitude of the vibrating mass such that the dynamic force remains constant for all of the preselected frequencies. The composite loading thus consists of a static load of 7.43 kN and a peak-to-peak dynamic force of 2.67 kN that oscillates about the static load.

Superposition Principles

The road rater loading is transmitted to the pavement by means of two feet symmetrically located on either side of a beam that extends ahead and carries the sensors. Superposition principles can be applied to the computation of the deflection at each sensor location. A combined load can be subdivided into its component loads. Superpositioning is applicable provided the deformations are small and do not substantially affect the action of external forces. If the principles of superpositioning are to apply, a linear relationship between displacement and external force must exist or be assumed to exist (8-10). When superposition principles are applied to the road rater, the deflection that results from the load applied to one foot must be added to the deflection that is due to the load applied by the other foot. For the symmetrical conditions of the road rater, deflection calculations need be made only for one foot and the radii corresponding to each sensor location.

The dynamic loading (sine wave) of the road rater can be approximated by a square wave such that the maximum value of the square wave is equal to $1/\sqrt{2}$ times the peak value of the sine wave. The peak-to-peak loadings of the road rater are 8.37 and 6.49 kN (1882 and 1458 lbf). From symmetry, the loads on each foot of the test head are equal to 4.19 and 3.24 kN (941 and 729 lbf). The dynamic deflection is defined by $\Delta_{total} = (\Delta_{4.19} - \Delta_{3.24}) \times 2$ where $\Delta_{4.19}$ and $\Delta_{3.24}$ represent the deflections calculated by using the Chevron computer program and the peak loading conditions.

Input Parameters for Simulation by Using Chevron Computer Program

In addition to the load, the inputs required by the Chevron computer program include a contact pressure corresponding to the load; the number of layers; and the thickness, Young's modulus, and Poisson's ratio of each layer. The contact pressure of the low and high loads are input to maintain the correct area for each foot. The constants used in simulating the road rater (11) are summarized in Table 1.

Reference Conditions

The modulus of elasticity of AC varies as a function of frequency of loading and temperature. Conditions for the current Kentucky thickness-design procedures and the method for conducting Benkelman beam tests correspond to a modulus of 3.31 GPa (480 000 lbf/in^2) at 0.5 Hz and a pavement temperature of 21°C (70°F). A reference frequency of 25 Hz was selected for the road rater; the corresponding AC modulus at 21°C is 8.27 GPa (1 200 000 lbf/in^2).

The modulus of a granular base (E_2) is a function of the moduli of the confining layers, i.e., the modulus of the AC layer (E_1) and the modulus of the subgrade (E_3). Estimation of the modulus of the crushed-stone layer (E_2) can be determined from the relationship $E_2 = F \times E_3$, where there is an inverse linear relationship between log F and log E_3. The ratio of E_2 to E_3 is equal to 2.8 at a California bearing ratio (CBR) of 7 and to 1 when E_1 equals E_3: $E_1 = E_2 = E_3$ (11)—which is the case of a Boussinesq semi-infinite half space. [E_3s (in lbf/in^2) can be approximated by the product of the CBR and 1500 (11-13), a method of estimating base moduli that appears adequate for normal design considerations up to a CBR of 18-20 (11-14).]

For a constant structure [depth of AC and depth of dense-graded aggregate (DGA)] and AC modulus, a theoretical relationship between deflection and subgrade modulus of elasticity can be developed from the simulated road rater deflections. An example of such a relationship is illustrated in Figure 1. There is a separate line for each sensor on the road rater. Figure 1 also con-

Table 1. Input parameters for simulation of road rater.

Input	Value
Poisson's ratio	
Asphalt concrete	0.40
Granular base	0.40
Subgrade	0.45
Contact pressure (MPa)	
Low (3.24-kN) load	0.183
High (4.19-kN) load	0.231
Layer thicknesses (mm)	
Asphalt concrete	50.8, 127, 203, 279, and 356
Dense-graded aggregate	50.8, 203, 356, 508, and 686
Full-depth asphalt concrete	102, 152, 203, 254, 305, 356, 406, 457, and 508
E (GPa)	
Asphalt concrete	1.38, 2.76, 4.14, 5.52, 6.90, 8.28, 9.66, 11.04, 12.42, and 13.80
Subgrade	0.041, 0.082, 0.123, 0.164, 0.205, 0.246, 0.287, 0.328, 0.369, and 0.41

Note: 1 MPa = 145 lbf/in^2; 1 kN = 225 lbf; 1 mm = 0.04 in.

tains a fourth line labeled no. 1 projection. This line was calculated by using the no. 2 and no. 3 deflections and will be discussed below.

ADJUSTMENTS FOR NONREFERENCE CONDITIONS

Moduli of Asphalt Concrete from Field-Test Data

Field measurements made include road rater deflections, surface temperature, time of day, and frequency of vibration. The surface temperature, time of day, and mean air-temperature history for the previous five days are necessary to determine the temperature distribution by using the method developed by Southgate and Deen (15, 16). The five-day mean air-temperature history can be obtained from weather records.

The modulus of elasticity of AC is a function of frequency of loading and mean pavement temperature, as illustrated in Figure 2 (17). Figure 2 can be used to develop a relationship between modulus and temperature for the reference frequency of 25 Hz or any other frequency desired, which may be representative of other dynamic loads. Thus, a distribution of the modulus through the AC layer for the reference frequency of 25 Hz can be determined for any temperature distribution. For layers thinner than 152 mm (6 in), the results were better when the pavement modulus was taken as the average of the moduli on 12.7-mm (0.5-in) intervals beginning at the 25.4-mm (1-in) level. For asphalt thicknesses greater than 152 mm, the most representative modulus appeared to be the mean of the moduli on 25.4-mm intervals beginning at the 25.4-mm level.

Adjustment Factors for Road Rater Deflections

Because of the significant effect of temperature on the modulus of elasticity of AC, it was necessary to develop a system with which to adjust the deflection measurements to a reference temperature and modulus. This adjustment-factor system uses ratios of deflections at reference conditions to deflections that result from arrayed variables of layer thicknesses and moduli.

For a given thickness of AC, the adjustment factors vary according to changes in the thickness of DGA and the value of E_3 but these variations are minimal when compared with the variation in adjustment factor for variations in AC thicknesses. Thus, the adjustment factors for all DGA thicknesses for a constant subgrade modulus and thickness of AC were averaged into a single line. Treating other thicknesses in the same manner produces similar relationships. Investigation of other subgrade moduli indicated only minor variation in adjustment-factor values for the same thickness of AC. The adjustment-factor curves shown in Figure 3a were produced by averaging the adjustment factors for each thickness of AC and across subgrade moduli.

Two-layered pavements show similar variations in adjustment factor relative to E_3s and AC thicknesses. The adjustment-factor curves shown in Figure 3b were produced by averaging the adjustment factors for all E_3s and a constant thickness of AC.

A mean pavement modulus can be found by using the distribution of AC moduli through the pavement. The necessary adjustment factor (a multiplier) required to bring the field deflection to a deflection at a reference modulus is determined by using the appropriate adjustment-factor chart (see Figure 3) and the mean pavement modulus of elasticity.

An alternative method of presenting the adjustment factors shown in Figure 3 is shown in Figure 4. The system shown in Figure 4 adjusts the deflections to a specific condition—25 Hz, a mean pavement temperature of 21°C, and E_1 = 8.27 GPa. The same method of calculating ratios of deflections was used to develop Figure 4 as was used to develop Figure 3. The only difference is that Figure 3 was developed on a basis of mean pavement modulus and Figure 4 was developed on a basis of mean pavement temperature. A reduction in frequency while holding pavement modulus constant results in a reduced pavement temperature. Thus, if the frequency is reduced, the adjustment-factor curves will not shift but the mean pavement-temperature scale will shift according to the chosen frequency. Also, mean pavement temperature is a function of AC thickness. The effects of AC thickness and subgrade modulus were averaged in the development of Figure 4. Figure 3 adjusts the road rater deflections to a reference modulus of E_1 = 8.27 GPa regardless of the the frequency of loading. Figure 4 adjusts the road rater deflections to a reference temperature and frequency and the corresponding AC modulus (25 Hz, 21°C, and E_1 = 8.27 GPa).

The adjustment-factor system presented in Figures 3 and 4 was developed by using theoretical deflection data corresponding to the no. 1 sensor of the road rater. A similar system could have also been developed by using deflection data corresponding to either the no. 2 or no. 3 sensors. For comparison, adjustment factors corresponding to the no. 2 and no. 3 sensors were developed for the same conditions and by using the same methodology. A comparison of the three different adjustment factors indicated an average difference of ±0.032 for the adjustment factors corresponding to the no. 1 and no. 2 sensors and an average difference of ±0.048 for the no. 1 and no. 3 sensors for a range of AC moduli of 1.38 to 13.79 GPa (200 000 to 2 000 000 lbf/in^2). The greatest differences in adjustment factors occurred at lower values of moduli and thin layers of AC. For example, a comparison of the differences in adjustment factors for moduli greater than 4.14 GPa (600 000 lbf/in^2) indicated differences of ±0.021 and ±0.037 for the no. 1 sensor versus the no. 2 and no. 3 sensors, respectively. Based on these analyses, the deflection adjustment-factor curves shown in Figures 3 and 4 were assumed to be adequate for use with the deflections of the no. 1, no. 2, and no. 3 sensors of the Kentucky road rater.

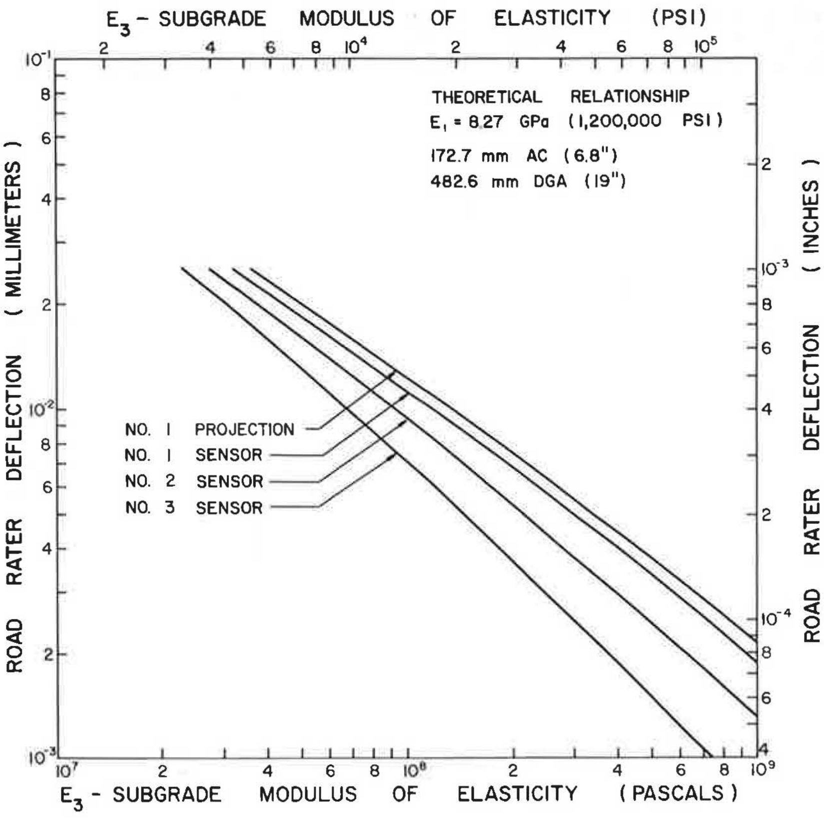


Figure 1. Theoretical relationships: road rater deflection versus modulus of elasticity for a constant structure and modulus of asphalt concrete.

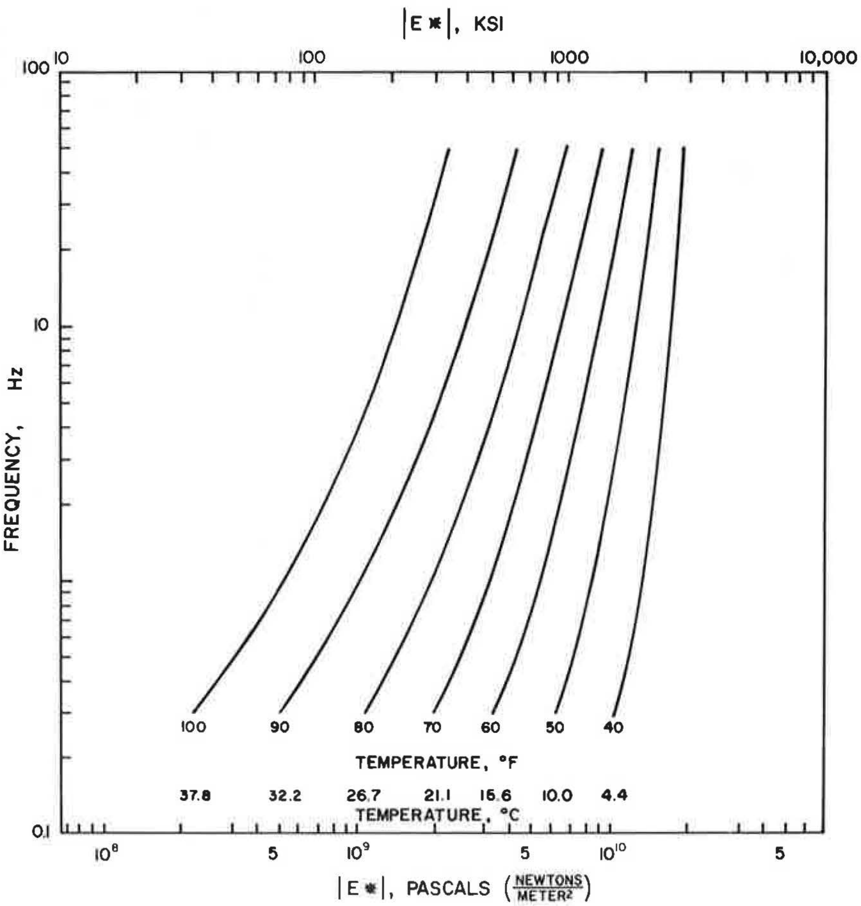


Figure 2. Effect of temperature and frequency on dynamic modulus of elasticity.

Figure 3. Relationship between thickness of asphalt concrete and temperature adjustment factor: (a) three-layered pavements and (b) two-layered pavements.

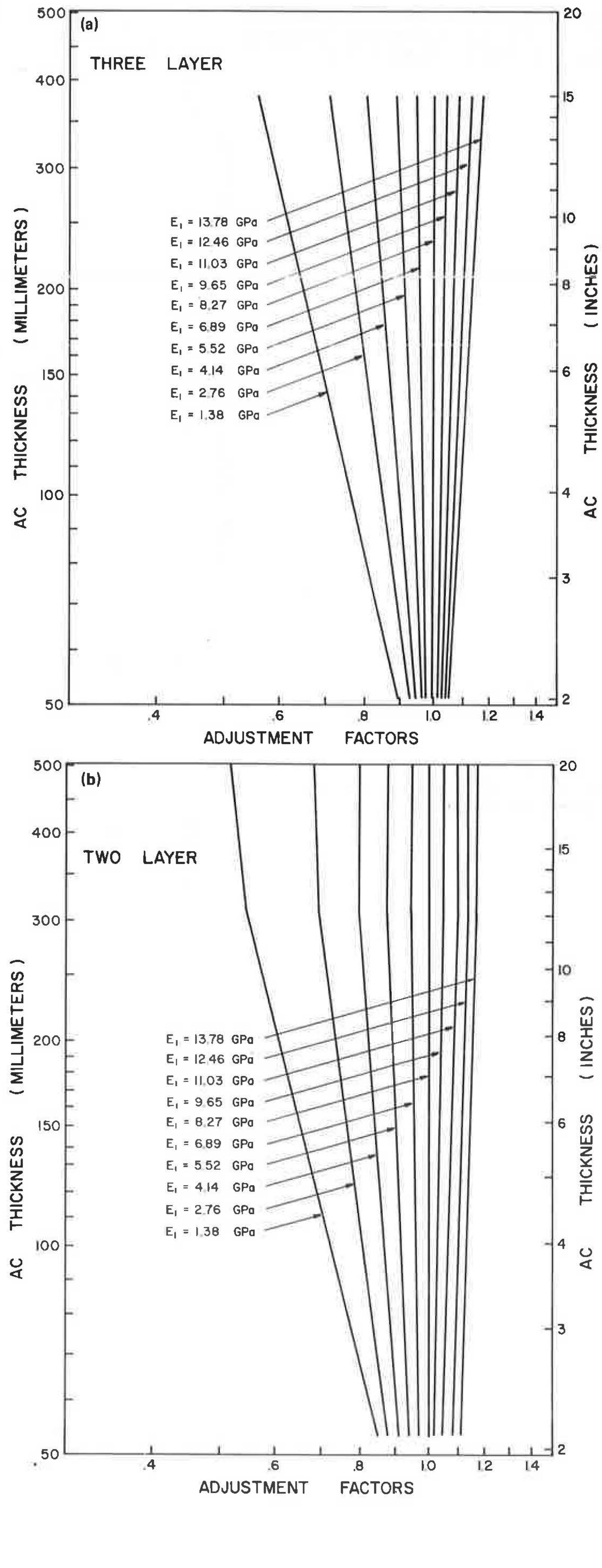
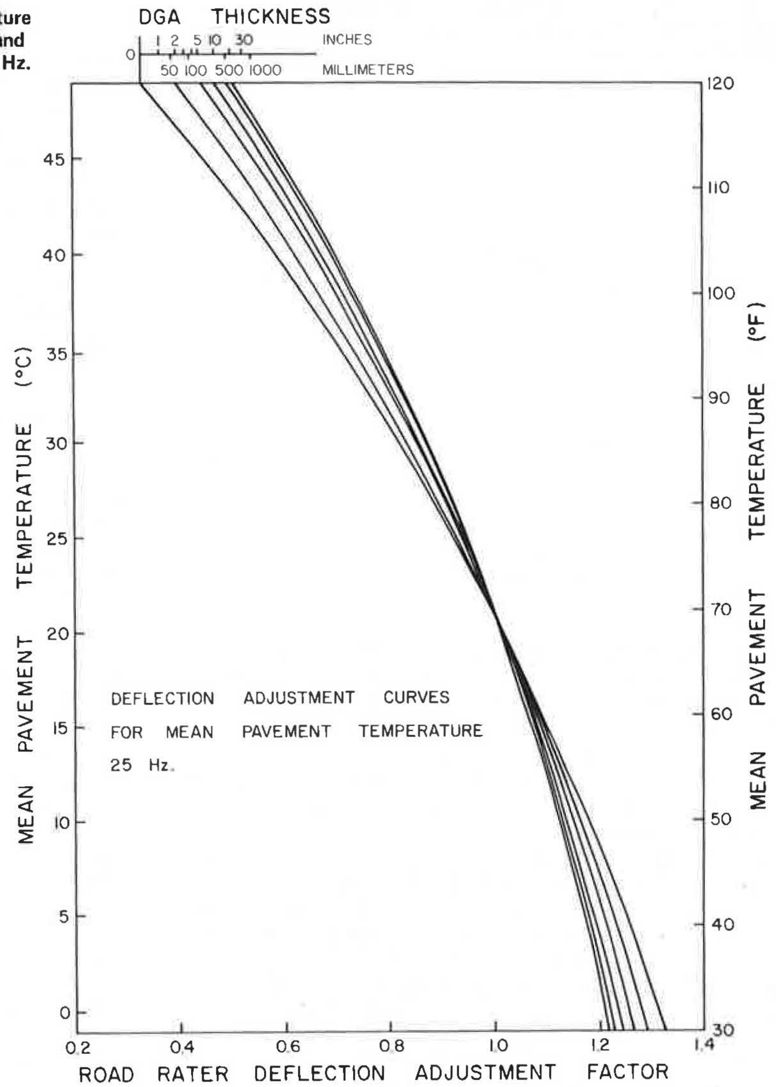


Figure 4. Relationship between mean pavement temperature and road-rater-deflection adjustment factors: full-depth and three-layered asphalt concrete pavements at 21°C and 25 Hz.



EVALUATION OF PAVEMENT STRUCTURE

Describing Shape of Deflection Bowl

An empirical evaluation of road-rater-deflection data involves extrapolating a straight line through the magnitudes of the deflections of the no. 2 and no. 3 sensors when the logarithm of the deflection is plotted against the arithmetic distance from the load head. Extrapolation of this line to the position corresponding the no. 1 sensor gives the no. 1 projection (Figure 5):

$$\text{No. 1 projection} = 10^{[(2\log \text{no. 2 deflection}) - \log \text{no. 3 deflection}]} \qquad (1)$$

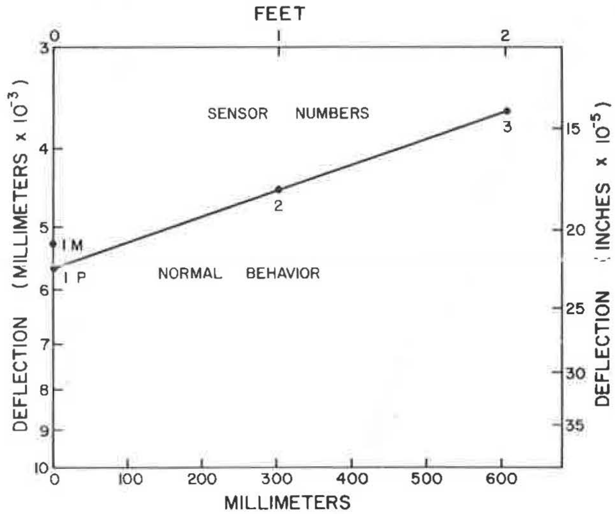
The slope of the semilog line (secant line), the difference in magnitude between the no. 1 projection and the no. 1 sensor deflection, and the magnitudes of all deflections are indicative of the shape of the deflection bowl. This concept can also be applied to theoretical deflections.

Estimating Subgrade Moduli

If the layer thicknesses are known, relationships can be developed (from elastic theory) between theoretical deflections and subgrade moduli. An example for one structure is shown in Figure 1. Field deflections for the no. 2 and no. 3 sensors and their corresponding no. 1 projections can be used as inputs in the subgrade-moduli estimation process to obtain three values for the subgrade modulus. The average modulus of the subgrade is calculated from the three estimates. The no. 1 sensor is closest to the point of application of the load and is most indicative of the condition of the pavement slab. For this reason, the deflection of the no. 1 sensor is not used in estimating the subgrade modulus. Sensors no. 2 and no. 3 are farther from the point of application of the load and are therefore more indicative of the condition of the foundation or supporting layers of the structure. The deflection of the no. 4 sensor is not used in the pavement evaluation process because there is little variability in its deflection with changes in structural conditions of the pavement.

Subgrade moduli corresponding to the no. 2 and no. 3 deflections and the no. 1 projections were estimated for four pavements (54 test sites). At the time of testing, each of these test pavements was less than two years old and showed no visible signs of deterioration. The average difference between subgrade moduli for any of the three predictors was 24.8 MPa (3600 lbf/in^2) with a standard deviation of 22.1 MPa (3200 lbf/in^2). When these three estimates of subgrade modulus were averaged and compared with the magnitude of the subgrade modulus estimated from the deflection of the no. 2 sensor,

Figure 5. Determination of no. 1 projection from relationship between deflection and distance from load head: example of normal pavement behavior.



the mean difference between the average subgrade modulus and the modulus estimated from the no. 2 sensor deflection was only 4.95 MPa (718 lbf/in^2) with a standard deviation of 7.58 MPa (1100 lbf/in^2). By using the data from these four pavements and in the interest of simplification of the system, the deflection of the no. 2 sensor was selected as the one to be used for the estimation of the subgrade modulus.

The variability in estimated subgrade modulus may be related to the operator's ability to read the correct deflection on the meters of the road rater, the selection of the most appropriate deflection adjustment factor, and the accuracy of the graphical interpolations in reading the subgrade modulus corresponding to a given deflection. Some error of interpolation for the correct structure could also be introduced during the development of the theoretical curves (Figure 1) from the matrix of conditions used in the road rater simulation.

A log-log plot of the sensor no. 1 deflections against the estimated subgrade moduli (from sensor no. 2) should be made for field deflections (see Figure 6). The sensor no. 1 measured deflection was selected because it showed the greatest sensitivity to the condition of the AC layer; the sensor no. 2 deflections were more indicative of the condition of the supporting foundation.

If the field deflections and the estimated subgrade moduli agree with the theoretical values for the original structure, the pavement is behaving as expected (Figure 6a). Over a length of pavement, it is normal to have a range in subgrade modulus because of variations in moisture content and soil type. If the pavement performance (deflections) does not agree with the original theoretical structure line, the pavement is behaving as a thinner effective structure (see Figure 6b).

The expression of deterioration in terms of reduced thickness is only one of the options available. Deterioration can also be expressed in terms of reduced layer moduli for constant layer thicknesses. Deterioration in terms of reduced thicknesses was selected because of its adaptability to overlay design. The effective structure, expressed in terms of reduced layer thicknesses that have properties similar to new pavement, can be used as an input parameter for overlay design.

Estimating Effective Structure

To evaluate effective structure, lines of equal deflection (no. 1 sensor) were drawn for a matrix of layer thicknesses and subgrade moduli for a constant reference modulus of AC (E_1 = 8.27 GPa) (see Figure 7). It was assumed that the effective structure is defined by the effective layer thicknesses and the modulus of the subgrade. In Figure 7, the subgrade modulus is held constant. One method (18) of estimating the amount of deterioration (percentage net worth) is shown in graphical form in Figure 8 in terms of percentage of residual or net worth versus percentage of design thickness. Figure 8 is a modification of a concept used in Florida. There, it was assumed that the AC had a residual value of 50 percent of its original value at a pavement serviceability index (PSI) of 1.5. Figure 8 is based on the assumption of 30 percent residual value at a PSI of 1.5. A relationship of percentage of original AC thickness versus percentage of the original DGA thickness was developed by using Figure 8 and is shown in Figure 9.

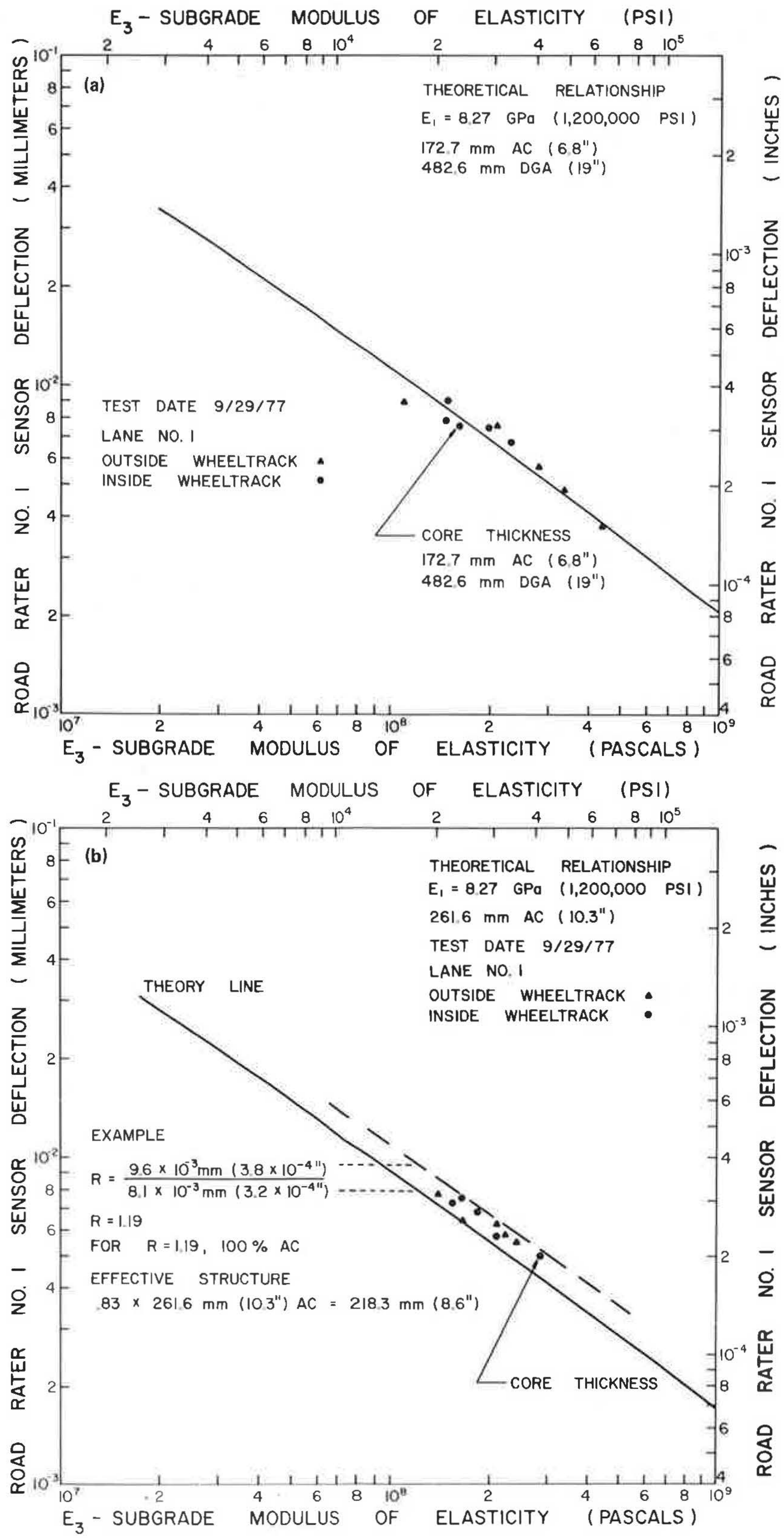
As the thicknesses of the individual layers decrease, the deflection along a deterioration curve (Figure 9) increases. Figure 10 illustrates the relationship between the ratio of the deflection of a deteriorated structure to the deflection of the original structure (an expression of the degree of deterioration) and the deteriorated structure in terms of percentage of the original thickness of each layer, where the modulus of the subgrade is constant. A sensitivity analysis was made of the ratio of deflections against the percentage of AC in the original design thickness as the subgrade modulus varied. The analysis showed that, for a normal range of subgrade moduli [42-206 MPa (6000 to 30 000 lbf/in^2) there was very little change.

Procedure for Evaluating Effective Structure

The effective structure is determined from plots of deflection versus subgrade modulus and of ratio of deflections versus percentage of original thicknesses by the following procedure.

1. For a given subgrade modulus, determine the theoretical deflection that corresponds to the original structure from the plot of the deflections of sensor no. 1 versus subgrade moduli (Figure 6b).
2. For the same subgrade modulus, determine the deflection that corresponds to a line of equal and parallel offset through the field deflection of greatest magnitude (Figure 6b).
3. Use the two deflections to compute the ratio of the field deflection (step 2) to the theoretical deflection (step 1).
4. Use the ratio (step 3) to determine (from Figure 10) the percentages of effective thicknesses of the asphalt and base layers.
5. Multiply these percentages by the original layer thicknesses to obtain the effective structure (Figure 6b).
6. Confirm the effective structure by using an iterative process of computing a new mean pavement temperature and modulus from the respective distributions, readjusting the field deflections for the new AC modulus (based on the thinner structure), and repeating the process of estimating the subgrade modulus. Figure 11 illustrates the confirmation of the example shown in Figure 6b and also compares the effective and original structures. The parallel line through the point of greatest offset from the theoretical deflection-subgrade-modulus line is a short-cut procedure that reduces the number of iterations required. Investigations (19) have shown that this procedure effectively reduces the iteration to one cycle.

Figure 6. Relationship between no. 1 sensor deflection and modulus of elasticity of subgrade: (a) normal pavement behavior and (b) abnormal pavement behavior and example of determination of effective structure.



Identification of Type of Deterioration

For a given pavement structure, AC modulus, and subgrade modulus, there is a difference between the no. 1 projection and the no. 1 sensor deflection for theoretical deflections (Figure 5). There will also be a difference between these values for field-measured deflections.

Normally, the differences between the no. 1 projected deflection and the no. 1 sensor deflection for both theory and field measurements are similar although the difference for field measurements should be greater than that for theoretical values. Slab deterioration is indicated when field measurements indicate a no. 1 sensor deflection greater than the no. 1 projection (see Figure

Figure 7. Example pavement-deterioration curves: contours of equal deflection of sensor no. 1 for matrix of asphalt concrete and dense-graded aggregate thicknesses.

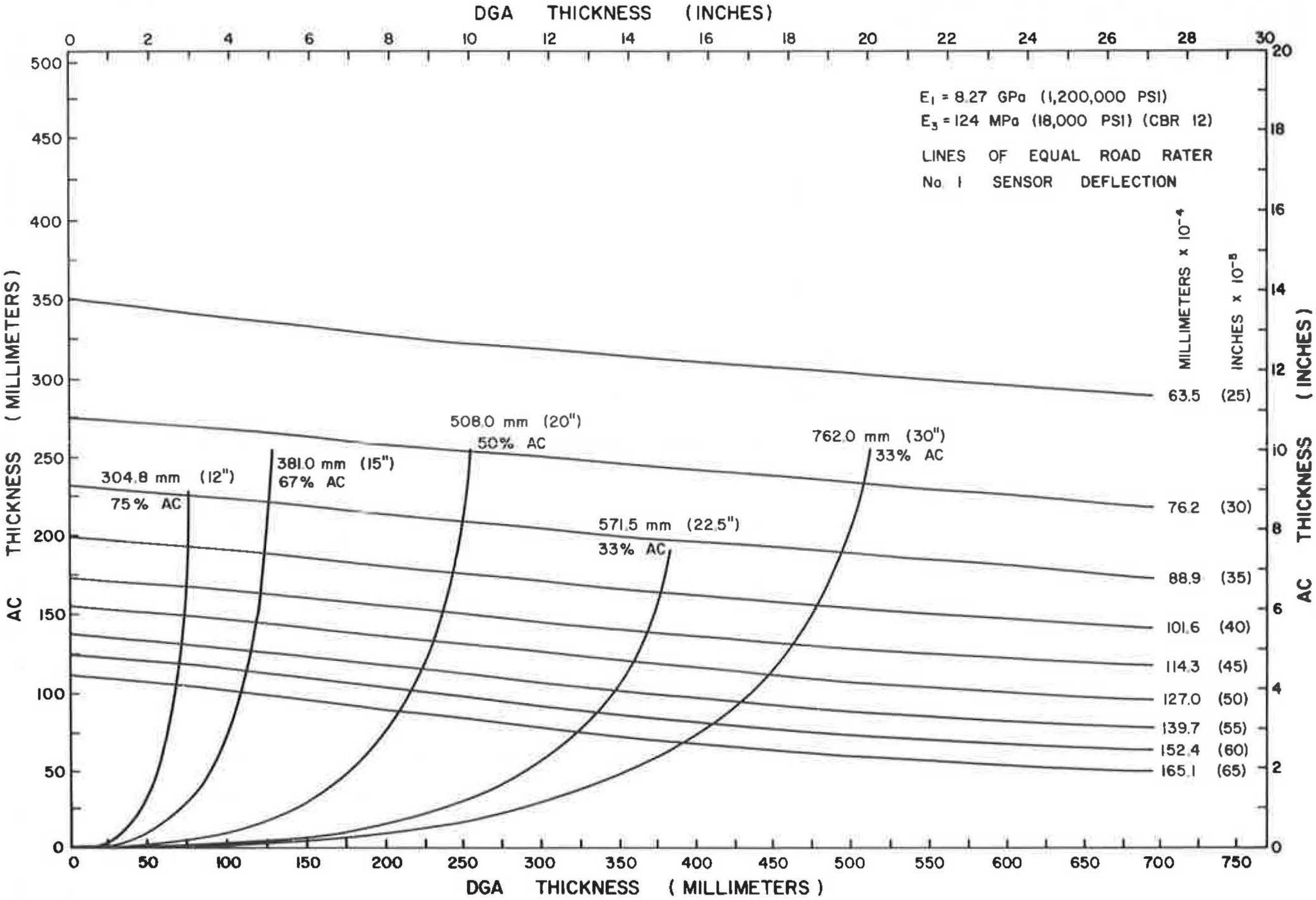
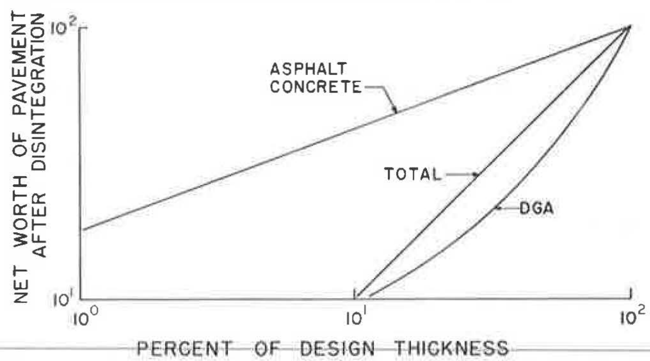


Figure 8. Relationship between percentage of net worth of pavement after beginning of disintegration and percentage of design thickness.



12a) and the difference between these values is greater than the difference for theoretical deflections. A foundation problem or lack of supporting capability is indicated by increased magnitudes of all field deflections and a no. 1 projection greater than the no. 1 sensor deflection (Figure 12b).

Log-log plots of no. 1 projected deflections versus no. 1 sensor deflections can be used to identify variations in pavement structure (see Figure 13). In these figures, the solid lines show the theoretical relationships of no. 1 projected deflections and no. 1 sensor deflections for a constant structure and AC modulus. Subgrade moduli vary along the line. The points about the line represent

Figure 9. Pavement-deterioration curve: relationship between percentage of original thickness of asphalt concrete and percentage of original thickness of dense-graded asphalt.

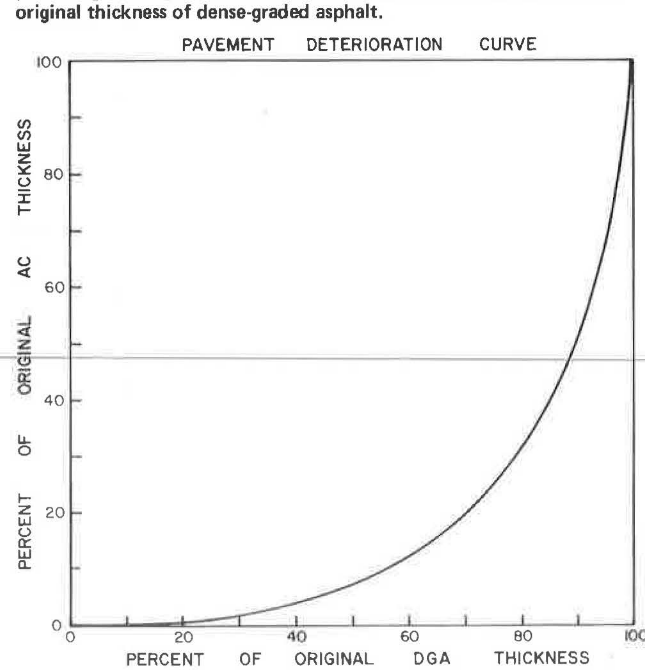
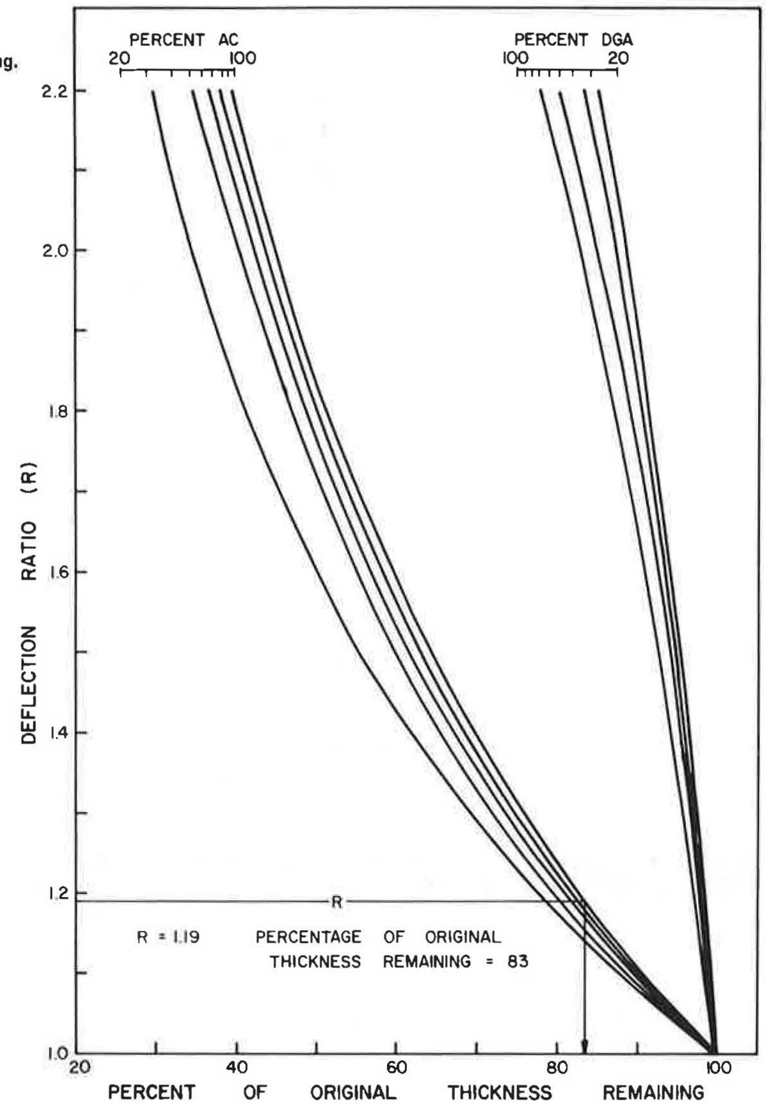


Figure 10. Relationship between ratio of deflection for effective behavior to deflection for theoretical original structure and percentage of original thicknesses remaining.



field-measured deflections. The variation in position of the theoretical line due to changes in the magnitudes of the deflections by ± one unit [0.254 μm (0.000 01 in)] and the associated change in calculated no. 1 projection is indicated by the two dashed lines. The zone inside these lines represents a normal variation due to reading the meters of the road rater.

The following situations have been observed from limited field evaluations.

1. Test data that lie within the zone of normal variation and show relatively low deflection magnitudes: This type of data is indicative of new construction that consists of high-quality materials and had good construction control.

2. Test data that plot on the lower side of the zone of normal variation: This type of data is indicative of a pavement structure in which the subgrade has remained in good condition but cracking or some other problem has caused deterioration of the slab.

3. Test data that plot in the higher range of the zone of normal variation: This type of data is indicative of either of two conditions—changes in type of soil with the pavement remaining in good condition and the layers acting in concert or a deteriorated slab coupled with excessive water content in the subgrade and, again, the layers acting in concert.

4. Test data that plot above the zone of normal variation: This type of data is indicative of subgrades that have an excessive water content.

The fourth condition and pattern of deflections was confirmed by test data obtained in Huntington Beach, California (20). In an investigation there, road rater tests were performed, the pavements were cored, subgrade samples were obtained, and the moisture contents of the subgrade were determined. In those locations that had high water contents (possibly free water) the difference between the no. 1 projected and measured deflections was considerably greater than the theoretical analyses would have indicated. One possible explanation is that water is a much better transmitter of sound or vibrations than is soil. Thus, vibrations are transmitted more easily and their magnitudes remain greater at a fixed distance from the source than those transmitted through normal subgrades. Therefore, the no. 2 and no. 3 sensors measure higher deflections for soils that have excessive water than for those soils that have normal water contents.

Figure 11. Confirmation of determination of effective structure.

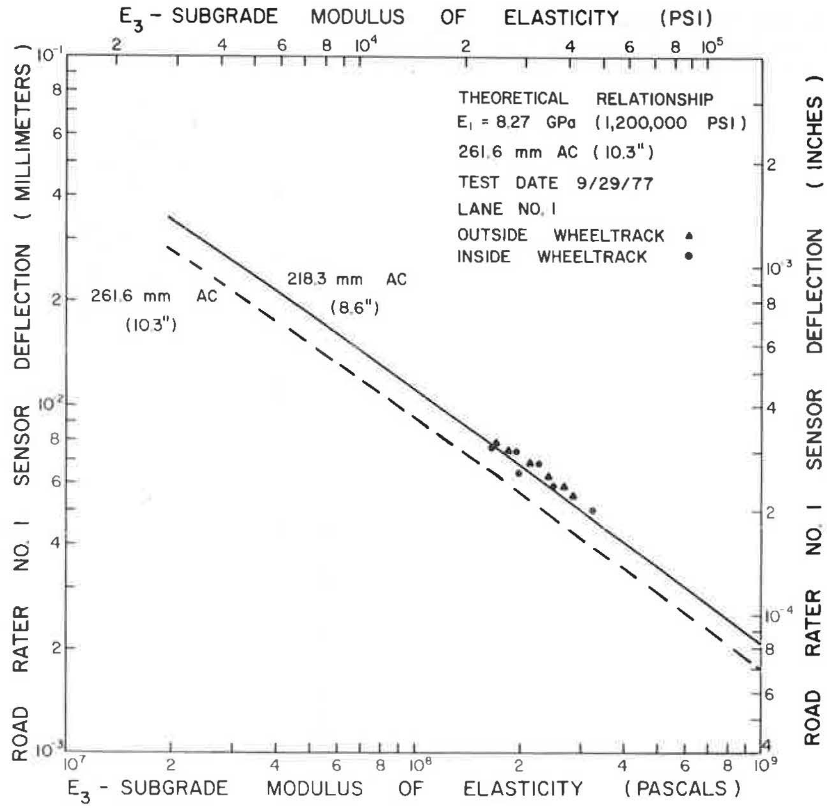
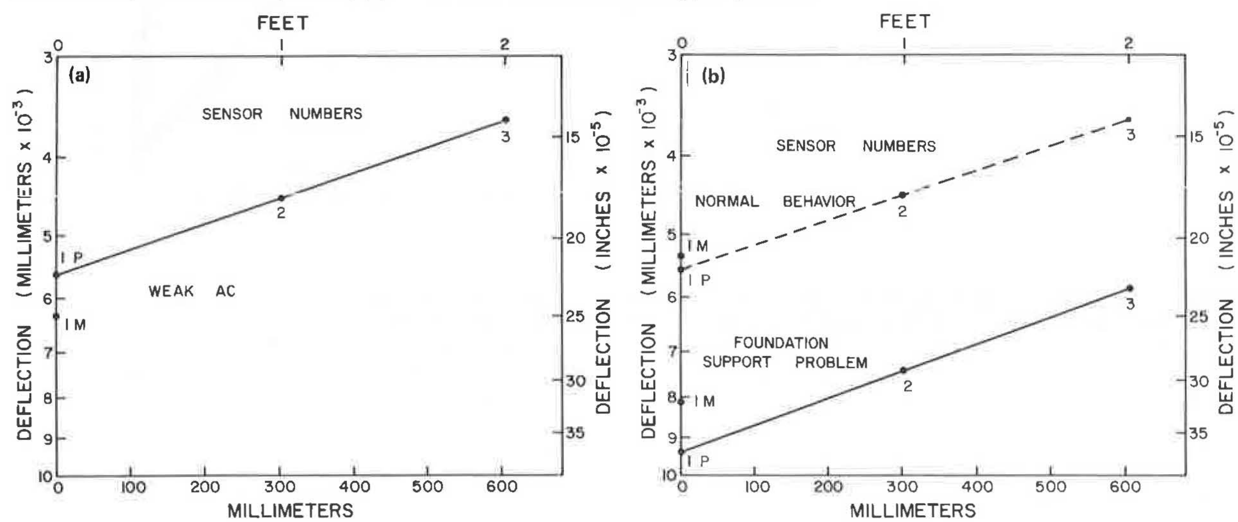


Figure 12. Relationship between deflection and distance from load head and determination of no. 1 projection: (a) pavement that has a weak asphalt concrete layer and (b) pavement that has a foundation-support problem.



SUMMARY

1. Dynamic deflections measured by the road rater have been rationally analyzed and duplicated by elastic theory.
2. Road rater deflections have been used to estimate in-place subgrade moduli.
3. A system has been developed that relates the deflection behavior of a pavement to its effective layer thicknesses of new-quality materials. These effective layer thicknesses can be considered as representative of the residual structure after deterioration and used as inputs for overlay design.
4. A method of analyzing road rater deflections has been developed that makes it possible to identify the type of deterioration in the pavement structure.

ACKNOWLEDGMENT

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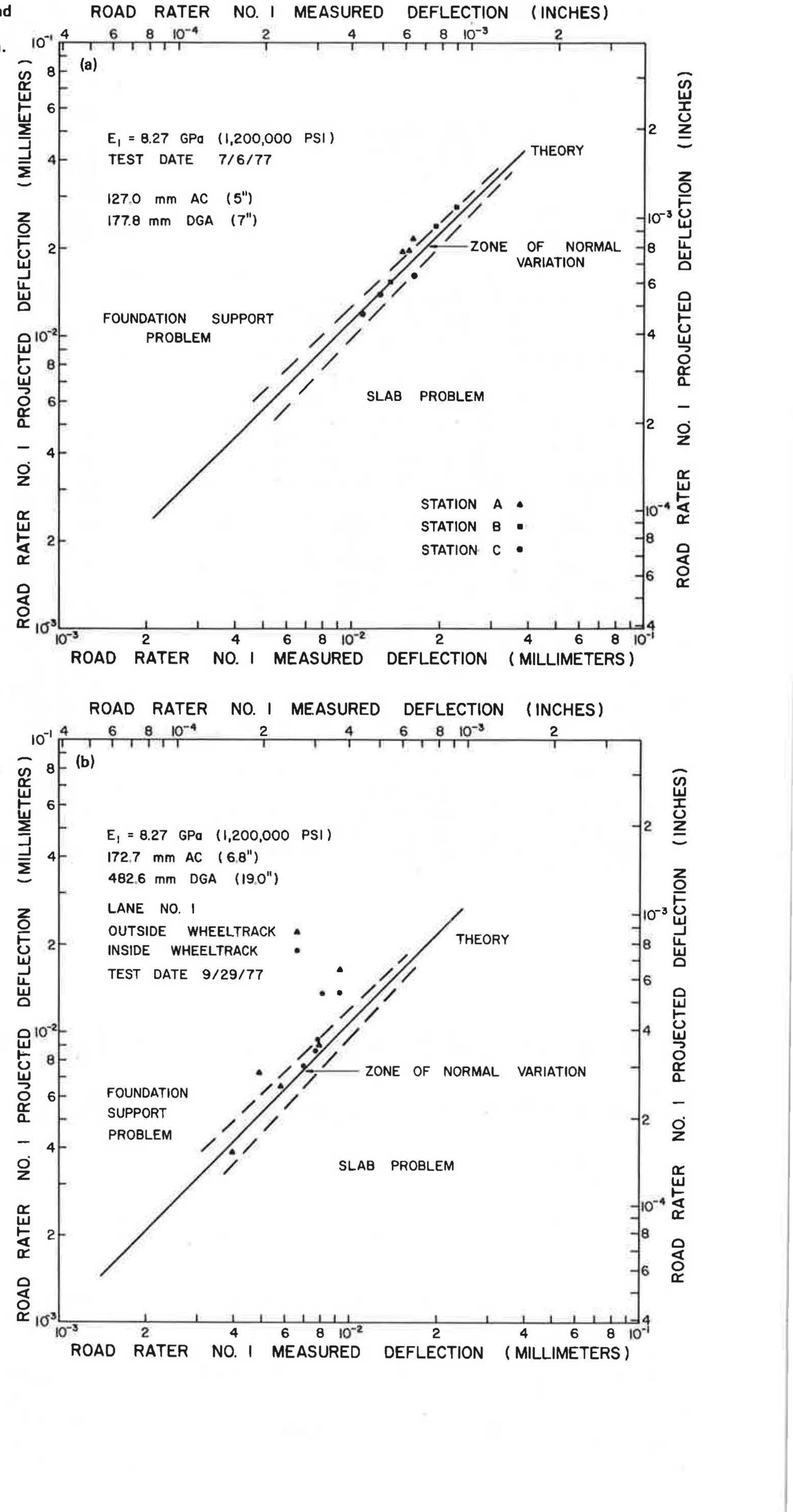
Figure 13. Relationship between no. 1 projection and no. 1 sensor deflection: (a) normal pavement and (b) pavement that has a foundation-support problem.



Kentucky Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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A Rational System for Design of Thickness of Asphalt Concrete Overlays

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A method for designing asphalt concrete overlays is presented that uses (a) the Kentucky proposed design curves, (b) an estimation of future traffic and the associated fatigue (five procedures are given according to type of information available), (c) the strength of the subgrade on the subject project (as determined by laboratory California bearing ratio tests or results of dynamic in-place tests such as road rater measurements), and (d) the present condition of the existing pavement (as determined from dynamic in-place tests, roughness measurements, or the present serviceability index). Deterioration is expressed as reduced thicknesses of new-quality materials that produce the same measured dynamic deflections. The overlay thickness required is the total thickness for the predicted traffic minus the effective or reduced thickness of the existing pavement.

The method for the design of overlay thicknesses presented in this paper has evolved from approximately 30 years of experience in thickness design. The earliest pavement-thickness design methods used in Kentucky were based on 22-kN (5000-lbf) equivalent wheel loads

(EWLs) (1). In 1973, a design procedure was proposed (2) that used 80-kN (18 000-lbf) axle loads and was similar to the procedure of the AASHO Interim Guide (3) (although the damage factors differed). The design of overlays (that is, the determination of required thicknesses) requires as inputs (a) a measurement of the load-carrying capacity of the subgrade, (b) an evaluation of the condition of the existing pavement, and (c) an estimation of expected traffic and associated fatigue.

Subgrade strength is determined by the California bearing ratio (CBR) test method. The CBR test procedure used in Kentucky differs from the American Society for Testing and Materials (ASTM) method only in the length of time of soaking before testing. The Kentucky method allows the sample to soak until swelling ceases and expresses the CBR values as Young's moduli by multiplying by 1500 (4). As expected, in-place dynamic test procedures generally give an estimated subgrade modulus greater than that obtained by the Kentucky laboratory CBR method because the in-place subgrade is not in the critical moisture-content state represented by the soaked conditions of the laboratory test. Thus, the overlay thickness should be determined by using the CBR curve equivalent to the weakest subgrade modulus obtained during in-place testing.

The proper design thickness for an overlay depends on the condition of the existing pavement. The existing condition can be expressed as a reduced modulus of the asphalt concrete (AC) or as reduced layer thicknesses of new materials that have the reference moduli. The concept of reduced thickness is used in this procedure (5, 6). The overlay thickness is that which must be added to the existing pavement so that there will be sufficient structural capacity to support the forecasted traffic or equivalent axle loads (EALs).

Normally, traffic volumes are estimated in connection with needs studies and in the planning stages for new routes and for major improvements of existing routes. However, although the anticipated traffic volume is an important consideration in the styling and geometric design of a roadway, the composition of the traffic in terms of axle loads (and possibly lane distributions) is essential for the structural design of pavements. Traffic volumes used for EAL computations should therefore be reconciled with other planning forecasts of traffic. Historically, actual growths of traffic have usually exceeded forecasts. Overriding predictions of traffic volumes may be admissible for purposes of EAL estimates when properly substantiated. Moreover, the design life of the pavement may differ from the geometric design period.

Basically, computation of the EALs involves forecasting the total number of vehicles expected on the road during its design life and multiplying by factors that convert traffic to EALs. The ideal approach would be to calculate and sum the yearly increments of EALs; this would permit including consideration of anticipated changes in legal weight limits, changes in styles of cargo haulers, and changes in routing.

DETERMINATION OF DESIGN EQUIVALENT AXLE LOADS

There are several methods of estimating the number of 80-kN EALs. For a particular design situation, the one that matches the data base available should be used.

Deacon and Deen Method

Deacon and Deen (7) have described the development and testing of a predictive method (calculation of EALs) for rural highways in Kentucky. The problem was treated as three separate but interrelated parts: (a) development of a proper methodology and identification of pertinent traffic parameters, (b) identification of relevant local conditions that could serve as indicators of the composition and loads of the traffic stream, and (c) development of significant relationships between the traffic parameters and the local conditions. The traffic parameters selected as most significant were the percentages of the various vehicle types and the average number of EALs per vehicle. These were empirically related by multiple regression and other techniques to the set of local conditions, which included road type, direction of travel, availability and quality of alternative routes, type of service provided, traffic volume, maximum allowable gross weight, geographical area, and season. The resultant methodology was judged to be sufficiently accurate, simple, reasonable, and usable to satisfy problem requirements. It is recommended for use, however, only when valid, long-term vehicle classification and load data are unavailable for the route under investigation. The relationships should be updated every two to five years to account for changes in use of vehicle types and changes in axle load limits.

Similar Situations Method

Another method of estimating EALs is by using data from similar facilities. Volume and classification data from parallel and feeder routes can be used when available. Where possible, the new facilities chosen as models should be ones for which there are recorded data representing conditions both before and after construction.

Traffic and Classification Counts

The Federal Highway Administration publishes W-4 tables each year for each state. These tables contain load data by classification of vehicle. The data are listed by site, combined into rural or urban tables, and then combined into total statewide values. If a weighing station is located near the facility being considered and the expected classification of traffic is approximately the same, the analyses should be based on that W-4 table. Otherwise, the statewide W-4 table or one covering groupings of similar sites may be more appropriate.

Several types of analyses can be made from the W-4 tables. The following procedure is suggested.

1. Express the vehicle classification counts as a ratio: C_i = classification count ÷ total number of vehicles counted, where i = vehicle classification.
2. From the W-4 tables, calculate an average damage factor (DF_i) for each vehicle classification by year by using Equation 1.

$$DF_i = \left(\sum_{j=1}^{m} N_j \times F\right) \Big/ (\text{number of weighed vehicles per classification}) \quad (1)$$

where

N = number of axles that have single axle load P_s or tandem axle load P_t (kips),
m = number of load categories (j) in the W-4 table, and
F = damage factor for AC, axle configuration, and axle load as determined from the table below.

Axle Load	F
Single	$1.2504^{(P_s - 18)}$
Tandem	$1.1254^{(P_t - 34)}$

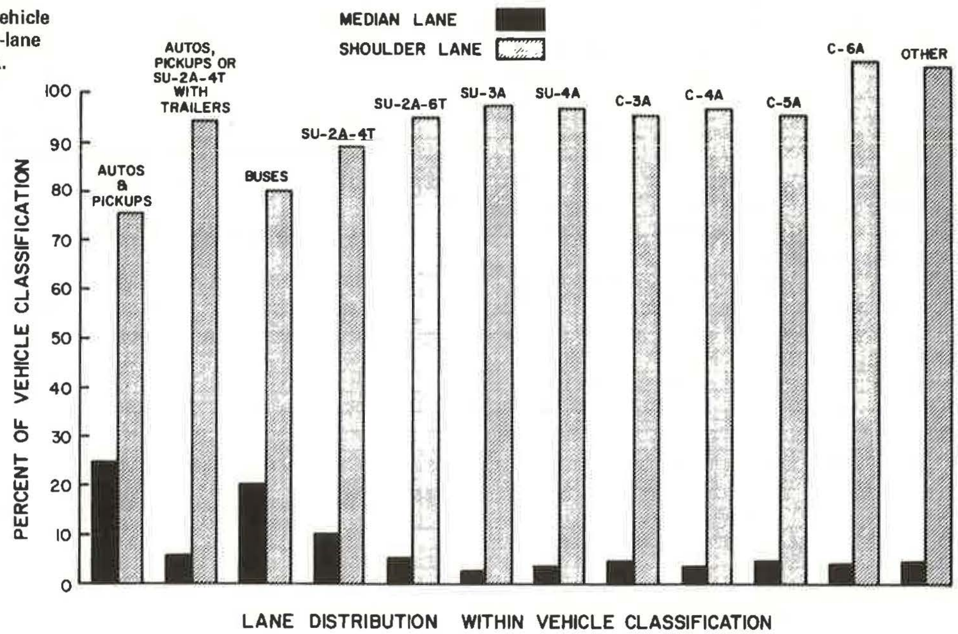
(The coefficients in the table above were determined for

Table 1. Damage factors by vehicle classification: asphalt concrete pavements.

Type of Vehicle	No. of Vehicles Weighed	Total No. of 80-kN EALS	Avg. No. of 80-kN EALS per Vehicle	Damage Factor by Year[a] M	B
Single unit: two axles, four tires	8 564	518.2	0.0605	0.008 310	-1.812 12
Single unit: two axles, six tires	19 058	5 627.6	0.2953	0.008 400	-1.198 76
Single unit: three axles	2 848	1 818.7	0.6386	0.042 940	-2.757 30
Combination unit: three axles	4 701	2 986.7	0.6353	0.008 466	-0.834 29
Combination unit: four axles	15 217	11 434.7	0.7514	0.009 622	-0.568 25
Combination unit: five axles	21 673	13 583.1	0.6267	0.012 298	-0.606 87
Automobiles and pickup trucks			0.0501		

Note: 1 kN = 225 lbf.
[a]Damage factor(year) = M(year - 1959) + B (for years after 1958).

Figure 1. Distribution of vehicle classifications by lane: four-lane facility at level of service A.



U.S. customary units only.) The average damage factors for Kentucky traffic from 1958 through 1975 for each vehicle classification are given in Table 1.

3. Estimate the lane distribution (LD_i) for highways that have four or more lanes for each vehicle classification. Figure 1 shows a typical set of factors for each vehicle classification [single unit: two axles, four tires (SU2A4T); single unit: two axles, six tires (SU2A6T); single unit: three axles (SU3A); combination unit: three axles (C3A); combination unit: four axles (C4A); combination unit: five axles (C5A); and automobiles and pickup trucks] for level of service A on a four-lane facility (2); similar figures have been developed for other levels of service and six-lane facilities (2, 5).

4. For each year, calculate the number of 80-kN EALs by using Equation 2.

$$EAL = 365 \times AADT \times \sum_{i=1}^{n_m} [C_i \times DF_i \times LD_i] \qquad (2)$$

where

AADT = annual average daily traffic and
n_m = maximum number of vehicle classifications used.

5. Add the EALs calculated in step 4 for each year since the pavement was opened to traffic to determine the total estimated EALs to date.

6. Plot the total EALs for each year against the year or fit an equation to the data.

7. Determine the design EAL. If the graphical method was used in step 6, draw a trend line through the data and project to the design year; if an equation was developed in step 6, solve it.

Volumes and Percentages of Trucks

This procedure should be used to estimate the number of 80-kN EALs when the only data available are the traffic volume and the percentage of trucks in the traffic stream.

1. Obtain the volumes from hand counts, recorded machine counts, or published AADT maps.

2. Obtain the percentage of trucks from classification counts made by survey teams.

3. From the W-4 table for a particular year, obtain the average number of axles per truck (APT) by using Equation 3.

$$APT = \sum_{i=1}^{n} A_i \times T_i/\Sigma T_i \qquad (3)$$

where

A_i = number of axles for vehicle classification i,
T_i = number of trucks in vehicle classification i, and
n = total number of vehicle classifications in the W-4 table.

4. From the W-4 table for a particular year, obtain the average axle load (AAL) by using Equation 4.

$$AAL = \sum_{j=1}^{m} [N_j \times AL_j]/[N_S + N_T] \quad (4)$$

where

N_j = number of axles in load category j,
AL_j = axle load for load category j,
m = number of load categories in the W-4 table,
N_S = number of single axles, and
N_T = number of tandem axles.

This provides only an approximation of the average axle load because actual axle loads may range from 8.9 to 267 kN (2000 to 60 000 lbf) depending on the axle configuration and truck style.

5. Calculate the damage factor (DF_{AAL}) for the average axle load by using Equation 5.

$$DF_{AAL} = 1.2504^{(AAL - 18)} \quad (5)$$

Errors involved in using this equation are minimal compared with those involved in predicting traffic volumes.

6. Obtain the lane-distribution factors from the appropriate portion of Table 2.

7. Plot graphs as a function of time or fit equations to the data for the following parameters: volume, percentage of trucks, APT, AAL, and lane-distribution factors. From the graphs or equations, obtain the data for missing years by interpolation and projection.

8. Determine the EAL for each year by using Equation 6.

$$EAL = [(\text{percentage automobiles} \times DF_A) + (\text{percentage trucks} \times APT \times DF_{AAL})] \times AADT \times 365 \quad (6)$$

where DF_A = damage factor for automobiles. Cumulate the EALs calculated for each year since opening to traffic plus the projections to estimate the total EALs that will be applied to the pavement through the design year.

Table 2. Lane distributions at different levels of service.

	Four-Lane Facility		Six-Lane Facility			
	Level of Service		Level of Service			
Lane	A	B	A	B	C	D
Shoulder	95	90	28	26	28	35
Center			45	43	38	32
Median	5	10	27	31	35	33

Annual Traffic Volumes

This procedure should be used if the only data available are those that can be obtained from historical AADT files or maps.

1. Convert the AADT values shown on the maps to one-way values, plot those values against the year, fit a smooth curve to the data, and project to the design year.

2. Use Figure 2 and the estimated AADT for each year to obtain the percentage of each vehicle classification (C_i).

3. Obtain the average damage factor for each vehicle classification by using the procedure described above or Table 1.

4. From the appropriate portion of Figure 1 [or, for other levels of service and for six-lane facilities, the curves given by Southgate and others (2)], obtain the lane-distribution factors (LD_i) for each vehicle classification.

5. Calculate and cumulate the EALs by using Equation 7.

$$\sum_{k=1}^{p} EAL = AADT_k \times C_i \times DF_i \times LD_i \times 365 \quad (7)$$

where

k = year in question minus year opened to traffic and
p = maximum year minus year opened to traffic.

Figure 2. Relationship between AADT and vehicle-classification percentages.

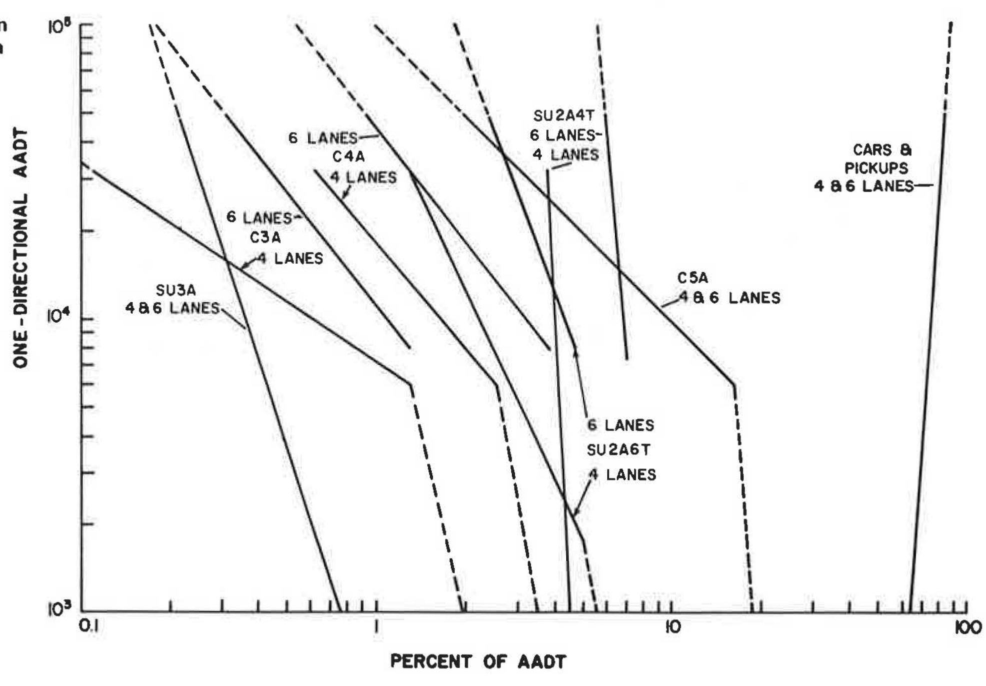


Figure 3. Relationship between roughness and time.

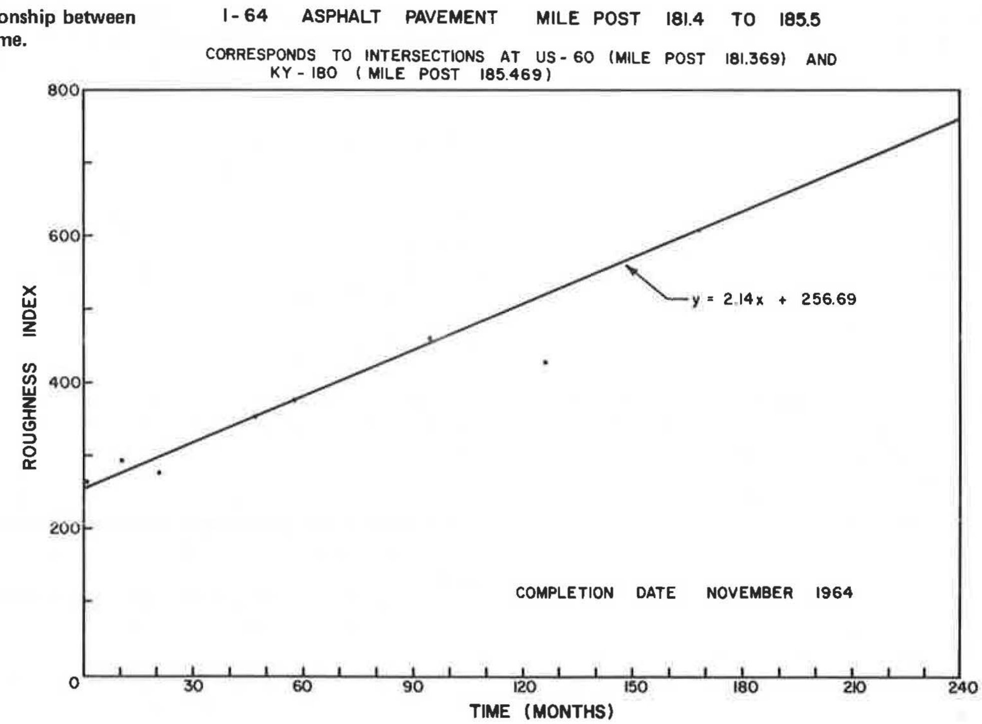
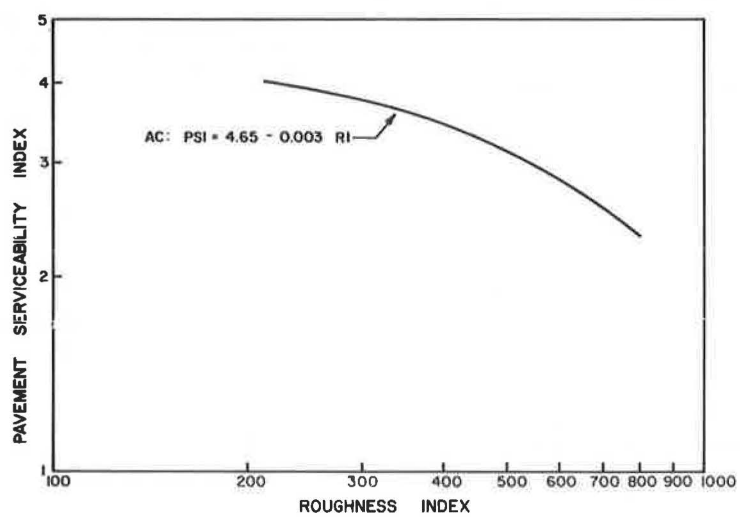


Figure 4. Relationship between serviceability and roughness indices.



6. Review the estimated total EALs for the design year and determine whether additional lanes or alternative routes should be considered.

Compound Interest Equation

If there are no volume data that seem appropriate for the facility under investigation, estimate the traffic volume by using the compound interest equation.

$$AADT_k = AADT_1 (1 + r)^p \qquad (8)$$

where

r = yearly growth factor and
p = number of years from the beginning.

Sum the $AADT_k$s through p years to estimate the total traffic over the design life.

CRITERIA FOR OVERLAY DESIGN

The proposed curves for thickness design (2) are the same as the curves for thickness design of new pavements. These design curves are based on elastic theory and permissible values of strains. The normal inputs into the overlay design procedure are a CBR value (or subgrade modulus), a design or projected number of 80-kN EALs, and the existing or equivalent crushed-stone-base [dense-graded aggregate (DGA)] thickness. For a constant DGA thickness, increasing the ratio of the AC thickness to the total thickness directly increases the AC thickness. Thus, the change in AC thickness is the AC overlay thickness.

METHOD FOR OVERLAY DESIGN

The following procedure can be used to design the thickness of an AC overlay to be applied to an existing AC pavement.

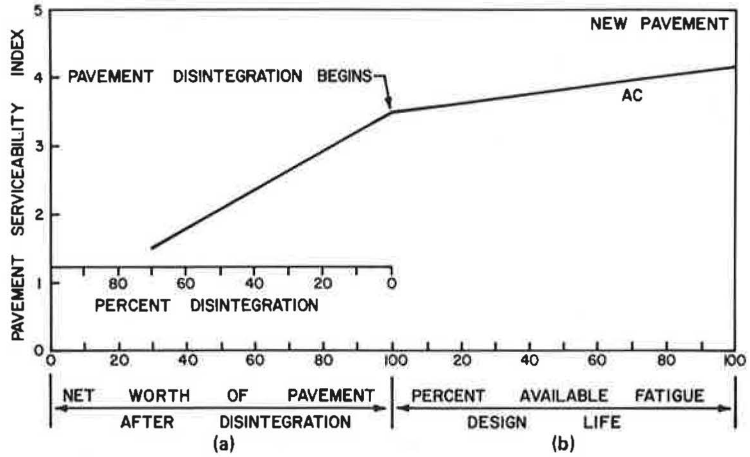


Figure 5. Relationship between serviceability index and (a) present worth of pavement structure after beginning of disintegration and (b) designed fatigue life.

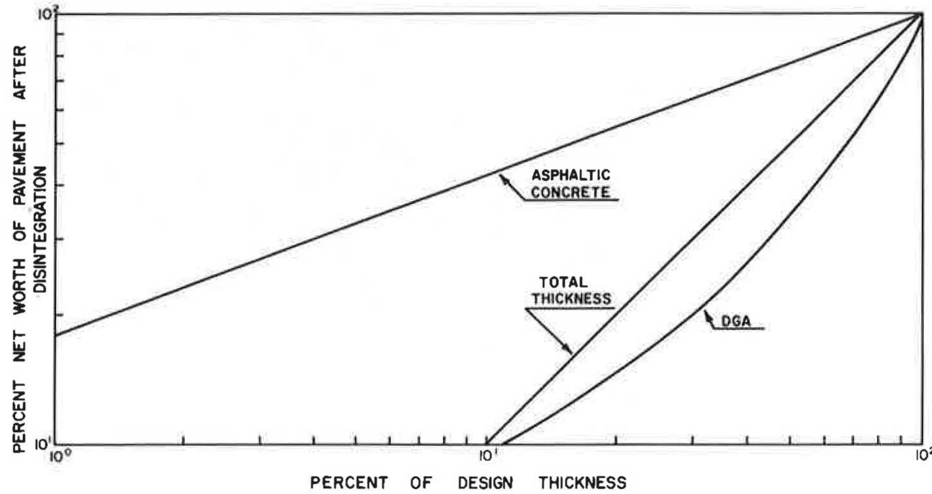
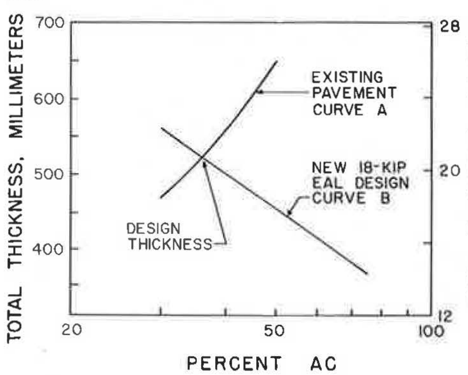


Figure 6. Relationship between percentage of net worth of pavement after beginning of disintegration and percentage of design thickness.

Figure 7. Relationship between total design thickness and percentage of total thickness due to AC.



1. Determine the estimated number of 80-kN EALs (accumulated and projected) by the most appropriate method.

2. Use pavement-roughness measurements (5, 8) to estimate the present serviceability index (PSI) and use this in turn to estimate the residual value (present worth), or remaining life, of the existing pavement structure. There are several methods of estimating the roughness index (RI): (a) historical RI data can be compiled for each project and plotted against time to obtain an estimation of when the critical RI might be expected [Figure 3 (8) shows an example of this procedure]; (b) if no RI data exist for the particular pavement, RI tests can be made; and (c) in lieu of RI tests, a Mays ride meter can be used to test the pavement for roughness and the Mays ride meter value (X) used to obtain the approximate RI value:

$$RI = 2.33X + 180 \quad [\leq 1975\ (6)] \tag{9a}$$

$$RI = 3.20X + 212 \quad [\geq 1976\ (5)] \tag{9b}$$

3. Convert the RI values to the estimated PSI by using Figure 4 (5, 8).

4. Estimate the existing pavement thicknesses from historical files or by using a road rater (5) or Dynaflect to determine an effective structure. If the road rater or Dynaflect is used, go to step 7.

5. After determining the PSI, estimate the present worth or residual value of the existing pavement structure from Figure 5 (5, 6).

6. Use the present worth of the pavement structure as determined from step 5 in Figure 6 and determine the adjustment factors (5) appropriate to the layers of the pavement system.

7. Obtain the equivalent layer thicknesses by using the adjustment factors obtained in step 6 and the original thickness obtained in step 5 in Equation 10.

$$\text{Total equivalent thickness} = (AF_{AC} \times \text{AC thickness}) + (AF_{DGA} \times \text{DGA thickness}) \tag{10}$$

where

Figure 8. Simplified thickness-design curves: thickness of AC layer = (a) 33, (b) 50, and (c) 67 percent, respectively, of total pavement thickness.

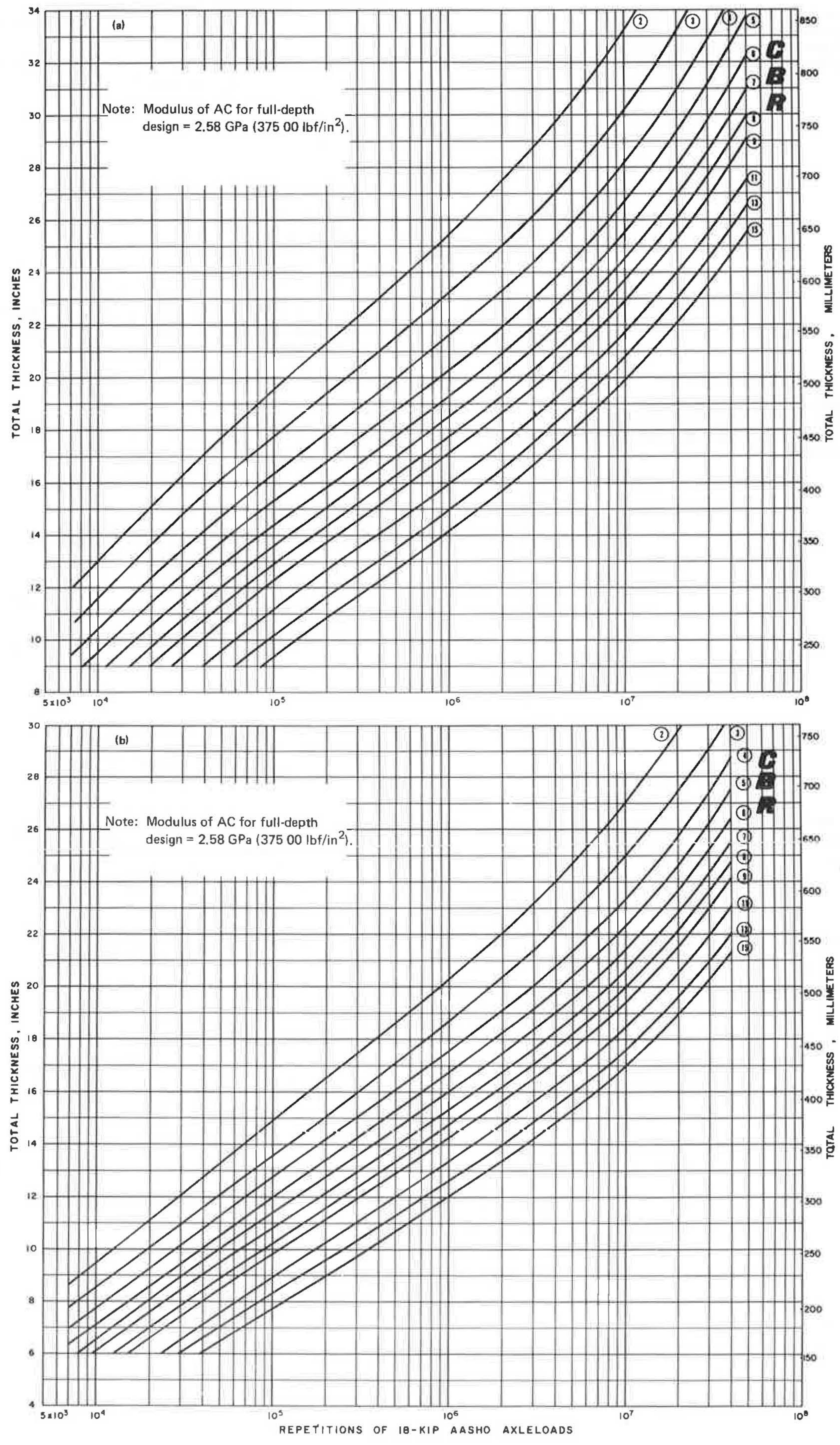


Figure 8. Continued.

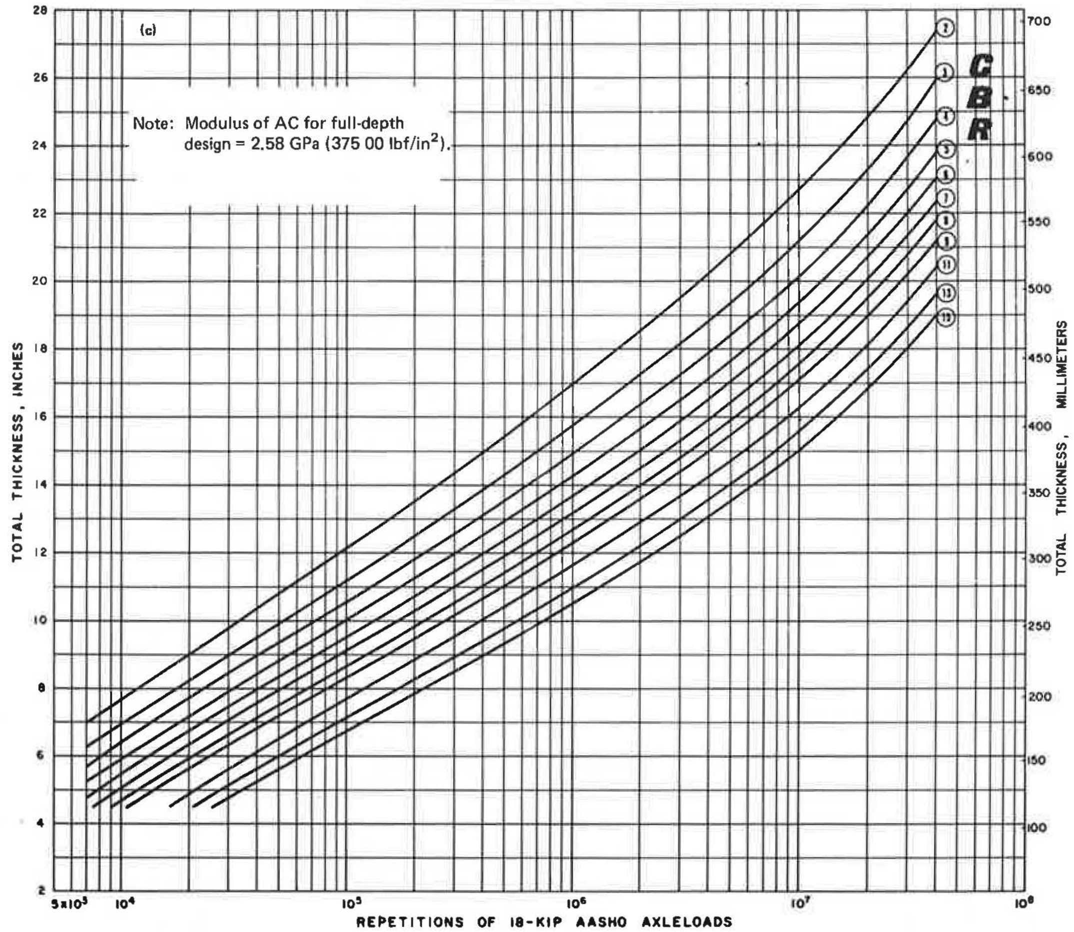


Figure 9. Coal-haul special truck.

AF_{AC} = adjustment factor for AC and
AF_{DGA} = adjustment factor for DGA.

8. Develop a design thickness curve by using the present-worth thickness of the DGA as the basic thickness (curve A of Figure 7) and determine the total thickness for the various AC-thickness percentages of the total thickness by using Equation 11.

Total thickness = 100 x adjusted DGA thickness
÷ (100 - percentage AC of design thickness) (11)

9. Determine the CBR design value for the subgrade by using a laboratory test, a soils survey, or a nondestructive testing instrument such as the Dynaflect, the falling weight deflectometer, or the road rater (5). In Kentucky, the weakest in-place subgrade modulus value as determined from dynamic tests is recommended for designing overlay thicknesses.

10. Use the number of EALs estimated in step 1, the CBR design value estimated in step 9, and Figure 8 to determine the design thicknesses. Plot these values against the AC percent of the total thickness as illustrated by curve B in Figure 7. [Figure 8 can also be used to determine the design thickness for a pavement that uses new material (2, 5).]

11. Determine the total pavement thickness (existing pavement and overlay) from the intersection of curves A and B in Figure 7.

12. Determine the overlay thickness by using Equation 12.

Overlay thickness = total thickness (from step 11)
- total equivalent thickness (from step 7) (12)

EVALUATION OF AN OVERLAID PAVEMENT

KY-33 is an access road to an electrical generating plant that uses coal as fuel and water from the Kentucky River for cooling. Future plans call for building a facility on the river for unloading coal barges and transferring the coal by truck to the plant over KY-33. Such a change in traffic conditions will require an appropriate upgrading of the pavement structure to support the anticipated loads.

The following assumptions were made:

1. The space available at the river will limit the size of trucks to single three-axle units [coal-haul specials (see Figure 9)].
2. This size and style of truck has a tandem rear axle, an empty loading of approximately 133.4 kN (30 000 lbf), and a gross loading capacity of 311.4 kN (70 000 lbf). A typical vehicle has a front axle load of 66.7 kN (15 000 lbf) and a rear tandem axle load of 244.7 kN (55 000 lbf). Its equivalent damage factor per trip is 22.5 EALs.
3. The capacity of the unloading machinery will be limited to 6 trucks/h (48 trips/day).
4. A barge will be located at the facility 125 working days a year.
5. The design should last for six years.
6. Volume of automobile traffic is considered to be relatively insignificant for this location.

The calculated number of 80-kN EALs is 48 trips/day × 125 days/year × 6 years × 22.5 EALs/trip = 4 810 000 EALs.

The Kentucky road rater was used to measure the deflections of the existing pavement. Historical records were searched to determine the thickness of each layer. Cores were taken at the test sites. Elevations were measured on 305-mm (12-in) intervals across the pavement at each test site. Surface temperature, time of day, frequency of testing, and road rater deflections were measured at each site. A compilation of the data recorded for one test site on KY-33 is shown in Figure 10. In this figure, the unadjusted field measurements, the layer thicknesses, and the mean air temperature for the previous five days (which can be obtained from U.S. Weather Bureau records) are shaded.

The method described by Sharpe in the previous paper of this Record and by Southgate (6) was used to evaluate the pavement. A temperature distribution for the AC layer was obtained by using the pavement surface temperature, the time of day, and the five-day mean air temperature. A corresponding distribution of moduli was obtained by using Figure 2 of Sharpe's paper. The mean pavement temperature and AC modulus were determined and used to select the appropriate factor to adjust the field-measured road rater deflections to the reference conditions [21°C (70°F), 25 Hz, and E_1 = 8.27 GPa (1 200 000 lbf/in^2)]. The mean pavement temperature, mean pavement modulus, the adjustment factor, and the road rater deflections adjusted to the reference conditions are shown in the unshaded areas in Figure 10. The graphs of temperature and modulus against pavement depth (temperature and modulus distributions) that were used to determine the mean pavement temperature and mean modulus are shown in Figure 11.

The relationship between the no. 1 sensor deflection and the no. 1 projection is shown in Figure 12a. The theoretical relationship between road rater deflections and subgrade modulus of elasticity for the no. 1 and no. 2 sensors is shown in Figure 12b. These graphs illustrate these relationships for the layer thicknesses, as determined from core measurements, and the reference conditions. Field-measured deflections adjusted to reference conditions are indicated by points. By using Figure 12b and the field-measured road rater no. 2 sensor deflections adjusted to reference conditions, the subgrade modulus corresponding to the no. 2 sensor deflection can be determined by using the line labeled "no. 2 sensor theoretical relationship," and this estimated subgrade modulus can be used to plot the no. 1 sensor deflection. The relationship between the field data for no. 1 sensor deflections and estimated subgrade moduli can be compared with the theoretical relationship. If the field deflections and the estimated subgrade moduli agree with the theoretical values for the original structure, the pavement is performing as expected. If pavement performance (deflections) does not agree with the original theoretical structure line, the pavement is performing as a thinner, effective structure. The relationship between the no. 1 measured (field) deflections and the corresponding no. 1 projections shown in Figure 12a can be used to identify variations in the pavement structure by comparing field data with the theoretical relationship.

The measured deflections and corresponding estimates of subgrade modulus (Figure 12b) do not agree with the theoretical relationship. The thinner, effective structure can be determined in the following way: A parallel line offset to the theoretical structure line (logarithm of the deflection versus logarithm of the subgrade modulus) is drawn through the field point of greatest magnitude. Then, the ratio of deflection (R) for field behavior to that of theoretical behavior is calculated for a constant subgrade modulus and this ratio is used to determine the effective or behavioral layer thicknesses. For the example shown in Figure 12b, the original layer thicknesses were determined by cores to be 114.5 mm (4.5 in) of AC on 127.0 mm (5.0 in) of DGA. However, the pavement was effectively behaving as 81 mm (3.2 in) of AC on 122 mm (4.8 in) of DGA.

Estimation of an effective structure is an iterative process. The first step involves an estimation of the effective structure. This step is accomplished by using the ratios of the deflections for field behavior to the deflections for the theoretical structure. The second step involves a comparison of field behavior with the theoretical behavior of the effective structure. This step is accomplished by completing a second analysis of the field data using the effective structure as the basis for the analysis. A new mean pavement temperature and modulus should be computed and used to determine the associated deflection adjustment factor. The original road rater deflections can now be adjusted to the reference conditions and used to estimate subgrade moduli. Field-measured no. 1 sensor deflections can be plotted against the predicted subgrade moduli and compared with the theoretical relationship for the effective structure. The data used to complete the estimation of the effective structure of the pavement described in Figure 10 are shown in Figure 13 and illustrated graphically in Figure 14. As Figure 14a indicates, all portions of the pavement structure are performing as expected. As can be seen from Figure 13b, the field deflection measurements are very nearly duplicated by the theoretical relationship for the effective structure of 81 mm of AC on 122 mm of DGA. If, for some reason, the field behavior does not agree with the theoretical behavior for the effective structure, then the estimation procedure is repeated until field behavior is duplicated by theory.

The line offset to the theoretical deflection-subgrade modulus line through the point of greatest magnitude provides a shortcut procedure that reduces the number of iterations. Investigations (6) have shown that this shortcut can reduce the number of iterations to one.

Approximately three months after construction of the overlay, the road rater was used to reevaluate the test site on KY-33. Elevations were taken at the same intervals across the pavement as before and were used to determine the average overlay thickness. The average overlay thickness was 76 mm (3.0 in). The procedure described above was used to analyze the road rater test data. The field data used are shown in Figure 15. The layer thicknesses used in evaluating the after-overlay data consisted of the residual or effective layer thicknesses before overlay plus the overlay thickness. The effective structure after overlay is 158 mm (6.2 in) of

Figure 10. Road-rater-data sheet: site no. 1 on KY-33— test data and analysis before overlay and assuming layer thicknesses from records.

Data Sheet
ROAD RATER MEASUREMENTS

Division of Research
Bureau of Highways
Kentucky Department of Transportation
Lexington, Kentucky

LOCATION KY33, MERCER CO., #1 — TIME OF TESTING 11:15 AM

DATE OF TESTING MARCH 25, 1975 — MEAN PAVEMENT TEMPERATURE 48°F

SURFACE TEMPERATURE 38°F — MEAN MODULUS OF ELASTICITY (ASPHALTIC CONCRETE) 2.18 X 10⁶ PSI

5-DAY MEAN AIR TEMPERATURE 58.5°F — DEFLECTION ADJUSTMENT FACTOR 1.10

LAYER THICKNESSES 4.5 IN. AC, 5.0 IN. DGA — FREQUENCY 25 HZ

FIELD DEFLECTION MEASUREMENTS (UNITS IN. x 10^{-5}) SENSORS

No. 1	No. 2	No. 3	No. 1 PROJECTED
59	37	21	65.2
69	41	23	73.1
88	54	30	97.2
59	37	22	62.2
73	43	24	77.0
80	47	23	96.0

ADJUSTED DEFLECTIONS (UNITS IN. x 10^{-5}) SENSORS

No. 1	No. 2	No. 3	No. 1 PROJECTED
64.9	40.7	23.1	71.7
75.9	45.1	25.3	80.4
96.8	59.4	33.0	106.9
64.9	40.7	24.2	68.4
80.3	47.3	26.4	84.7
88.0	51.1	25.3	105.7

PREDICTED SUBGRADE MODULUS OF ELASTICITY (UNITS PSI)
19,500
17,500
12,500
19,500
16,500
15,000

Figure 11. Distributions of temperature and modulus of elasticity with depth of AC: site no. 1 on KY-33 before overlay.

TEMPERATURE AND AC MODULUS DISTRIBUTIONS
STATION No. 1
KY 33 MERCER COUNTY
TEST DATE MARCH 25, 1975
STRUCTURE 114.5 mm AC (4.5"); 127.0 mm DGA (5.0")
EFFECTIVE STRUCTURE 81.3 mm AC (3.2"); 121.9 mm DGA (4.8")

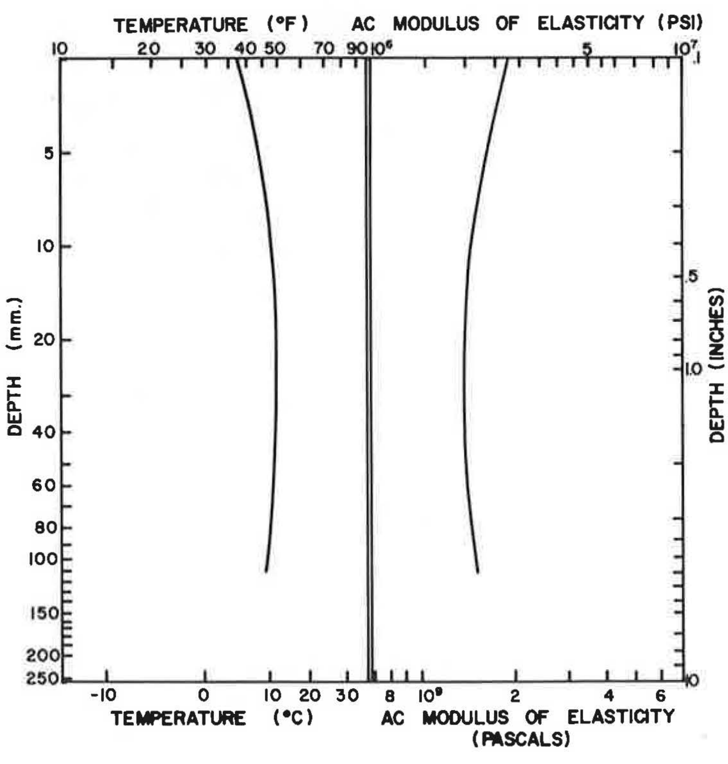


Figure 12. Analysis of road rater data (data from Figure 10): site no. 1 on KY-33 before overlay.

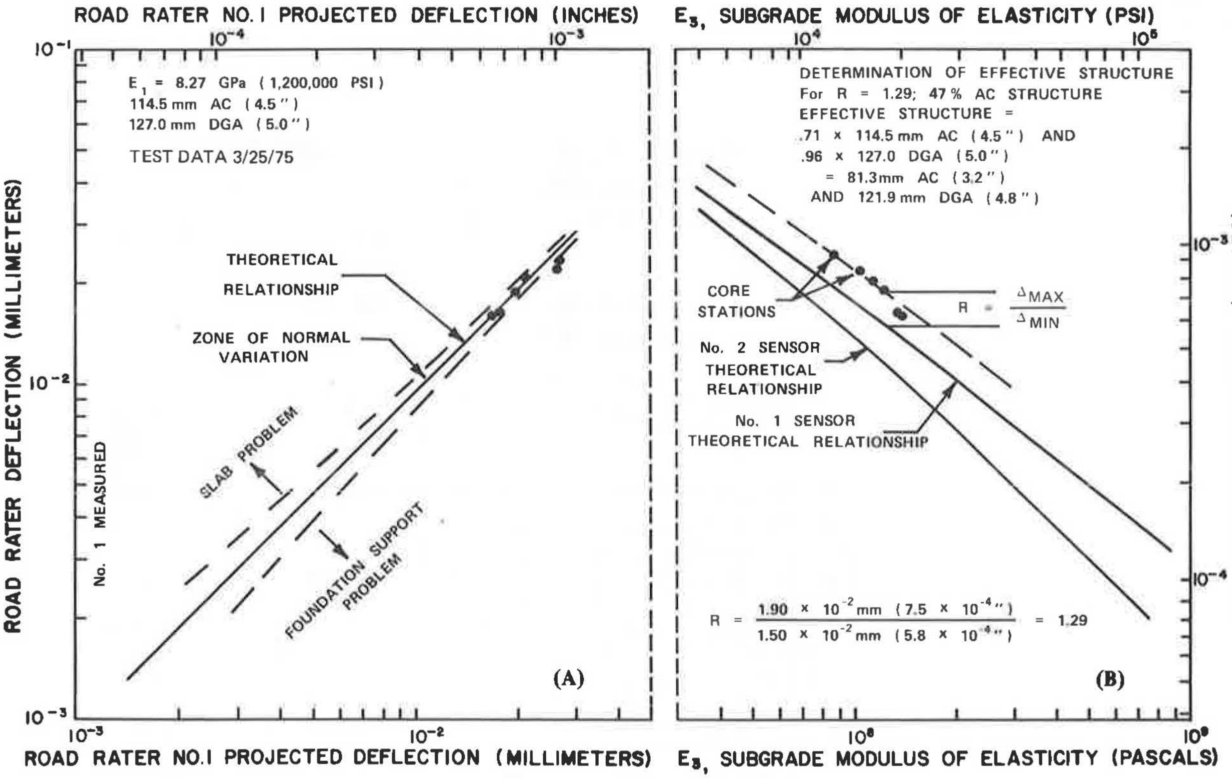


Figure 13. Road-rater-data sheet: site no. 1 on KY-33– test data and analysis before overlay assuming adjusted effective layer thicknesses as determined in Figure 12b.

Data Sheet
ROAD RATER MEASUREMENTS

Division of Research
Bureau of Highways
Kentucky Department of Transportation
Lexington, Kentucky

LOCATION KY 33, MERCER CO., #1 — TIME OF TESTING 11:15 AM

DATE OF TESTING MARCH 25, 1975 — MEAN PAVEMENT TEMPERATURE 50°F

SURFACE TEMPERATURE 38°F — MEAN MODULUS OF ELASTICITY (ASPHALTIC CONCRETE) 2.08 x 10^6 PSI

5-DAY MEAN AIR TEMPERATURE 58.5° F — DEFLECTION ADJUSTMENT FACTOR 1.08

LAYER THICKNESSES 3.2 IN. AC / 4.8 IN. DGA — FREQUENCY 25 HZ

FIELD DEFLECTION MEASUREMENTS [UNITS IN. x 10^-5] SENSORS				ADJUSTED DEFLECTIONS [UNITS IN. x 10^-5] SENSORS				PREDICTED SUBGRADE MODULUS OF ELASTICITY [UNITS PSI]
No. 1	No. 2	No. 3	No. 1 PROJECTED	No. 1	No. 2	No. 3	No. 1 PROJECTED	
59	37	21	65.2	63.7	40.0	22.7	70.4	21,500
69	41	23	73.1	74.5	44.3	24.8	78.9	19,250
88	54	30	97.2	95.0	58.3	32.4	105.0	14,250
59	37	22	62.2	63.7	40.0	23.8	67.2	21,500
73	43	24	77.0	78.8	46.4	25.9	83.2	18,250
80	47	23	96.0	86.4	50.8	24.8	103.7	16,500

Figure 14. Analysis of road rater data (data from Figure 13): site no. 1 on KY-33 before overlay.

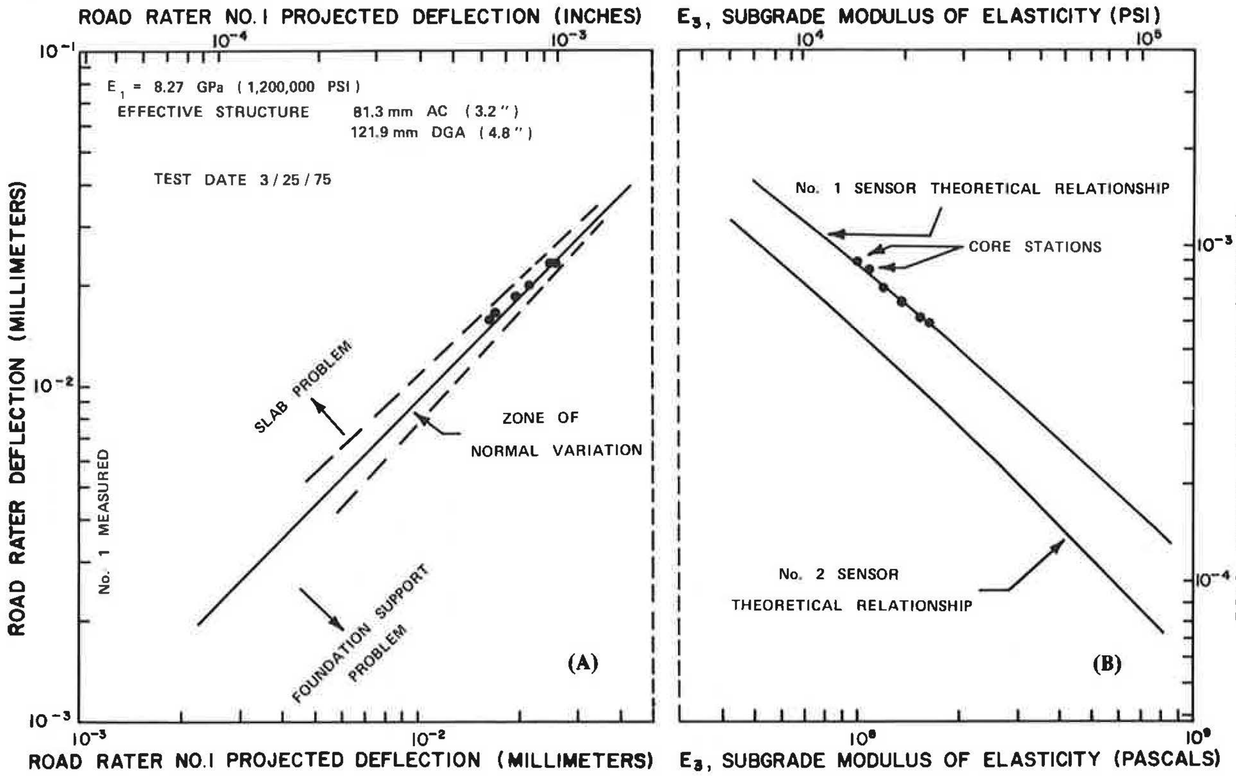


Figure 15. Road-rater-data sheet: site no. 1 for KY-33—test data and analysis after overlay assuming adjusted effective layer thicknesses as determined in Figure 12b plus overlay thickness.

Data Sheet
ROAD RATER MEASUREMENTS

Division of Research
Bureau of Highways
Kentucky Department of Transportation
Lexington, Kentucky

LOCATION KY 33, MERCER CO, #1 — TIME OF TESTING 10:00 AM

DATE OF TESTING NOVEMBER 6, 1975 — MEAN PAVEMENT TEMPERATURE 65°F

SURFACE TEMPERATURE 67°F — MEAN MODULUS OF ELASTICITY (ASPHALTIC CONCRETE) 1.38×10^6 PSI

5-DAY MEAN AIR TEMPERATURE 62.6°F — DEFLECTION ADJUSTMENT FACTOR 1.025

LAYER THICKNESSES AFTER OVERLAY 6.2 IN. AC, 4.8 IN. DGA — FREQUENCY 25 HZ

FIELD DEFLECTION MEASUREMENTS (UNITS IN. $\times 10^{-5}$) SENSORS			
No. 1	No. 2	No. 3	No. 1 PROJECTED
44	31	20	48.1
30	23	15	35.3
37	26	16	42.4
41	32	20	51.2
31	23	16	33.2
35	27	16	45.7

ADJUSTED DEFLECTIONS (UNITS IN. $\times 10^{-5}$) SENSORS			
No. 1	No. 2	No. 3	No. 1 PROJECTED
45.1	31.8	20.5	49.3
30.8	23.6	15.4	36.2
37.9	26.7	16.4	43.5
42.0	32.8	20.5	52.5
31.8	23.6	16.4	34.0
35.9	27.7	16.4	46.8

PREDICTED SUBGRADE MODULUS OF ELASTICITY (UNITS PSI)
21,250
30,500
26,000
20,500
30,500
25,000

Figure 16. Distributions of temperature and modulus of elasticity with depth of AC: site no. 1 on KY-33 after overlay.

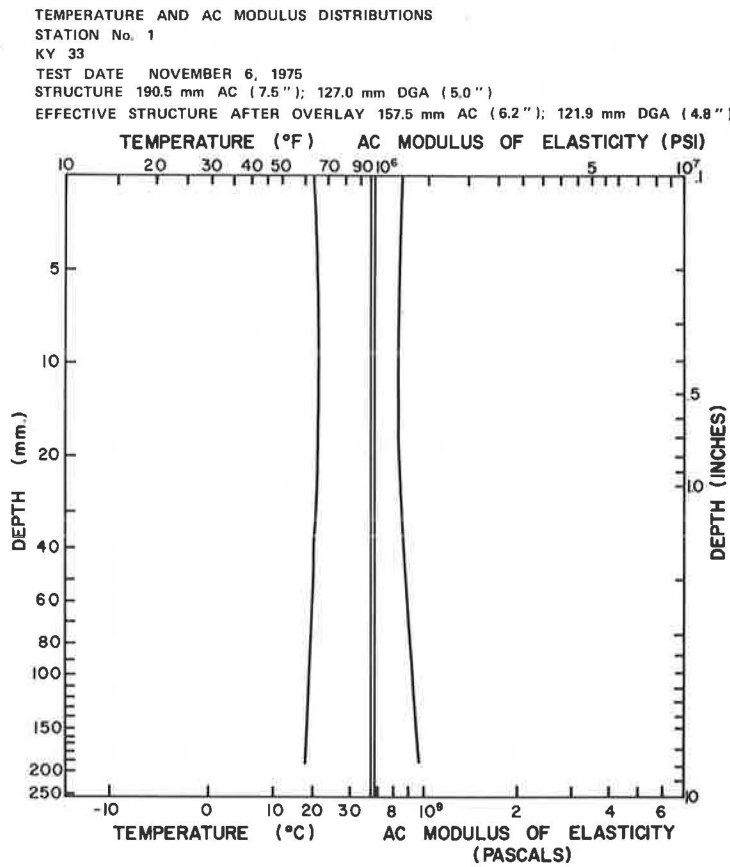
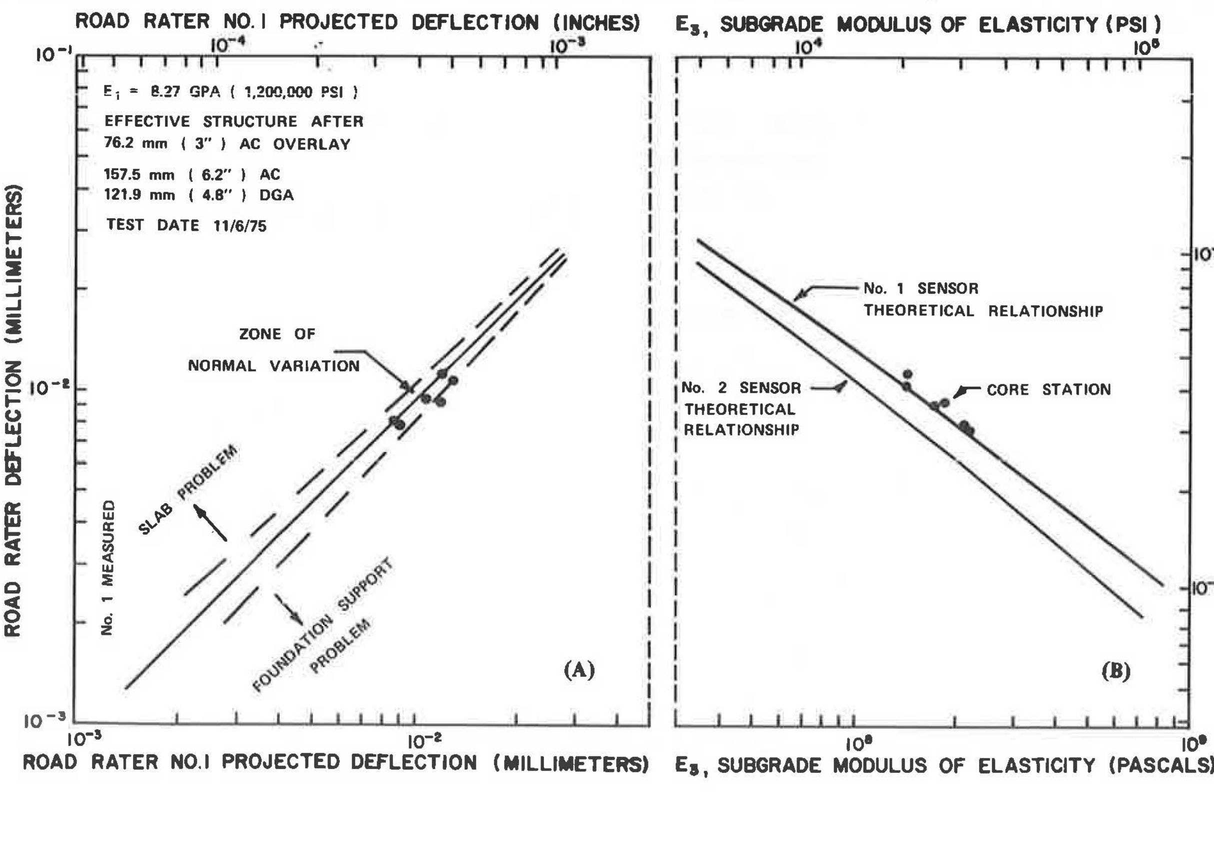


Figure 17. Analysis of road-rater-data sheet (data from Figure 15): site no. 1 on KY-33 after overlay.



AC on 122 mm (4.8 in) of DGA. Temperature and moduli distributions and the associated mean pavement temperature and modulus were determined. The mean pavement temperature and modulus were used to determine the deflection factor needed to adjust the field deflections to reference conditions. Plots of temperature and AC modulus distributions are shown in Figure 16. The relationships between measured and projected deflections and subgrade moduli for both theory and field behavior are shown in Figure 17; the after-overlay test data shown in Figure 17b indicate a behavior equivalent to the effective structure plus the overlay thickness.

SUMMARY

A system for the rational design of an AC overlay has been presented in a step-by-step format. Evaluation of one of many test sites has been presented to illustrate the before-and-after conditions and the agreement between the test data and the theory.

ACKNOWLEDGMENT

The concepts, data, and analyses reported in this paper are a result in part of a research study on the development of a rational overlay design method for pavements that was conducted as a part of a program funded by the Federal Highway Administration and the Kentucky Department of Transportation. The contents of the report reflect our views; we are responsible for the facts and accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the Kentucky Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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Overlay Design Based on Falling Weight Deflectometer Measurements

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The technique used for measuring deflections in an asphalt pavement by means of a falling weight deflectometer is described in some detail. Two models of the deflectometer that have different force ranges have been developed at Koninklijke/Shell-Laboratorium, Amsterdam. The deflectometer is used for the routine evaluation of pavements. The data it produces are of sufficient quantity and quality to serve as input for an analytical method of overlay design. The validity of the data and the interpretation method has been verified by wave-propagation measurements. The basic principles of the new Shell design method are outlined, with specific reference to the determination of overlay thicknesses. It is shown that the required thickness of an overlay depends on one of two criteria, subgrade strain and asphalt-fatigue strain, and that all designs must be checked to determine which of the two criteria is the limiting one. To illustrate this, several examples are given. Some possible refinements to the basic overlay design procedure are discussed, such as the incorporation of various mix characteristics and the procedure for use if the type of mix to be used for the overlay differs significantly from that of the existing pavement.

The economic growth of the 1950s and early 1960s was accompanied by rapid expansion of the existing road network in almost all of the countries of North America and western Europe. Many of the roads constructed at that time, however, are now nearing the end of their structural design lives and in need of major repair.

The structural strength of a pavement refers to its ability to limit strains to such an extent that, during its design life, virtually no cracking occurs in any part of the structure and there is no excessive permanent deformation in the subgrade.

Structural strength is not the only factor that determines the serviceability of a road. Skid resistance and rut depth, for example, are also important in determining the acceptability of a pavement as a riding surface. The recently published Shell Pavement Design Manual (1) specifically recognizes that rut depth due to permanent deformation of the asphalt (and the prediction thereof) is a separate criterion; this has been discussed in several other publications, for example, Van de Loo (2). This paper is concerned solely with structural aspects.

First, a method is discussed that enables the road engineer to determine, in situ, those factors from which the mechanical properties of an existing pavement can be determined. This method is based on deflections measured with an instrument known as a falling weight deflectometer (FWD).

Second, the way is discussed in which these mechanical properties can be used as a basis for a quantitative determination of the residual life of an existing pavement and of the strengthening measures (in terms of overlay thickness) that may be required for the desired future service life.

Most nondestructive techniques for testing flexible pavements are based on measurements of deflections of the pavement under a known load. Empirical techniques of interpretation derive overlay thicknesses more or less directly from the deflection amplitude. More analytical techniques use this amplitude to determine certain parameters significant for the design life of the pavement (e.g., moduli of elasticity of the component layers of the pavement) and then use these parameters in a design model to calculate the thickness of overlay required.

Falling weight deflectometers, which have been used at the Koninklijke/Shell-Laboratorium in Amsterdam over the past three years, have proved to be particularly suitable for the routine evaluation of pavements. At the same time, the information they yield about the mechanical properties of a pavement provides a sound basis for the calculation of the overlay thickness required, for example, by using the Shell design method.

The technique used for interpreting the FWD measurements has been validated on a number of pavements by wave-propagation measurements with the heavy road-vibration machine, the development of which began at Amsterdam some 35 years ago, and with the Goodmans vibrator.

Preferably, pavement properties determined by a pavement evaluation technique should be used in an analytical pavement model from which the required overlay thicknesses for a given future design life can be quantified.

The pavement model that provides the basis for the method described in the Shell Pavement Design Manual is a three-layer structure: an upper asphalt layer, a middle layer of either unbound or bound material, and a lower subgrade layer. Previous publications (3, 4) have discussed the details of the method and its presentation extensively. In this paper, therefore, only a brief outline is given of the pavement-design principles; the discussion is limited to the part concerned with overlay design. It is stressed that, in the three-layer design model, there are two criteria that may govern the design—subgrade strain and asphalt strain—and an overlay-thickness design must be checked for each criterion separately. To illustrate this point, several examples are given.

Asphalt-mix properties can differ widely; moreover, the properties of the mix to be used for an overlay are not necessarily those of the existing pavement. Thus, the design method includes a procedure by which allowance for differing mix characteristics can be incorporated in an overlay design.

MEASURING DEFLECTIONS WITH THE FALLING WEIGHT DEFLECTOMETER

The basic principle of the FWD, as described by Claessen (5), is that of a mass falling on a footplate that is connected to a baseplate by a set of springs (see Figure 1). The peak force (F) thus exerted on the pavement is

$$F = \sqrt{2Mghk} \tag{1}$$

where

M = mass of the falling weight (kg),
h = drop height (m), and
k = spring constant (N/m).

There are several methods of varying the magnitude of the maximum force.

1. Changing the mass of the falling weight: This is impractical for routine investigations where many measurements must be made as quickly as possible to obtain a meaningful impression of the pavement under investigation rapidly.
2. Changing the drop height: This is feasible for routine investigations if the design of the mechanical method of setting the drop height permits it.
3. Changing the spring constant: When mechanical springs are used, the only way to do this is to substitute a set of springs that have different characteristics (which is not normally feasible in the course of routine investigations).

Both changing the mass and changing the springs also affect the pulse width of the force. This means that, if a constant pulse width at different force levels is required and the method by which the force is changed is by substitution of a different mass, there must also be a change of springs.

It is obvious, therefore, that the most practical way to change the force level is to change the drop height.

It should be noted that Equation 1 assumes a linear spring constant, which is not correct for rubber springs. However, a linear spring constant can be assumed if only a small range of the spring characteristic is used (see Figure 2).

The Koninklijke/Shell-Laboratorium currently uses two automated FWDs. The first, shown in Figure 3, has been in use since 1975 and drops a mass of 150 kg from a height that can be varied from 0.04 to 0.40 m. The peak force exerted on the pavement can thus be varied from 15 to 48 kN, a force level representative of the actual wheel loads of most commercial vehicles.

The characteristic of the set of springs has been chosen in such a way that a pulse width of 0.028 s is produced. Numerous measurements on actual pavements have shown that this corresponds to the pulse width produced by commercial traffic traveling at approximately 60 to 70 km/h.

The second, shown in Figure 4, has been developed and constructed recently for use on heavy pavement structures, e.g., airfield pavements. It drops a mass of 407 kg from a height set between 0.04 and 0.40 m and exerts a force that varies between 40 and 125 kN at the same pulse width of 0.028 s. To improve its versatility, this FWD also has a 240-kg mass that is kept in reserve and, together with a set of springs with modified characteristics, covers a peak-force range of 23-90 kN, again at the 28-s pulse width.

The same measuring technique (and interpretation of results) is used for both FWDs. The effect of the force exerted is to deflect the pavement under and around the area of loading. The deflection in the center of the area is a function of the properties and dimensions of the pavement structure but, as is illustrated in Figure 5, this is not sufficient for an exact interpretation because pavements that have entirely different deflection bowls and thus entirely different pavement properties can very well show the same deflection in the center of the area of loading. Therefore, in addition to the deflection at the center of the area of loading (δ_0) the deflection at at least one other point must be measured. This point can be chosen arbitrarily but in routine investigations is usually fixed at 0.3, 0.6, 1.0, or 2.0 m, depending on the type of pavement structure.

The interpretation of the measurements requires two deflection values: the deflection at the center of the area of loading and a deflection value approximately half this. The distance (r) from the center of the loading area at which this latter value is found must also be known. A sensitivity analysis has shown that the interpretation technique yields the most accurate results on the basis of these two deflections.

The deflections are measured by velocity transducers (geophones). The transducers use the inertia of a mass; because their original (predeflection) position serves as reference, they do not require any rigid support from a base outside the deflection bowl.

The deflection signals are projected on a screen in the instrumentation van, where they are evaluated by an operator for acceptability before being printed on a continuous paper sheet or stored on magnetic tape for later

Figure 1. Schematic representation of falling weight deflectometer.

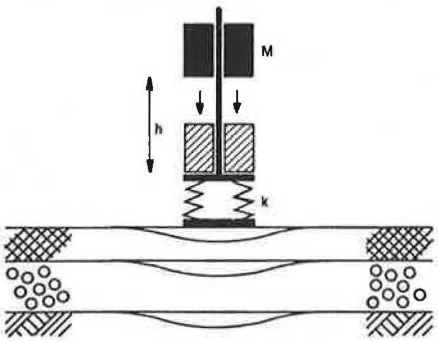


Figure 2. Spring characteristic: rubber springs of second falling weight deflectometer.

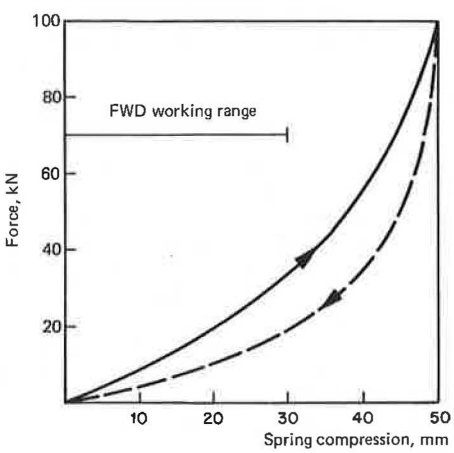


Figure 3. First falling weight deflectometer.

Figure 4. Second falling weight deflectometer.

Figure 5. Schematic representation of pavement deflection.

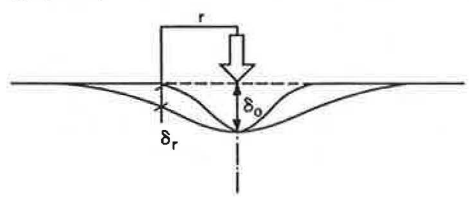
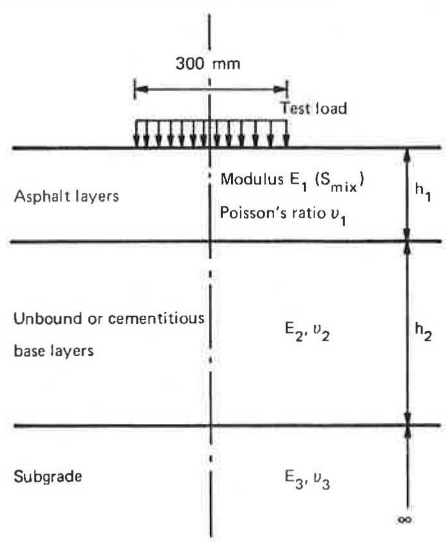


Figure 6. Schematic representation of a pavement structure under a test load.



automated processing (or both).

Every deflection measurement is accompanied by an indication of the location of the measuring point. The force level, pulse width, and asphalt temperature are checked at intervals and recorded.

The force level and pulse width are measured by an accelerometer that can be attached to the falling mass. The accelerometer registers the deceleration of the mass after it has hit the footplate.

The temperature of the asphalt pavement under investigation is measured at regular intervals in the course of the day. This is done by taking a temperature reading of a spike that is shot into the asphalt to a specific depth by using a special gun developed for building practice. Experiments have shown that the heat generated by insertion of the spike dissipates in less than one minute, which makes this method of measuring asphalt temperature practicable for routine use. The unsatisfactory method of estimating the asphalt-layer temperature from a measured surface temperature is thus avoided.

The temperature measurements are used for estimating the modulus of elasticity (E) of the asphalt pavement from known mix characteristics; normally, the values obtained by this route are accurate enough. If, however, a higher degree of accuracy is required, the E-modulus of the asphalt pavement can be determined by high-frequency (80-3000 Hz) wave-propagation measurements. For this purpose, both FWD carriers also contain Goodmans vibrator equipment. However, this procedure is rather time consuming and it is therefore not used unless the higher accuracy is specifically required.

INTERPRETATION OF FALLING WEIGHT DEFLECTOMETER DATA

The pavement structure is schematized as a three-layer model, as shown in Figure 6. The top layer represents the asphalt layer, the middle layer represents the base materials, be they granular (unbound) or cementitious (bound), and the third layer, taken as being of infinite dimensions, represents the subgrade or original soil.

The materials of which the separate layers consist are assumed to behave in a linear elastic way; this has proved an acceptable assumption for the (short) loading times in question. The layers are further characterized by the following properties:

1. For the asphalt layer, an E-value (E_1 or S_{mix}), a Poisson's ratio (ν_1), and a layer thickness (h_1);
2. For the base layer, an E-value (E_2), a Poisson's ratio (ν_2), and a layer thickness (h_2); and
3. For the subgrade layer, an E-value (E_3), and a Poisson's ratio (ν_3) (the layer thickness is taken as infinite).

If these values are known, the stresses and strains, and thus the shape of the deflection bowl of a pavement under a given load (6), can be calculated, for example by using the BISAR computer program (7, 8).

In the interpretation of the FWD measurements, some of these values are assumed or estimated as closely as possible from coring or from existing records (e.g., construction reports). The Poisson's ratios and the layer thicknesses of the base layers are assumed because small variations in these values have little effect. The E-moduli of cementitious base layers can be derived from past experience or measurements. Actually, the unbound base layers will show an increasing modulus from the subgrade up. This range of moduli can be replaced by an effective modulus of the total unbound base layer that is a function of the subgrade modulus E_3:

$$E_2 = 0.2 \times h_2^{0.45} \times E_3 \qquad (2)$$

where h_2 is in millimeters and is subject to the limits ($2 \le 0.2 \times h_2^{0.45} \le 4$). This effective modulus can only be used for calculation of stresses and strains in layers other than the unbound base layer itself.

In the normal interpretation procedure, the E-modulus of the asphalt layer is determined from the stiffness (E_1) modulus of the asphalt mix and the temperature of the asphalt during the FWD measurements, provided that the type of mix used is known or can be estimated very closely [for example, by using a nomograph given in the Shell design manual (1)]. Determining the E-modulus of the asphalt layer by wave-propagation measurements is recommended only in cases where the greater accuracy is specifically required.

The remaining two pavement properties—the thickness of the asphalt layer and the subgrade modulus—can be calculated from the measured values of δ_0/F and δ_r/F (where F is in Newtons and δ_0 and δ_r are in 10^{-10} meters per Newton). The value of the asphalt-layer thickness thus calculated is called the effective asphalt-layer thickness ($h_{1\,eff}$) because it incorporates and compensates for errors in the estimation of the stiffness modulus of the asphalt layer and/or the deterioration of the asphalt layer.

Another feasible procedure would be to use the actual asphalt-layer thickness (from cores or old records) and calculate the effective E-modulus of the asphalt layer. The two procedures do not differ significantly.

It is not possible to determine the residual life of a pavement solely from deflection measurements. The reason for this is shown in Figure 7. The change in E-modulus as the number of load repetitions increases has been observed in laboratory fatigue tests and is corroborated by deflection measurements in practice. Deflection values are almost constant over a long period; however, when the pavement approaches the end of its design life, the deflections increase quite sharply. It is, however, possible to determine the original design life, in terms of the number of repeated applications of a standard axle load. This is the reason that the standard FWD practice is to make the measurements at points where traffic loading is slight (such as between the wheel tracks).

Occasionally, a check is made by measuring the deflections in one of the wheel tracks. If the deflection values measured in the wheel track are significantly larger than those measured between the wheel tracks, this is a definite indication that the pavement is approaching the end of its service life.

For a length of road pavement, the average values of δ_0 and δ_r/δ_0 (Q_r) are calculated (see Figure 8), together with their standard deviations. Next, the 85th percentile value of δ_0 and the 55th percentile value of Q_r are calculated. Then, these and the known value of E_1 are compared with a series of interpretation graphs (of which Figure 9 is an example) to determine the values of E_3 and h_1. The value of h_1 obtained in this way should be interpreted as the effective asphalt-layer thickness ($h_{1\,eff}$).

Formerly, the interpretations of all possible combinations of 15th and 85th percentile values of δ_0 and Q_r were checked in terms of the corresponding required overlay thickness. However, experience has shown that, in nearly all structures, the 15th percentile value of δ_0 in combination with the 85th percentile value of Q_r leads to a safe overlay design.

Interpretation graphs such as that shown in Figure 9 are based on the results of several BISAR computer program calculations.

VALIDATION AND EXAMPLES

There have been several experiments performed with the FWD to check the validity of the results obtained by using the method of interpretation described above. Some of the results are shown in Figures 10 and 11, and further confirmation is provided by the results of a recent experiment (see Table 1). Constructions 1-4 were different sections of the same road, constructions 5 and 6 were located in a second road, and constructions 7 and 8 in a third, all situated relatively close to each other in one municipality. The results of the FWD measurements performed on these pavements are shown in Figure 12.

For constructions 1-6, the deflections measured at a distance r = 1 m from the center of the load were used in the interpretation. In the case of constructions 7 and 8, the deflection bowl was wide and shallow because of the thick layer of hydraulically bound slag and the deflection at r = 2 m was used.

In these interpretations, all the base materials were assumed to be unbound material to keep the interpretation technique simple. The particular properties of any bound base materials manifested themselves in terms of an additional asphalt-layer thickness over and above the actual asphalt-layer thickness. This is obvious for constructions 1 and 4 and even more so for constructions 7 and 8. Also, the difference in effective thickness of constructions 7 and 8 can only partly be explained by the difference in actual asphalt thickness, which means that the slag in the base layers of construction 7 must have reached a higher degree of hydraulic binding. Thus, it may be concluded that there is generally good conformity between the actual pavement thicknesses and the derived effective asphalt-layer thicknesses.

As an additional check, low-frequency (<80 Hz) wave-

Figure 7. Relationship between E-modulus and number of load repetitions.

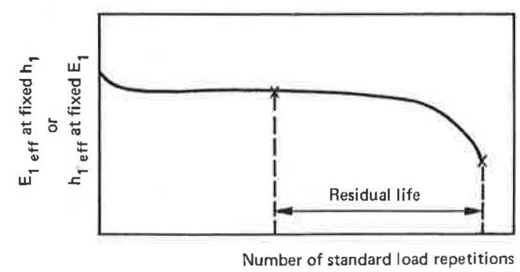


Figure 8. Graphical representation of δ_0 and Q_r.

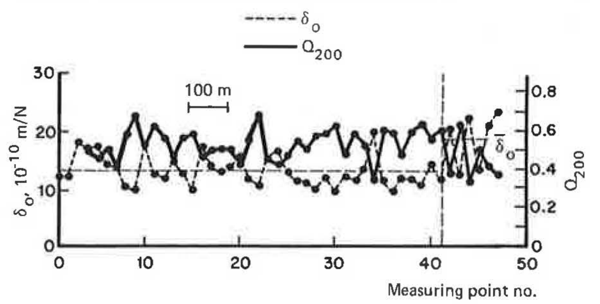


Figure 9. Deflection interpretation chart.

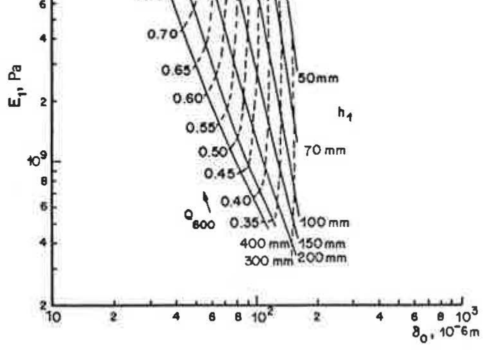


Figure 10. Comparison between subgrade modulus obtained from sinusoidal-wave-propagation measurements (E_{3SW}) and that obtained by FWD (E_{3FWD}) measurements.

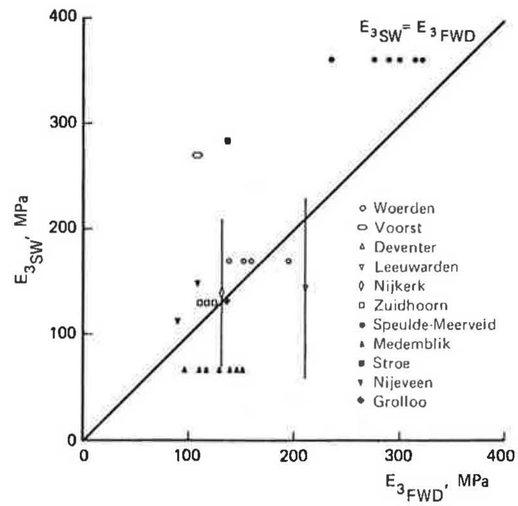


Figure 11. Comparison between asphalt layer thickness determined from FWD deflections ($h_{1\ FWD}$) and actual layer thickness ($h_{1\ actual}$).

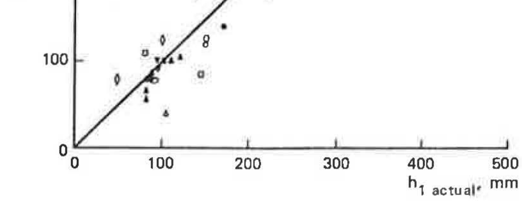


Figure 12. Deflection measurements made with the falling weight deflectometer: constructions (I-VIII).

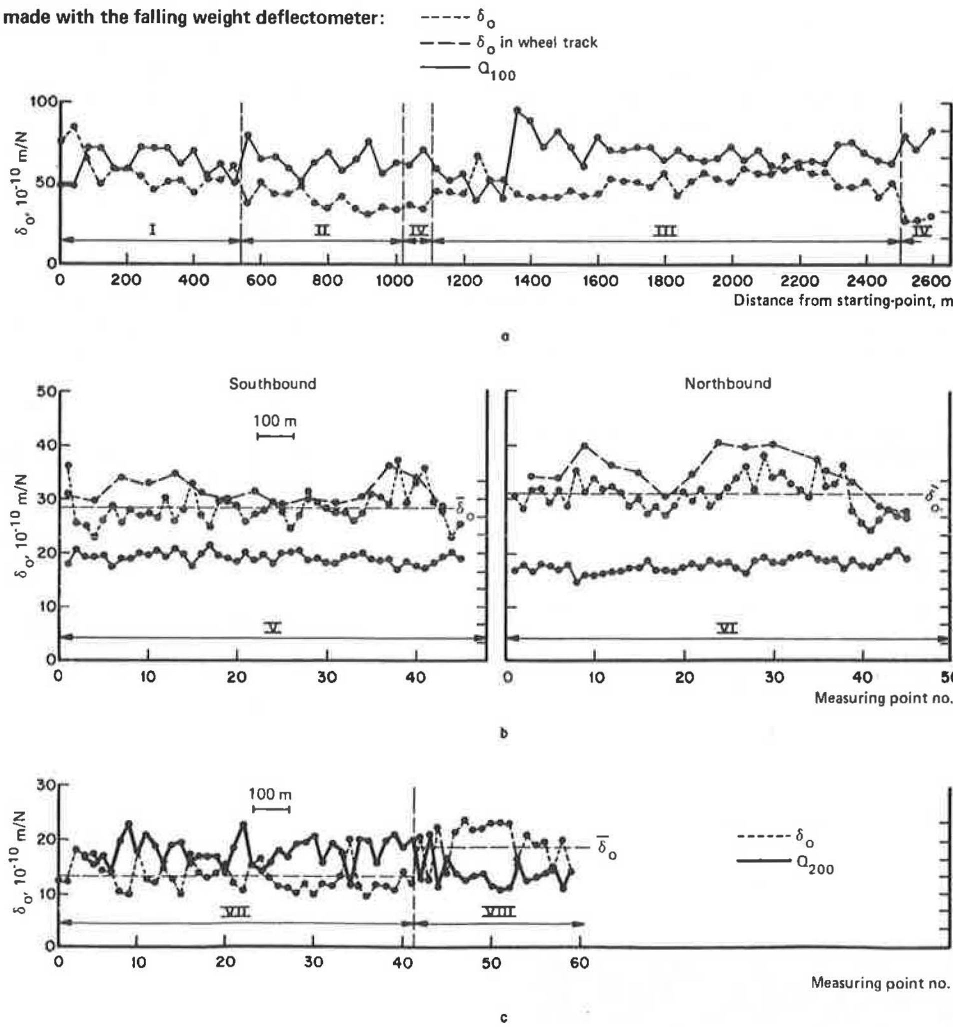


Table 1. Confirmation of validity of interpretation method.

	Construction							
Item	1	2	3	4	5	6	7	8
Thickness of bitumen-bound material (cm)	7	14	14	14	24	24	15	11
Road base material	~5 cm of old pavement (cracked) on 20 cm of crushed gravel	10 cm of blast furnace slag	20 cm of crushed gravel	Sand cement			5 cm of crushed gravel and 30 cm of blast-furnace slag on 50 cm of sand	5 cm of crushed gravel and 30 cm of blast-furnace slag on 50 cm of sand
Subgrade	Sand	Sand	Sand	Sand	Sand	Sand	Clay and sand	Clay and sand
$h_{1\,eff}$ (FWD) (cm)	12	15	13	17.5	27.5	25	65	35
E_3 (MPa)								
FWD	120	190	150	190	163	182	194	200
RVM							150	150

propagation measurements were performed on constructions 7 and 8 by using the road vibration machine (RVM) to determine the value of E_3. The values obtained in this way (150 MPa) correspond fairly well to those obtained by using the FWD technique. [The somewhat lower value might be explained by the fact that the stresses induced by the RVM in the subgrade are lower (by about 50 percent) than those induced by the FWD.]

SHELL DESIGN CHARTS

The Shell Pavement Design Manual published in 1978 includes a large number of thickness-design charts that can be used in the design of both new pavements and overlays. The design model on which the charts are based (and consequently the model from which the overlay thickness can be calculated) is the three-layer model described above.

The failure criteria that govern the design are threefold (see also Figure 13):

1. The compressive strain at the surface of the subgrade: If this strain is excessive, permanent deformation will eventually occur at the top of the subgrade, re-

sulting in permanent deformation at the pavement surface as well.

2. The tensile strain in the asphalt layer: The maximum tensile strain generally occurs at the bottom of the asphalt layer; if this strain is excessive, cracking of the asphalt layer will occur.

3. The permanent deformation within the asphalt layer: This may lead to rutting to such an extent that the acceptability of the pavement as a riding surface is impaired.

Figure 13. Pavement design model.

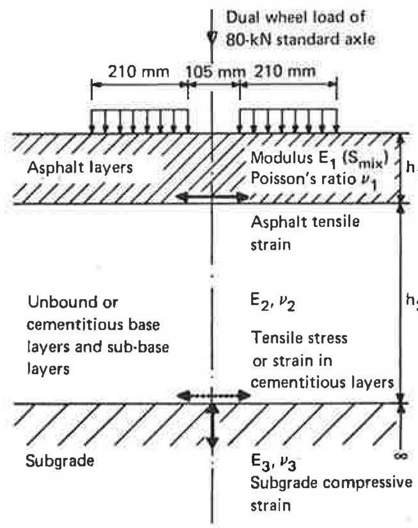
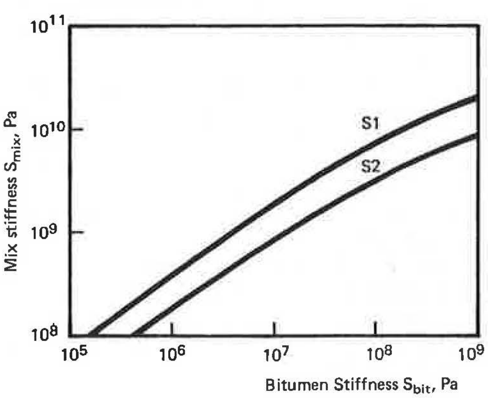


Figure 14. Standard stiffness characteristics.



One of the basic principles of the pavement-design theory on which the manual is based is that the structural design life of a pavement is dependent on either the subgrade-strain criterion or the asphalt-strain criterion.

In determining the thickness required for an overlay, the subgrade-strain and the asphalt-strain criteria should each be considered separately; it is quite possible that the design criterion that did not govern the original pavement design will become limiting for the overlay thickness. Consider, for example, a structure originally governed by the subgrade-strain criterion. If, after a certain proportion of the service life has been consumed, an overlay is applied, the permanent deformation due to both asphalt and subgrade deformation will automatically be eliminated and the original design life will be restored. However, the maximum asphalt strain must also be taken into account. In the original pavement, this occurred at the bottom of the asphalt layer. After the overlay has been applied, it will still occur in the same place, but its magnitude will be less because the pavement is thicker.

The level of this asphalt strain must be low enough to reduce the rate of consumption of the residual fatigue life so that the original pavement will last the future design life without cracking. This can be calculated by using a fictitious design life in interpreting the charts.

The design manual contains close to 300 design charts that were developed on the basis of the three-layer model of the pavement and incorporate the criteria of subgrade strain and asphalt strain. The permanent-deformation criterion is not incorporated in the charts and must be dealt with by a separate procedure.

In the design procedure, certain (standard) asphalt-mix types are used. The mix stiffness (E_1 or S_{mix}) is characterized in relation to bitumen stiffness by the curves S1 and S2 as shown in Figure 14, and the fatigue behavior is standardized as F1 and F2 as shown in Figure 15. Two standard grades of bitumen—hardened 50 and 100 pen—are used; their properties are given below.

Bitumen	$T_{800\ pen}$ (°C)	Pen_{25} (0.1 mm)	Penetration Index
50 pen	59	35	0
100 pen	53	60	0

Thus, charts are generally given for eight different standard mix codes (all possible combinations of S1 or S2 with F1 or F2 and 50 or 100 pen bitumen); for example, a mix that has good stiffness behavior (an E-modulus of S1) and good fatigue behavior (F1) made with 45-60 pen bitumen would be represented by the mix code S1-F1-50.

Other input parameters for design are subgrade mod-

Figure 15. Standard fatigue characteristics.

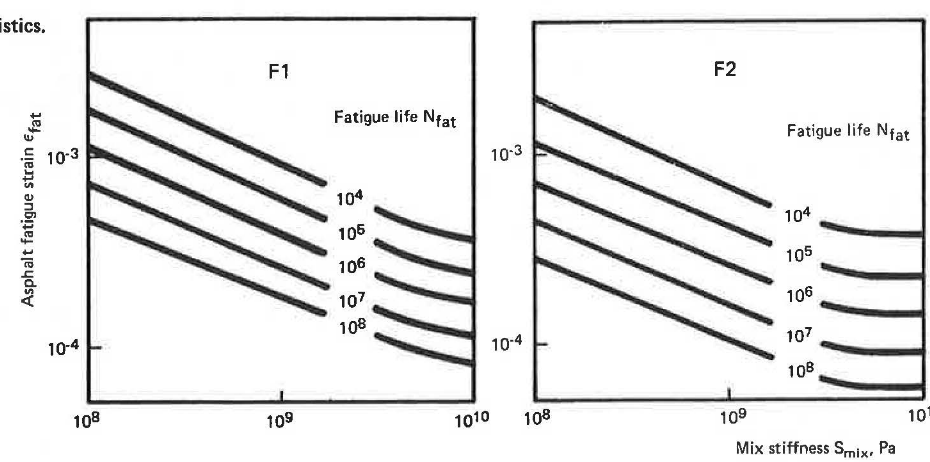


Figure 16. Axle-load conversion.

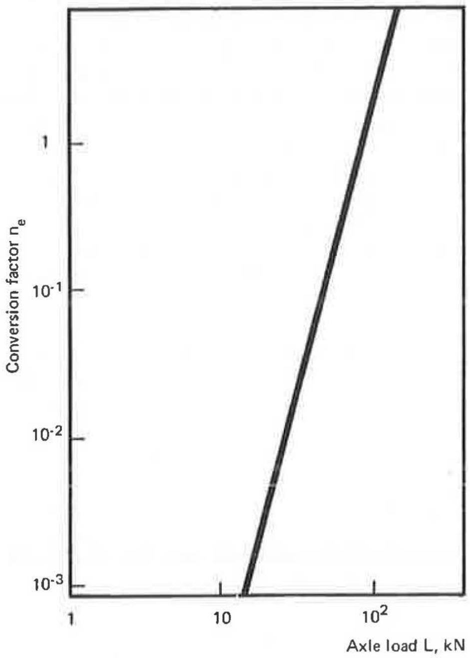
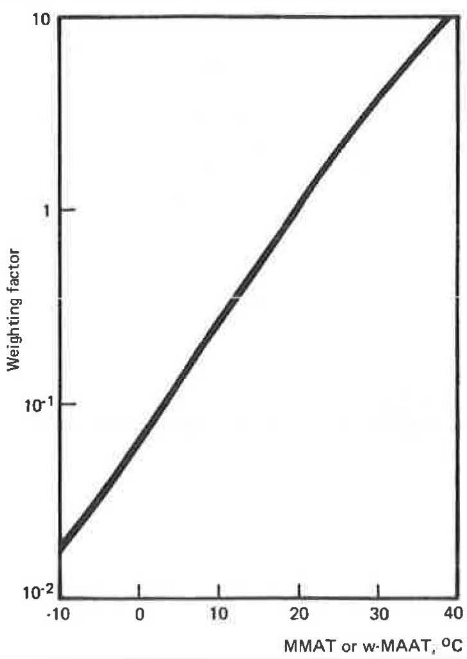


Figure 17. Chart W: temperature-weighting curve.



ulus, design life, and climate.

The design life (N) is expressed as the number of repetitions of a standard axle load of 80 kN. Any given axle spectrum can be converted into an equivalent number of 80-kN standard axle loads by using the graph illustrated in Figure 16. Based on 80 kN = 1.00, this curve represents the relative damage done to the pavement by an axle load different from 80 kN.

The climate is introduced as a weighted mean annual air temperature (w-MAAT) that can be calculated from the mean monthly air temperatures (MMATs) by use of the weighting-factor curve shown in Figure 17 (chart W of the design manual). This weighting curve was ob-

Table 2. Determination of w-MAAT.

Month	MMAT, °C	Weighting Factor	Month	MMAT, °C	Weighting Factor
January	1.4	0.08	September	20.3	1.1
February	2.2	0.09	October	14.2	0.50
March	6.7	0.18	November	8.1	0.20
April	12.2	0.38	December	2.8	0.10
May	18.1	0.82	Total		8.85
June	22.8	1.5			
July	25.3	2.1	Avg		0.74
August	23.9	1.8			

tained by using BISAR calculations for a selection of representative multilayer pavement structures. The asphalt layers of these structures were further subdivided to account for the stiffness gradient that results from a temperature gradient in an otherwise homogeneous asphalt layer. The term "weighted" thus indicates that the daily and monthly temperature gradients in the asphalt layer in a certain climate have been taken into account. For example, by using the data given in Table 2 and Figure 17, the w-MAAT for Washington, D.C., is found to be $17.5 \approx 18°C$.

There are four types of design charts (see Figure 18).

1. Type HN (see Figure 18a): This type of chart shows the relationship between required asphalt thickness and the required thickness of the unbound base layers for different design lives, expressed as N. There are 128 design charts of the type HN in the manual, covering w-MAAT values of 4°C, 12°C, 20°C, and 28°C and subgrade moduli of 25-200 MPa.
2. Type HT (see Figure 18b): This type of chart most clearly illustrates the effect of climate on design. In general, a warm climate requires a thicker asphalt layer. However, this is not always the case; for example, when the asphalt strain is the governing criterion, the larger permissible asphalt strain of the warmer asphalt (see also Figure 15) may reverse this effect. There are 72 charts of the type HT in the manual.
3. Type TN (see Figure 18c): This type of chart shows the effect of climate in a different way, making possible, for example, the extrapolation of an empirically proven construction to a different climate. There are 48 charts of the type TN in the manual.
4. Type EN (see Figure 18d): This type of chart is useful not only for extrapolating asphalt thicknesses to areas that have different subgrade moduli, but also for the determination of overlay thicknesses. There are 48 charts of the type EN in the manual.

In principle, all four types of chart can be used for both the determination of overlay thicknesses and the design of new pavement structures. The presentation of a given type of information in different ways means that, whatever the form in which a problem manifests itself, the designer should be able to find the answer in a reasonably direct manner.

USE OF FALLING WEIGHT DEFLECTOMETER DATA WITH THE SHELL DESIGN CHARTS

The charts most frequently used for overlay design are those of the type EN, which are very suitable for the purpose. Because the number of charts had to be limited for practical reasons, there are EN charts for unbound-base-layer thicknesses of 0 and 300 mm only, for climates that have w-MAATs of 12°C, 20°C, and 28°C only. For other thicknesses of unbound-base layers and other

Figure 18. Example design charts: (a) type HN, (b) type HT, (c) type TN, and (d) type EN.

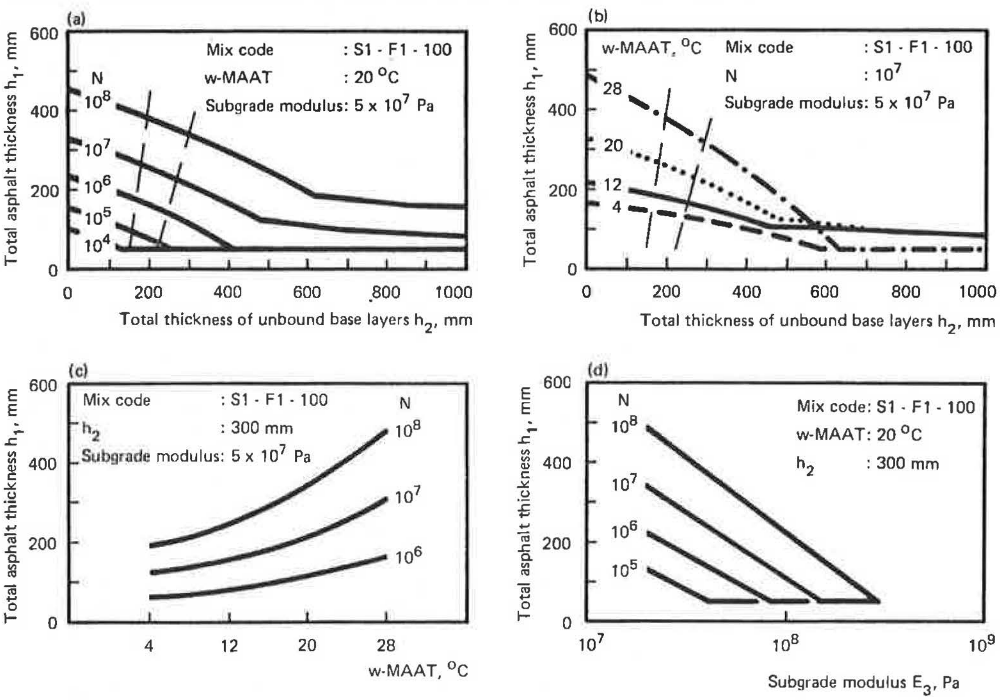
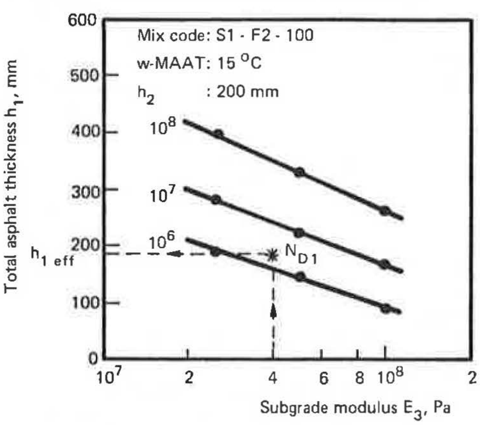


Figure 19. Determination of original design life.



climates, therefore, charts of the type EN must be constructed by interpolation of values read from charts of the type HN, HT, or TN. (The manual makes provision for this by supplying blank EN charts in the form of EN chart grids on a transparency.)

If the value of h_2 is known or can be assumed and the values of E_3 and $h_{1\,eff}$ of the pavement can be established, for example, from interpretation of the FWD measurements, the original design life of the pavement can be determined from a chart of the type EN.

This is illustrated in Figure 19 for a climate that has a w-MAAT of 15°C (which means that interpolation is necessary). If the mix code of the asphalt of the existing pavement is S1-F2-100, h_2 = 200 mm, the calculated E_3 = 40 MPa, and the calculated $h_{1\,eff}$ = 190 mm, then the original design life of that pavement must have been 2 000 000 passes of an 80-kN standard axle load. The number of standard axle passes to date can be calculated from data on the traffic intensity and axle-load distribution by using the weighting factors shown in Figure 16. The residual life of the existing pavement can then be established, and it can be determined whether this residual life suffices or whether an overlay is needed to produce the required future service life.

If an overlay is required, three separate calculation procedures are required:

1. One for an overlay-thickness design based on the subgrade-strain criterion;
2. One for an overlay-thickness design based on the asphalt-fatigue-strain criterion, taking into account the design life already consumed by the traffic to date; and
3. One to check that the thicknesses derived by the first two calculations do not exceed the thickness required when the existing pavement is taken as having deteriorated to such an extent that it must be regarded as an addition to the unbound-base layers only.

If the "check" thickness is found to be less than the larger one resulting from the first two procedures, it should be used because it is the most economical one, while still being adequate from the point of view of structural strength. If it is not less, the larger thickness from the first two procedures should be used.

Because subgrade strain manifests itself as a permanent deformation that is automatically eliminated by any overlay thickness, no allowance need be made for traffic passed to date and the original design life (from the point of view of subgrade strain) is restored and even increased. But if, in the existing pavement, the asphalt strain was not critical, it is possible that, under certain combinations of design parameters, it may become critical in the overlaid structure. The reason for this is that the maximum asphalt strain will occur at the underside of the existing asphalt pavement, both in the original and in the overlaid structure. In both cases, allowance should be made for the traffic passed to date, because this has consumed part of the asphalt fatigue life of the existing pavement.

The way that the consumption of asphalt fatigue life is accounted for is by substitution of a design number (N_{D2}) for the actual number of standard axle-load repetitions (N_{A2}) to be expected in the required future design period. The relationship between N_{D2} and N_{A2} can be derived as follows: If the original total design life (on the basis of asphalt strain) is N_{D1} and the number of standard axle-load repetitions that has occurred to date is N_{A1}, the relative consumption of life is N_{A1}/N_{D1}. The relative residual fatigue life is then $1 - N_{A1}/N_{D1}$.

This relative residual fatigue life can be consumed by the number of future standard axle-load repetitions (N_{A2}). If N_{A2} exceeds the absolute residual fatigue life ($N_{D1} - N_{A1}$), it is clear that an overlay is necessary. However, in the overlaid structure, the maximum asphalt strain will occur in the same place as in the existing structure (i.e., at the underside of the asphalt layer). If N_{A2} were used directly in the charts, the asphalt thickness obtained would be that for a new pavement, as though the existing pavement had not suffered any damage.

For the purposes of overlay design, therefore, the notional design number (N_{D2}) is introduced. This design number can be regarded as a fictitious number of expected future standard-axle-load repetitions that incorporates an allowance for the proportion of the fatigue life of the existing pavement that has already been consumed.

The value of N_{D2} is derived from the known data on the basis that the relative consumption of fatigue life by N_{A2}, expressed as N_{A2}/N_{D2}, is equal to the relative residual fatigue life $(1 - N_{A1}/N_{D1})$. Thus,

$$N_{D2} = (N_{D1} \times N_{A2})/(N_{D1} - N_{A1}) = N_{D1} \times N_{A2}/N_R \qquad (3)$$

Subsequently, N_{D2} is handled in the same way as N. Once the maximum, required, total future asphalt thickness is known, the overlay thickness can be obtained simply by subtracting the existing asphalt-layer thickness therefrom.

The check calculation should be made to ensure that the overlay thickness thus obtained not only enables the pavement to carry the future traffic but also involves no unnecessary application of asphalt. This situation could arise in an overlay design governed by the asphalt-strain criterion in which the existing pavement is quite close to the end of its fatigue life, i.e., N_{A1} is approaching N_{D1}. In this case, the overlay thickness necessary to limit strains occurring at the underside of the existing pavement to such a degree that it will last out the future number of load repetitions may be excessively large.

EXAMPLES OF OVERLAY DESIGN

The procedure described above can best be explained by examples.

Example 1

Assume that FWD or wave-propagation measurements have shown that an existing pavement has an E_3 of 60 MPa and an $h_{1\,eff}$ of 250 mm and that investigation of a core taken from the pavement indicates that the mix code can be designated as S1-F2-100, and that this is also the code for the type of mix to be used for the overlay. The climate of the location can be represented by a w-MAAT of 18°C. Old records indicate an h_2 of approximately 200 mm.

Because the design manual gives specific charts of the type EN for w-MAATs of 4°C, 12°C, 20°C, and 28°C only, it will be necessary to develop a chart of the type EN for a w-MAAT of 18°C by interpolation of data given in charts of the type HT at a w-MAAT of 18°C. This should be done for the two design criteria (subgrade strain and asphalt strain) separately for various design lives in terms of a number of standard axle passes (see Figure 20). Figure 20a shows the interpolated design chart that gives the required asphalt thicknesses based on the subgrade-strain criterion, and Figure 20b shows the design chart based on the asphalt-strain criterion.

Inserting the FWD results (E_3 = 60 MPa and $h_{1\,eff}$ = 250 mm) into Figure 20a gives point A, which indicates that the original design life based on the subgrade-strain criterion of the pavement (N_{D1s}) was 18 000 000 standard 80-kN axle loads. It can be calculated from traffic data that N_{A1} is 15 000 000. This means that the residual life in terms of standard axle passes (N_{Rs}) based on the subgrade-strain criterion will be 18 000 000 - 15 000 000 = 3 000 000.

Let it be assumed in this example that the road authority requires an overlay that will make the road pavement last for another 30 000 000 standard axle loads (i.e., N_{A2} = 30 000 000). It is obvious that the residual life of the existing pavement is insufficient. Point B in Figure 20a shows that a total asphalt thickness of h_1 = 290 mm will be required. Because there is already an $h_{1\,eff}$ of 250 mm and any overlay will automatically eliminate all ill effects of the old surface, an overlay thickness (h_0) of 290 - 250 = 40 mm will be sufficient from the point of view of the subgrade-strain criterion.

In the old construction, the maximum asphalt strain occurred at the bottom of the asphalt layer, and this will still be the case after the overlay has been applied. It must now be checked whether this asphalt strain will become the governing criterion during the future life of the pavement. For this purpose, the same FWD results are used in Figure 20b. This gives point C, the original asphalt fatigue life (N_{D1a}) of 30 000 000 standard axle loads, which is thus not the governing criterion. Therefore, the residual asphalt fatigue life (N_{Ra}) will be 30 000 000 - 15 000 000 = 15 000 000 standard axle loads, which is insufficient for the future design period.

Thus, a fictitious number of standard axle loads for the future design period (N_{D2}) must now be calculated by using Equation 3: $N_{D2} = N_{D1a} \times N_{A2} \div N_{Ra}$ = 30 000 000 × 30 000 000 ÷ 15 000 000 = 60 000 000 standard axle loads. The asphalt thickness required for a design number of 60 000 000 can then be determined from Figure 20b (point D) and is found to be 280 mm. Thus, the required overlay thickness from the point of view of the asphalt-strain criterion (asphalt fatigue) is 280 - 250 = 30 mm, which is less than that required on the basis of subgrade strain. This leads to the conclusion that the subgrade strain is the governing criterion and that the required overlay thickness is 40 mm.

Because the subgrade strain remains decisive, it is not necessary to check the approach in which the old asphalt layer is regarded as having deteriorated to the extent that it can be taken as part of the unbound base. Had this check been made by using, for example, an HT chart, a required (overlay) thickness of more than 200 mm would have been found, proving that the old pavement has a significant residual value. The overlay thickness required thus remains 40 mm.

Example 2

Assume that deflection measurements have shown that an existing pavement has an E_3 of 70 MPa and an $h_{1\,eff}$ of 200 mm. This time the mix code of the existing pavement and the intended overlay can be designated as S1-F2-50, and h_2 is again 200 mm. The climate is represented by a w-MAAT of 18°C. By interpolation from HT or HN charts, charts of the type EN can be developed as shown in Figure 21. Figure 21a shows the required asphalt thicknesses based on subgrade strain, and Figure 21b shows those based on asphalt strain.

Inserting the FWD data (E_3 = 70 MPa and $h_{1\,eff}$ = 200 mm) in Figure 21a gives a value of N_{D1s} of 30 000 000 standard axle loads. If the traffic to date is again assumed to be 15 000 000 standard axle passes, the re-

Figure 20. Interpolated design chart: example 1—(a) based on subgrade-strain criterion and (b) based on asphalt-strain criterion.

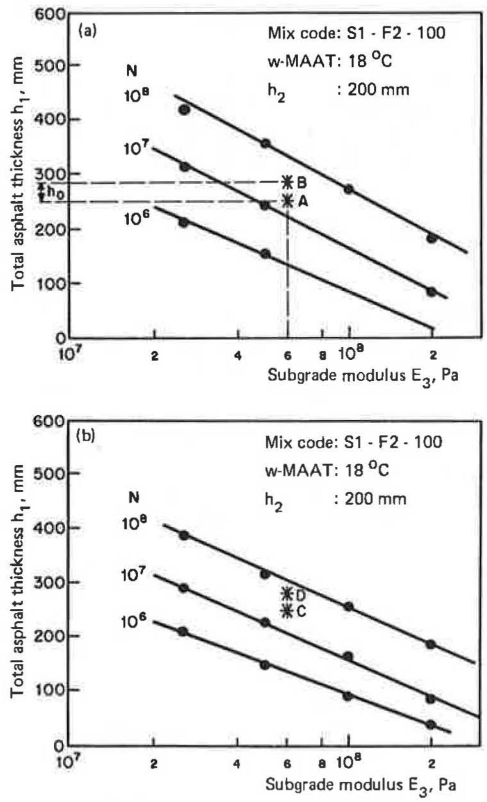


Figure 21. Interpolated design chart: examples 2 and 3—(a) based on subgrade-strain criterion and (b) based on asphalt-strain criterion.

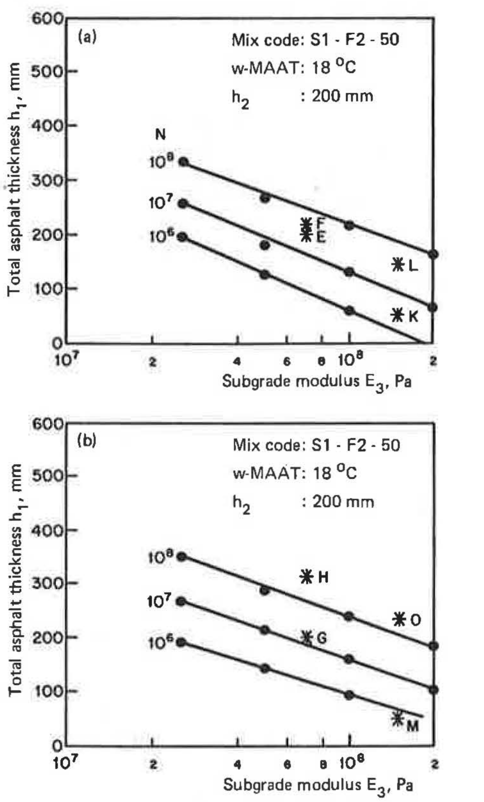
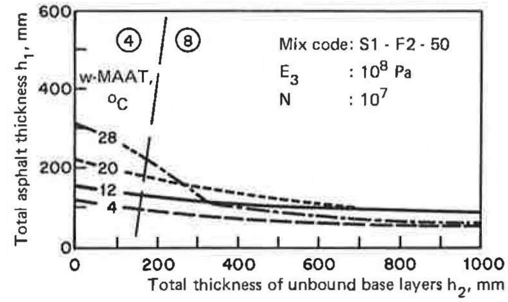


Figure 22. Calculation of overlay thickness by using chart HT 58 and assuming complete failure of existing pavement: example 2.



sidual life (N_{Rs}) is 15 000 000, which, for a required future life (N_{A2}) of 40 000 000 standard axle passes, gives a required total asphalt thickness of 210 mm (point F) and hence a required overlay thickness of 210 - 200 = 10 mm, from the point of view of the subgrade-strain criterion.

Inserting the same FWD data in Figure 21b (point G) gives a value of N_{A1a} of 17 000 000 standard axle passes, which thus is not decisive in the design of the original pavement. Therefore, N_{Ra} = 17 000 000 - 15 000 000 = 2 000 000 standard axle loads, and $N_{D2} = N_{D1a} \times N_{A2} \div N_{Ra}$ = 17 000 000 × 40 000 000 ÷ 2 000 000 = 340 000 000 standard axle loads. The required total asphalt thickness for the design number N_{D2} = 340 000 000 can be determined from point H in Figure 21a and is 320 mm. The required overlay thickness from the point of view of the asphalt strain is thus 320 - 200 = 120 mm. This means that the asphalt strain becomes decisive for the overlaid structure, even though this was not the case in the original pavement.

This time the checking procedure must be carried out to establish whether it would be more economical to consider the old pavement as having deteriorated to the point of failure and to regard the old asphalt layer as part of the unbound-base material. A construction that has 400 mm of base requires an asphalt thickness of more than 120 mm, as can be determined from chart HT 58, shown in Figure 22, which is the standard chart most closely corresponding to the design parameters of the example. Apparently, it is advantageous to use the residual (fatigue) life of the old pavement; the required minimum overlay is 120 mm.

Example 3

Assume, this time, an E_3 of 150 MPa, an $h_{1\,eff}$ of 50 mm, an h_2 of 200 mm, and a climate that has a w-MAAT of 18°C. The interpolated EN charts used for example 2 can again be used (Figure 21). Assume also that N_{A1} = 600 000 standard axle loads and that N_{A2} = 40 000 000 standard axle loads.

Inserting the FWD data (E_3 = 150 MPa and $h_{1\,eff}$ = 50 mm) into Figure 21a shows that N_{D1s} = 3 000 000 standard axle loads (point K). Thus, N_{Rs} = 3 000 000 - 600 000 = 2 400 000 standard axle loads and, from Figure 21a, the required total asphalt thickness for the future design life is 150 mm (point L). Therefore, the required overlay thickness from the point of view of the subgrade-strain criterion is 150 - 50 = 100 mm.

Inserting the same FWD data into Figure 21b shows that N_{D1a} = 800 000 standard axle loads (point M). This means that $N_{Ra} = N_{D1a} - N_{A1}$ = 200 000 standard axle loads. The fictitious design number (N_{D2}) for an N_{A2} of 40 000 000 standard axle passes can be calculated as follows: N_{D2} =

Figure 23. Effect of writing off existing pavement on overlay thickness: example 3.

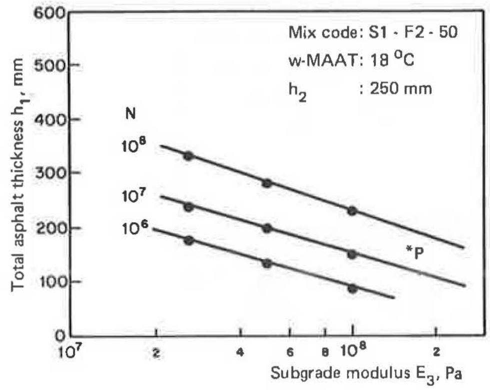
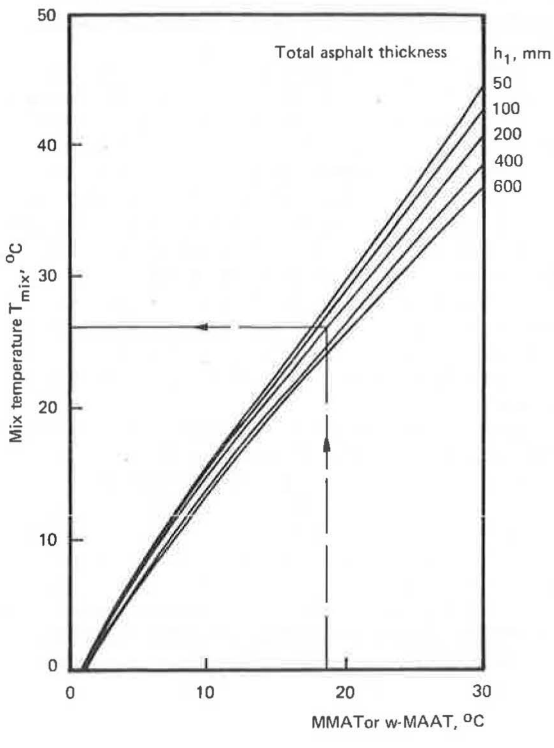


Figure 24. Determination of effective asphalt temperature by using chart RT.



800 000 × 40 000 000 ÷ 200 000 = 150 000 000 standard axle loads. This design number (Figure 21b) means that a total asphalt thickness of 230 mm (point O) is required and that the required overlay thickness from the point of view of the asphalt-fatigue criterion will be 230 - 50 = 180 mm.

Thus, the asphalt strain was the governing criterion in the original pavement and will remain so in the overlaid structure. This is self-evident because the maximum strain occurs in the same place both before and after overlaying, i.e., at the underside of the original asphlat layer. In this case, therefore, the separate determination of the required overlay thickness from the point of view of subgrade strain could actually have been omitted.

What remains to be done is to check whether it is advantageous to regard the old pavement as having failed completely. If it is so regarded, the total thickness of the unbound base becomes 200 + 50 = 250 mm. Another interpolated EN chart can now be developed, this time for an h_2 value of 250 mm. This chart (see Figure 23) is found, not surprisingly, to be based on the asphalt strain. The subgrade-strain criterion simply is not decisive under these circumstances. The required asphalt thickness is 170 mm (point P).

Hence it is economical, although not very much so, to write off the old pavement completely and apply an overlay of 170 mm, as compared with the 180 mm that would be needed to prevent the existing pavement from cracking.

OVERLAYS OF DIFFERENT MIX TYPES

There will be many cases in which the overlay will be of a different mix type from that used in the existing pavement. This does not immediately invalidate the approach that assumes one mix type for the entire construction, i.e., existing pavement and overlay together. If, for example, the difference is one of composition only, the combination of different grades of bitumen could very well produce a mix that has nearly the same stiffness level at the temperatures present in the pavement in the given climate.

For example, a structure approximately 200 mm thick in a climate that has a w-MAAT of 18°C has an effective asphalt temperature of about 26°C (this value was obtained from chart RT of the manual, as shown in Figure 24). At 26°C, a mix that has a stiffness characteristic S2 but is made with 50 pen bitumen produces the same stiffness level as a mix that has a stiffness characteristic of S1 but is made with 100 pen bitumen (see Figure 25).

Furthermore, if the fatigue characteristic of the overlay mix differs from that of the existing pavement, it will normally be possible simply to use one fatigue characteristic because the maximum asphalt strain after the overlay has been applied will still occur in the same place, i.e., at the underside of the existing pavement. For example, the thickness of an overlay of an S1-F1-100 mix on an existing pavement of an S2-F2-50 mix can be calculated by assuming the mix code S2-F2-50 for the entire construction.

If the stiffness level of the overlay mix differs significantly (by a factor greater than 2) from that of the existing pavement, the necessary overlay thickness is first determined as described above, on the basis of the mix code of the existing pavement. Then, equivalency factors are calculated that indicate the thickness of the overlay mix that would be needed to replace a given thickness of the mix type of the existing pavement. Briefly, the procedure is as follows:

Because the subgrade strain depends not only on the stiffness level of the pavement on top of it but rather on the combination of stiffness level and layer thickness of the pavement, the procedure for calculating on the basis of the subgrade-strain criterion is fairly simple and straightforward.

Consider, for example, a design whereby a 50-mm overlay is to be laid on top of an existing pavement 180 mm thick and assume a mix code for both of S1-F2-100, so that the total depth of asphalt will be 230 mm (based on the subgrade-strain criterion). The asphalt mix envisaged for the overlay, however, has a mix code of S1-F1-50. For a mix of this type, the total depth required to limit the subgrade strain to the level given by that criterion will be 180 mm, which leads to the conclusion that 180 mm of S1-F1-50 replaces 230 mm of S1-F2-100. The required overlay thickness of 50 mm of S1-F2-100 can thus be replaced by 180 × 50 ÷ 230 = 40 mm of S1-F1-50.

Figure 25. Derivation of mix stiffness from mix temperature by using chart M-2.

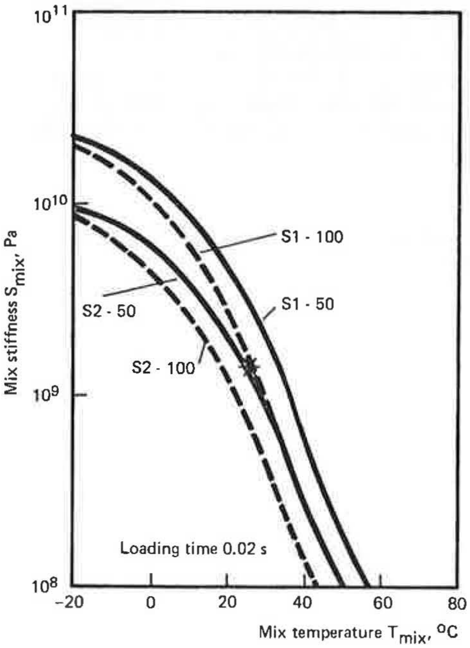
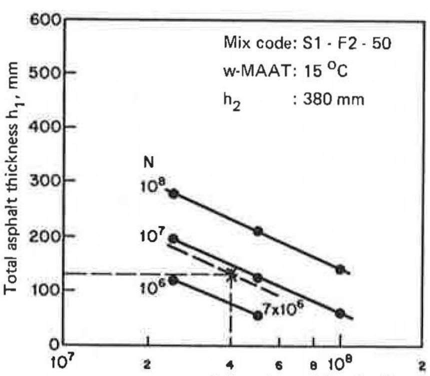


Figure 26. Determination of overlay thickness assuming complete failure of existing pavement (overlay of different mix type).



Again, it must be checked whether the asphalt strain may become the governing factor. Here the matter is complicated by the fact that, on the one hand, the level of permissible strain is governed by the combination of pavement thickness and stiffness level but that, on the other hand, it also depends on the stiffness level itself. Therefore, the equivalency factor must be determined at the same stiffness level. This can be done by comparing the pavements at different temperatures, choosing these temperatures in such a way that both mixes have the same stiffness level and thus the same permissible asphalt strain.

Let us assume that, on the basis of asphalt fatigue life, a total depth of 250 mm is required of a mix that has a code of S1-F2-100 and the existing pavement thickness is 180 mm of a mix that has a code of S1-F2-100. However, the mix intended for the overlay has a code of S1-F1-50, with which the required future design life could be obtained by using a total asphalt depth of only 130 mm. The effective temperature in the pavement can be determined from the w-MAAT and the approximate thickness of the existing pavement plus the overlay (Figure 24).

Suppose that, for this particular pavement, the effective pavement temperature is 22°C. As shown in Figure 25, the mix that has a code of S1-F2-100 has the same stiffness level at 16°C as the mix that has a code of S1-F1-50 has at 22°C. The effective temperature of 16°C in the given approximate pavement thickness corresponds to a w-MAAT of 11°C. According to the charts, this gives a required total thickness of a mix coded S1-F2-100 (at a w-MAAT of 11°C) of 220 mm (based on asphalt fatigue). The equivalency factor is thus 220 ÷ 250 = 0.88, and the required overlay thickness of 250 - 180 = 70 mm of S1-F2-100 mix can be replaced by 0.88 × 70 ≈ 60 mm of S1-F1-50 mix.

In this example, the overlay thickness based on the subgrade-strain criterion was decisive. It still needs to be checked, of course, whether the approach whereby the old asphalt layer is taken as having deteriorated to failure would give a more economical overlay thickness. Figure 26 illustrates that, for the circumstances of this example, a thickness of 130 mm of S1-F1-50 is required on top of an unbound base layer of 200 + 180 = 380 mm, so this is clearly not the case.

CONCLUSIONS

Deflection measurements made with a falling weight deflectometer can provide the road engineer with meaningful data on a pavement structure. From these data, the state of the pavement (e.g., in terms of residual life) can be evaluated in an analytical way, and, if necessary, the structural restrengthening measures (e.g., in terms of overlay thickness) that should be undertaken can be determined. The data provided by the FWD are sufficiently accurate to tailor the design to the individual circumstances but, at the same time, are produced quickly enough for routine investigations.

The charts in the Shell Pavement Design Manual can be used by the designer, without resort to computer calculations, to determine analytically the overlay thicknesses required for a variety of circumstances and can even introduce a large measure of refinement if this is required.

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Mechanistic Method of Pavement Overlay Design

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This paper presents a synopsis of a comprehensive procedure for the rehabilitation design of overlays of both flexible and rigid pavements. The procedure includes an evaluation of the existing pavement by using modern nondestructive deflection testing, condition surveys, and materials sampling and testing. The analytical model on which the computer method is based is elastic-layer theory. This model is used in both the pavement-evaluation and the overlay-thickness analyses. This design and evaluation analysis is unique for various categories of pavement condition. The final overlay thicknesses are selected on the basis of fatigue and rutting criteria where applicable. The entire procedure is automated in a series of three computer programs.

This paper describes the use of a universal procedure for the design of pavement overlays. The detailed development of the criteria for the procedure is discussed and documented in several reports (1-4).

The procedure covers flexible overlays of flexible pavements and both flexible and rigid overlays of rigid pavements. It includes both jointed and continuous rigid pavements and both bonded and unbonded overlays. It covers existing pavements that have remaining life, those that are substantially cracked, and those that are so badly deteriorated that they could be broken mechanically into small pieces. The procedure infers that overlay materials and construction specifications will not differ significantly from those currently used in highway construction. However, it does include some nonconventional materials testing methods.

The comprehensive procedure provides for rehabilitation of existing portland cement concrete (PCC) and asphalt concrete (AC) pavements and is divided into three basic steps: (a) evaluation of the existing pavement, (b) determination of the design inputs, and (c) overlay-thickness analysis. The procedure is illustrated in flowchart form in Figure 1. Evaluation of the existing pavement is accomplished by a condition survey and deflection measurements. This information enables the designer to distinguish among different segments of the existing pavement based on their condition. Each segment becomes a design section and is analyzed separately. Thus, the most economical rehabilitation may involve varying the overlay thickness along the roadway according to the existing pavement condition.

The design inputs include both past and future traffic, environmental considerations, and materials testing and analysis. The results of deflection measurements also serve to aid in establishing properties of the subgrade material.

The overlay-thickness analysis is based on the concepts of failure by excessive rutting (flexible pavements) and by excessive fatigue cracking (rigid and flexible pavements). Stresses and strains in the pavement are computed by using linear elastic-layer theory (5). The overlay life is determined by entering these stresses into a fatigue or rutting equation that relates stress or strain magnitude and repetitions to failure. The overlay thickness that satisfies the fatigue and rutting criteria is selected as the design thickness.

The design procedure is automated in the form of three separate computer programs—PLOT2, TVAL2, and POD1. The programs require the designer to make only minor hand computations, and these are only aids in determining computer-program input data.

GENERATION OF DESIGN-PROCEDURE INPUTS

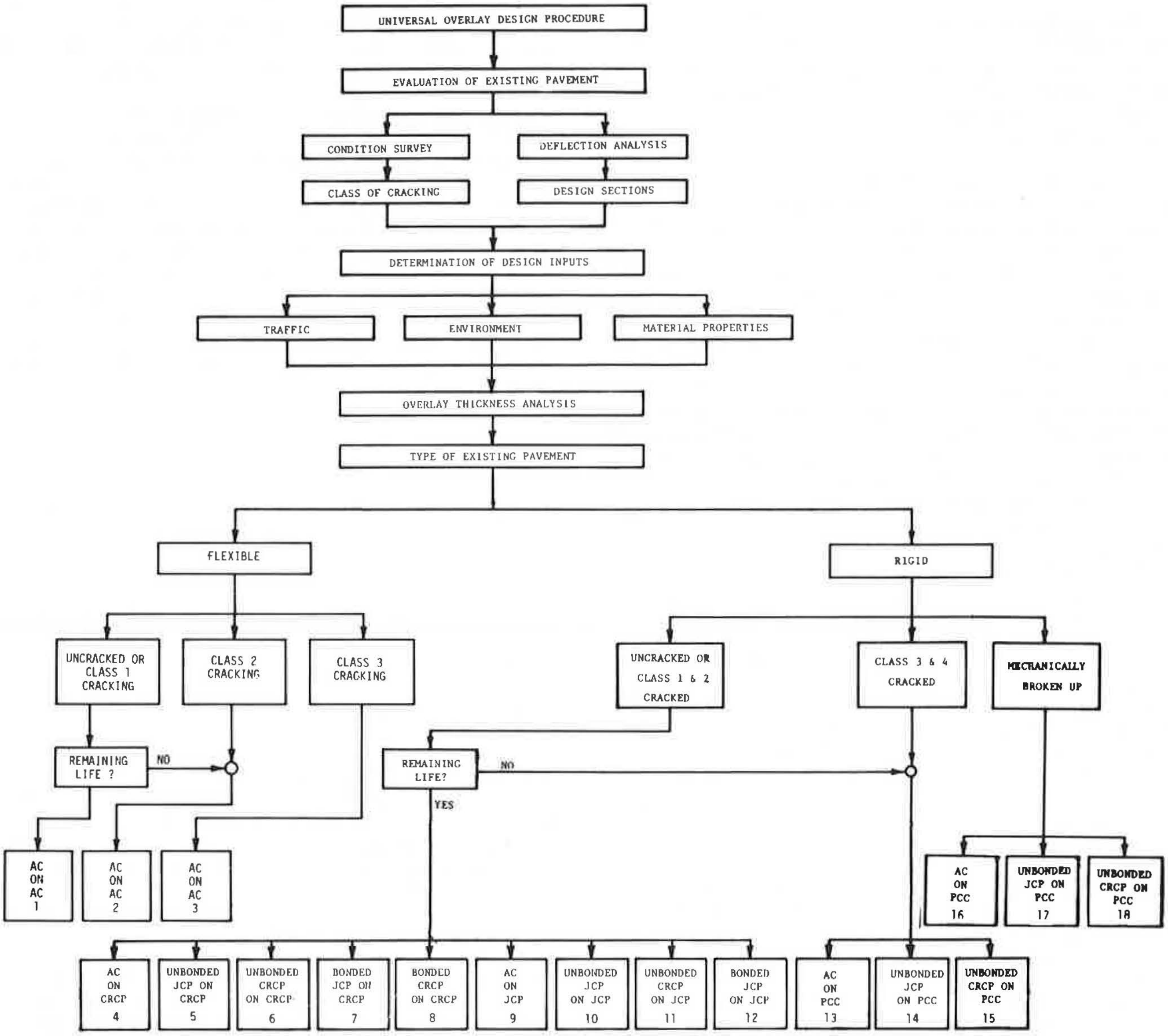
The design procedure requires input from the following three areas: deflection testing, condition surveys, and traffic data.

Deflection Testing

Deflection testing is used to measure the response of the in-service pavement to load. From this behavior pattern, areas that have equal or similar performance and materials properties can be determined.

1. Type of equipment: Any type of deflection-measuring equipment (such as the Dynaflect or the road rater) that gives satisfactory deflection results can be used (6). This type of equipment lends itself to rapid testing, thus making it economically possible to investigate the pavement structure thoroughly. Deflections measured with a Benkleman beam and an 80-kN (18 000-lbf) single-axle load can also be used.

Figure 1. Flowchart of pavement overlay design procedure.



2. Recommended testing conditions: The design procedure is based on measurements that represent the time of year when the deflection values are maximum. It is recommended that the user measure deflections during this time of year. If measurements are made during other seasons, they must be corrected to relate to the worst condition.

3. Sampling frequency and procedure: The recommended testing procedure includes determining at least one deflection profile along the outer wheel path of the existing roadway. The spacing between the measurements should be a maximum of 30.5 m (100 ft). For undivided roadways, it is desirable to obtain two deflection profiles, one in each outside wheel path. The measurements for each profile should be spaced 30.5 m apart, but those for the two profiles should be staggered to provide profile data that has 15.2-m (50-ft) spacings. For divided highways, deflection profiles should be obtained in the outside lanes of both roadways on a staggered basis.

For undivided highways, the two deflection profiles should be combined into one that represents the entire width of roadway. However, for divided highways, the profiles for the pavements on either side of the median should be considered separately. Two profiles will give adequate coverage of most highways. The measurements, however, should be made at locations between cracks or joints in a good portion of the pavement and at regular intervals and so documented. A suggested guideline for spacings for deflection tests is given below (1 m = 3.3 ft).

Type of Location	Spacing (m)
Rolling terrain	30.5
Numerous cut-to-fill transitions	30.5
Level with uniform grading	76.2

In addition to the measurements for the determination of a deflection profile, it is also necessary to make measurements at the corners of a jointed concrete pavement (JCP). These data are used in determining the degree of load transfer. Corner measurements should be kept separate and not included in the deflection profile, but should be made at the same time as the interior measurements.

Condition Surveys

Condition-survey information should be obtained that includes such items as an accurate inventory of the different types and amounts of cracking, rutting, spalling, joint condition, faulting, pumping, and blowups and some inventory of roughness. [Condition-survey techniques are described elsewhere (7).]

1. Cracking in rigid pavements: Cracking is defined and recorded according to the American Association of State Highway Officials (AASHO) definitions, i.e., class 1, class 2, class 3, and class 4 (8).

Class 1 includes fine cracks that are not visible under dry surface conditions by a person who has good vision and is standing at a distance of 4.6 m (15 ft). Class 2 cracks are those that can be seen at a distance of 4.6 m but exhibit only minor spalling such that the opening at the surface is less than 6.4 mm (0.25 in). A Class 3 crack is defined as a crack opened or spalled at the surface to a width of 6.4 mm or more over a distance equal to at least one-half the crack length, except that any portion of the crack opened less than 6.4 mm at the surface for a distance of 0.9 m (3 ft) or more is classified separately. A class 4 crack is defined as any crack that has been sealed.

2. Cracking in flexible pavements: As for rigid pavements, cracking is defined and recorded according to the AASHO definitions. Class 2 cracking (commonly referred to as alligator cracking) is defined as that which has progressed to the stage where the cracks have connected together to form a grid-type pattern. Class 3 cracking is the progression from class 2 in which the class 2 cracks spall more severely at the edges and lose integrity between blocks and the segments of pavement surface loosen and move or rock under traffic.

The condition surveys provide important data for explaining variations in the deflection profiles and also differences in materials properties determined in laboratory investigations. A comparison of the deflection profile and the observed distress should be considered in formulating the materials sampling plan. Furthermore, the type of cracking observed on the existing surface becomes a decision criterion relative to the method of characterization of the existing pavement and the kind of analysis to be performed.

3. Rutting: The rutting measurements in wheel paths on existing AC surfaces are included in the condition survey to give (a) insight into the selection of an allowable rut depth and (b) an estimation of the leveling up that will be required on the existing surface before it is overlaid. It is recommended that the rut depth be measured every 152 m (500 ft) in both wheel paths and that the averages for the two wheel paths be determined for the same pavement lengths as the design sections established from the deflection profiles as described below. These measurements can be made by simple mechanical devices similar to those used at the AASHO Road Test. An alternative method is to lay a stringline or other long straightedge across the wheel path and measure the rut depth with a scale.

4. Environmental data: If the existing pavement is AC, it is necessary to obtain temperature information. The number of days per year that the average daily temperature exceeds 18°C (64°F) must be determined for use in the rutting analysis. These data can usually be obtained from past weather records.

Traffic Information

The traffic information required for the design procedure is described in terms of the number of 80-kN equivalent single-axle loads (ESALs) determined in accordance with the Interim Guide for the Design of Pavement Structures (9). The number of load applications already experienced on the existing surface must be estimated. The number of load applications must also be projected for the anticipated life of the overlay.

If the traffic projection represents the total of all lanes for both directions of travel, the traffic must be distributed by direction and lane for design purposes. Directional distribution is normally made by assigning 50 percent to each direction unless special conditions warrant some other distribution. In regard to lane distribution, the outside lane is generally the controlling lane. If an agency has available lane-distribution factors for facilities that have two or more lanes in each direction, these should be used. If not, the guideline below can be used; if there is doubt as to which factor to apply, it is suggested that the more conservative range be used.

Number of Lanes in One Direction	Lane-Distribution Factor
2	1.0
3	0.8-1.0
More than 3	0.4-0.6

SELECTION OF DESIGN SECTIONS

By using the nondestructive deflection test data, a highway can be divided into different design sections, i.e., areas where the pavement responds differently to load.

Deflection Profiles

The deflection data obtained in the site investigations, excluding joint deflections, are plotted in the form of profiles throughout the length of the roadway as shown in Figure 2. Profiles from separate lanes should be combined according to location or station number. These plots can be made manually or by using the computer program PLOT2.

Preliminary Design Sections

The deflection profiles are divided into areas that have similar deflections. Information from the condition survey can be used as an additional guideline for dividing the profile into sections. These sections should also be compared with cracking surveys to show whether there are differences in deflection and performance of the pavement. Areas that have significantly different cross sections should be assigned different sections of deflection profile.

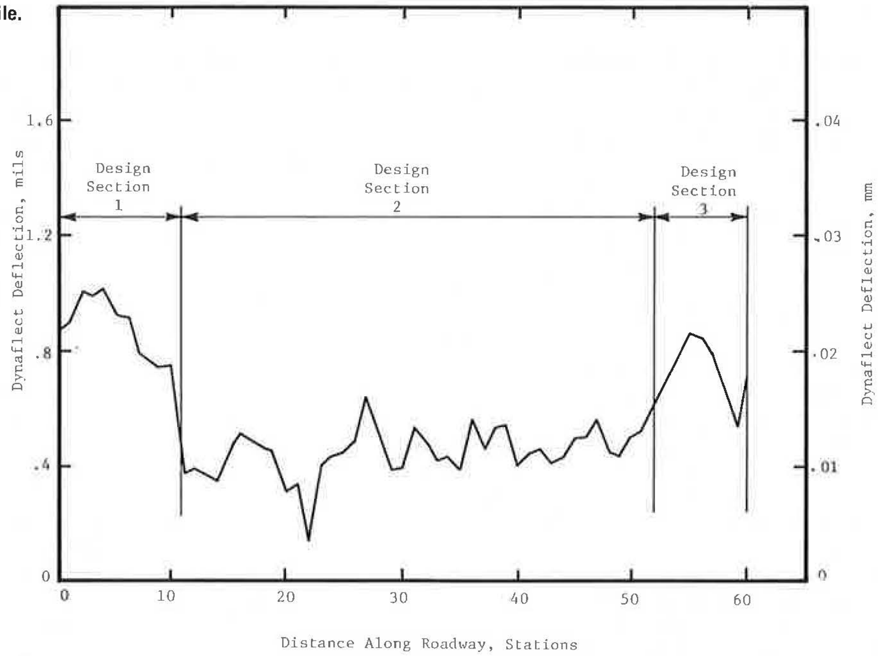
Statistical Hypothesis Testing

Adjacent design sections that have the same cross section should be tested to determine whether they are significantly different or whether they are from the same population of data. This can be done by using a standard statistical method for testing the significance of the difference between two samples, such as the hypothesis tests for equal means (10).

The designer selects the significance level at which the deflection differences are to be tested; a level of 5 percent is recommended for general use. The statistical test can be made either by hand or by use of the computer program TVAL2 (7).

If two adjacent sections are not significantly different, they should be combined into one and that one tested against the next adjacent section. This procedure establishes the design sections, each of which becomes

Figure 2. Sample deflection profile.



a separate design problem.

Determination of Design Deflection

The standard deviations of the deflections are an indication of the variations that exist within the design sections. The design deflection for any given roadway section is a function of the mean deflection, the variation, and the confidence level selected for design:

$$W_\alpha = \bar{w} + zS_{dw} \qquad (1)$$

where

- W_α = design deflection based on Dynaflect or Benkleman beam measurements (in),
- $\bar{W}$ = mean deflection (in),
- z = distance from mean to selected significance level on a normal distribution curve, and
- S_{dw} = standard deviation of mean deflection (in).

(The z-values for use with Equation 1 have been determined for U.S. customary units only.)

The z-values corresponding to various design confidence or reliability levels are given below.

Confidence Level	z-Value
50	0
75	0.674
90	1.282
95	1.645
97.5	1.960
99	2.330

MATERIALS SAMPLING AND TESTING

Sampling Plan

Normally, the design sections should be established before sampling is planned. It is recommended that at least one boring be made in each design section and, for extremely long sections, more than one boring may be desirable.

The borings should include cores of any paving layers that are intact, such as (a) existing asphalt or concrete, (b) cement-stabilized materials, (c) asphalt-stabilized materials, or (d) other chemically treated materials. Any granular or gravel layers should be sampled by collection of augered materials from the drill hole. Unbound materials should be sampled in sufficient quantity for remolding of specimens; this requires the in-place moisture content and the density, which are easily obtained if nuclear equipment is available. For materials where it is possible to push tubes, undisturbed samples should be obtained. The drill hole should be carefully logged so as to accurately document the layer thick nesses in the existing pavement structure. Normally a total depth of 1.5-2.1 m (5-7 ft) is sufficient for pavement borings.

Asphalt Concrete and Portland Cement Concrete Testing

The materials properties required for the AC are its Poisson's ratio and modulus of elasticity; those for the PCC are its Poisson's ratio, modulus of elasticity, and flexural strength.

1. Modulus of elasticity: The dynamic modulus of elasticity of the AC material should be determined. Currently, there is no American Society for Testing and Materials standard for this test, but there are established procedures. The designer should determine the modulus over a range of temperatures and then use the modulus based on his or her selected temperature(s). A temperature of 21°C (70°F) is suggested for design. Recommended procedures are given elsewhere (7). The modulus of elasticity for PCC can be determined according to ASTM C469, and the flexural strength can be determined according to ASTM C78.

2. Poisson's ratio: Normally, tests are not performed for Poisson's ratio because it does not vary significantly. It is recommended that a value of 0.3 be used for AC and of 0.15 for PCC in the design analyses. The overlay-design computer program has default values of 0.3 and 0.15 built in for the Poisson's ratios of asphalt and concrete, respectively.

Base-Material Testing

The moduli of elasticity of all base and subbase materials must be determined. Poisson's ratio tests are not necessary. Default values in the computer program are 0.20 for stabilized bases and 0.40 for granular bases.

1. Unbound materials: Normally, base and subbase materials will be disturbed samples and thus require recompaction. The in-place density and moisture content should be determined if possible and the materials remolded at these values; otherwise, samples should be recompacted at the optimum moisture content and not less than 95 percent of the density corresponding to that moisture content as used for construction control. Base and subbase materials should be tested at confining pressures equal to the overburden pressure unless that is less than 6.9 kPa (1 lbf/in^2), when the tests should be unconfined. The tests should be performed at a deviator stress of 138 kPa (20 lbf/in^2) if the total concrete thickness is 15 cm (6 in) or less or 69 kPa (10 lbf/in^2) if it is greater than 15 cm (7).
2. Subgrade materials testing: Disturbed subgrade samples should be treated similar to base materials. Undisturbed subgrade samples should be tested at confining pressures equal to the overburden and over a range of repeated deviator stresses [e.g., 13.8-82.7 kPa (2-12 lbf/in^2)]. The laboratory tests should be performed at a minimum of four levels of deviator stress; 13.8, 34.4, 55.2, and 82.7 (2, 5, 8, and 12 lbf/in^2) are recommended. A default value of 0.45 is used for Poisson's ratio of the subgrade.

DESIGNATION OF OVERLAY DESIGN CATEGORY

Existing Pavement Classification

The use of the design procedure requires that each design section of the existing pavement be classified into one of the following categories:

1. Remaining-life PCC—a PCC pavement that is uncracked or has class 1 or 2 cracking as defined in the AASHO guide,
2. Cracked PCC—a PCC pavement that has class 3 or 4 cracking as defined in the AASHO guide (the program can change a design section originally in category 1 to this category if the calculated remaining life of the existing pavement is less than a preestablished minimum),
3. Mechanically broken PCC—a PCC pavement in such poor condition that the designer feels it should be broken up to serve as a base material before overlay (repair or removal and replacement of the damaged portions may instead be used to upgrade the section to category 2),
4. Remaining-life AC—an AC pavement that is uncracked or shows less than 5 percent class 2 cracking,
5. Mildly cracked AC—an AC pavement that has more than 5 percent class 2 cracking but less than 5 percent class 3 cracking [if the cracked areas are removed and replaced to meet the conditions specified for category 4, then the analysis for category 4 (remaining life) can be used], and
6. Severely cracked AC—an AC pavement that shows more than 5 percent class 3 cracking (pavements in this category can be upgraded to category 5 or category 4 by appropriate repair or removal and replacement of the damaged portions).

Types of Overlay Analysis

The category assigned to the existing pavement and the types of materials for the existing pavement and the overlay, all of which are required design inputs, determine the type of overlay analysis. In addition, for pavement sections designated as remaining-life pavements, the number of 80-kN ESALs to date affects the internal section of the analysis in the calculation of the fraction of remaining life. If this fraction is less than a preassigned minimum, the section is no longer considered to be a remaining-life case.

A total of 18 overlay analysis types are considered: 9 for PCC remaining-life pavements, 3 for PCC that has class 3 or 4 cracking, 3 for PCC that will be mechanically broken up, 1 for AC remaining-life pavements, 1 for mildly cracked AC, and 1 for severely cracked AC.

If the existing pavement is a remaining-life CRCP, AC, bonded or unbonded jointed concrete pavement (JCP), and bonded or unbonded CRCP are acceptable overlays. If the existing pavement is a remaining-life JCP, AC, bonded or unbonded JCP, and unbonded CRCP are acceptable overlays. If the existing pavement has class 3 or 4 cracking or will be mechanically broken up, AC, unbonded JCP, and unbonded CRCP are acceptable overlays, but bonded JCP and bonded CRCP are not. Only AC overlays are permitted on AC existing pavements. Rigid overlays over existing flexible pavements must be treated as new pavement designs, not as overlays.

USE OF OVERLAY-DESIGN COMPUTER PROGRAM POD1

The program POD1, described in detail elsewhere (7), is used to determine the overlay thickness needed to satisfy the design criteria. For PCC existing pavements, only a fatigue-cracking criterion is used; for AC existing pavements, both fatigue and rutting criteria are used, and the larger of the thicknesses required by the two criteria is used in the final design.

Outline of Program Operation

POD1 performs the following operations:

1. It determines the subgrade modulus under the design load from the design deflection, the measured characteristics of the subgrade soil, and the characteristics of the deflection and design loads.
2. It computes the fraction of remaining life in the existing pavement from stresses in the pavement before overlay, when appropriate.
3. It calculates the stress (strain for AC pavements) in the pavement system for the design load (an 80-kN ESAL) for overlay thicknesses of 7.6-30.5 cm (3-12 in).
4. It determines the fatigue life from the stress or strain for each overlay thickness and the rutting life (life to specified rut depth) for AC pavements in categories 5 and 6 (the rutting model is not applicable to category 4).
5. It plots lifetimes against overlay thicknesses and interpolates for thicknesses corresponding to the design-lifetimes input.

Summary of Input Information

The information needed to determine input values for POD1 is summarized below:

1. The design deflection as determined by using PLOT2, TVAL2, and Equation 1 for the design deflection;
2. The load magnitude, tire pressure, and wheel

configuration of the deflection-measuring device;

3. The condition of the existing pavement surface, i.e., whether or not it is cracked, the type of cracking if present, and whether it will be mechanically broken before overlay;
4. If the existing pavement is JCP, the ratio of the corner deflection to the interior deflection;
5. The presence or absence of voids beneath the existing pavement;
6. The number of 80-kN ESALs the pavement has experienced to date and the number that it is being designed to accept before failure;
7. If the existing pavement is AC, the allowable rut depth before rutting failure is assumed and the number of days per year that have a mean temperature greater than 17°C;
8. The material type, thickness, Poisson's ratio, and modulus for each layer in the existing system;
9. For the subgrade material, the laboratory-determined deviator stresses and corresponding modulus values;
10. If the existing pavement is PCC, the flexural strength;
11. The type of overlay and its modulus, Poisson's ratio, and flexural strength; and
12. The type of bond breaker, if used, and its thickness, modulus, and Poisson's ratio.

The program contains default values for the Poisson's ratio values based on material types and for bond breaker thickness and modulus. If the condition survey has shown the existing pavement to be a class 3 or 4 cracked PCC or one that will be mechanically broken up or a class 2 or 3 cracked AC, the modulus value that is input for the surface layer will automatically be defaulted to a predetermined value.

Program POD1 has been written so that the required data can be input in a simple, yet logical, manner. Problems that deal with nearly similar situations can be stacked by, for each problem, inputting only the directives (data input cards) that contain the item that is changed from the first problem. For any one problem, the directives can appear in any order except that a PROBLEM directive must begin the data for every problem and an END directive must follow the data for the last problem.

Program Execution Information

POD1 requires approximately 50 000 octal (about 21 000 decimal) words of memory on a CDC CYBER 74 or CDC 6600 computer and uses 8-10 s of central processing unit time for a complete problem for a rigid pavement. Flexible pavement problems may take somewhat longer, especially if rutting computations are involved. If the subgrade modulus for the first problem of several stacked together is applicable for the remaining problems, those remaining will execute in approximately 4 s each.

No peripheral equipment is required except a card reader and a line printer. If the program is on a permanent file, it can be executed from a remote terminal; the output is relatively compact and can be printed easily.

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Pavement Evaluation and Overlay Design: Summary of Methods

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Methods of pavement evaluation and overlay design are summarized and compared.

A general framework for pavement maintenance and rehabilitation is shown in Figure 1, and Figure 2 lists a number of available alternatives. Maintenance refers to those processes—both preventive and corrective—that do not involve major alterations in the pavement structure. Rehabilitation encompasses the areas of reconstruction, overlays, and recycling and their combinations and can be used to restore or to improve the serviceability of the pavement structure.

To assist the engineer in deciding what pavement maintenance or rehabilitation to do and when to perform it, pavement performance must be measured on a systematic and continuing basis. In this performance evaluation, both the functional and the structural performance of the pavement system should be considered.

Functional performance describes how well the pavement serves the user, and structural performance is related to its ability to sustain load (which in turn affects its ability to serve the user). If the pavement becomes too rough, for example, it will be difficult for the user to operate the vehicle and functional performance will be unsatisfactory. However, although these characteristics are related, there is currently no well-defined relationship between structural distress and functional performance. Thus, at present, judgment must be used in deciding when structural deterioration will lead to a level of functional performance below that considered reasonable by the user of the facility (which obviously will vary among vehicles, users, and types of facilities).

For street and highway pavements, particularly, it is appropriate to separate the performance-evaluation process into two phases (Figure 1). In the first, termed the network-monitoring phase, condition surveys are used to provide a basis for segregating those pavements that clearly do not require maintenance or rehabilitation from those that may and for which further information is required.

The second phase is composed of more detailed or diagnostic investigations that can provide the data required to determine an appropriate strategy. These include measurements of physical condition (surface characteristics and structural response) and assessment of special problems such as drainage difficulties or thermal cracking.

Visual condition-evaluation procedures described in this Record in the papers by Shahin, Darter, and Kohn for airfield pavements and by Phang for highway pavements are a necessary part of the network-monitoring process and can serve as a guide to the type of maintenance or rehabilitation to be accomplished. Also, as noted by Phang, these surveys can assist in the overall investment-programming process for maintenance and rehabilitation.

The use of visual condition surveys is well established and should be a part of the maintenance and rehabilitation methodology of every organization that has responsibility for pavements.

Structural performance can be measured by a number of nondestructive testing devices that have been developed in recent years and are being used as a part of the overlay-design methodologies of many organizations. The majority of these devices provide some measure of surface deflection. The various devices currently in use are summarized below.

Method by Which Load is Applied	Device	Organization by Which Used
Slow-moving wheel	Benkelman beam	Asphalt Institute, College Park, Maryland
	Traveling deflectometer	California Department of Transportation
	Deflectograph	U. K. Transportation and Road Research Laboratory, Crowthorne, England, and National Institute for Transport and Road Research, Pretoria, South Africa
Vibratory load	Light vibrators Road rater	Kentucky Department of Transportation
	Dynaflect	Utah Department of Transportation, Louisiana Department of Transportation and Development, and the Federal Highway Administration (Austin Research Engineers, Inc.)
	Heavy vibrators	U.S. Army Corps of Engineers Waterways Experiment Station
Falling weight	Falling weight deflectometer	Shell Research B. V., Amsterdam

There is some concern about the use of light vibratory loading for measuring the structural response of heavy-duty pavements. When light vibrators (e.g., Dynaflect and road rater) are used for such pavements, careful interpretation of the results is required, such as that incorporated in the procedure developed for the Federal Highway Administration by Austin Research Engineers (ARE) that recognizes the stress sensitivity of pavement materials and is described by Treybig in another paper in this Record.

One of the purposes of structural evaluation is to provide data for the design of pavement overlays.

Overlay-pavement design can be accomplished by using tests of representative samples of pavement components, deflection measurements at the pavement surface, or a combination of both.

Figure 3 represents a general framework for overlay designs based on deflection measurements. In this type of system, performance considerations are usually limited to a single factor (such as fatigue cracking, rutting, or riding comfort). Each method makes the assumption that, if the specific design factor being considered is adequately controlled, other forms of distress or performance will also be controlled. For example, it may be assumed that, if fatigue cracking is minimized, protection against rutting will be adequate and riding comfort will be satisfactory.

As shown in Figure 3, by using deflection data and condition-survey information, homogeneous analysis

Figure 1. General framework for pavement maintenance and rehabilitation.

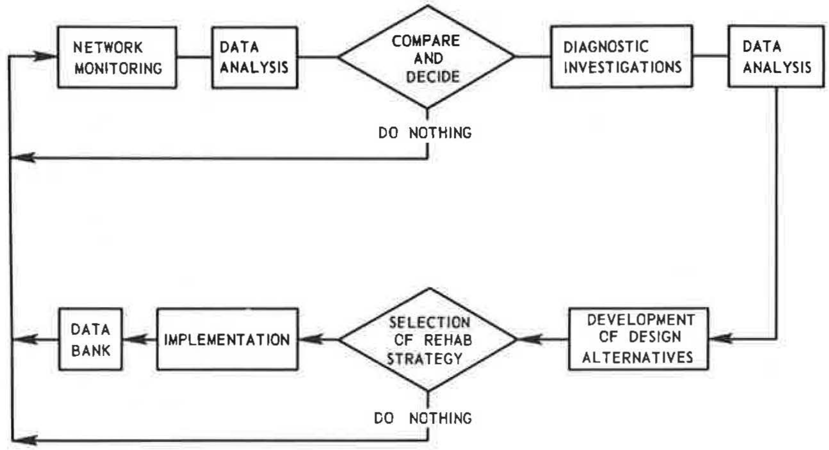
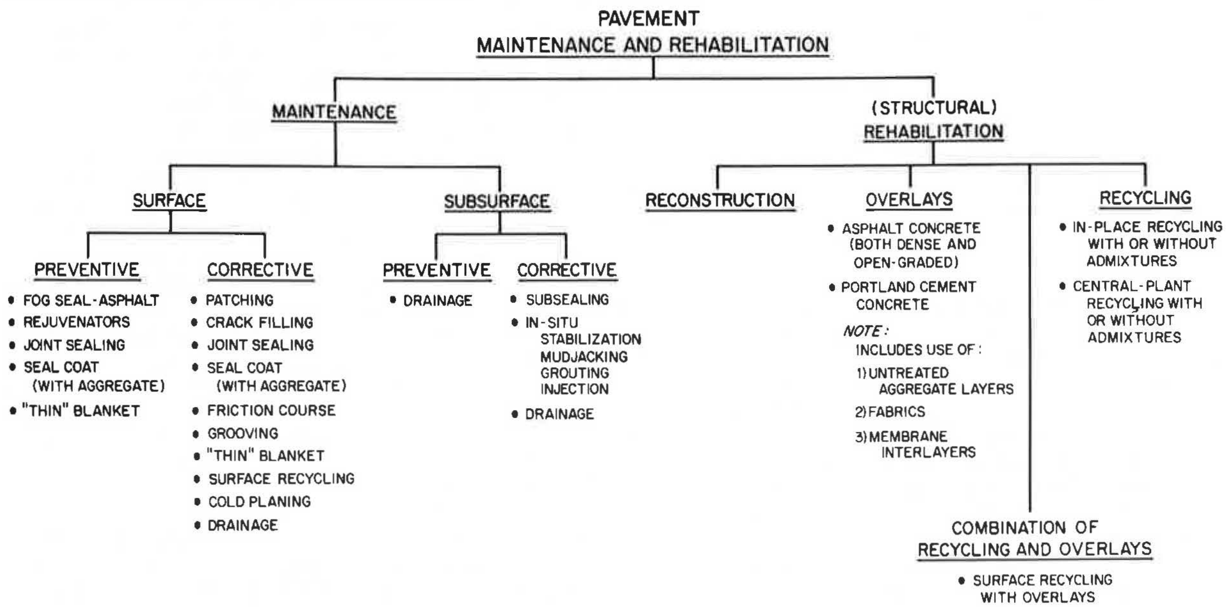


Figure 2. Maintenance and rehabilitation alternatives.



sections that have uniform stiffness characteristics can be identified for further analysis. For each of these sections, representative or design deflections can in turn be established; these are selected to represent the variability of in situ conditions and can be adjusted to a particular season of the year. Representative deflections can be used to estimate the remaining life of a pavement, although little information is currently available to establish the reliability of such predictions. Generally, however, these deflections are used to determine an overlay design by estimating future traffic and selecting the overlay thickness from an appropriate design relationship that is a function of the materials used in the existing pavement and the pavement deflection before overlay. These design relationships have usually been established on the basis of field observations and reflect the reduced deflection resulting from the addition of the overlay.

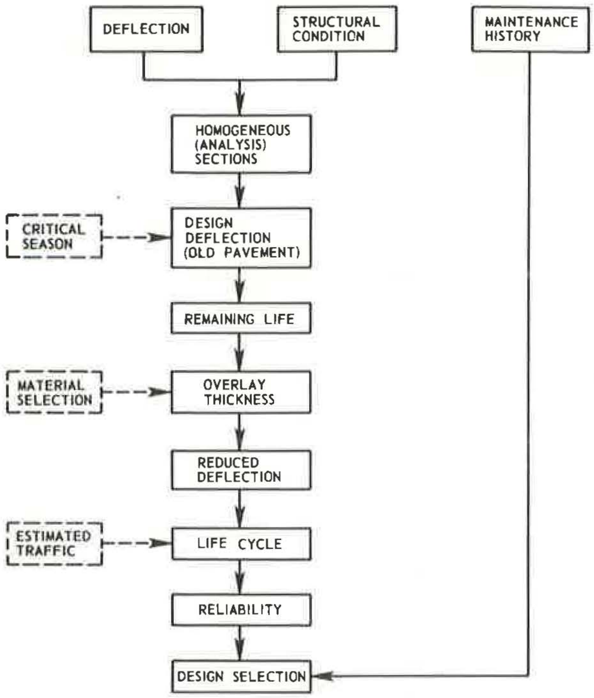
Although such a procedure is comparatively simple, it must be adequately qualified with respect to type of pavement. Moreover, extensive correlations are necessary to extrapolate to increased loading conditions, different materials, and different environments from those for which a specific procedure was developed.

There are also a number of recently developed procedures that follow in some manner the framework shown in Figure 4. It is believed that these procedures can improve the overlay-design process and minimize some of the deficiencies noted above. These procedures build on already established procedures and incorporate the results of recent pavement research. Nondestructive pavement evaluation (for example, deflection measurements) and condition surveys must be made in these procedures. In addition, some measure of the stiffness properties and distress characteristics of the various materials constituting the specific pavement structure are required.

The various procedures provide guidelines for the establishment of representative (analysis) sections, and the Shell and ARE methodologies discussed in the papers by Koole and Treybig, respectively, in this Record provide criteria based on statistical treatment of the deflection data.

In the Shell procedure; the Kentucky procedure described in the papers by Sharpe, Southgate, and Deen and by Southgate, Sharpe, and Deen in this Record; and the U.S. Army Corps of Engineers Waterways Experiment Station (WES) procedure described in the paper by Weiss in this Record, the material characteristics are deduced from the deflection measurements. In the

Figure 3. Framework based on deflection measurements for overlay design.



WES procedure, the expression for subgrade stiffness is estimated from the relationship: subgrade stiffness (in lbf/in^2) = 1500 × the California bearing ratio, which is only a very approximate relationship. For example, in its original development, the stiffness varied from 750 to 3000 times the California bearing ratio.

Of the procedures discussed in this Record, only the ARE method requires that in situ samples of materials be obtained and stiffness determinations be made in the laboratory. Moreover, this procedure uses both the deflection data and the laboratory-measured properties to ensure that the stiffness values obtained for the pavement components are reasonable.

The other procedures deduce in some manner the subgrade characteristics from deflection measurements by using the center deflection together with some offset value(s). For pavements that contain granular layers, the Shell procedure uses the same assumption relative to the ratio of base to subbase stiffness as used in its design methodology for new pavements. The others attempt to determine the subgrade stiffness through a form of solution of the multilayer elastic system that uses the surface deflection.

There is not yet adequate evidence to indicate that pavement properties can be ascertained from deflection-basin measurements. Thus, it is important that stiffnesses of in situ materials also be determined in the laboratory to permit comparisons to be developed. It is possible that, when such data become available, the technique of deducing properties from nondestructive test data will be proved valid but, until such evidence is available, laboratory testing should be an important part of the evaluation process.

It is important, until more data become available, to measure not only the stiffness of the subgrade but also the stiffnesses of the other pavement layers, particularly untreated and treated granular bases and subbases.

Various models are used to represent pavement response to load. In a number of the procedures that follow the framework shown in Figure 3, e.g., the WES dynamic-stiffness-modulus method, thickness-selection procedures based on existing methods are used as a part of the overlay-thickness-selection process.

In four procedures—those of Shell, WES, ARE, and Kentucky—the pavement is represented as a layered-elastic solid, and computer solutions such as the CHEVRON and BISAR programs are used to estimate stresses and deformations.

In the WES procedure, there is some attempt to consider dynamic effects as well, which would permit improved estimates of material properties to be deduced from the dynamic vibratory measurements made by using the heavy vibrator. However, this approach, while interesting, is not considered to be capable of implementation at present.

In using the methods of overlay design illustrated in Figure 4, distress criteria must be established for the various components of the pavement structure. For asphalt pavements, load-associated cracking (fatigue) of the asphalt-treated layer is controlled by the magnitude of the tensile strain that is repeatedly applied and rutting is controlled by limiting values of subgrade strain (Shell, Kentucky, WES, and ARE procedures), of stresses in the other layers (ARE procedure), and of strain in the asphalt-bound layer (Shell procedure). For portland cement concrete pavements, load-associated cracking is controlled by the magnitude of the repeatedly applied tensile stress.

In the Shell and ARE procedures, remaining life can be estimated in existing pavements by using the linear summation of cycle ratios as the cumulative-damage hypothesis. For this analysis to be effective, however, good traffic information is required! In addition, when this approach is used in designing an overlay for an existing pavement, one must be careful if the remaining life of the existing pavement approaches relatively small values (e.g., 10 percent or less). In this situation, as demonstrated in the Shell procedure, a thinner overlay may result if the existing pavement is considered to be part of the granular layer.

Both the Shell and ARE procedures make provision for the treatment of an existing, cracked asphalt concrete layer. In the Shell procedure, such a layer is treated as a part of the granular layer and, in the ARE procedure, different levels of stiffness modulus are assigned that depend on the extent of cracking.

The procedure illustrated in Figure 4 has not been limited to asphalt pavements in the WES and ARE procedures. The ARE procedure, particularly, includes provision for a range of types of existing portland cement concrete pavements.

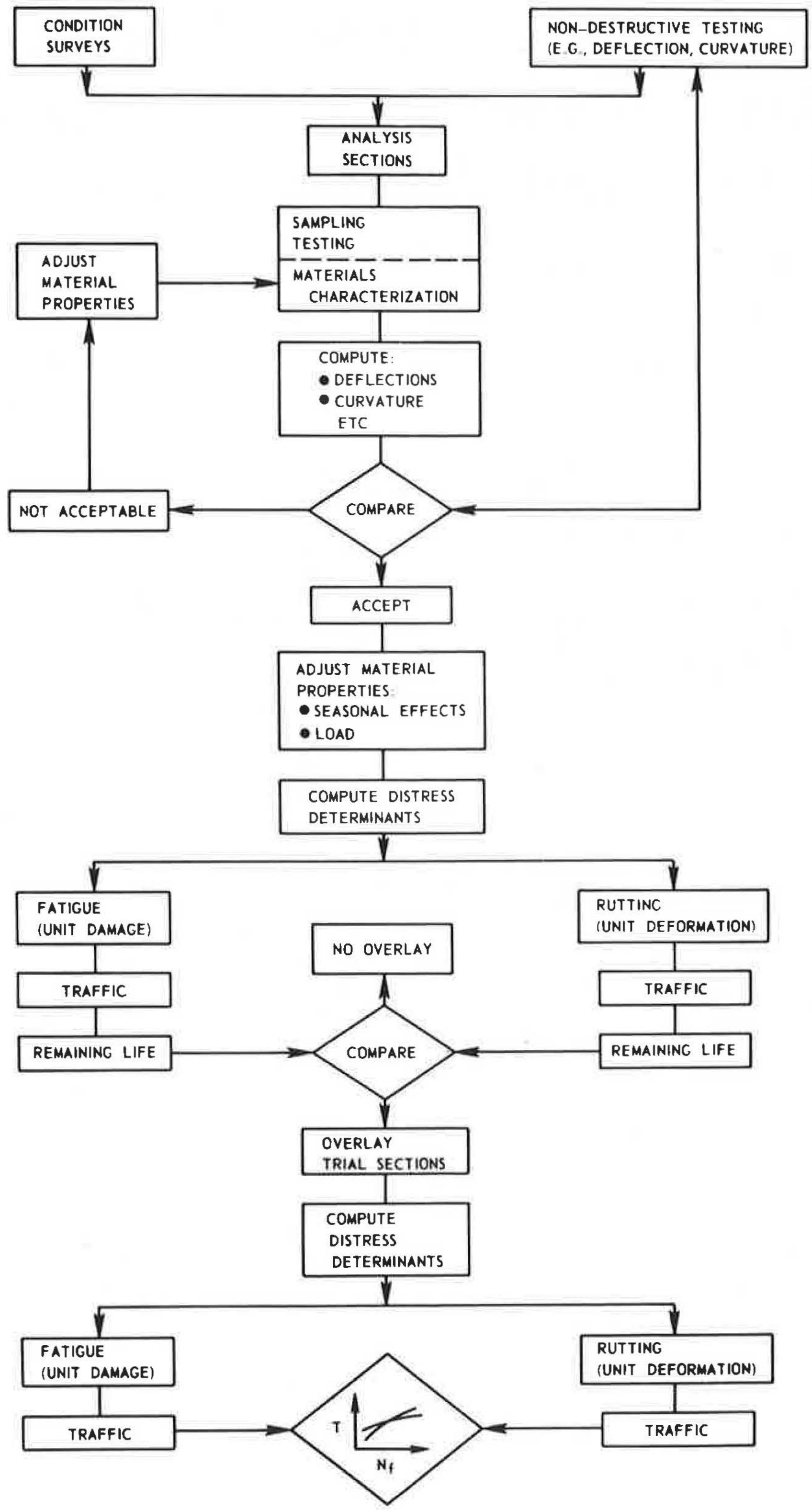
Although considerable progress has been made, there are a number of important problems that must be solved in order to develop improved evaluation and overlay-design techniques.

More effort should be directed toward developing comparisons between stiffness properties estimated by deflection or other nondestructive measuring techniques and laboratory-determined stiffness values. Of the methods compared here, only the ARE procedure uses laboratory tests on samples of in situ materials.

Considerable effort should be directed toward solving the reflection-cracking problem in overlays. Some attempt has been made in the ARE procedure, this must be considered a very crude first attempt and, while noteworthy, must be used with considerable caution.

There is a lack of performance data for overlays. Because this type of information requires time to accumulate, if it is not already being done, efforts should be directed immediately toward this aspect of performance evaluation.

Figure 4. Improved framework for overlay design.



However, there are also many positive features in the overlay-design procedures discussed in this Record. Well-defined techniques are available for the performance of condition surveys. Guidelines are available for the delineation of analysis sections (Figures 3 and 4) by using deflection measurements and statistical treatments of deflection area. It is evident that the general framework for overlay design that follows the format of Figure 4 and is embodied in the ARE, Kentucky, and Shell procedures for highway pavements and in the Shell and WES procedures for airfield pavements will lead to improved designs. Although there are limitations in this methodology, the potential for better-engineered pavements by using this emerging technology has already been demonstrated.

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Procedure for Design of Overlays for Rigid Pavements for Texas State Department of Highways and Public Transportation

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The Texas State Department of Highways and Public Transportation method for the design of overlays for rigid pavements described in this paper is a design method that consists of fatigue- and reflection-cracking subsystems. The fatigue-cracking subsystem considers the remaining life of the existing pavement, uses fatigue principles, and determines the required overlay thickness for a specific design life. Miner's linear damage hypothesis is used in the process. The reflection-cracking subsystem provides a rational way of analyzing an overlay for the possible occurrence of reflection cracking. This design procedure was developed by adapting (through evaluation, modification, improvement, and simplification) the recently developed Federal Highway Administration overlay-design procedure for rigid pavements. The revisions include modifications to (a) the computer programs, (b) the input guides for the computer programs, and (c) the materials-characterization procedures. This procedure provides a rational way to design a wide variety of overlays on rigid pavements.

Many of the pavements in the Interstate system are approaching the end of their originally programmed design lives. In recognition of the fact that rehabilitation and overlays will become increasingly important in the future, the Federal Highway Administration (FHWA) recently sponsored a research effort (1) to develop a new method of overlay design. This method had the following goals:

1. To develop overlay thickness design procedures for rehabilitation of all common pavement types and
2. To develop design procedures for eliminating or reducing reflection cracking of pavement overlays.

Subsequently, the Texas State Department of Highways and Public Transportation (SDHPT), with the goal of developing and implementing design, construction, and rehabilitation methods for rigid pavements, modified and adapted the FHWA method for flexible and rigid overlays on rigid pavements for specific use in Texas (2).

This paper highlights the main features of the resulting Texas SDHPT rigid-pavement overlay-design method, discusses some of the interesting results of the evaluation of the FHWA method, touches on some of the features included in the User's Manual for the Texas method, and describes the usefulness of this procedure as a research and design tool.

OVERVIEW OF DESIGN METHOD

The design method discussed herein (2) is based on an FHWA procedure developed by ARE Inc. (2, 3). The FHWA method was first evaluated and then modified, simplified, and adapted to Texas needs. The procedure, which is outlined in Figure 1, contains three basic steps:

1. Evaluation of the existing pavement,
2. Determination of the design inputs, and
3. Overlay thickness analysis.

Evaluation of the Existing Pavement

The existing pavement is evaluated by a deflection survey and a condition survey. The deflection-survey information is used to divide the roadway under consideration into design sections that will behave differently from one another under load and to select design deflections for each section. The condition-survey information is used to classify the pavement into one of three categories:

1. Pavements that have remaining-life potential,
2. Pavements so severely cracked that they would not be considered to have remaining life, and
3. Pavements that will be mechanically broken up before being overlaid.

For pavements where reflection cracking is a problem, additional condition-survey information is needed, such as differential vertical deflection and the amount of horizontal movement with temperature change at joints or cracks.

Determination of Design Inputs

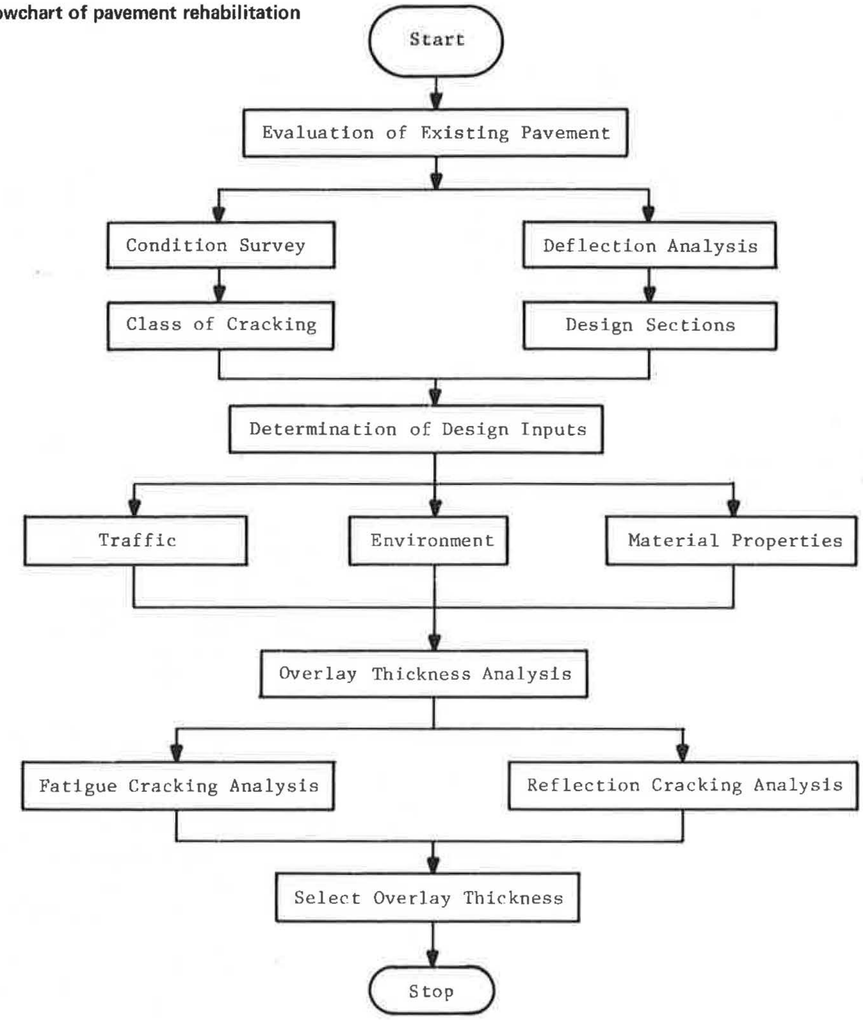
The required design inputs include estimations of past and projected future traffic in terms of number of 80-kN [18 000-lbf/in^2 (18-kip)] equivalent single-axle loads (ESALs), environmental considerations, material properties, and dimensions of layers. Elastic properties of various pavement materials are determined in the laboratory. Deflection measurements and a laboratory determination of the resilient modulus at different levels of deviator stress are used to characterize the subgrade material. For the reflection-cracking analysis, additional data such as creep modulus of asphalt materials, thermal coefficients, and temperature information are also required.

Overlay Thickness Analysis

The overlay thickness analysis involves two subsystems: (a) a fatigue-cracking analysis and (b) a reflection-cracking analysis.

The fatigue-cracking analysis involves the use of linear elastic-layer theory to characterize the subgrade material and to compute stresses, strains, and deflections. McCullough (4) showed in 1969 that "a computer-oriented solution to layered theory is the most appropriate solution for overlay design." He found that the results of the layered-theory approach were favorably correlated with the results of the generally accepted Westergaard theory over a wide range of parameters for rigid pavements. The remaining life of the existing pavement is taken into account by using Miner's linear

Figure 1. Flowchart of pavement rehabilitation procedure.



damage hypothesis. The governing stresses are assumed to be the horizontal tensile stresses due to applied wheel loads; they are assumed to be at the bottom of the overlay for pavements that do not have remaining life and at the bottom of the existing pavement for pavements that do. Stresses computed by the linear elastic-layer program are taken to be interior stresses, and stress factors derived by using a discrete-element theory program [Westergaard and Picket theory (1)] are used to determine the maximum stress at the critical point for a specific type of pavement-overlay combination. Continuous pavements are designed for edge loading, and jointed pavements are designed for corner loading. Void factors derived by using slab theory (1) are used to account for increased stresses due to voids under the pavement.

The fatigue-cracking analysis is computerized by the program RPOD2, which can handle both asphalt concrete (AC) and portland cement concrete (PCC) overlays on concrete pavements. The output of the program is the required overlay thickness for a specific design life.

The reflection-cracking analysis is primarily intended for AC overlays on rigid pavements (5) although other types of overlays can be analyzed by reviewing the procedure. The RFLCRI computer program provides a rational procedure for evaluating the susceptibility of an overlay to reflection cracking. At joints or cracks in the existing pavement, the program computes (a) the horizontal tensile strain in the overlay due to thermal movement and (b) the vertical-load-associated shear strain in the overlay. These computed strain values are then compared with allowable maximum values. The program provides for the possible use of bond breakers, intermediate layers, or reinforcement in the overlay if these maximum criteria are violated.

EVALUATION OF FATIGUE-CRACKING SUBSYSTEM

In the process of developing the Texas SDHPT procedures from the FHWA procedure, various studies were conducted on the fatigue-cracking subsystem, RPOD1. Subsequently, the revised computer program RPOD2 was developed for Texas SDHPT.

Nayak and others (6) have conducted an extensive sensitivity analysis of the program RPOD1. Both fractional-factorial and single-factorial results are reported. The pavement conditions used in the analysis are described in Table 1, and the sensitivity results are summarized in Table 2. In general, the following were concluded to be important variables:

1. The modulus of the subbase is a very important variable. This rather surprising result indicates one of the advantages of using layered theory in the analyses, i.e., that factors outside the slab are accounted for more accurately.
2. Design deflection is another very important variable. The design deflection is also used to characterize the subgrade material. In this sensitivity study, the

Table 1. Description of pavement conditions for sensitivity analysis.

Pavement Condition	Type of Overlay	Type of Existing Pavement	Bonding Condition	Voids	Cracking Condition
1	Continuously reinforced concrete pavement	Continuously reinforced concrete pavement	Bonded	No	Classes 1 and 2
2	Jointed concrete pavement	Jointed concrete pavement	Unbonded	No	Classes 1 and 2
3	Asphalt concrete pavement	Continuously reinforced concrete pavement	Bonded	No	Classes 1 and 2
4	Continuously reinforced concrete pavement	Continuously reinforced concrete pavement	Unbonded	No	Mechanically broken
5	Jointed concrete pavement	Continuously reinforced concrete pavement	Unbonded	No	Classes 1 and 2
6	Continuously reinforced concrete pavement	Jointed concrete pavement	Unbonded	No	Classes 1 and 2
7	Jointed concrete pavement	Jointed concrete pavement	Unbonded	No	Classes 3 and 4
8	Jointed concrete pavement	Continuously reinforced concrete pavement	Unbonded	Yes	Classes 1 and 2
9	Jointed concrete pavement	Jointed concrete pavement	Unbonded	Yes	Classes 1 and 2

Table 2. Results of sensitivity analysis.

	Pavement Condition								
	Fractional-Factorial Experiment				Single-Factorial Experiment				
Input Variable	1	2	3	4	5	6	7	8	9
Modulus of subbase	1	2	2	4	1	6		2	5
Design deflection	2	1	1	2	2	5	1	1	2
Thickness of surface	3		3		5			5	
Modulus of surface	4	4			6	1	2	6	1
Thickness of subbase	5		4		3			3	
Poisson's ratio of surface	6				4			4	
Modulus of subbase times design deflection		3		6					
Poisson's ratio of overlay		5		3		4	4		3
Modulus of overlay		6		1		2	3		4
Modulus of surface times thickness of subbase			5						
Modulus of subbase times thickness of base			6						
Modulus of bond breaker				5		3	5		6
Poisson's ratio of bond breaker					6				
Thickness of bond breaker							6		

stress sensitivity of the subgrade material was not a factor because the design load was the same as the deflection load. The importance of the design deflection therefore indicates that the stress sensitivity of the subgrade material might be an important factor (and this proved to be so under certain circumstances).

3. Other variables that are important are the thickness and modulus of the surface layer, the modulus of the overlay, the thickness of the subbase, and the modulus and thickness of the bond breaker or stress-relieving layer, if used.

4. The Poisson's ratios of the overlay, surface layer, and bond breaker are important variables in some instances.

5. It can be assumed that flexural strength is an important variable: In this study, concrete modulus and flexural strength were varied together [which is feasible because an increase in modulus will normally be accompanied by an increase in flexural strength (1, 7)] and therefore the effect of the concrete modulus represents the combined effect of both variables.

Effect of Remaining Life on Overlay Thickness

The concept of using the remaining life of the existing pavement in designing overlays was introduced by McCullough in 1969 (4) and is used in the Shell method for overlay design on flexible pavements (8), the FHWA method for flexible pavements (9), and by Zaniewski (10). The remaining-life concept is defined as follows:

$$R_L(x,t,l,e,m) = 1 - \Sigma\,(n_i/N_i)\,(x,t,l,e,m) \quad (1)$$

where

R_L = remaining life;
n_i = number or load applications of level i experienced from the beginning to time t;
N_i = number of load applications of level i required to cause failure in simple loading; and
(x, t, l, e, m) = notation to describe the subject relations as a matrix function of space, time, loading, environment, and materials properties.

The effect of the percentage of remaining life on the required overlay thickness for a 203-mm (8-in) concrete pavement that has a 203-mm stabilized subbase on a subgrade is shown in Figure 2. The moduli for the concrete (both pavement and overlay) and for the subbase were taken to be 31.7 and 3.45 GPa (4 600 000 and 500 000 lbf/in^2), respectively, and Poisson's ratios of 0.2, 0.2, and 0.4 were assumed for the concrete, stabilized subbase, and subgrade, respectively.

By varying the assumed traffic before overlay, the percentage of remaining life can be varied. It should be noted that, when the remaining life of the existing pavement is taken into consideration, the required overlay thickness is significantly reduced. On the other hand, if the fact that some of the life of the existing pavement has already been consumed by traffic is not recognized, the resulting overlay may be too thin.

The remaining-life concept is used in conjunction with the fatigue equation:

For PCC,

$$N = 23\,440\,(f/\sigma)^{3.21} \quad (2a)$$

where

N = number of 80-kN ESALs until failure,
f = flexural strength of concrete (lbf/in^2), and
σ = computed tensile stress due to design load (lbf/in^2).

For AC,

$$N = 9.7255 \times 10^{-15}\,(1/\epsilon)^{5.16267} \quad (2b)$$

where ϵ = computed strain due to design load (in/in). (The equations given in this paper are designed for U.S. customary units only.)

These equations indicate that, for very low values of remaining life, it might be more economical to consider the existing pavement not to have any and the governing stress to be at the bottom of the overlay. For pavements that have remaining life, the governing stress is considered to be at the bottom of the existing pavement. In the Texas method, there is a modification where the existing pavement is considered both to have remaining life and to not have remaining life so that the more economical overlay thickness can be selected.

Effect of Subgrade Resilient Modulus on Overlay Thickness

In this study, the pavement structure was the same as that described above except for the use of an unbonded overlay. The stress-relieving layer was taken to be 50.8 mm (2 in) thick and to have an elastic modulus of 689 MPa (100 000 lbf/in^2). Overlay thicknesses were determined for different subgrade resilient moduli. Both RPOD1 and manual (by using the linear elastic-layer program ELSYM5 to calculate stresses, strains, and deflections) calculations were made. Figure 3 shows the relationship between overlay thickness and subgrade resilient modulus. It can be seen that the overlay thickness for an existing pavement that has remaining life is much more sensitive to a change in subgrade resilient modulus than is that for an existing pavement that does not have remaining life. Schnitter and others (2) have pointed out that this reduction in overlay thickness with increase in subgrade resilient modulus when the existing pavement has remaining life is due to the combined effects of having the governing stress lower down in the pavement system and the increase in remaining life.

Effect of Stress Dependency of Subgrade Resilient Modulus on Overlay Thickness

The resilient moduli of subgrade materials are generally stress dependent. In the Texas method, as in the FHWA method, the subgrade modulus is determined by a combination of repetitive-load triaxial testing and deflection measurements.

When plotted on a log-log scale, the relationship between the modulus and the deviator stress for subgrade soils is generally close to a straight line (1, 5, 9, 10). Zaniewski (10) indicates that, as the confining pressure increases, the resilient modulus of a subgrade material increases, but in such a way that individual curves for different confining pressures are parallel. Mathematically, the relationship can be expressed as follows:

$$M_R = a(\sigma_1 - \sigma_3)S_{SG} \tag{3}$$

where

M_R = resilient modulus (lbf/in^2),
a = intercept on the subgrade-modulus axis,
S_{SG} = slope of the line determined by the log-log plot of the resilient modulus versus deviator stress,
σ_1 = applied vertical stress (lbf/in^2),
σ_3 = applied horizontal stress (lbf/in^2), and
$\sigma_1 - \sigma_3$ = deviator stress (lbf/in^2).

The slope (S_{SG}) is generally negative for clayey materials and positive for granular materials (2). For materials that included clays, silty clays, sandy silts, clayey silts, and very fine-grained sand, a practical range for S_{SG} was found to be between -1.2 and 0.

Figure 4 shows the required overlay thicknesses for different values of S_{SG} for the pavement structure shown in Figure 5. A Dynaflect design deflection of 0.014 mm (0.000 565 in) was used and traffic before overlay was kept constant on 4 million 80-kN ESALs. The overlay was designed for 7 million 80-kN ESALs. The results of this study indicate that, if the existing pavement does not have remaining life, the predicted overlay thickness is relatively insensitive to variations in S_{SG}. For pavements that have remaining life, the variation in S_{SG} has a considerable influence on overlay thickness.

The reason for this phenomenon is that, in characterizing the subgrade material by using the measured deflection, different resilient moduli are obtained for materials that have different stress dependencies (S_{SG}). The more stress dependent the material, the lower the resilient modulus to be used with the design load (for negative values of S_{SG}). This also affects the remaining life of the existing pavement.

These results suggest that relatively more effort should be spent in characterizing the subgrade materials of pavements that have remaining life than of those that do not.

Effect of Change in Stress Level in Subgrade, Due to the Overlay, on Overlay Thickness

In the fatigue-cracking subsystem, the subgrade modulus is determined under the design load on the existing pavement and then used throughout the rest of the overlay-design process. The overlay, however, will reduce the stress level in the subgrade, which will result in a higher subgrade modulus, for a soil that has a negative S_{SG}. This will cause the design to be conservative. This effect is illustrated in Figure 6. The computer program used in the FHWA method (RPOD1) was used to determine overlay thicknesses for different values of S_{SG}, and these results were compared with values obtained by manual calculations that included the effect of the reduction in subgrade stress due to the overlay. However, although the RPOD1 results are somewhat conservative as expected, it was decided that the increased computer time an additional iteration process would require would not be justified.

Asphalt Concrete Overlays on Portland Cement Concrete Pavements

The RPOD1 computer program (1) does not include provision for the design of AC overlays on PCC pavements that do not have remaining life. Schnitter and others (2) have shown that, for these pavements, the effective modulus (3.45 GPa) assumed for a pavement that exhibits classes 3 and 4 cracking can easily be higher than the modulus of the overlay. The governing stress is considered to be at the bottom of the overlay, which can, according to layer-theory solutions, even be in compression and have significant stresses at the bottom of the cracked pavement. This is not an easy problem to deal with by using layer theory.

In the Texas method, this problem is solved by characterizing the subgrade material in the normal way of using measured deflections and laboratory testing and then determine the modulus of a semi-infinite half space that will have the same deflection under design load as the existing pavement structure. The overlay is then designed on this half space.

EVALUATION OF REFLECTION-CRACKING SUBSYSTEM

The reflection-cracking subsystem was evaluated by a limited sensitivity analysis (2) and will not be discussed here.

IMPLEMENTATION OF RESULTS OF EVALUATION STUDIES

After the evaluation studies of the RPOD1 computer

Figure 2. Relationship between overlay thickness and remaining life of existing pavement.

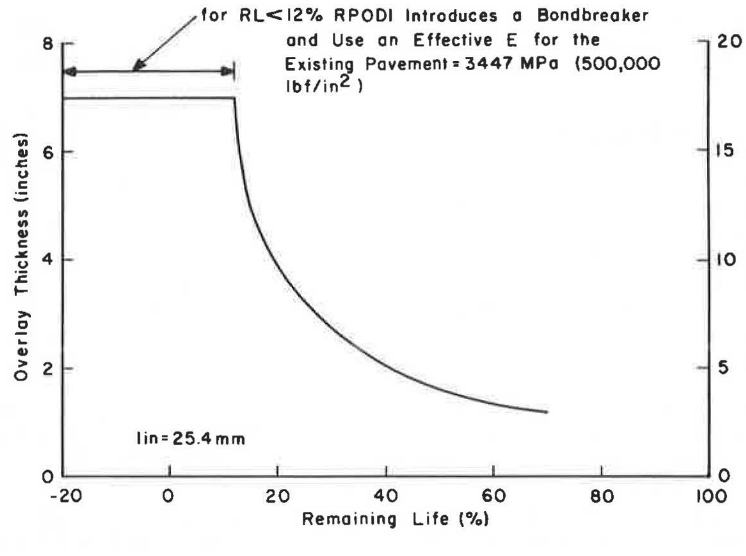


Figure 3. Relationship between overlay thickness and subgrade resilient modulus.

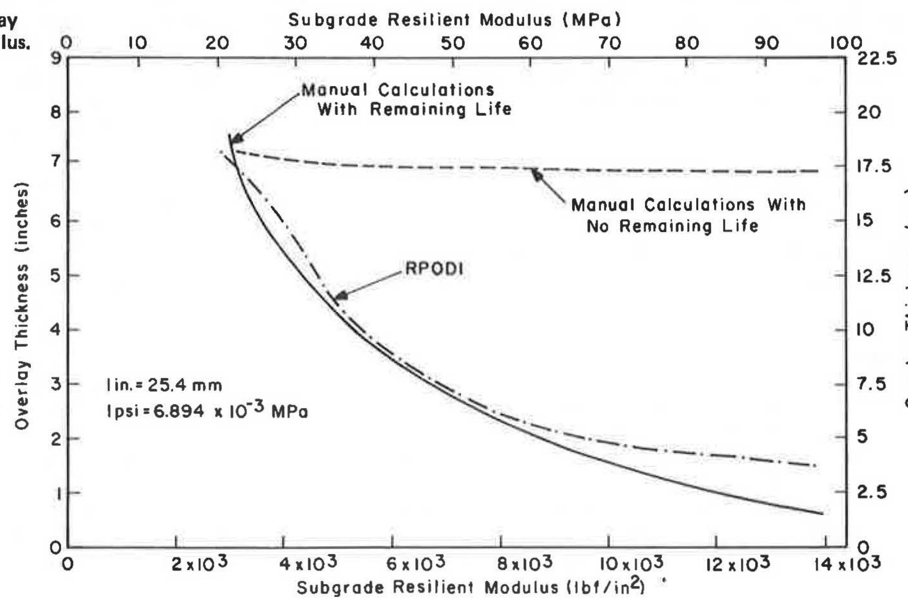
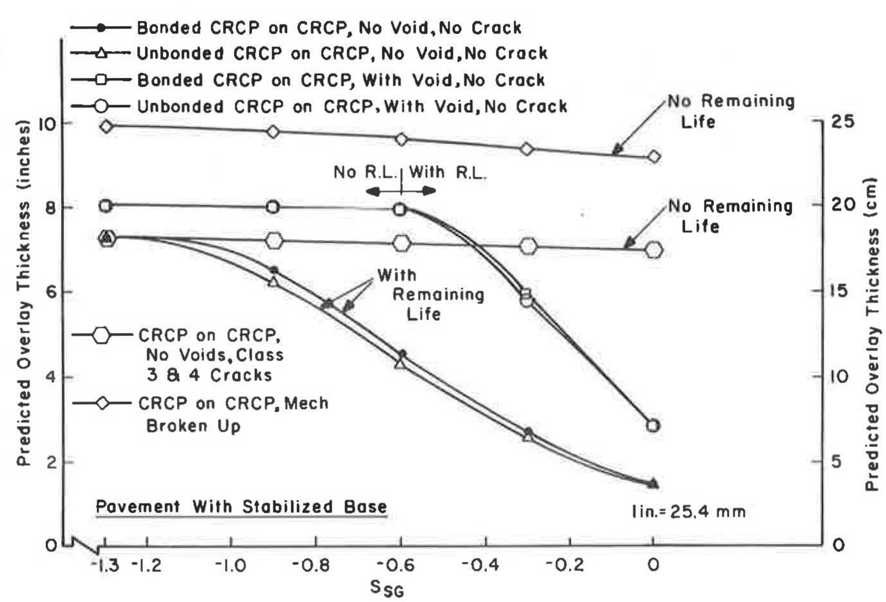


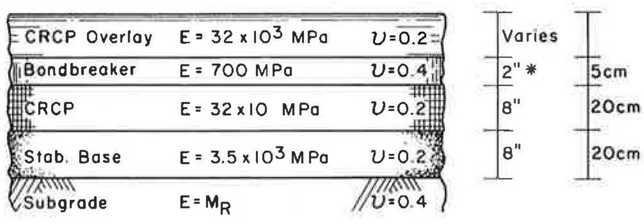
Figure 4. Sensitivity of RPOD1 response to slope of line that describes relationship between subgrade resilient modulus and deviator stress (on log-log scale).



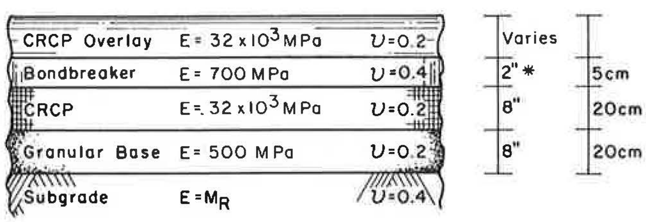
program, a revised program called RPOD2 was developed that includes the following modifications:

1. RPOD2 includes the design of AC overlays on pavements that do not have remaining life by using the concept of a semi-inifinite half space that results in the same deflection under the design load as the existing pavement.
2. RPOD2 allows for the input of values of flexural strength of both the existing pavement and the overlay (in RPOD1 only a single value could be specified).
3. Because it is more economical under certain conditions to consider a pavement that has a low percentage of remaining life to not have any, RPOD2 considers both possibilities for selection of the more economical thickness.
4. Because overlay thicknesses on pavements that do not have remaining life are less sensitive to the stress dependency of the subgrade modulus, RPOD2 provides an alternative way to describe the relationship between the laboratory-determined resilient modulus and the deviator stress.
5. Because the Dynaflect is widely used in Texas for deflection measurements, Dynaflect loads are used as default values in RPOD2.
6. RPOD2 sets limiting elastic-modulus values for subbases of pavements that have classes 3 and 4 cracking and mechanically broken up pavements because it is unlikely that, for example, a cement-stabilized base under a mechanically broken up pavement would be intact.

Figure 5. Pavement structures used to study effect of S_{SG} on overlay thickness.



a. Pavement structure with stabilized subbase.



b. Pavement structure with granular subbase.

* Dimension in Cases Where a Bondbreaker Has Been Used

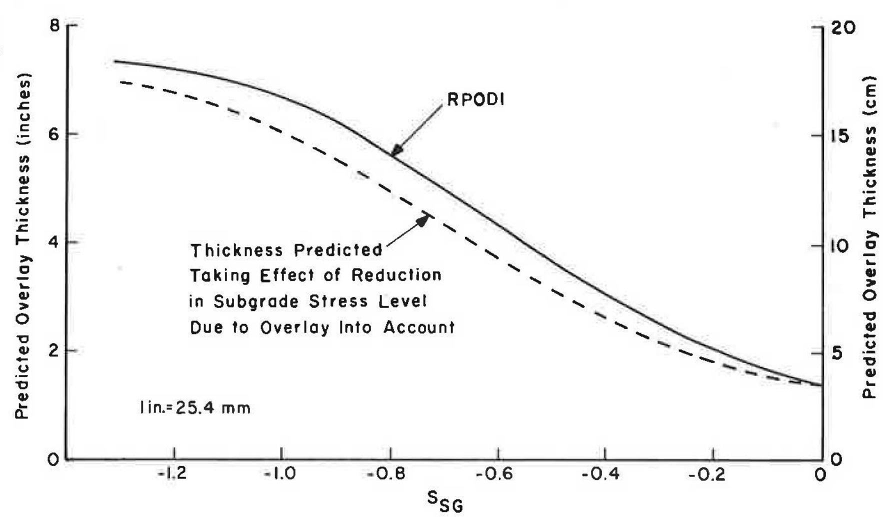
in.= 25.4 mm
1 psi = 6.894 x 10^{-3} MPa

Table 3. Input variables for program RPOD2 for different existing-pavement conditions.

Variable	Pavement Condition[a] A	B	C	D
Traffic before overlay	R	R	-	-
Existing pavement				
Concrete flexural strength	R	R	-	-
Condition	R	R	R	R
Modulus	R	R	F	F
Poisson's ratio	F	F	F	F
Thickness	R	R	R	R
Subbase				
Modulus	R	R	R	R
Poisson's ratio	F	F	F	F
Thickness	R	R	R	R
Subgrade				
Modulus	-	-	-	-
Poisson's ratio	F	F	F	F
Thickness[b]	R[b]	R[b]	R[b]	R[b]
Laboratory data (M_R versus σ)	R	R	E	E
Design deflection	R	R	R	R
Deflection-load magnitude	F	F	F	F
Deflection-load positions	F	F	F	F
Corner-to-interior stress ratio[c]	R[c]	R[c]	R[c]	R[c]
Overlay				
Modulus	R	R	R	R
Poisson's ratio	F	F	F	F
Concrete flexural strength	-	R	R	R
Bonding condition	R	R	-	-
Bond breaker[d]				
Modulus	R[d]	R	R	R
Poisson's ratio	F	F	F	F
Thickness	R	R	R	R
Design traffic	R	R	R	R

Note: R = required, F = fixed (can be changed by using supplement to input guide), and E = estimate (a good estimate of this value is sufficient).
[a]Pavement condition: A = has remaining life; B = uncracked or classes 1 and 2 cracks, does not have remaining life; C = classes 3 and 4 cracks; D = mechanically broken up.
[b]If bedrock is specified.
[c]If existing pavement is JCP.
[d]If bond breaker is specified.

Figure 6. Comparison of predicted thicknesses: by RPOD1 program and by manual calculations that include effect of reduction in level of subgrade stress due to overlay.



The limited sensitivity analyses of the reflection-cracking program RFLCR1 indicated that it gives reasonable results; therefore, no modifications were required to that program.

TEXAS SDHPT USER'S MANUAL

A step-by-step User's Manual has been provided for the use of the Texas State Department of Highways and Public Transportation. The manual is divided into four sections:

1. Evaluation of the existing pavement,
2. Fatigue-cracking analysis,
3. Reflection-cracking analysis, and
4. Selection of overlay thickness.

Input guides for the computer programs are also provided. For simplicity, several of the less sensitive variables required by the procedure have been assigned default values and need not be input. A supplement to the input guide provides a way to change these default values should the designer so desire. Such variables include the Poisson's ratio values, the deflection loads, and the positions for the deflection measuring device.

The manual also contains

1. An indication of the variables that are required for each combination of overlay and pavement (see Table 3),
2. A way of determining S_{SG} by using deflection measurements at two different deflection loads, and
3. A tentative way of determining a maximum allowable value for the repeated shear strain due to traffic loads used in the reflection-cracking analysis.

IMPLEMENTATION OF TEXAS SDHPT OVERLAY-DESIGN PROCEDURE

Because field verification is always an important aspect of a new design procedure, this procedure will be implemented for trial use on real overlay-design problems as soon as possible. This will be done by designing a number of overlay sections, constructing them, and then monitoring their performance.

Because the procedure is computerized, it can be easily adapted to rigid-pavement management systems also (11). Future research will be directed toward this goal.

This overlay design method can also be a useful research tool for parameter and sensitivity analyses and is currently being used by personnel of the Center for Highway Research, University of Texas at Austin, and the Texas SDHPT in a study to determine the most economical time or condition to overlay pavements.

It is hoped that this overlay-design procedure will eventually provide pavement designers in Texas with a sound practical method of designing structural overlays in all classes of rigid pavements.

CONCLUSIONS

In the process of adapting the FHWA procedure for the design of rigid pavement overlays for the Texas SDHPT, the FHWA method has been thoroughly evaluated. Some of the evaluation studies on the fatigue-cracking subsystem are discussed in this paper, and the following can be concluded:

1. In general, the most important input variables are design deflection and the elastic moduli and thicknesses of the various layers.
2. Taking the remaining life of the existing pavement into consideration reduces the required overlay thickness considerably.
3. The subgrade modulus has a much larger effect on required overlay thicknesses for existing pavements that have remaining life than for those that do not.
4. The effect of the stress dependency of the subgrade resilient modulus is much greater for existing pavements that have remaining life than for pavements that do not.
5. Relatively more effort should be given to the determination of the subgrade modulus of pavements that have remaining life than of pavements that do not.
6. The effect of ignoring the reduction in the subgrade stress due to the overlay is to make the design conservative for subgrades that have negative S_{SG} values.
7. The problem of modeling AC overlays on PCC pavements by using elastic-layer theory has been overcome.

ACKNOWLEDGMENT

This research was conducted at the Center for Highway Research, University of Texas at Austin. We wish to thank the Texas State Department of Highways and Public Transportation, whose sponsorship is gratefully acknowledged. The contents of this report reflect our views; we are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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Overlay Design Based on Visible Pavement Distress

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Data collected on 111 Interstate highway projects in Virginia were analyzed by using a multiregression procedure, and the rating coefficient for each type of distress was determined. From these coefficients, the total distress and the resultant maintenance rating for each pavement were calculated. The types of distress that were found to affect the maintenance rating are longitudinal cracking, alligator cracking, rutting, pushing, raveling, and patching. A method for designing the required thickness of an overlay was developed based on taking the thickness equivalency of an asphalt concrete overlay in Virginia as equal to 0.5 and the overlay thickness as a function of the ratio of the traffic, in terms of the number of 80-kN [18 000-lbf (18-kip)] equivalent loads, carried by the pavement before the overlay to the traffic it would carry after the overlay, depending on the durability of the asphalt mix. This design method does not require the use of a deflection-measuring device.

In Virginia, the decision to provide an overlay over a flexible pavement conventionally is based on a visual inspection that does not make reference to any defined criterion for pavement evaluation. However, to comply with the Reconstruction, Rehabilitation, and Resurfacing Program of the Federal Highway Administration, the states now need procedures by which the necessity for an overlay can be validated and its required thickness can be estimated so as to obtain federal participating funds.

In Virginia and some other states, mechanistic methods for determining the required thicknesses for overlays have been developed. However, these methods are based on deflection data (1, 2) and their use would require that all districts have deflection equipment such as the Dynaflect available, along with a technician, for the collection of data. Similarly, the methods for quantifying total pavement distress based on rating systems require the use of some technique for measuring distress by mechanical means. Consequently, there is a need for a method by which to establish a relationship between the total pavement distress, the accumulated traffic and the structural strength of the pavement that can be used to design overlays without the necessity of using pavement-deflection (or any other) measuring devices.

OBJECTIVE AND SCOPE

The objective of the investigation reported here was the development of a method for designing the thickness of overlays for flexible pavements that would be based on maintenance ratings of the pavements as determined by visual observations and sound engineering judgment. These overlays would be designed for the sole purpose of improving the structural strength of the pavement. Defects in the pavement surface that did not affect its strength would not be considered.

As outlined in the working plan (3), the study was designed to accomplish the following tasks:

1. To develop a pavement-maintenance-rating system based on the total observed pavement distress;
2. To develop a relationship between the maintenance rating, the accumulated traffic {in terms of 80-kN [18 000 lbf (18-kip)] equivalents}, and the structural strength of the pavement (in terms of its thickness index) that could be used to evaluate the performance of the pavement before and after the overlay;
3. To determine the thickness equivalency of the overlay; and
4. To develop a method for determining the required thickness of the overlay.

PAVEMENT-MAINTENANCE-RATING SYSTEM

The pavement-maintenance-rating technique that was developed is based on the same principle as the serviceability index (SI) included in the American Association of State Highway and Officials (AASHO) Road Test results. The SIs of the new pavements at the AASHO Road Test varied from 3.9 to 4.5, with an average value of 4.2. For the design of overlays in Virginia, it is proposed that a maintenance-rating factor (MR) of 100 for a new pavement be adopted. Thus, an AASHO SI of 4.2 would equal an MR of 100, and an SI of 0 would equal an MR of 0. As distress to the pavement increases, factors assigned to various types and degrees of distress are subtracted and the MR decreases. The MR for a new pavement will decrease from 100 as the accumulated traffic, and hence the distress, increases.

Although the pavement distress over the first few years that a road is open to traffic is so small that it is not discernible to the naked eye, it can be measured by a Dynaflect or a roughometer. However, measurement of this indiscernible distress is not necessary for the design of overlays. In the rating system developed, an SI of 3.9 or an MR of 93 [i.e., $(3.9/4.2) \times 100 = 93$] is

Table 1. Interstate flexible pavement distress ratings and overlay data.

Serial No.	Pavement Distress: 1974-1975 LC	AC	Ru	Pu	Ra	MR	Year Constructed	First Overlay Year	No. of 80-kN Equivalent Loads (000 000s)	D
1	2	1	0	1	0	88	1961	1971	1.55	12.3
2	1	3	1	1	0	78	1960	1971	2.00	12.3
3	3	2	2	0	0	83	1963	1977	2.72	14.0
4	3	2	2	0	1	83	1963	1975	2.24	13.8
5	1	0	0	0	0	93	1962	1975	2.43	13.8
6	3	3	0	0	3	78	1963	1976	2.44	13.8
7	0	0	0	0	0	93	1963	1974	2.02	13.6
8	2	0	0	0	0	93	1963	1974	1.97	13.6
9	2	0	0	0	0	93	1963	1969	1.09	13.8
10	1	0	0	0	2	91	1963	1969	1.16	13.8
11	1	0	0	0	2	91	1963	1970	1.27	13.8
12	3	3	3	0	0	78	1963	1976	2.27	13.8
13	0	0	0	0	0	93	1964	1974	1.77	13.8
14	0	0	0	0	0	93	1964	1973	1.57	13.8
15	0	0	0	0	0	93	1964	1973	1.55	13.8
16	0	0	0	2	0	91	1964	1972	1.42	13.8
17	3	3	3	0	0	78	1965	1975	1.61	13.8
18	1	0	0	0	0	93	1968			15.2
19	1	0	0	0	0	93	1961	1969	1.36	13.4
20	0	0	0	0	0	93	1962	1970	1.39	13.4

Note: 1 kN = 225 lbf.

considered as the maximum value of incipient visible distress for the following reasons:

1. The minimum value of the AASHO SI for a new pavement was 3.9, which is equal to an MR of 93;
2. The rate of decrease of the MR as the traffic increases is constant to an MR of approximately 93 and, below that value, accelerates (i.e., at an MR of 93, the deterioration of the pavement begins to accelerate);
3. Statistical analysis gives higher values of correlation coefficients when pavements that do not have visible distress are assigned an MR of 93—in the present investigation, all pavements that did not have visible distress were assigned an MR of 93, irrespective of their age, because pavements that have MRs of 93 or higher are never considered for overlays.

The types of distress that contribute to pavement deterioration are longitudinal cracking (LC), alligator cracking (AC), rutting (Ru), pushing (Pu), raveling (Ra), and patching (Pa). For these types, it is recommended that the ratings given below be adopted.

Distress	Not Severe	Severe	Very Severe
None observed	0	0	0
Rarely observed	1	2	3
Occasionally observed	2	4	6
Frequently observed	3	6	9

On Interstate highways, overlays are applied while the distress is still not severe but, on low-traffic primary roads, the distress may be rated severe or very severe before overlays are placed. The amount and severity of distress considered indicative of a need for an overlay will require clear specification before the rating system can be used by field engineers.

In 1974-1975, McGhee carried out a survey of 111 flexible-pavement projects on 886 km (521 miles) of the Interstate highway system and visually determined the MRs (4) shown in Table 1. A multiregression analysis based on Equation 1 of these data, in which it is assumed that none of the distress recorded was rated as being severe,

$$MR = a_0 + a_1(\text{LC rating}) + a_2(\text{AC rating}) + a_3(\text{Ru rating}) + a_4(\text{Pu rating}) + a_5(\text{Ra rating}) + a_6(\text{Pa rating}) \quad (1)$$

gives Equation 1a which has a correlation coefficient of 0.96 and standard error of 0.39.

$$MR = 92.6 - 2.4LC - 2.3AC - 1.0Ru - 1.0Pu - 0.9Ra \quad (1a)$$

Because none of the projects on the Interstate highways considered had any patched areas, no coefficient for patching was included in Equation 1a. Patching is usually provided to cover severe or very severe distress, generally in the form of alligator cracking. If patching were considered in Equation 1a, the coefficient for it would be 2.3, the same as that for alligator cracking. However, patching is here classified as not severe and is rated only by the amount observed.

The data given below, taken from serial no. 4 in Table 1, can be used to illustrate the method for determining the MR of a pavement.

Type of Distress	Amount	Severity	Rating
LC	Frequent	Not severe	3
AC	Occasional	Not severe	2
Ru	Occasional	Not severe	2
Pu	None		0
Ra	Rare	Not severe	1
Pa	None		0

By using these data and Equation 1a, $MR = 92.6 - (2.4 \times 3) - (2.3 \times 2) - (1.0 \times 2) - (1.0 \times 0) - (0.9 \times 1) - (2.3 \times 0) = 77.9$. None of the MRs of the 111 Interstate projects cited above were lower than 78.

The MRs for the 111 projects were determined in June 1975. Pavements that had values between 78 and 83 were overlaid in 1975 or 1976, except for a few that were overlaid in 1977. Thus, there is an indication that the rating system determined in the investigation is in line with field practice. However, the establishment of priorities based on the system might lead to improvements in the utilization of funds. As shown in Table 1, (a) one project that had an MR of 83 in 1975 was overlaid in 1977; (b) two projects that had MRs of 78 in 1975 were overlaid in 1976; and (c) three projects that had MRs of 78, 83, and 93, respectively, were overlaid in 1975. If priorities had been established by using the rating system, the pavements that had the

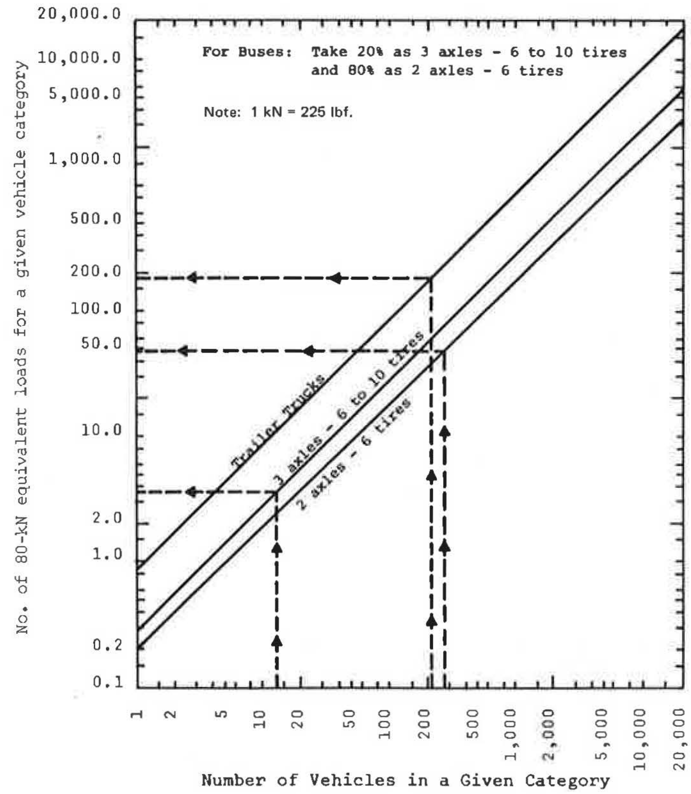


Figure 1. Determination of number of 80-kN equivalent loads from traffic counts.

Table 2. Thickness equivalencies of materials used in Virginia for Interstate, arterial, and primary roads.

Location	Material	Thickness Equivalent
a_1	Asphalt concrete	1.0
a_2	Asphalt concrete	1.0
	Untreated aggregate base material (crushed or uncrushed, specification numbers 20, 21, and 22)	0.35
	Type 1 select material directly under asphalt concrete mat and over good quality subbase	0.35
a_3	Types 1, 2, and 3 select material	
	In Piedmont area	0.0
	In Valley and Ridge area and coastal plain	0.2
	Soil cement or soil lime	0.4
	Cement-treated aggregate base directly over subgrade	0.6

lower MRs would have been overlaid first.

The SI limits recommended by the AASHO committee (5) for use in decisions as to when overlays should be applied were correlated with the MR system as shown below.

Road Classification	SI	MR
Interstate	≤3.5	≤83
Arterial	≤3.0	≤71
Primary	≤2.5	≤60
Low primary or secondary	≤1.5	≤36

Thus, it is seen that, for the Interstate highway pavements in Virginia that had MRs of 83 or less, overlays are justified. Pennsylvania has used the same approach to pavement-maintenance rating (6).

RELATIONSHIP BETWEEN MAINTENANCE RATING, ACCUMULATED TRAFFIC, AND STRUCTURAL STRENGTH

The rate and amount of pavement deterioration are a function of the pavement strength and the accumulated traffic (in terms of the number of 80-kN equivalent loads) and can be determined by using Equation 2 (7).

$$\log \text{no. of 80-kN equivalent loads} = A + B(\text{thickness index}) \qquad (2)$$

where

A = f(MR), a function of the maintenance rating and constant for a given MR value, and
B = a constant for any given MR value.

The number of 80-kN equivalent loads can be determined from a traffic count by using Figure 1 (8). The annual traffic counts are prepared by the Traffic and Safety Division of the Virginia Department of Highways and Transportation (9).

The thickness index (D) is a number that shows the intrinsic strength of the pavement (i.e., without the subgrade support). It is a nondimensional quantity and is obtained by using Equation 3.

$$D = a_1 h_1 + a_2 h_2 + a_3 h_3 + \ldots \qquad (3)$$

where

h_1, h_2, and h_3 = thicknesses of asphalt concrete surface layer, base layer, and subbase layer, respectively, and
a_1, a_2, and a_3 = strength coefficients of the layers h_1, h_2, and h_3, respectively.

The values of a_1, a_2, and a_3 are given in Table 2.

Because there were no MR data for pavements in

Figure 2. Relationship between maintenance rating and cumulative traffic at different values of thickness index.

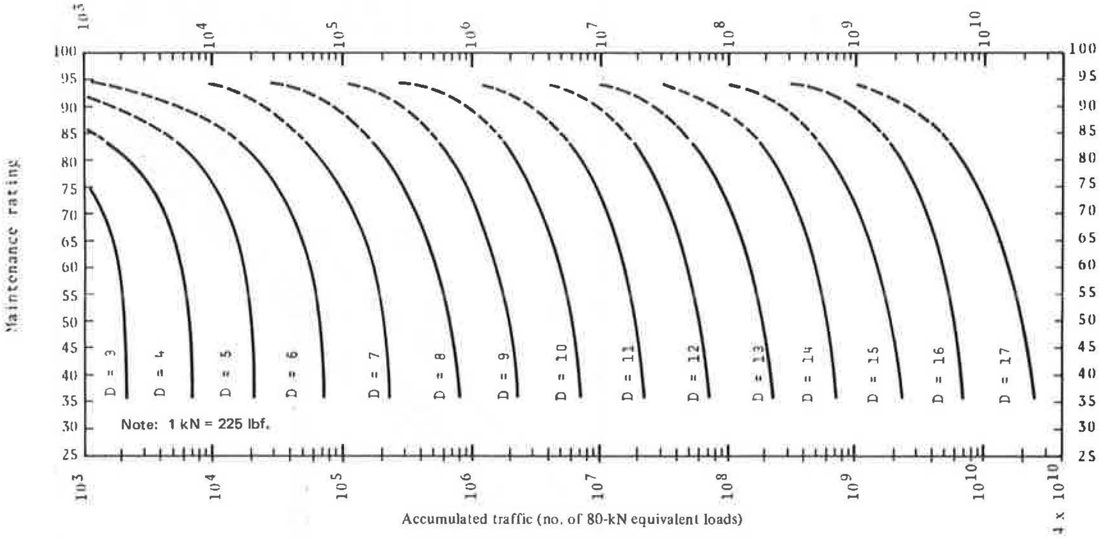
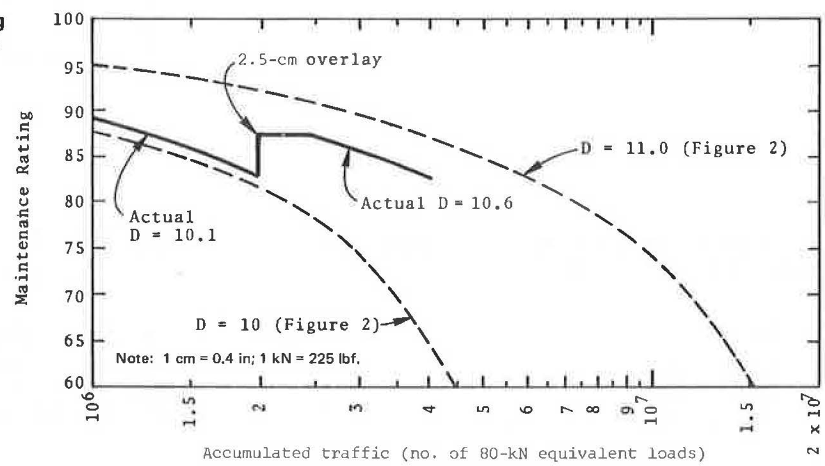


Figure 3. Relationship between maintenance rating and cumulative traffic of an Interstate pavement in Virginia before and after an overlay.



Virginia available for evaluation, the raw data from the AASHO Road Test pavements were used. The AASHO results give data on 270 projects that had different pavement cross sections. On each of the projects, traffic in terms of 80-kN equivalent loads is given for MRs of 83, 71, 60, 48, and 36 (SI values of 3.5, 3.0, 2.5, 2.0, and 1.5), respectively. The D-value index of each project was obtained by using the strength coefficients given in Table 2.

$$D = (1.00h_1 + 0.35h_2 + 0.20h_3)/2.50 \tag{3a}$$

(h_1, h_2, and h_3 are measured in centimeters), and an equation based on Equation 2 had the form

$$\text{log no. of 80-kN equivalent loads} = A + 0.5(\text{thickness index}) \tag{2a}$$

The values of A so obtained and the correlation coefficients (Rs) and standard errors (SEs) are summarized below.

MR	A	R	SE
83	1.213	0.87	0.71
71	1.582	0.92	0.49
60	1.742	0.94	0.41
48	1.823	0.94	0.39
36	1.871	0.94	0.39

These correlation coefficients show that an excellent relationship exists between the MR, the traffic, and the structural strength (see Figure 2).

STRENGTH COEFFICIENT

No MR data are available for overlaid pavements in Virginia; however, the AASHO Road Test gives basic data on 99 overlaid projects. These data have been evaluated and the results reported elsewhere (7). The evaluation showed that the strength coefficient of an overlay should be taken as one-half that of the asphalt concrete for new construction. In Virginia, the thickness equivalency of asphalt concrete for new construction is equal to 1 (as shown in Table 2). The strength coefficient of asphalt concrete for an overlay in Virginia is, therefore, 0.5.

Of the 111 projects analyzed in the investigation, 8 were overlaid in 1975. The average MR of these 8 projects was 83, and the average traffic on them before the overlay was about 2 million 80-kN equivalent loads. The average thickness of the overlays on these 8 projects was 2.5 cm (1 in). A 2.5-cm overlay on a new pavement in Virginia usually lasts as long as did the pavement before the overlay. Hence, it is assumed that these 8 pavements will be able to carry an additional 2 million 80-kN equivalent loads before a second overlay

Figure 4. Relationship between traffic-carrying capacity and overlay thickness.

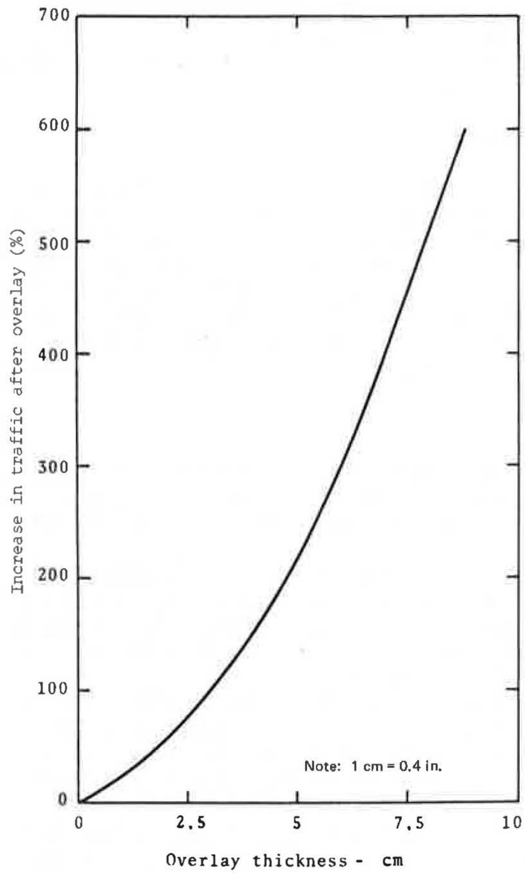


Table 3. Traffic growth rate and accumulated traffic (assuming 5 percent growth/year).

Period of Traffic (years)	Growth Rate	Accumulated Traffic	Period of Traffic (years)	Growth Rate	Accumulated Traffic
1	1	365	11	1.62	5 169
2	1.05	748	12	1.70	5 789
3	1.10	1 149	13	1.78	6 438
4	1.16	1 572	14	1.87	7 120
5	1.22	2 017	15	1.97	7 839
6	1.27	2 480	16	2.07	8 595
7	1.34	2 969	17	2.17	9 387
8	1.40	3 480	18	2.28	10 219
9	1.47	4 016	19	2.39	11 091
10	1.54	4 578	20	2.51	12 007

is needed. The relationship of the MRs to the traffic history of the average of these 8 pavements is shown on an exaggerated scale in Figure 3, which shows that the averages of the thickness indices of these 8 pavements before and after the overlays were 10.1 and 10.6, respectively. Thus, a 2.5-cm overlay gives a strength coefficient of 10.6 - 10.1 (or 0.5). Hence, it appears that the conclusion reached in the evaluation of the overlay strength coefficient for AASHO road projects (7) could also be applied to overlays in Virginia.

THICKNESS OF AN OVERLAY

By using Equation 2a, the traffic carried by an overlaid pavement can be calculated as

$$\text{Traffic} = \text{antilog}(Aa + 0.5Da) - \text{antilog}(Ab + 0.5Db) \quad (4)$$

where

Ab and Aa = constants for the MR before the overlay and at the end of the overlay service and
Da and Db = thickness indices of the pavement before and after the overlay.

As described above, for a given type of highway, the MRs before the overlay and at the end of the overlay service are the same; that is, Aa = Ab. In such a case, Equation 4 reduces to

$$\text{Traffic after overlay} = \text{traffic before overlay} \times \{\text{antilog}\,[0.5\,(\text{overlay thickness}/2.5) \times \text{thickness equivalency of overlay}] - 1\} \quad (4a)$$

(when overlay thickness is measured in centimeters) or

$$\text{Traffic after overlay/traffic before overlay} = [\text{antilog}\,(0.1 \times \text{overlay thickness}) - 1] \quad (4b)$$

or

$$\text{Percentage increase in traffic after overlay} = [\text{antilog}\,(\text{overlay thickness}/10) - 1] \times 100 \quad (4c)$$

Figure 4, which is based on Equation 4c, shows the percentage increase in the number of 80-kN equivalent loads versus the overlay thickness in centimeters and can be used to determine the required thickness of an overlay. This figure shows that the traffic capacities for overlay thicknesses of 2.5, 5.1, and 7.6 cm (1, 2, and 3 in) are, respectively, 78, 217, and 464 percent of the traffic before the overlay.

Deflection studies carried out before and after the application of asphalt concrete overlays have shown that overlay thicknesses of 2.5 cm or more do contribute to an increase in the structural strength of the pavement. It is therefore recommended that the overlays provided for increasing the structural strength of pavements be a minimum of 2.5 cm thick. The following method is recommended for designing the thickness of an overlay.

The design for the thickness of an overlay is dependent on the durability of the asphalt concrete mix used and the way it is affected by such factors as age, hardening, and stripping of the asphalt. An overlay made from a properly placed, well-designed mix can perform satisfactorily for 10-15 years without surface rejuvenation. For determining the required thickness of an overlay, the use of a 12-year service life for the mix is recommended. The procedure for determining the overlay thickness is as follows:

1. Determine the accumulated traffic in terms of the number of 80-kN equivalent loads that the pavement has carried from the date of construction to the date of the proposed overlay, irrespective of any previous overlays. If necessary, use Figure 1 to convert the traffic count into 80-kN equivalent loads.
2. Determine the accumulated traffic in terms of the number of 80-kN equivalent loads the pavement will carry in the 12 years following the overlay.
3. From Figure 4, determine the thickness of the overlay for a given percentage increase in traffic after the overlay, taking the percentage increase as (number of 80-kN equivalent loads after the overlay ÷ number of 80-kN equivalent loads before the overlay) × 100.

For an Interstate highway pavement that had been built in 1967 and had a maintenance rating of 76.5 in

1977, an overlay probably would be justified. The thickness of the overlay can be calculated as outlined below.

1. (a) Determine the daily traffic in 80-kN equivalent loads. From the average daily traffic (ADT) volume records published annually, obtain the ADT for 1976 and use these data and Figure 1 to calculate the number of 80-kN equivalent loads as shown below.

Type of Vehicle	ADT	No. of 80-kN Equivalent Loads
Two axle, 6 tire	320	58
Three axle, 10 tire	50	14
Trailer trucks	2850	2500
Buses (assume 20 percent three-axle and 80 percent two-axle vehicles)	40	6
Total		2578

(In these calculations, automobiles and two-axle, four-tire vehicles are not considered because their damaging effect on the pavement is almost negligible.) Thus, for a four-lane highway, the design traffic = 2578 × 0.5 × 0.8 = one thousand thirty-one 80-kN equivalent loads, where the reported traffic counts include both directions of travel, one-half the reported traffic is assumed to travel in each direction, and 80 percent of the truck traffic is assumed to use the outside (design) lane.
(b) Determine the accumulated traffic before the overlay. These data can be determined from the traffic records or estimated on the assumption that the traffic has increased as the rate of 5 percent/year (a widely accepted standard) by using Table 3, which gives the growth rate for each year for a 20-year period (e.g., the ADT after 9 years = 1.47 × ADT during the first year) and the accumulated traffic for each year for a 20-year period (the accumulated traffic after 9 years = 4016 × ADT during the first year). Thus, in the above example, the accumulated traffic on the road in 1977 (i.e., at the end of 11 years of service and before the overlay) = design daily traffic in 1977 × accumulated traffic rate ÷ growth rate = 1031 × 5169 ÷ 1.62 = 3.29 million 80-kN equivalent loads.

2. Determine the accumulated traffic after the overlay. From the daily traffic just before the overlay (i.e., 2578 equivalent loads), the traffic to be carried by the overlay is 2578 × accumulated traffic rate (12 years) ÷ growth rate (12 years) = 2578 × 5789 ÷ 1.70 = 8.78 million 80-kN equivalent loads.

3. Determine the overlay thickness. The required overlay thickness can be determined by using Figure 4. The percentage increase in daily traffic to be sustained by the overlay is (8.78 million ÷ 3.29 million) × 100 = 267 percent and the required overlay thickness is approximately 130 mm (5.1 in).

CONCLUSIONS

1. A simplified method based on visual inspections can provide uniformity in decisions regarding the stages at which pavements would be overlaid in an economical manner.

2. In Virginia, the thickness equivalency for an asphalt concrete overlay is 0.5.

3. A method for designing the thickness of overlays has been developed.

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Overlay Design Based on AASHO Road Test Data

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The raw data on the 99 overlays tested at the American Association of State Highway Officials Road Test were evaluated. In the process, the raw data on the pavements that were overlaid were also evaluated. The relationship between the serviceability index of the pavement, the accumulated traffic in terms of 80-kN [18 000-lbf/in² (18-kip)] equivalent loads, and the thickness index of the pavement before the overlay was determined. This relationship was found to apply to the overlaid pavements also. Based on this relationship, the strength coefficient of the overlay was determined and a method of designing the thickness of an overlay was developed. This design method does not require the use of pavement-deflection data.

Since 1962, the American Association of State Highway Officials (AASHO) Road Test results (1) have provided the fundamental guidance for the design of pavements in this country. However, although the Road Test included studies on 99 overlays, these failed to produce conclusive results and, thus, the test results provide no guidance for the design of overlays. The conclusions from the study of overlays stated in part that "Attempts at mathematical analysis designed to establish a specific relationship between performance and overlay design were unsuccessful." But recently, the need for suitable methods of designing overlay thicknesses and of predicting their performance has been recognized, and it has become imperative that the AASHO results be further investigated to provide suitable guidance for the design of overlays.

OBJECTIVE AND SCOPE

The objective of this investigation was to use the raw data from the AASHO Road Test for the determination of the strength coefficient of overlays and the design of overlay thicknesses. The objective was met by accomplishing the following three tasks:

1. The development of a relationship between the serviceability of a pavement, the accumulated traffic, and the structural strength of the pavement before and after an overlay,
2. The determination of the strength coefficient of the overlay, and
3. The development of a method for designing the thickness of overlays.

VARIABLES

The extent and type of distress that a pavement undergoes depends on the traffic it carries and its structural strength. These three variables are discussed below.

Distress, in the AASHO Road Test, is defined by the serviceability index (SI). The SI of a new pavement decreases as the accumulation of traffic increases. The rate of decrease depends on the structural strength of the pavement; the higher the structural strength, the lower the rate of decrease. Traffic is defined in terms of the number of accumulated 80-kN [18 000 lbf (18-kip)] equivalent single-axle loads. The structural strength is defined in terms of the design thickness index (D), which is defined as

$$D = a_1 h_1 + a_2 h_2 + a_3 h_3 \tag{1}$$

where

h_1, h_2, and h_3 = thicknesses of the surface layer, the base layer, and the subbase layer, respectively, and
a_1, a_2, and a_3 = strength coefficients of the layers h_1, h_2, and h_3, respectively.

The SI is measured by pavement roughness, cracking, patching, and rutting.

RELATIONSHIP BETWEEN ACCUMULATED TRAFFIC AND D

Before Overlay

The AASHO Road Test report gives raw data—including the cross section, total traffic, and axle loads—on 270 pavements for five values of the SI (3.5, 3.0, 2.5, 2.0, and 1.5). These raw data were used to determine the value of D for each pavement and its accumulated traffic as discussed below.

Thickness Index

The AASHO Road Test results give the strength coefficients of the materials used in the pavement sections as 0.44 for an asphalt concrete surface layer, 0.14 for an untreated stone base, and 0.11 for an untreated material in the subbase. Because all of the 270 pavements tested in the AASHO Road Test consisted of these three materials only, in this investigation, Equation 1 can be written as

$$D = (0.44h_1 + 0.14h_2 + 0.11h_3)/2.5 \tag{1a}$$

(when h_1, h_2, and h_3 are measured in centimeters).

Accumulated Traffic

The axle-load equivalency factors given by the AASHO Road Test results were used to determine the 80-kN equivalent load for a given axle load. The accumulated number of 80-kN equivalent loads for each of the five SIs on each project were then determined by multiplying the accumulated number of axle-load repetitions by the axle load in terms of the 80-kN equivalent load.

Linear multiregression analyses of the D-values and the accumulated traffic for each of the five SIs were carried out separately by using Equation 2.

$$\text{log no. of 80-kN equivalent loads} = C + E\ (\text{thickness index}) \tag{2}$$

where

C = intercept of the D-axis for a given SI and
E = slope of the linear portion of the curve of the accumulated traffic versus D for a given SI.

Figure 1. Relationship between serviceability index and cumulative traffic at different values of thickness index.

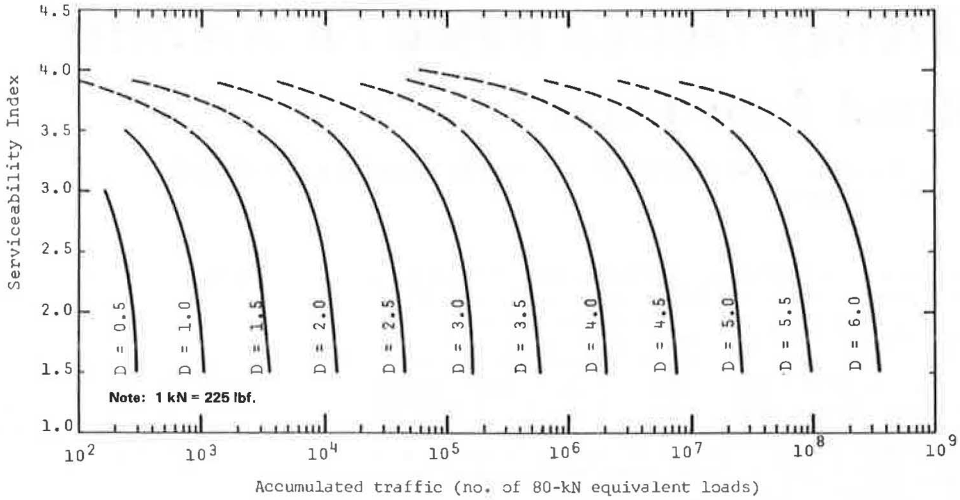
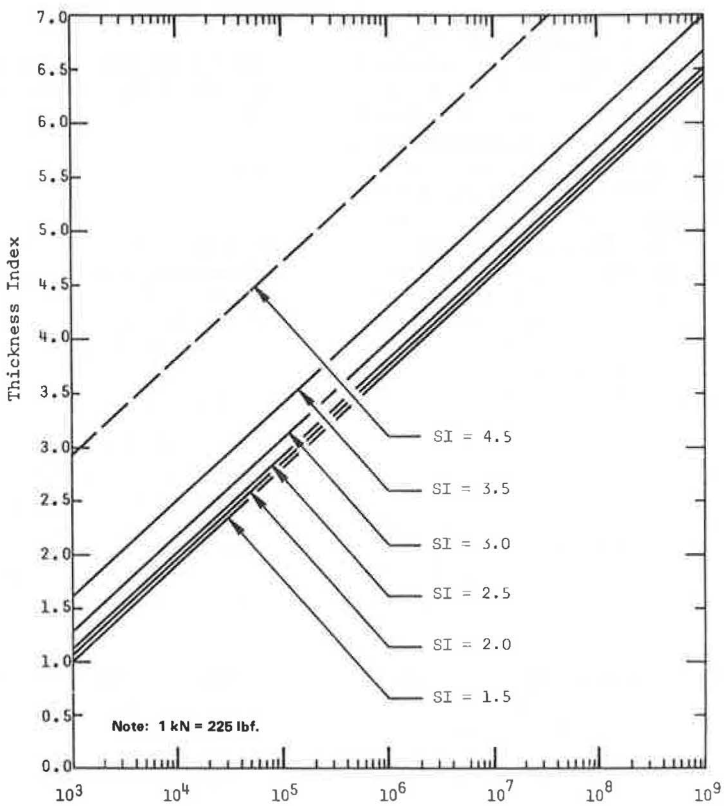


Figure 2. Relationship between thickness index and cumulative traffic at different values of serviceability index.



The values of C and E in the equations so developed and the correlation coefficients (Rs) of these equations are given below.

SI	C	E	R
3.5 (N = 270)	1.140	1.128	0.88
3.0 (N = 258)	1.702	1.063	0.93
2.5 (N = 239)	1.810	1.080	0.95
2.0 (N = 230)	1.814	1.106	0.95
1.5 (N = 216)	1.834	1.116	0.95

Thus, the values of E in Equation 2 for the five SIs are almost identical. Therefore, its value was taken as its average (i.e., 1.1) and the values of C in Equation 2 were redetermined.

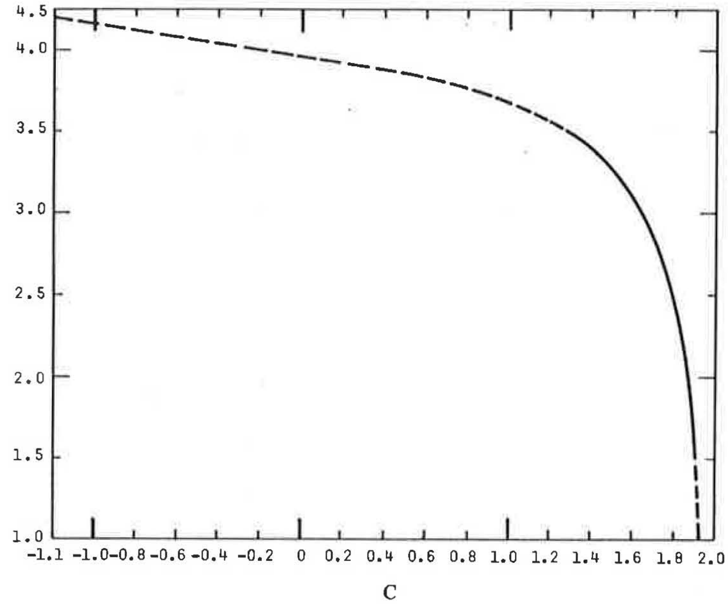
$$\log \text{no. of 80-kN equivalent loads} = C + 1.1(\text{thickness index}) \tag{2a}$$

The values of C and the Rs and standard errors of estimate (SEs) of the equations so developed are given below.

SI	C	R	SE
3.5	1.27	0.88	0.69
3.0	1.63	0.93	0.47
2.5	1.79	0.95	0.38
2.0	1.87	0.95	0.36
1.5	1.92	0.95	0.36

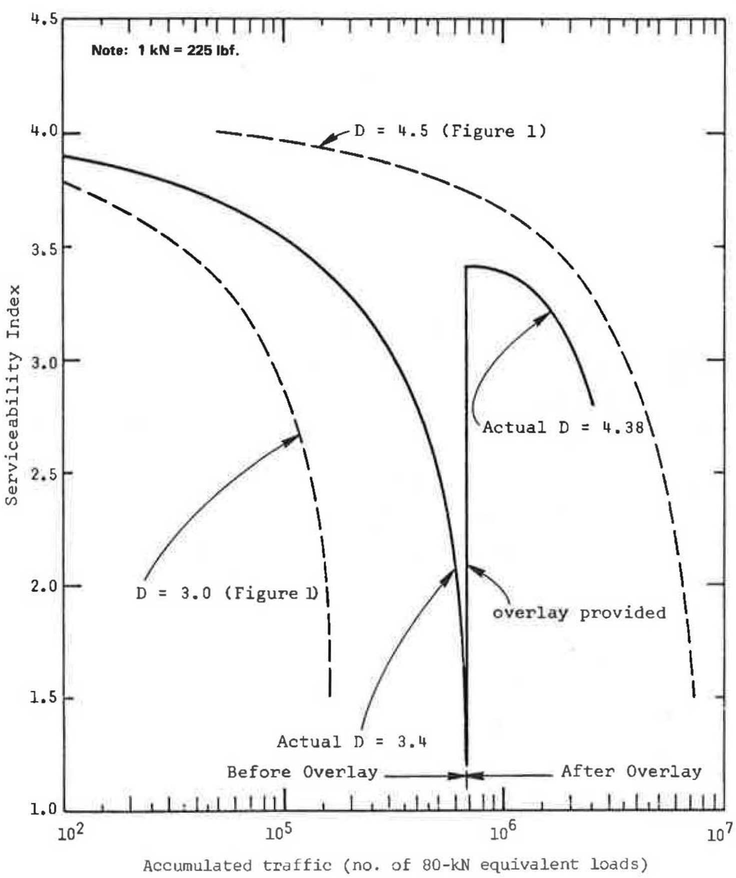
Figures 1 and 2 show the relationship between the SI, the accumulated traffic, and D throughout the life of a

Figure 3. Relationship between serviceability index and C.



SI	C
1.0-	1.92
1	1.92
1.25	1.91
1.50	1.90
1.70	1.89
1.80	1.88
2.0	1.87
2.1	1.85
2.2	1.84
2.3	1.83
2.4	1.81
2.5	1.78
2.6	1.76
2.7	1.74
2.8	1.71
2.9	1.68
3.0	1.64
3.1	1.59
3.2	1.54
3.3	1.47
3.4	1.39
3.5	1.27
3.6	1.14
3.7	0.97
3.8	0.70
3.9	0.32
4.0	-0.2

Figure 4. Example of pavement and overlay behavior (loop 5: AASHO Road Test).



flexible pavement. In these two figures, the values of the SIs were extrapolated by plotting C versus the SI as given in Equation 2a and shown in Figure 3. By using Figure 3, the value of C can be obtained for any value of the SI.

After Overlay

The AASHO Road Test results give raw data on 99 overlay projects. From these data, the following were obtained: (a) the values of the SI before the overlay, immediately after the overlay, and at the end of the overlay service and (b) the accumulated traffic before the overlay and at the end of the service life of the overlay. The accumulated traffic immediately after the overlay is equal to the accumulated traffic immediately before the overlay. The three data points so obtained for each project can be plotted in Figure 1 and extrapolated parallel to the curves given there and, thus, the values of the SI, the accumulated traffic, and D for the pavement

Table 1. Data on AASHO overlay projects.

			Before Overlay			After Overlay	At End of Service			
Loop	AASHO Design D	Overlay Thickness (cm)	Accumulated Traffic [80-kN equivalent loads (000s)]	SI	Actual D	SI	Accumulated Traffic [80-kN equivalent loads (000s)]	SI	Actual D	Mean Strength Coefficient of Overlay
5	3.04	7.5	214	0.6	3.07	3.23	2894	3.65	5.15	0.693
	3.04	7.5	183	1.4	3.03	2.97	2413	2.85	4.30	0.423
	3.06	7.5	281	1.2	3.18	3.20	2894	2.35	4.26	0.360
	3.06	7.5	341	1.6	3.30	3.27	2297	2.85	4.30	0.333
	3.04	7.5	147	0.9	2.94	3.83	292	2.00	3.43	0.163
	3.04	7.5	132	1.4	2.94	3.10	271	2.00	3.40	0.153
	3.50	7.5	375	0.4	3.32	3.80	2798	2.60	4.31	0.330
	3.50	7.5	385	1.5	3.36	3.80	2208	3.65	5.08	0.573
	3.48	7.5	362	0.9	3.28	3.00	2849	2.00	4.21	0.310
	3.48	7.5	337	1.7	3.31	3.17	2297	2.54	4.22	0.303
	3.92	7.5	1667	0.1	3.88	3.40	1224	2.45	4.24	0.120
	3.92	7.5	975	1.4	3.68	3.10	1053	2.85	4.20	0.173
	3.06	7.5	161	1.5	2.98	3.60	330	2.00	3.48	0.167
	3.06	7.5	170	1.1	2.96	3.75	167	1.55	3.31	0.117
	3.46	7.5	364	1.4	3.30	3.50	2848	3.60	5.02	0.573
	3.46	7.5	334	1.7	3.31	2.90	240	2.00	3.54	0.077
	3.90	7.5	1654	1.4	3.87	3.50	735	3.60	4.95	0.360
	3.50	7.5	378	1.6	3.35	3.63	2797	3.85	5.72	0.790
	3.50	7.5	304	1.7	3.26	3.30	2296	2.55	4.23	0.323
	3.48	7.5	370	1.3	3.30	4.07	2798	3.95	5.93	0.877
	3.48	7.5	942	1.1	3.66	3.55	1053	2.50	4.10	0.147
	3.48	7.5	378	1.5	3.33	4.00	2798	3.10	4.50	0.390
	3.92	7.5	2001	0.9	3.95	3.53	895	2.15	4.20	0.083
	3.46	7.5	231	1.0	3.11	3.00	2894	2.55	4.27	0.387
	3.46	7.5	319	1.6	3.28	3.37	2257	2.20	4.16	0.293
	3.90	7.5	1852	0.8	3.93	3.30	1192	3.30	4.61	0.227
	3.90	7.5	1544	0.6	3.85	2.97	882	2.95	4.30	0.150
	3.90	8.8	1492	1.5	3.88	3.30	1283	3.15	4.50	0.207
	3.90	7.5	1334	1.2	3.80	3.33	946	3.45	4.60	0.267
	3.48	7.5	994	1.0	3.68	3.67	1005	3.60	4.85	0.390
Mean	3.48	7.5	674	1.2	3.40	3.40	1696	2.79	4.38	0.325[a]
Mean of 99 projects	3.35	7.5	760	1.2	2.93	3.26	2338	2.82	3.79	0.30[b]

Note: 1 cm = 0.4 in; 1 kN = 225 lbf.
[a]SD = 0.203. [b]SD = 0.30.

before and after the overlay can be determined. An example of this is shown in Figure 4.

A study of these data for each project showed that all pavements behave in the manner shown by the solid line in Figure 4 (which is an example of the mean values of pavements on loop 5 as given in Table 1).

In this example, the pavement had deteriorated to an SI of 1.2 before the overlay. Because the overlay covered all the observed types of distress, the SIs increased without a change in traffic. When an overlaid pavement is first opened to traffic, the rate of decrease in the SI as the traffic increases is constant but the duration of this situation depends on the thickness of the overlay. After some time, the reduction in the SI accelerates in the same manner as for a new pavement, and the curve of the SI versus the traffic follows the general trend shown for new pavements before the overlay. This behavior of the overlaid pavement is shown in Figure 4.

In practice, the SI of the pavement and the accumulated traffic carried by the pavement before the overlay are known. If the additional value of D contributed by the overlay could be determined, the pavement behavior in terms of the SI versus the traffic after the overlay could be predicted, as shown in Figure 4. The D-value of the overlay can be determined if its strength coefficient is known.

DETERMINATION OF STRENGTH COEFFICIENT OF AN OVERLAY

To determine the strength coefficient of an overlay, the raw data on the 99 AASHO overlay projects were used. The data on each of the 99 projects needed for this investigation were used in their original form or after conversion. The data showed the following:

1. The pavements overlaid had a minimum D-value of 1.28, a maximum of 4.82, and an average of 3.35. The actual thicknesses of the pavements ranged from 12.7 to 53 cm (5 to 21 in). Thus, the strength-coefficient values used in this investigation covered a broad range of pavement strengths.
2. All of the pavements had reached an SI of about 1.5 or lower, with an average of 1.2, before they were overlaid. Usually, pavements—especially those that are heavily trafficked—are overlaid at an SI of 2.5 or higher. The overlay data based on low terminal SIs will not affect the results of this investigation and could be applied to pavements that have high terminal SIs because, as shown in Figure 1, the traffic carried at an SI of 2.5 is not a great deal more than the traffic carried at an SI of 1.5 or lower.
3. Except for one case of an overlay that was 10 cm (4 in) thick, the overlay thickness varied from 5 to 8.9 cm (2 to 3.5 in) and averaged 7.6 cm (3 in). The results of this study must therefore be assumed to be applicable for overlay thicknesses greater than 5 cm until further data are available for verification.

The strength coefficient of an overlay can be obtained by using Equation 3.

$$D_a = D_b + ha_o \tag{3}$$

where

D_b and D_a = actual D-values before and after the overlay,

Figure 5. Method for determining thickness index from accumulated traffic and serviceability index (example: loop 5–AASHO Road Test).

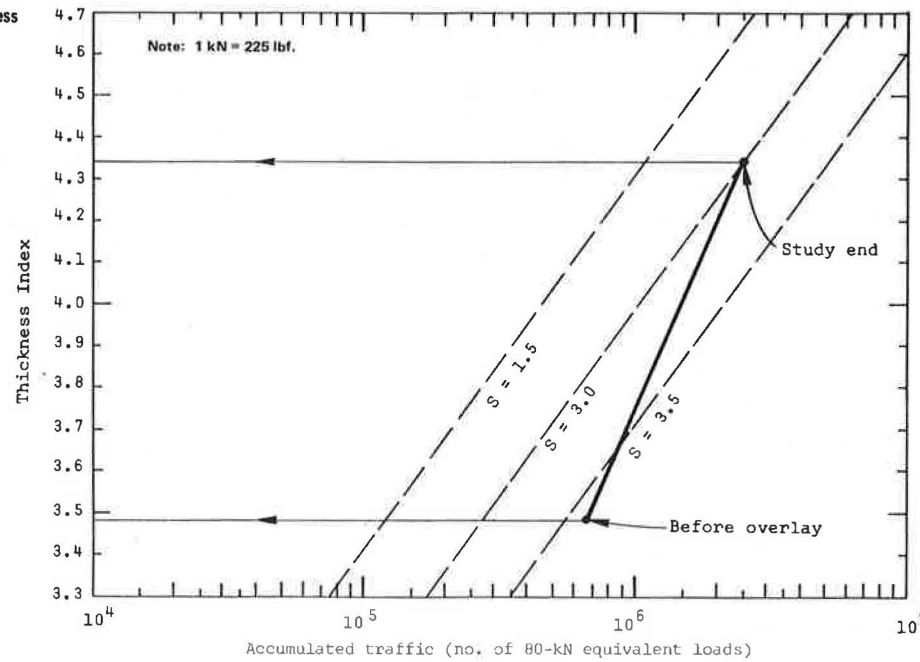
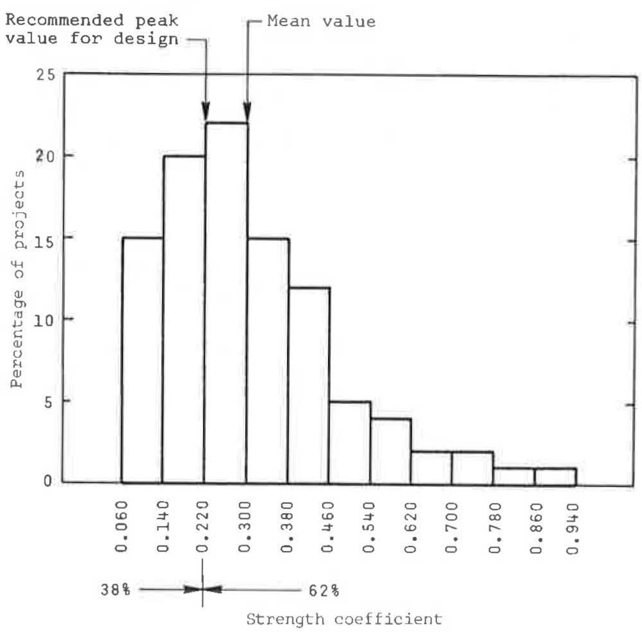


Figure 6. Histogram of thickness equivalency values of overlays on AASHO overlay test pavements.



h = thickness of the overlay, and

a_0 = strength coefficient for one unit thickness of the overlay.

The value of D_b before the overlay may not be exactly the same as the design value obtained by using Equation 1a. This difference may be due to various factors such as subgrade support, material variations, and construction techniques. Thus, in the example shown in Figure 4 for the mean of values of pavements on loop 5, the mean design D-value of the pavements obtained by using Equation 1a was 3.48. When plotted, the data of the SI versus the traffic showed that the actual value of D_b was 3.40. It is therefore necessary that the actual value of D_b be determined for the design of overlays.

The actual value of D_b can be obtained by using the data on the SI and the accumulated traffic as given by (a) Equation 2a and Figure 3, or (b) Figure 1, or (c) Figure 2. The use of these three methods can be shown by taking as an example the mean of the values on loop 5 wherein the mean values of the SI and the accumulated traffic before the overlay are 1.2 and 674 thousand 80-kN equivalent loads. From Figure 3, the value of C for an SI of 1.2 is 1.92. Hence, D_b in Equation 2a = (log 674 000 - 1.92)/1.1 = 3.6. From Figure 1, the value of D_b (as shown on an enlarged scale in Figure 4) is 3.4. From Figure 2, the value of D_b (as shown on an enlarged scale in Figure 5) is 3.48.

In a similar way, the value of D_a in Equation 3 can be determined by using Figures 1 or 2. This can be shown by the example of the mean of the values on loop 5, wherein the mean values of the SI and of the accumulated traffic after the service life of the overlay are 2.79 and 2.370 million 80-kN equivalent loads. The value of D_a from Figure 1 (as shown on an enlarged scale in Figure 4) is 4.38 and that from Figure 2 (as shown on an enlarged scale in Figure 5) is 4.34.

By using the average values of D_b and D_a obtained from Figures 4 and 5, we obtain D_b = 3.44 and D_a = 4.36. The mean thickness of an overlay on loop 5 is 7.5 cm (3 in). Thus, the strength coefficient of the overlay is (4.36 - 3.44)/3 = 0.31.

The strength coefficients for the 99 overlay projects are given in Table 1. The average value is 0.30. A histogram of the strength coefficients of the overlays (see Figure 6) indicates that the population is not normally distributed. If the mean value of 0.30 is adopted as the strength coefficient for overlays, 50 percent of the design projects will be satisfied. To cover a greater percentage of projects, a value of 0.22 is recommended. This value will cover 62 percent of the design projects for AASHO pavements that had been reduced to a terminal SI of 2.5 or lower before an overlay was provided. For roads and highways for which overlays are provided at higher terminal indices, a strength coefficient of 0.22 should satisfy a much larger percentage of the design projects. The value of 0.22 is exactly half the value of the strength coefficient of asphalt concrete for new pavements. It is, therefore, recommended that, for design purposes, the strength coefficient for an overlay be taken as half the strength coefficient of asphalt concrete.

Taking the strength coefficient of an asphalt concrete overlay as half the value for new construction can be justified as follows. As a pavement ages and is trafficked, it becomes fatigued and weak. When an underlying layer becomes weaker than the overlying one, the thickness equivalency of the overlying layer decreases. This is illustrated by the practice in Virginia of taking the thickness equivalency of a cement-treated aggregate placed directly over a raw subgrade as 0.6 times its thickness equivalency when placed over a strong subbase or a base course.

THICKNESS OF AN OVERLAY

By using Equation 2a, the traffic carried by an overlaid pavement can be calculated as

$$\text{Traffic} = \text{antilog}(C_a + 1.1D_a) - \text{antilog}(C_b - 1.1D_b) \quad (4)$$

where C_b and C_a = values of C in Equation 2a before the overlay and at the end of the overlay service period, respectively.

In the AASHO Road Test, the SIs before the overlay and at the end of the service period of the overlay were not the same. In practice, these values are the same, depending on the road classification; i.e., $C_a = C_b$, and Equation 3 reduces to

$$\text{Traffic after overlay} = \text{traffic before overlay} \times \{\text{antilog}\,[0.11 \times (\text{overlay thickness}/2.5) \times \text{strength coefficient of overlay}] - 1\} \quad (5a)$$

(when overlay thickness is measured in centimeters) or

$$\begin{aligned}\text{Traffic after overlay/traffic before overlay} &= \{\text{antilog}\,[0.11 \times 0.22 \times (\text{overlay thickness}/2.5)] - 1\} \\ &= [\text{antilog}\,(0.01 \times \text{overlay thickness}) - 1] \quad (5b)\end{aligned}$$

or

$$\text{Percentage increase in traffic after overlay} = \text{antilog}\,(0.01 \times \text{overlay thickness}) - 1 \times 100 \quad (5c)$$

As shown by Vaswani in Figure 4 of the previous paper in this Record, the relationship between the percentage increase in the accumulated traffic and the overlay thickness can be used to determine the required thickness of an overlay. This figure shows that the traffic capacities for overlay thicknesses of 2.5, 5.1, and 7.6 cm (1, 2, and 3 in) are, respectively, 78, 217, and 464 percent of the traffic before the overlay.

If these percentage increases in traffic are examined carefully, it is seen that the percentage increase in traffic would be the same if the overlay were applied in several thin layers rather than in one thick layer. Thus, one 7.6-cm-thick layer would carry the same traffic as three 2.5-cm-thick layers.

CONCLUSIONS

1. The strength coefficient of an asphalt overlay is less than the strength coefficient of asphalt concrete for new pavements. It is recommended that, in the design of overlays, the strength coefficient for an asphalt overlay should be taken as half (0.22) the strength coefficient of asphalt concrete for new pavements (0.44).
2. The method for designing an overlay developed in this investigation could be used to determine the thickness of an overlay.

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Analytical Study of Minimization of Reflection Cracking in Asphalt Concrete Overlays by Use of a Rubber-Asphalt Interlayer

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The problem of the reflection cracking that is associated with the rehabilitation of existing cracked pavements by the application of an overlay is considered. A general-purpose finite-element program was used to determine the stresses in the overlay at the discontinuities in the underlying pavement, focusing on the effect of a rubber-asphalt stress-absorbing-membrane interlayer on these stresses. A number of variables—the thickness and stiffness of the overlay, interlayer, cracked layer, and subgrade as well as the crack width—were investigated for a specific load condition.

It is shown that, under certain conditions, the inclusion of an interlayer membrane will significantly reduce the crack-induced stresses in the overlay and, hence, by inference, the probability of reflection cracking. Most of the analyses were directed at traffic-load-associated stresses, but a single thermal-stress analysis indicated that an interlayer is effective in reducing these stresses also.

Asphalt concrete (AC) overlays are widely used for the rehabilitation of AC and portland cement concrete (PCC) pavements in need of additional load-carrying capacity. There are many empirically and theoretically based methods for the design of such overlays; in general, these methods give thickness requirements adequate to provide the structural strength needed to accommodate anticipated traffic volumes and loadings for a projected design life. However, although these overlays are structurally functional, they are susceptible to the development of cracks caused by the reflection of cracking patterns in the underlying pavement.

The mechanisms for the development of these reflection cracks in overlays are not fully understood but are believed to be related to the transfer of high stresses to the underside of the overlay at discontinuities in the underlying pavement. An approach to relieving these stresses that appears promising is the placement of a thin [6.25 to 9.5-mm (1/4 to 3/8-in)] asphalt-rubber membrane of low stiffness and high deformability—a so-called stress-absorbing-membrane interlayer at or near the interface between the underlying pavement and the overlay. Recently reported studies of the field performance of overlaid pavements in Arizona (1, 2) that contain one type of rubber-asphalt-membrane interlayer appear to indicate the merit of such an approach and have led to the proposal of testing methods for this material (3).

The analytical study reported in this paper was undertaken to provide some insight as to the efficacy of such an approach and some guidelines for the use of these interlayers.

In this study, responses were investigated of overlaid pavements with and without rubber-asphalt-membrane interlayers to surface loads representative of heavy truck traffic and to temperature changes at the pavement surface. The pavement system was represented by a finite-element idealization, and the distribution of stress in the overlay system in the vicinity of a crack in the existing pavement was examined.

The effects were determined of a number of variables on the stresses resulting from a specific load applied to the surface of the overlay directly above the crack. The variables used include the following:

1. Thickness and stiffness of the rubber-asphalt layer,
2. Thickness and stiffness of the AC overlay,
3. Stiffness of the existing cracked surfacing, and
4. Crack width in the existing surface.

Luther, Majidzadeh, and Chang (4) have also examined the stresses in the vicinity of a crack by using a prismatic-solid finite-element program and a fracture-mechanics approach.

In addition, the stresses in the vicinity of the crack were investigated for a temperature reduction of 22°C (40°F) at the surface of the overlay. This problem has also been examined by Chang, Lytton, and Carpenter (5), who used a fracture-mechanics approach in a study of pavements in west Texas.

The rubber-asphalt membrane characterized for use in this study was a 78:02:20 mixture by weight of paving-grade asphalt, rubber-extender oil, and ground rubber blended together in a conventional distributor truck at a temperature of 175-200°C (350-450°F) and spray applied to the surface. The ingredient imparting the membrane properties is the 20 percent ground rubber fraction, which is a dry, free-flowing blend of 40 percent powdered reclaimed (i.e., devulcanized) rubber and 60 percent ground vulcanized scrap of high natural rubber content.

PAVEMENT SYSTEM

A schematic representation of the pavement system examined in this study is shown in Figure 1. The pavement consists of an AC overlay with or without a rubber-asphalt layer and an existing pavement consisting of a PCC layer, a 300-mm (12-in) thick base course, and a subgrade assumed to have an infinite stiffness at finite depth.

The stiffness characteristics were measured for the rubber-asphalt material and assumed for the other materials. The ranges in stiffness values and layer thicknesses and the crack widths for the PCC layers are summarized below (1 MPa = 145 lbf/in^2 and 1 mm = 0.04 in).

Item	Symbol	Value
AC		
Stiffness (MPa)	E_{ac}	690-10 300
Thickness (mm)	t_{ac}	50-100
Rubber-asphalt interlayer		
Stiffness (MPa)	E_{ra}	6.9-138
Thickness (mm)	t_{ra}	3.2-12.5
PCC		
Stiffness (MPa)	E_{pcc}	6900-27 600
Thickness (mm)	t_{pcc}	100 and 200
Crack width (mm)		6.25 and 12.5
Base course		
Stiffness (MPa)		138
Thickness (mm)		300
Subgrade		
Stiffness (MPa)		34.5 and 69.0
Thickness (mm)		300 and 450

STIFFNESS CHARACTERISTICS OF RUBBER-ASPHALT MATERIAL

Stiffness characteristics of the rubber-asphalt material were determined at two modes of loading—creep and constant rate of strain.

The creep tests were performed in July 1977 on a retained sample of the rubber-asphalt membrane placed on McDowell Road, west of Phoenix, in Maricopa County, Arizona, and obtained in March 1977. These tests gave stiffnesses of 52-25 MPa (7500-3600 lbf/in^2) for loading times representative of moving traffic (0.02-0.05 s) at 21°C (70°F). Based on these data, a value of 34.5 MPa (5000 lbf/in^2) was selected and used for the majority of the analyses. More recently (December 1977), the constant-rate-of-strain tests at 24°C (75°F) gave values of rubber-asphalt stiffness of 9.0-4.5 MPa (1300-650 lbf/in^2) for the same loading time, based on the data plotted in Figure 2. Although a satisfactory explanation for the differences is not available at present, a few additional analyses were performed using stiffnesses for the rubber-asphalt layer as low as 6.9 MPa (1000 lbf/in^2).

FINITE-ELEMENT ANALYSES

The pavement structure was analyzed for traffic loads by using the finite-element program ANSR-1. The two-dimensional finite-element idealization of the pavement structure of Figure 1 is shown in Figure 3. All materials are assumed to be linear elastic and isotropic; a planar strain condition is also assumed (for this assumption, the pavement and load extend to infinity in a direction normal to the plane of the diagram). The actual mesh used to analyze the structure is not shown because some very small elements [as small as 1.6×0.4-mm ($^1/_{16}$ by $^1/_{64}$-in) were required at the crack tip in the overlay where very high stress gradients were expected.

Table 1 summarizes the various conditions that were analyzed; there were 48 computer determinations. A total of 106 finite elements were used for cases 1-4, and 115 elements were used for case 5. Approximately 15 additional computer runs were made to validate the results obtained by using the ANSR-1 program. In these comparisons both a layered-elastic-system program (ELSYM5) and the 2D-SAP program were used. Considering the various assumptions required, these comparisons provided sufficient evidence that the ANSR program was giving a reasonable representation of the situation analyzed.

The output of the finite-element program allows plotting the stress or strain distributions throughout the

Figure 1. Schematic representation of pavement structure.

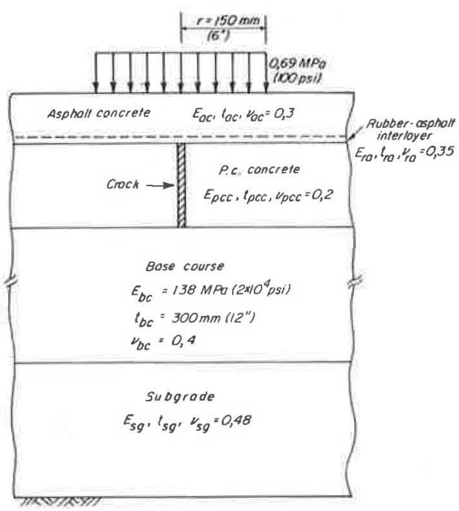


Figure 2. Determinations of creep compliance of rubber asphalt: constant strain rate tests at 21°C.

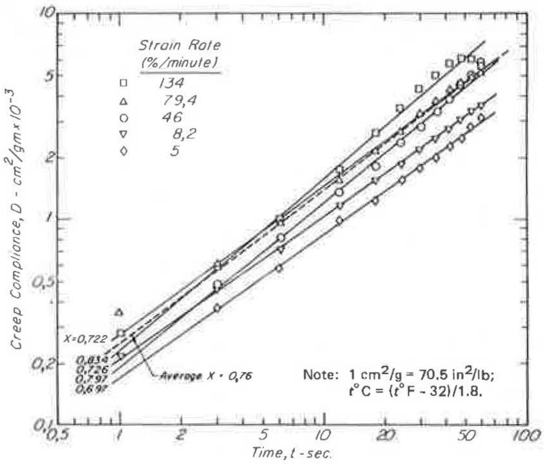


Figure 3. Schematic finite-element representation of pavement structure.

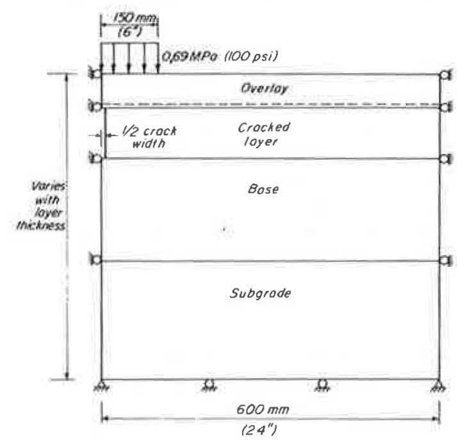


Table 1. Analysis conditions.

	Modulus (MPa)				Thickness (mm)				
Case	AC	Rubber Asphalt	PCC	Sub-grade	AC	Rubber Asphalt	PCC	Sub-grade	Crack Width (mm)
1	690	34.5	13 800	69.0	50	6.25	200	300	12.5
	690	34.5	13 800	69.0	50	6.25	100	300	12.5
	690	34.5	13 800	69.0	75	6.25	200	300	12.5
	690	34.5	13 800	69.0	100	6.25	200	300	12.5
	2 070	34.5	13 800	69.0	50	6.25	200	300	12.5
	3 450	34.5	13 800	69.0	50	6.25	200	300	12.5
	5 175	34.5	13 800	69.0	50	6.25	200	300	12.5
	6 900	34.5	13 800	69.0	50	6.25	200	300	12.5
	10 350	34.5	13 800	69.0	50	6.25	200	300	12.5
	6 900	34.5	6 900	69.0	50	6.25	200	300	12.5
	6 900	34.5	20 700	69.0	50	6.25	200	300	12.5
	6 900	34.5	27 600	69.0	50	6.25	200	300	12.5
	690	6.9	13 800	69.0	50	6.25	200	300	12.5
	690	20.7	13 800	69.0	50	6.25	200	300	12.5
	690	69.0	13 800	69.0	50	6.25	200	300	12.5
	690	103.5	13 800	69.0	50	6.25	200	300	12.5
	690	138.0	13 800	69.0	50	6.25	200	300	12.5
	690	34.5	13 800	69.0	50	6.25	200	300	6.25
2	690	34.5	13 800	34.5	50	6.25	200	300	12.5
	690	34.5	13 800	34.5	50	6.25	100	300	12.5
3	690	34.5	13 800	34.5	50	6.25	200	450	12.5
	690	34.5	13 800	34.5	50	6.25	100	450	12.5
4	690	34.5	13 800	69.0	50	6.25	200	450	12.5
	690	34.5	13 800	69.0	50	6.25	100	450	12.5
5	690	34.5	13 800	69.0	50	3.13	200	300	12.5
	690	34.5	13 800	69.0	50	3.13	100	300	12.5
	690	34.5	13 800	69.0	50	9.5	200	300	12.5
	690	34.5	13 800	69.0	50	12.5	200	300	12.5

Note: 1 MPa = 145 lbf/in^2; 1 mm = 0.04 in.

Figure 4. Effective stress distribution: case 1—a 200-mm portland cement concrete pavement (a) with a rubber-asphalt interlayer and (b) without a rubber-asphalt interlayer.

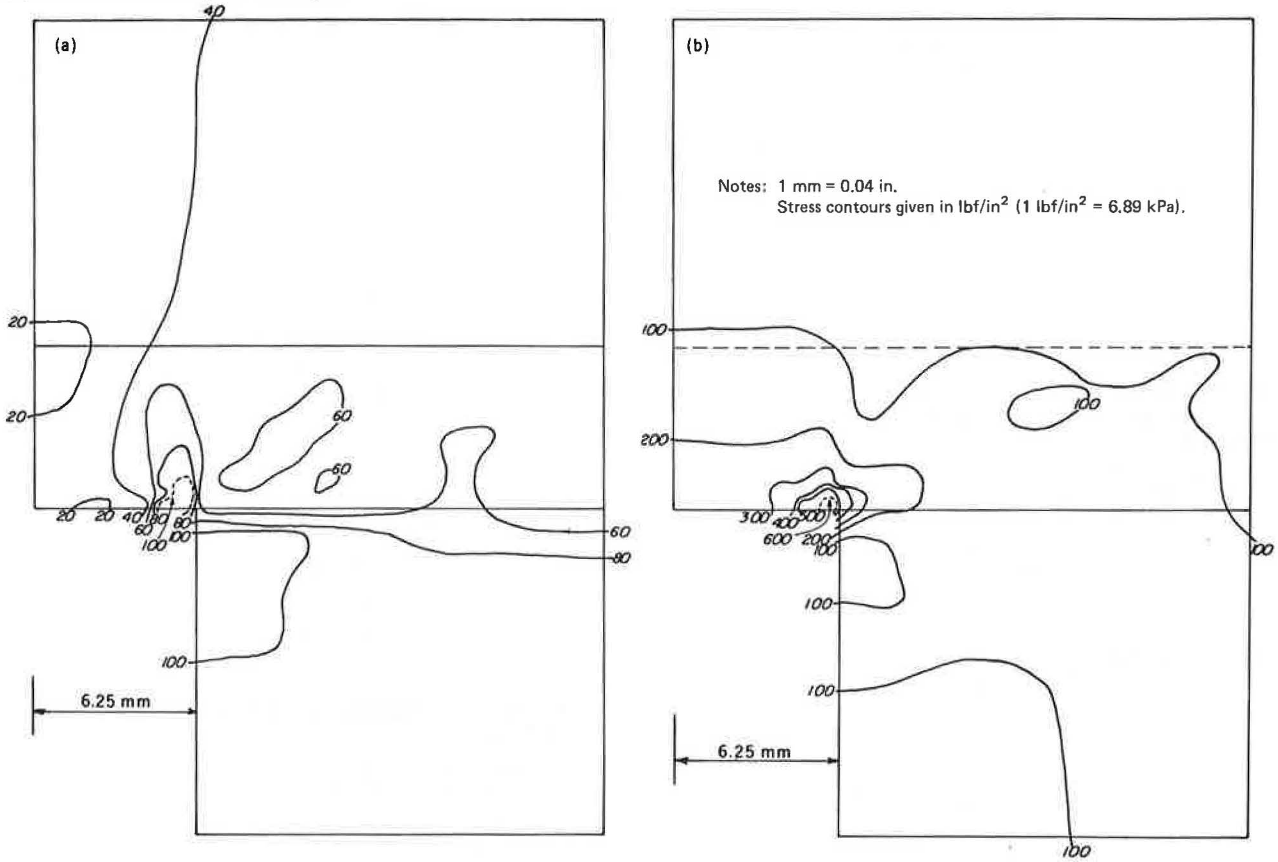
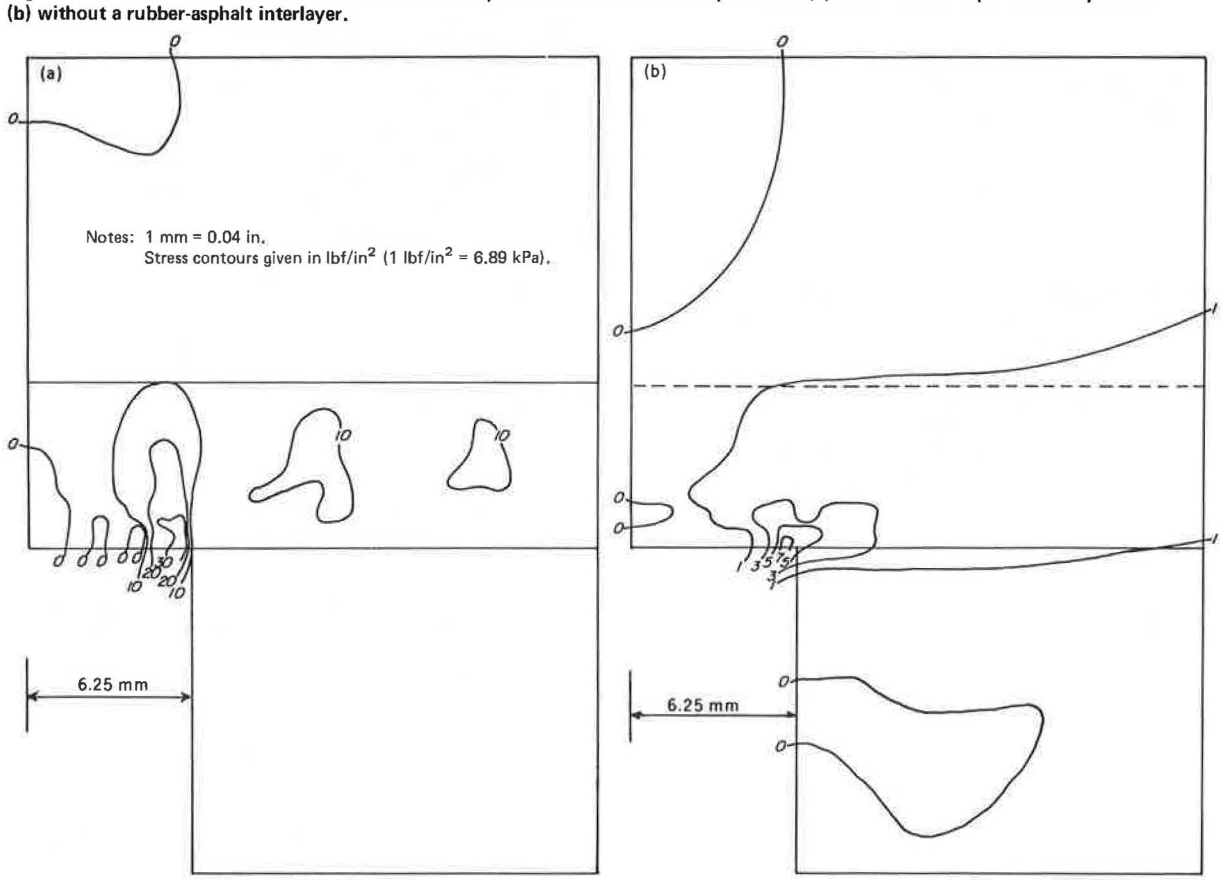


Figure 5. Shear strain distribution: case 1—a 200-mm portland cement concrete pavement (a) with a rubber-asphalt interlayer and (b) without a rubber-asphalt interlayer.



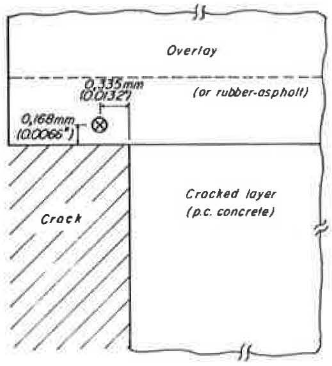
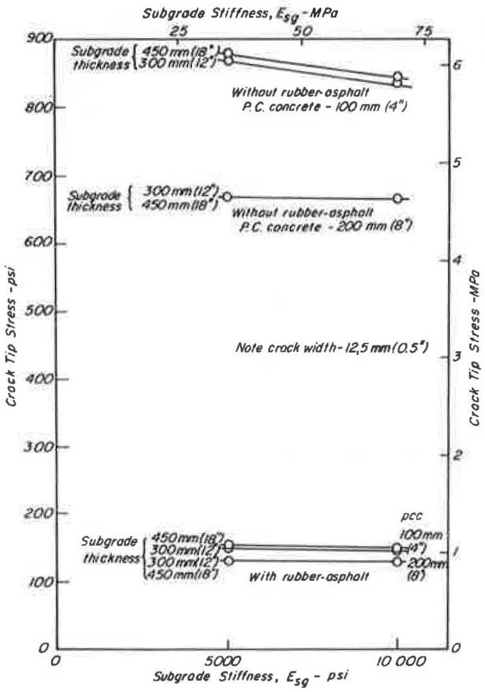


Figure 6. Position (x) at which crack-tip stress is determined.

Figure 7. Effect of subgrade stiffness and thickness and thickness of portland cement concrete on effective stress at crack tip.



analyzed section. Figures 4 and 5, which represent the small section at the crack tip, are typical of these and illustrate the stress-concentrating effect of the discontinuity. (The finite-element program ANSR was designed for U.S. customary units only; therefore, values in Figures 4, 5, and 16 are not given in SI units.) Figure 5 also begins to provide insight into the so-called stress-absorbing mechanism; viz., that the low-modulus material apparently stores strain energy at low stress and high strain levels, but does not transfer the high strains to the AC overlay or rupture. Because of the difficulty of comparing plots of stress distributions, the results of the analysis presented below, i.e., the effective stresses defined according to the von Mises criterion as

$$(1/\sqrt{2})[(\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]^{1/2} \qquad (1)$$

are for a specific location (see Figure 6) in the overlay or the rubber asphalt, when it is present, adjacent to the crack. This is the point closest to the crack tip for which output is available from the ANSR program. These stresses were selected because they were considered to

Figure 8. Effect of overlay thickness on effective stress at crack tip.

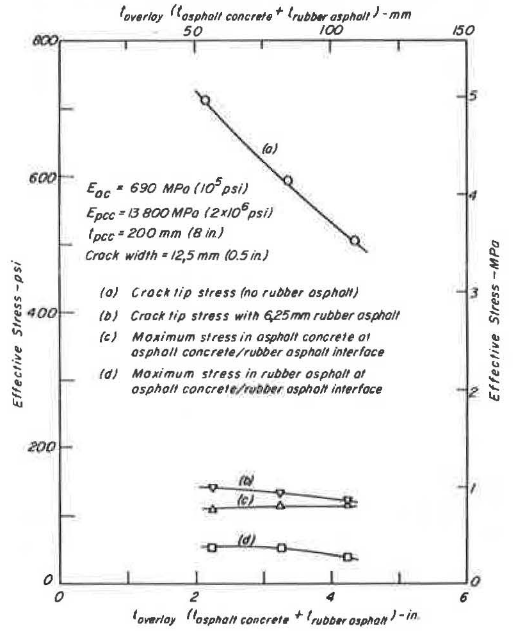
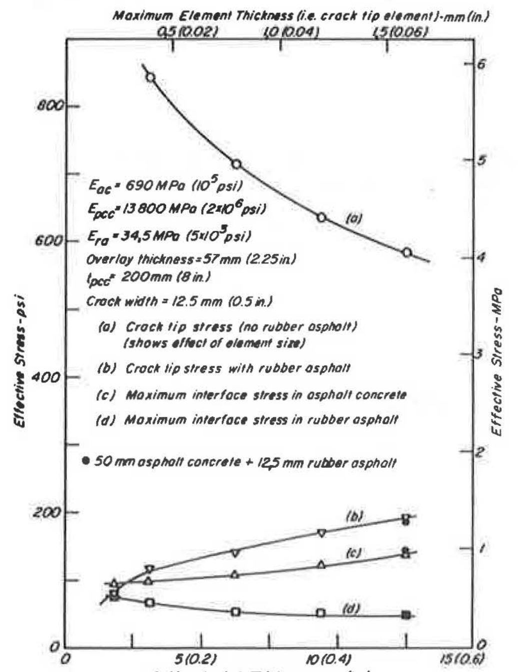


Figure 9. Effect of interlayer thickness on the effective stress at crack tip.



be a realistic determinant for fracture (cracking) under the triaxial stress state existing in the overlay pavement.

COMPUTATIONAL RESULTS

Figures 7-14 illustrate the effects of the various conditions described in Table 1 on the effective stress at the crack tip.

Figure 7 shows that the effect of the rubber-asphalt layer is significant; the stresses in the pavement that contains this layer are one-sixth to one-eighth those in

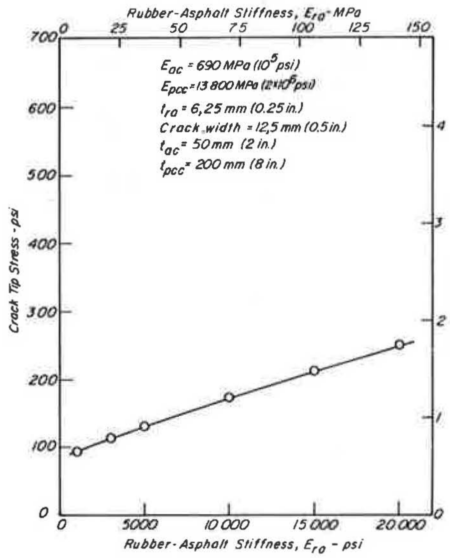


Figure 10. Effect of rubber-asphalt stiffness on crack-tip stress.

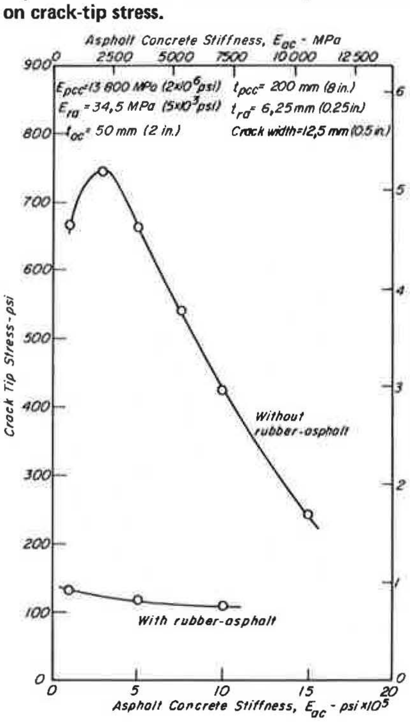


Figure 11. Effect of stiffness of asphalt concrete on crack-tip stress.

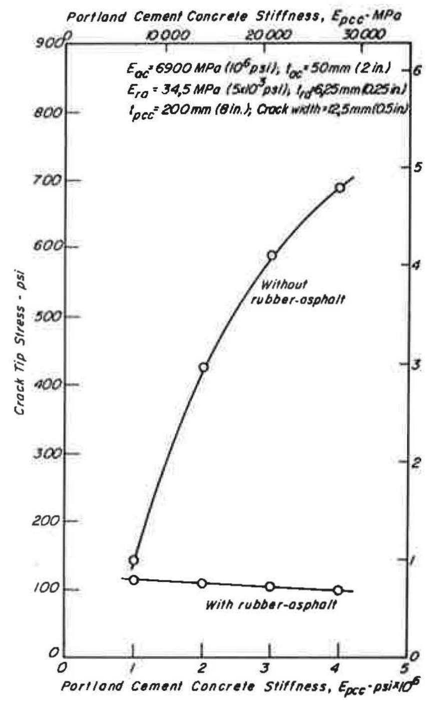


Figure 12. Effect of stiffness of portland cement concrete on crack-tip stress.

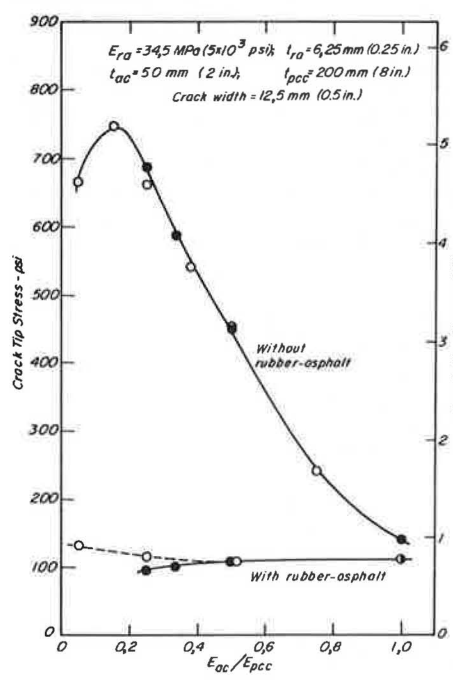


Figure 13. Effect of ratio stiffness of asphalt concrete to stiffness of portland cement concrete on crack-tip stress.

the pavement that does not contain such a layer. The thickness of the PCC layer has only a small effect on the stresses, and the effects of subgrade stiffness and thickness are minimal.

Figure 8 illustrates that the thickness of the AC overlay ($t_{overlay}$) has a significant effect on the stress at the crack tip in the pavement that does not have a rubber-asphalt layer, but little effect on that which does. [In Figure 8, interlayer thickness is constant at 6.4 mm (0.25 in).] An analysis of this type could be most helpful in assessing the trade-off between the thickness of the AC layer and the inclusion of the rubber-asphalt layer. For example, as a rough approximation for the particular situation represented by Figure 8, there appears to be a stress relief of about 0.7 MPa (100 lbf/in^2) at the crack tip for each additional 25 mm (1 in) in thickness of AC. Because there is an approximately 3.8-MPa (550-lbf/in^2) difference in stress levels between curves (a) and (b) at $t_{overlay}$ = 57 mm (2.25 in), it would seem that the pavement that does not have an interlayer would require an additional 140 mm (5.5 in) of AC [i.e., $t_{overlay}$ = 57 mm + 140 mm = 197 mm (7.75 in)] to achieve the same crack-tip stress level as 50 mm (2 in) of AC and a 6.4-mm rubber-

Figure 14. Effect of crack width on crack-tip stress.

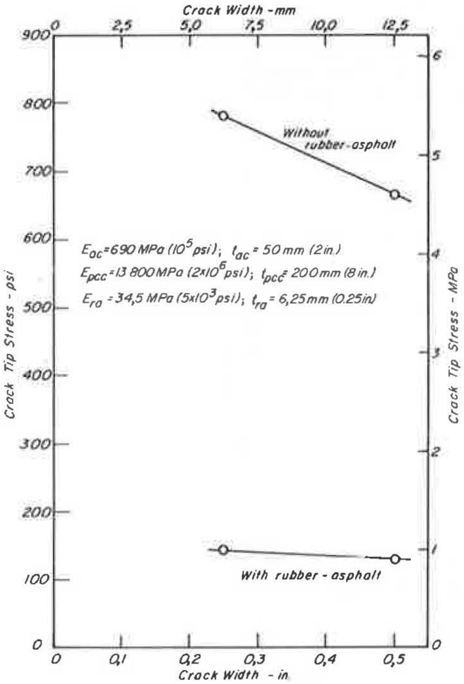
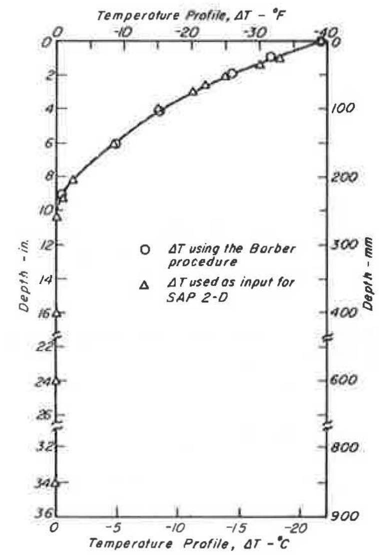


Table 2. Material properties used in thermal-stress determination.

Layer	Modulus (MPa)	Poisson's Ratio	Coefficient of Linear Expansion per °C	Thickness (mm)
Asphalt concrete	690	0.3	22.5×10^{-6}	50
Rubber asphalt	13.8	0.35	27.0×10^{-6}	6.25
Portland cement concrete	13 800	0.2	7.0×10^{-6}	200
Aggregate base	138	0.4	18.0×10^{-6}	300
Subgrade	69	0.48	18.0×10^{-6}	300

Note: 1 MPa = 145 lbf/in²; t°C = (t°F - 32)/1.8; 1 mm = 0.04 in.

Figure 15. Temperature profile for 22.2°C decrease in temperature at pavement surface.



asphalt interlayer. It is interesting that, for this thickness of rubber asphalt, at least, the maximum stress in the AC at its interface with the rubber asphalt is about the same independent of overlay thickness.

Curve (a) of Figure 9 illustrates the effect of element size on crack-tip stress for an overlay thickness of 57 mm on a pavement that does not have a rubber-asphalt layer. The elements varied in size from 1.6×0.24 mm (0.0625×0.009 375 in) (an aspect ratio of 6.7:1) to 1.6×1.6 mm, (an aspect ratio of 1:1). Generally, an aspect ratio of 2:1 or less is recommended because the increased accuracy expected from the smaller elements in the thinner layers will offset the effect of the less desirable aspect ratio.

Curve (b) of Figure 9 shows that the crack-tip stress decreases as the interlayer thickness decreases. This is expected in this case because the total overlay thickness was maintained constant; a decrease in interlayer thickness is equal to a corresponding increase in AC thickness. (The effect of element size, as noted above, must be considered here as well.)

As the interlayer thickness decreases, it would appear reasonable that the stresses at the interface of the AC and the rubber asphalt should approach the stresses of curves (a) and (b); i.e., the interface stress of curve (d) (in rubber asphalt) should converge with the crack-tip stress (curve b), which in fact it appears to do.

Similarly, it would be expected that the maximum interface stress in the AC (curve c) would first decrease with decreasing interlayer thickness (increasing AC thickness) and then to increase rapidly as the AC layer approaches the high strain zone at the crack tip; i.e., curves (a) and (c) should converge. From an examination of Figure 9, it would appear that this may occur. At an interlayer thickness of 2.25 mm (0.09 in), the AC interface stress is larger than the crack-tip stress in the rubber asphalt.

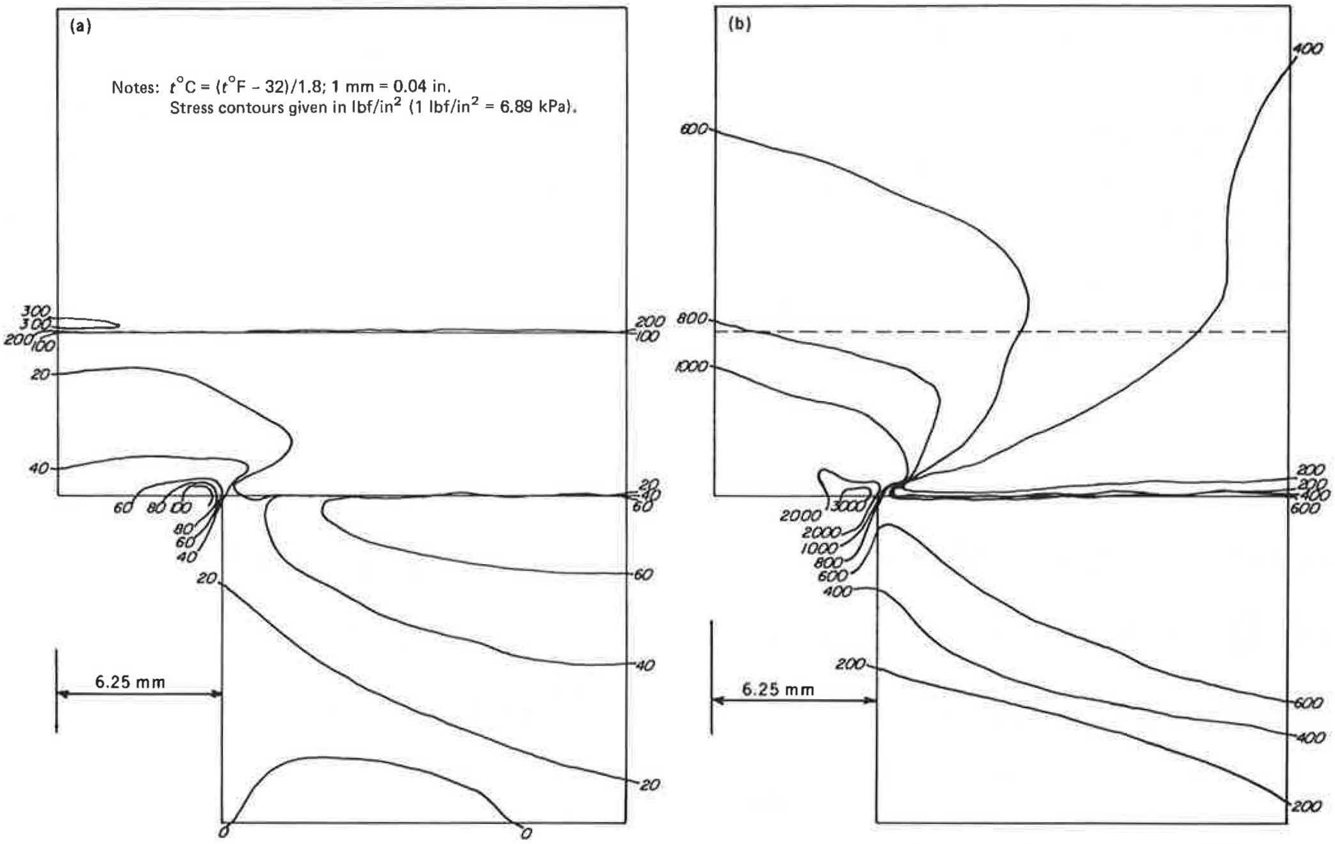
Figure 9 also shows the results of an analysis that compared stresses for 50 mm (2 in) of AC plus 12.5 mm (0.5 in) of rubber asphalt with those for 44.5 mm (1.75 in) AC plus 12.5 mm (0.5 in) of rubber asphalt. The results are essentially the same.

Figure 10 shows that, as the modulus of the rubber asphalt decreases, the crack-tip stress also decreases; this leads, in turn, to the conclusion that the estimated stresses at the crack tip in the pavements that contain rubber asphalt given thus far are probably larger than those that actually occur. As pointed out in the analysis of the creep-test data, a rubber-asphalt-modulus value of 34.5 MPa is probably too high; a value of 10.3-13.8 MPa (1500-2000 lbf/in²) may (possibly) be more acceptable. If this is the case, then from Figure 9, it can be expected that the effect of the lower modulus would be even more significant than already shown.

Figure 11 illustrates the effect of AC stiffness on crack-tip stress for a constant cracked-layer modulus of 13 800 MPa (2 000 000 lbf/in²). Figure 12 illustrates the effect of the modulus of the cracked layer at an AC modulus of 6900 MPa (1 000 000 lbf/in²). Figure 13 combines the data from Figures 11 and 12 and normalizes them by considering the ratio of the modulus of the AC layer to that of the cracked layer. The resulting curve is similar to that shown in Figure 11, and the points in Figure 13 appear to fit this curve well, indicating that this ratio is very important in determining crack-tip stress level, more so than the actual modulus of either layer. It will be noted that, the closer the modulus ratio is to unity, the better the performance of the overlay that does not have a rubber-asphalt interlayer as compared with the overlay that does.

Figure 14 illustrates the effect of variation of crack width. This is the most difficult task to perform on the finite-element mesh. However, the increase in stress found as the crack width decreased is consistent with what a theoretical (fracture mechanics) approach would predict.

Figure 16. Horizontal stresses resulting from 22.2°C decrease in temperature at overlay pavement surface: a 200-mm portland cement concrete pavement (a) with a rubber-asphalt interlayer and (b) without a rubber-asphalt interlayer.



A limited analysis was conducted by using the SAP program to estimate the stresses that result from temperature changes. The materials properties of the system examined are given in Table 2, and its temperature profile is shown in Figure 15. The stress distributions in the vicinity of the crack are illustrated in Figure 16 for a temperature decrease of 22°C at the pavement surface at a constant AC modulus. However, as shown in Figure 15, this temperature change is attenuated with depth and, in reality, the modulus of AC is a time-dependent parameter. Thus, it is probable that the differences in stress between the two cases are not as large as indicated in Figure 16, although the effect of the interlayer in reducing thermally induced stresses in the overlay is obviously significant.

SUMMARY

Consideration of Figures 4, 5, and 16 shows that the reflection-cracking problem can, in the first stage at least, be directly attributed to the high stresses that will develop in the overlay as a result of the discontinuities in the cracked layer. It is suggested that the problem of load-associated reflection cracking should be considered in two stages: viz., first, attention should be directed to the attenuation of the high stress that occurs as a result of the cracks and, second, because the maximum vertical deflections of the overlay occur at these cracks, a fatigue analysis should be carried out for the overlay at these points after the stress-concentrating effect of the cracks has been nullified.

For thermal stresses, of course, only the first stage is of interest. As is apparent from Figures 7-14 and 16, a low-modulus interlayer (rubber asphalt) can significantly inhibit both load- and temperature-change-induced reflection cracking, particularly when the modulus of the proposed overlay is of the order of 0.1-0.25 that of the cracked-layer modulus. This investigation has also shown that crack width, interlayer modulus, and overlay thickness appear to have significant effects on crack-tip stress, but that the ratio of the modulus of the overlay to that of the cracked layer appears to be the major factor involved.

It should be noted that this study has focused on crack-tip stress. When a rubber-asphalt interlayer is used, this crack-tip stress occurs in the interlayer; if the strength of the interlayer material is such that the crack-tip stress does not cause it to yield, then the crack-tip stresses for the cases that do not have interlayers should be compared with the maximum stress in the AC layer for the case that has an interlayer. In general, this stress is lower than the crack-tip stress, as can be seen by comparing curves (b) and (c) in Figures 8 and 9. Furthermore, because the rubber-asphalt modulus used for these analyses is probably too high, the crack-tip stresses when the interlayer is included are conservative (high).

The finite-element representation used results in high stresses at the lower tip of the crack, i.e., in the granular base course. To simulate the yielding, further analyses were made in which the restraints on this material were released to such an extent that the stress in the base course was at an acceptable level. The effect of this was to marginally increase the crack-tip stresses, so that the values plotted can be taken to be valid.

Finally, it should be pointed out that the analyses typified the interlayer as a low-modulus material exhibiting linear elastic behavior. Stress and strain distributions (e.g., Figures 4 and 5) indicate also that the material should be able to withstand high strain levels without rupturing. From the limited tests performed on the rubber asphalt, it appears to have all the desirable properties, an observation that is supported by reports (1, 2) of its successful performance in practice.

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Attempts to Reduce Reflection Cracking of Bituminous Concrete Overlays on Portland Cement Concrete Pavements

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Studies of methods used in Virginia to reduce the incidence of reflection cracking when portland cement concrete pavements or bases are overlaid with asphalt concrete are reported. The methods discussed are (a) the use of sand to break the bond between the portland cement concrete pavement and the asphalt overlay and (b) the use of a fabric that has a high tensile strength as a stress-relieving layer between the asphalt layer and the concrete base. The studies showed that neither the sand bond breaker nor the high-strength fabrics are effective in reducing reflection cracking where differential vertical joint movements are a significant factor. Further studies showed that high-strength fabrics can delay the onset of reflection cracking but that such cracking will eventually develop under the application of repetitive wheel loadings.

The transverse joints in rigid pavements commonly reflect through bituminous concrete overlays in a short time. Many highway engineers believe that these reflection cracks are detrimental to pavement riding quality, and others believe that they are generators of future maintenance problems because they provide surface water ready access to subsurface pavement layers (1). Recent studies support this latter belief; it has been reported that a crack only 0.9 mm (0.035 in) wide will admit 70 percent of the surface water that falls on a pavement sloped 1.25 percent under a 50-mm/h (2-in/h) rate of precipitation (2).

METHODS USED

Numerous attempts to reduce reflection cracking have been reported in the literature. A good summary of those that have been at least partially successful is given in the National Cooperative Highway Research Program Synthesis on Pavement Rehabilitation (1). In that document, most of the methods attempted are grouped into four general classifications: (a) increased thickness of the asphalt concrete (AC) overlay, (b) special treatment of the existing PCC pavement, (c) special consideration of the AC overlay design, and (d) treatment of joints and cracks.

In Virginia, most of the methods in categories a through c have been rejected for economic or other reasons. The category d methods used in Virginia all consist of some method of breaking the bond or otherwise relieving the stress between the PCC and the bituminous concrete overlay. The first attempts to provide a bond breaker were reported by Hughes, who found that a thin layer of sand spread on either side of the PCC pavement joints before the application of a bituminous concrete overlay was partially successful in reducing reflection cracking. In his studies, an asphalt-emulsion tack coat was applied at a rate of 0.23-0.46 L/m^2 (0.05-0.10 gal/yd^2) for a distance of 225-300 mm (9-12 in) on either side of the transverse joints, and the class A sand sieved to pass a 9.5-mm (3/8-in) sieve was applied over the tack coat to a thickness of approximately 6 mm (1/4 in). A 59 to 95-kg/m^2 (100 to 175-lb/yd^2) AC overlay (85-100 penetration grade asphalt) was applied over the pavement surface and the sanded joints. Joint spacings were 9 m (30 ft). However, of the three projects treated in such a manner, only one showed any indication of fewer reflection cracks on the joints treated with sand. There was no apparent reason for any differences in performance among the three projects, and nine years later, some of the joints still had not reflected through the best-performing project, that located on US-13.

The next significant attempt to reduce reflection cracking, also reported by Hughes, involved the use of a nonwoven polypropylene fabric spanning the reflective cracks on a previously overlaid concrete pavement on US-460 (2). The polypropylene had a high tensile strength and was reported to prevent horizontal overstressing of the overlay. Supposedly, at points of stress concentration such as transverse joints or cracks, the material would prevent reflection cracking. Again, the joint spacing was 9 m; the fabric was applied by using approximately 1.1 L/m² (0.25 gal/yd²) of cationic asphalt-emulsion tack coat. The fabric was applied in 0.9-m (3-ft) wide strips approximately centered on the cracks and running lengthwise with the cracks. A total of 99 joints, all of which had discernible cracking in the previous overlay, were treated in this manner. A 68-kg/m² (125-lb/yd²) AC (85-100 penetration grade asphalt) overlay was applied after the fabric had been under traffic for about 12 hours.

The performance of the fabric-treated section, like that of the sanded sections, was disappointing. After three months under traffic, many of the joints were reflected through the second overlay (although there was somewhat more cracking in an adjacent section where no fabric had been used).

As a result of these partially successful experiments, field-test studies were undertaken in 1972 to determine the mechanism of reflection cracking on overlays of jointed PCC pavements. This paper summarizes these studies.

US-460 PROJECT

Horizontal Joint Movements

Before the second overlay on the US-460 project was placed, control and test sections were chosen for studies of horizontal joint movements. It was hoped that, by monitoring the horizontal hydrothermal movements of typical joints in both the section that had a fabric reinforcement and a control section that did not have such a reinforcement, it would be possible to determine the effect of such movement on the ability of the fabric to reduce reflection cracking. Five consecutive joints in both the control and the test sections were selected for monitoring. After the placement of the overlay on the control section and the fabric and overlay on the test section, gauge points were embedded in the overlay such that a nominal 250-mm (10-in) gauge length would span the area subject to reflection cracking. These gauge points were established at each of the 10 previously selected joints. Initial readings of the exact measurements between the gauge points were taken on August 25, 1971, the day after the overlay was placed. At the same time, readings were taken on reference (calibration) points embedded in the AC at places where no cracking was expected to occur. Thus, it would be possible to correct the measurements spanning the reflection cracks for the length change occurring in a 250-mm segment of uncracked pavement. A realistic measure of crack movement was anticipated through this adjustment.

Readings of both the test and calibration points were taken at monthly intervals for 30 months subsequent to the overlay. During the first 3 months, reflection cracks developed between the gauge points at three locations in each section. At one location in each section, a reflection crack developed outside the limits of the gauge points. At the end of the 30-month period, only one joint in each section had not reflected through the overlay. The results of these studies are shown in Figure 1; a positive number on the ordinate of this figure indicates joint opening and a negative number indicates joint closure. It is apparent that the fabric-reinforced test section and the control section behaved in a similar manner; there is no evidence from these tests that the stress-relieving layer provided any advantage in preventing reflection cracking. Once the cracks had formed, their behavior was generally as would be expected for the first year and seasonal movements were clearly evident. However, there is no ready explanation for the strange behavior of the measurements after the first year. Obviously, the indication that the cracks (which were clearly visible) took on negative widths is ridiculous. It is evident that, for unknown reasons, the distances between the gauge points became somewhat less than the nominal 250 mm originally established. This anomaly may be related to the humping effect noted at many transverse reflection cracks in Virginia. In such cases, a gradual upheaval or accumulation of AC at the reflection cracks results in noticeable roughness.

Vertical Joint Movements

The realization that most of the reflection cracking in the US-460 project had occurred during the first three months, when the measured horizontal movements had been minimal, led to the consideration of other factors that might contribute to the cracking. Because the pavement showed significant evidence of joint faulting and pumping, it was considered probable that vertical movement of the joints might be such a factor.

In April 1972, deflection tests were conducted at the joints on both the fabric-reinforced and the control sections. The procedure for these tests is indicated in Figure 2. A Benkelman beam (A in Figure 2) is placed on the shoulder of the road with its point near the edge of a reflection-cracked joint or of a joint that has not reflected through. A dump truck loaded to 80 kN (18 000 lbf) on its rear axle is positioned on the opposite side of the joint. At this point (point 1, Figure 2), an initial beam reading is taken. The truck is then driven slowly across the joint, and beam readings are taken as points 2 and 3 are traversed. The edge-deflection reading for point 2 (D_2) indicates the deflection when the wheel load is directly at the joint. The comparison between the reading for point 1 (D_1) and D_2 indicates the load-transfer efficiency, and the reading for point 3 is used to ensure that the Benkelman beam, still located at point 2, is no longer within the area of influence of the wheel load.

The results of these tests are shown below (1 mm = 0.04 in).

Section	Joints No.	Joints %	Avg. Deflection (mm) D_2	D_1	$D_2 - D_1$
Fabric treated					
Cracked	57	58	0.35	0.23	0.12
Uncracked	42	42	0.23	0.20	0.03
Control					
Cracked	90	73	0.38	0.25	0.13
Uncracked	34	27	0.30	0.25	0.05

The differential joint deflection ($d = D_2 - D_1$) can be interpreted as a function of the load-transfer capability of the joint; i.e., if the load transfer is 100 percent, $d = 0$.

These tests were run when the overlay was approximately eight months old. Traffic records show that the test sections sustain an average of more than 600 vehicles/day in the 2-axle, 6-tire-or-larger truck and bus categories.

As shown above, the fabric-treated section had somewhat less reflection cracking than the control section (58 and 73 percent of joints cracked, respectively). A later survey (September 1974) showed 61 and 75 percent of joints cracked, respectively. The net joint deflection (D_2) may have some effect on the cracking; the average deflections of the uncracked joints are somewhat less than those of the cracked joints in both the fabric-treated and the control sections. More descriptive, however, is the differential joint deflection (d); the uncracked joints have very low average d-values [0.03-0.05 mm (0.001-0.002 in)] but the cracked joints average 0.13 mm (0.005 in) in both sections.

An analysis of the distribution of d-values is given in

Figure 1. Movement of cracks with age.

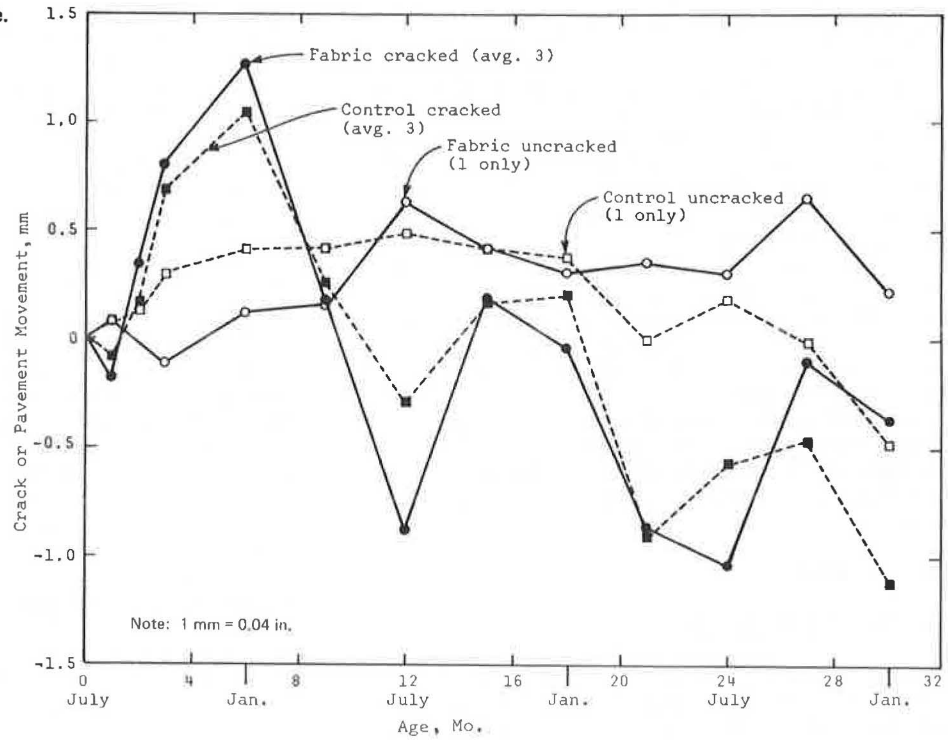
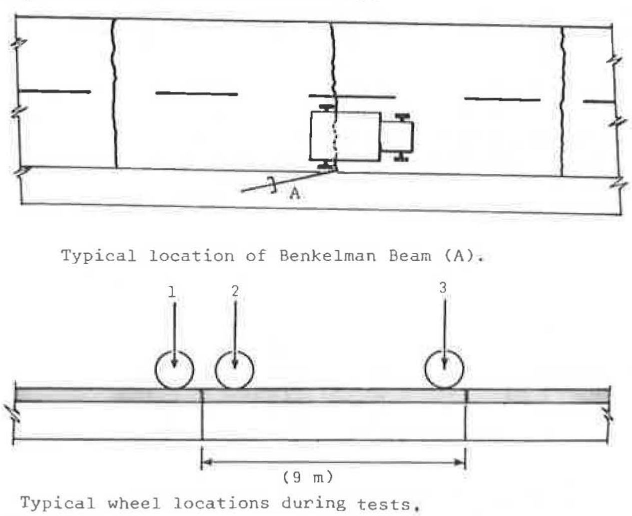


Figure 2. Schematic of deflection-testing procedure.



Note: 1 m = 3.3 ft.

Table 1, where cracking frequency is given as a function of differential deflection in increments of 0.05 mm (0.002 in) (the smallest reading of the Benkelman beam used). When the differential deflection was 0 (load transfer = 100 percent), the fabric had a marked effect on reflection cracking; of 20 treated joints, none were cracked, although 4 of the 9 joints in the control section were cracked. Similarly, but less dramatically, when d was 0.05 mm, 29 and 54 percent of the fabric-treated and the control sections, respectively, had reflection cracks. Finally, when d was greater than 0.20 mm (0.008 in), all joints in both the control and the fabric-reinforced sections had reflection cracks.

Clearly, when joints have essentially 100 percent load-transfer capability, the reason for the absence of reflection cracking could be that the joints simply are not functioning. In such a case, no stress concentrations or cracking would be expected. This may well explain the absence of cracking at the 5 untreated joints where the differential deflection was 0. The 4 untreated joints where cracking did occur may be working joints where load transfer is fully effective. Thus, it is likely that many of the 20 fabric-treated joints that were uncracked and had a differential deflection of 0 were working joints where the fabric served its intended purpose of reducing overlay stresses to the point that no cracking occurred. Conversely, it is likely that, for those joints that had higher differential deflections, the fabric, a thin sheet, could not sufficiently distribute the sheer stresses and was thus unable to reduce reflection cracking significantly.

If this hypothesis is accepted, it follows that much, if not most, of the reflection cracking on the treated joints was the result of concentrations of sheer stress induced as wheel loads traversed the joints and caused differential deflections. Luther and others (3) have since established that reflection cracking of asphalt overlays is due to multimodal fatigue fracture. Their paper, based on laboratory-model studies, states in part:

> It was observed that these [reflection] cracks propagate from the surface under mixed mode conditions arising from compressive bending stresses and high sheer stresses induced by differential vertical movement between the underlying rigid concrete layer.

If, as seems to be the case, reflection cracking is fatigue in nature and differential vertical movements are a major cause, it is reasonable to assume that the rate of crack development will be a function of the frequency of wheel loadings and the magnitude of the vertical movements (differential deflection) caused by these loadings. This concept seems to have been substantiated on the US-460 project, where, with approximately 600 heavy loads/day, there was a 30 percent increase in cracking between April 1972 and September 1974, but the joints that had very low differential deflections in 1972 were still uncracked in 1974.

Table 1. Cracking and differential deflection: US-460.

Differential Deflection (mm)	No. of Joints Cracked		No. of Joints Uncracked		Percentage of Joints Cracked	
	Fabric Treated	Control	Fabric Treated	Control	Fabric Treated	Control
0	0	4	20	5	0	44
0.05	7	20	17	17	29	54
0.10	23	35	3	12	88	74
0.15	15	11	2	0	88	100
0.20	12	20	0	0	100	100

Note: 1 mm = 0.04 in.

Figure 3. Core through new overlay (top), polypropylene fabric, and old overlay.

Cores

During the September 1974 crack survey, five 100-mm (4-in) diameter cores were removed from the US-460 pavement. Each core was taken at a reflection crack in the fabric-treated section and was located so as to intercept the crack approximately as a core diameter. These five cores were returned to the laboratory for study. The results of these studies are indicated in Figure 3, where it can be seen that the core through both AC layers is held together by the polypropylene fabric. The reflection crack is plainly visible both above and below the fabric. Attempts to separate the AC from the fabric showed that all were firmly bonded together so that some effort was required to remove either AC layer. When the AC had been removed, the fabric showed no evidence of damage but had a slight wrinkle that corresponded to the location of the reflection crack.

The observation that there were no tears or other signs of elongation of the fabric was taken as further evidence that the reflection cracking had been caused primarily by vertical joint movements.

Joint Pumping

A coincidental observation of the 1972 and 1974 surveys was that the polypropylene fabric spanning the transverse joints might also have helped to reduce pumping. At both times about 15 percent of the joints in the control section were observed, on the basis of the ejection of fines from the subbase or subgrade, to be pumping, but no cases of pumping were observed in the fabric-treated section during either survey. It may be conjectured that the fabric, which had been asphalt-impregnated during its manufacture and applied with a heavy tack coat of liquid asphalt, served as a barrier to surface water entering the joints and thus prevented pumping. However, because joint pumping had not been a consideration early in the study, no data are available on the incidence of pumping before the fabric was applied and, thus, no firm conclusions can be offered on this matter.

US-13 PROJECT: VERTICAL JOINT MOVEMENTS

The apparent relationship between vertical joint movements and the effectiveness of the stress-relieving layer on the US-460 project led to speculation that such movements might also be significant where sand had been used as a bond breaker between an AC overlay and a jointed PCC pavement. Because, as noted above, the sand had been partially successful on the US-13 project, it was decided to conduct joint-deflection tests at that site. These tests were conducted, in the manner described above, in June 1972, when the test section was six years old. At that time, of 60 control or untreated joints, 100 percent exhibited reflection cracking and, of 232 sanded joints, 155 (66 percent) had such cracking. The results of the tests are summarized in Table 2.

Thus, although a sand layer can be effective in reducing reflection cracking, the degree of this effectiveness is strongly affected by the magnitude of the differential deflection. For example, after six years, the sand layer appears to have been 76 percent effective where there was no differential deflection but only 7 percent effective where the differential deflection was as much as 0.15 mm (0.006 in).

A more recent survey of this project showed that, after nine years, 93.5 percent of the sanded joints exhibited reflection cracking. Thus, it appears that the fatigue nature of reflection cracking is again shown on this project, where traffic volumes include an average of 335 vehicles/day in the 2-axle, 6-tire-or-larger truck and bus categories.

I-95 PROJECT

As a result of the studies described above, which indicated that stress-relieving layers could be effective in delaying reflection cracking where differential vertical joint movements were minimized, several test sections were placed on a segment of I-95 under construction in northern Virginia in July 1972. This project, called the Mixing Bowl, is a part of the multilane highway network near the Pentagon and has a composite pavement overlying a very rigid foundation. The pavement design

Table 2. Cracking and differential deflection: US-13.

Differential Deflection (mm)	No. of Joints Cracked		No. of Joints Uncracked		Percentage of Joints Cracked	
	Sand Treated	Control	Sand Treated	Control	Sand Treated	Control
0	4	1	13	0	24	100
0.05	58	15	43	0	57	100
0.10	66	28	19	0	77	100
0.15	27	14	2	0	93	100

Note: 1 mm = 0.04 in.

Table 3. Fabric-reinforced cracks available for study: September 1975.

Site	Location	Date Overlaid	No. of Cracks		
			Polypropylene Treated	Nylon Treated	Control
1	VA-27: northbound lane	9/18/72	22	0	0
2	I-95: southbound lane	9/18/72	25	25	8
3	I-95: southbound lane	10/31/72	0	0	29
Total			47	25	37

Figure 4. Core through bituminous layers and polypropylene fabric: I-95 project.

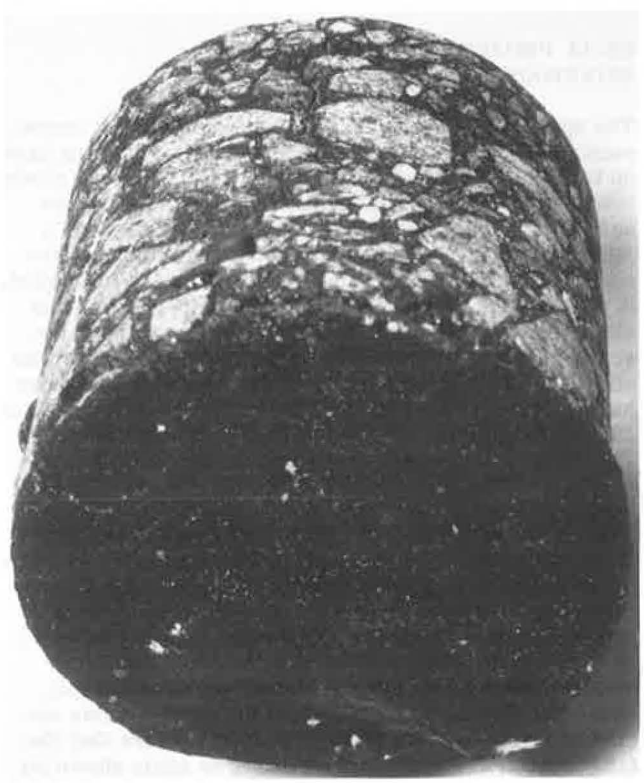


Figure 5. Core through bituminous layers and nylon fabric: I-95 project.

features are described below (1 kg/m = 1.85 lb/yd and 1 mm = 0.04 in).

Pavement Feature	Material	Thickness (mm)
Surface	54-kg/m^2 bituminous concrete	13
Binder	136-kg/m^2 bituminous concrete	19
Base	Plain PCC	200
Subbase	Cement-stabilized material	200

It was expected that the extremely rigid base and subbase layers would reduce vertical joint motions to a minimum so that the provision of a stress-relieving layer between the bituminous concrete layer and the PCC base would reduce the incidence of reflection of the shrinkage cracks in the concrete base. Plans called for the installation of two fabric stress-relieving materials, each on approximately 100 shrinkage cracks.

The details of the installation have been reported by McGhee and Hughes (4), and some of the more important features are summarized below along with the results of three years of performance studies.

Materials and Application

The materials applied were (a) an asphalt-impregnated,

Table 4. Cracking by lane: site 2—September 1975.

Land	Cracks Reflected					
	Polypropylene Treated		Nylon Treated		Control	
	Percentage	Function A*	Percentage	Function A*	Percentage	Function A*
Acceleration					100	8 of 8
Traffic	71	5 of 7	83	5 of 6		
Middle	55	5 of 9	67	6 of 9		
Passing	33	3 of 9	60	6 of 10		

*Function A = number of reflection cracks as function of total number of treated cracks in PCC base.

nonwoven polypropylene fabric and (b) a nonwoven, spun-bonded nylon. After the concrete base was old enough to develop shrinkage cracks [which occurred at approximately 9-m (30-ft) intervals] but before the application of bituminous concrete layers, the cracks were located for installation of stress-relieving materials with respect to permanent reference points on the roadway or median. Before the materials were placed, each crack was tacked for its full length [a 3.6-m (12-ft) lane width] and for 0.45 m (18 in) on either side with approximately 1.1 L/m^2 (0.25 gal/yd^2) of cationic asphalt emulsion. After the tack had cured for 1-3 h, the fabric was broomed into place to ensure good adhesion; the polypropylene appeared to absorb the tack better and to adhere more uniformly to the base course than did the nylon.

Because of numerous problems, many of the fabric-treated cracks were not suitable for evaluation by the time the bituminous concrete layers had been placed. Since the overlay was placed, many of the treated cracks have been under traffic volumes of more than 40 000 vehicles/day, and no evaluation has been possible. The result is that at present only two test sections and one control (no fabric) section are available for evaluation. The results are summarized in Table 3.

Results

Periodic surveys of the three test sites showed an early difference in the number of reflection cracks. For example, in February 1973, when little traffic had used the sites, there were 0, 5, and 16 reflection cracks detected in the binder course on sites 1, 2, and 3, respectively. Thus, there was a clear indication that the fabric on sites 1 and 2 was being effective in reducing the incidence of reflection cracking at an early age of the overlay. In April 1973, soon after the final surface had been applied, no cracks could be detected in any of the three sections, and no significant cracking developed in the surface course during the summer of 1973. However, during the winter of 1973/74, when hydrothermal pavement movements could be expected to be most conducive to reflection cracking, numerous cracks began to develop in the control section and in site 2. It also became clear during this period that new cracks were developing in the unreinforced concrete base and were, in turn, being reflected through the AC surface. As a result, by July 1974, there were 32 reflection cracks in the control section (site 3) and 45 cracks in site 2, many of which were newly developed at the base-course level. At the same time, there were only 2 cracks in site 1.

Also, in July 1974, deflection measurements were made. Similar to the results for the US-460 project described above, the average differential deflection on visible cracks was 0.05 mm.

Cores removed from site 2 during July 1974 showed results similar to those described above for the US-460 project. For both the polypropylene- and the nylon-fabric-treated cracks, the cores showed that the cracks were directly above cracks in the PCC base and that the fabric was still intact and showed no signs of distress (see Figures 4 and 5).

Final surveys of the cracking on the I-95 project were conducted in September 1975. Again, there were a number of cracks that had developed in the base concrete and reflected through the bituminous layers after the installation of the test sections. However, as shown in the table below, which is based on the original cracking in the three test sites, both fabrics were somewhat effective in at least delaying the onset of reflection cracking.

Site	Traffic (vehicles per day)		Percentage of Cracks Reflected		
	Total	Large Trucks and Buses	Polypropylene Treated	Nylon Treated	Control
1	19 000	270	41		
2	42 500	3050	52	68	100
3	42 500	3050			90

There is also some evidence that the polypropylene was more effective than the nylon. The traffic characteristics given above are indicative of the service conditions but cannot be used to establish a relationship between traffic volume and reflection cracking. Although site 1 had been subjected to the indicated traffic for most of the 3-year period, because of construction-related detours, sites 2 and 3 had had only sporadic traffic.

However, the effects of traffic volume and fatigue on the rate of development of the reflection cracking are shown in a detailed study of site 2. The frequency of reflection cracking is greatest in the lanes subject to the most traffic, particularly trucks [see Table 4 (where the outermost lane has been designated as the acceleration lane, the second as the traffic lane, the third as the middle lane, and the innermost as the passing lane)]. Clearly truck traffic will be heaviest on the acceleration and traffic lanes. On this site, there was a marked decrease in reflection cracking on the lanes where there would be less truck traffic. There was also a significant difference in cracking between the polypropylene- and the nylon-fabric-treated cracks, although there is no explanation for this difference. Finally, all of the untreated cracks in the acceleration (control) lane have reflected through.

CONCLUSIONS

The following conclusions appear to be warranted from the studies reported above.

1. Neither sand as a bond breaker nor high-strength fabrics as stress-relieving layers are effective in reducing reflection cracking where vertical joint movement (differential deflection) is a significant factor.
2. At differential deflections greater than approximately 0.05 mm (0.002 in), reflection cracks from very early. Such cracking is delayed at lower differential deflections but will occur as the magnitude and frequency of wheel loadings increase.
3. When placed to span the joints in a PCC pavement or the cracks in a PCC base and then covered with an AC overlay, both asphalt-impregnated, nonwoven poly-

propylene and nonwoven, spun-bonded nylon fabrics can sustain the formation of reflection cracking in the overlaying layer without being damaged themselves.

4. An asphalt-impregnated, nonwoven polypropylene fabric spanning the joints in a PCC pavement and placed between the pavement and an asphalt overlay can be effective in reducing the infiltration of surface water to pavement sublayers. There is some evidence that pavement pumping may be reduced by this method.

5. Both asphalt-impregnated, nonwoven polypropylene and nonwoven, spun-bonded nylon fabrics can delay the formation of reflection cracking. There is strong evidence, however, that such cracking is fatigue in nature and will eventually develop under the application of repetitive wheel loadings.

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