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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE

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ASPHALT OVERLAY DESIGN PROCEDURES

TRANSPORTATION RESEARCH BOARD
National Research Council

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

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PREFACE

A vast storehouse of information exists on nearly every subject of concern to highway administrators and engineers. Much of this information has resulted from both research and the successful application of solutions to the problems faced by practitioners in their daily work. Because previously there has been no systematic means for compiling such useful information and making it available to the entire highway community, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize useful knowledge from all available sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series reports on various practices, making specific recommendations where appropriate but without the detailed directions usually found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems. The extent to which these reports are useful will be tempered by the user's knowledge and experience in the particular problem area.

FOREWORD

By Staff
Transportation
Research Board

This synthesis will be of interest to pavement designers and others concerned with the design of asphalt concrete overlays. Information is presented on reasons for overlaying a pavement and on the various methods available for design of an asphalt overlay.

Administrators, engineers, and researchers are continually faced with highway problems on which much information exists, either in the form of reports or in terms of undocumented experience and practice. Unfortunately, this information often is scattered and unevaluated, and, as a consequence, in seeking solutions, full information on what has been learned about a problem frequently is not assembled. Costly research findings may go unused, valuable experience may be overlooked, and full consideration may not be given to available practices for solving or alleviating the problem. In an effort to correct this situation, a continuing NCHRP project, carried out by the Transportation Research Board as the research agency, has the objective of reporting on common highway problems and synthesizing available information. The synthesis reports from this endeavor constitute an NCHRP publication series in which various forms of relevant information are assembled into single, concise documents pertaining to specific highway problems or sets of closely related problems.

A pavement overlay may be required because of inadequate ride quality, excessive pavement distress, reduced friction between tire and pavement, high user costs, or inadequate structural capacity for planned use. This report of the Transportation Research Board discusses the current methods used for designing asphalt concrete overlays with emphasis on deflection-based and analytical procedures.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the researcher in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis is an immediately useful document that records practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As the processes of advancement continue, new knowledge can be expected to be added to that now at hand.

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Information on current practice was provided by many highway and transportation agencies. Their cooperation and assistance were most helpful.

ASPHALT OVERLAY DESIGN PROCEDURES

SUMMARY

Resurfacing or overlaying a pavement may be required for one or more of the following:

- Inadequate ride quality
- Excessive pavement distress
- Reduced coefficient of friction between tire and pavement
- Unacceptable user costs
- Inadequate structural capacity for planned use

The guidelines that need to be considered in selecting a resurfacing treatment should include the following factors: present condition, volume and weight of traffic, structural evaluation of existing pavement, rainfall and temperature range, surface and subsurface drainage, design life of the treatment, and materials in original construction and those to be used for overlay.

The traditional approach for the design of asphalt overlays before 1960 was based on engineering judgment or component analysis. Since 1960 nondestructive (deflection-based) procedures have gained wide acceptance. In the mid 1970s quasi-mechanistic (analytically based) procedures were developed for overlay design.

A pavement's functional performance, structural performance, structural capacity, and safety contribute to the need for an overlay. Functional performance refers to the ability to provide a serviceable ride quality. Structural performance is the ability of a pavement to maintain structural integrity without exhibiting distress (i.e., fracture, distortion, disintegration). Structural capacity is the pavement's ability to accommodate traffic. Safety is evaluated in terms of skid resistance and hydroplaning potential.

Component-analysis, overlay design procedures require samples of material from the existing pavement; laboratory tests on these samples are performed to develop design parameters. The design parameters include a measure of the strength of the subgrade soils, characteristics of the paving materials, traffic estimates, and, sometimes, a regional environmental factor.

Deflection-based, overlay design procedures require some type of deflection testing of the existing pavement. In addition, a pavement condition survey will establish the need for maintenance or rehabilitation, identity homogeneous segments (analysis sections), indicate design requirements, and give special conditions that will influence the overlay design.

Analytically based design procedures also require deflection testing, condition surveys, and traffic estimates. In addition, some measure of the stiffness properties and distress characteristics of the existing pavement are required.

The life cycle of an overlay will depend on the accuracy of traffic estimates, the ability to measure structural capacity, and the durability of the materials used in the overlay.

Each of the three types of overlay design procedures has advantages and disadvantages, which are enumerated in the synthesis. The analytically based procedure should be developed further because it provides the most comprehensive approach to evaluation of the existing structural capacity and to the design of overlays. Regardless of which procedure is used, it is recommended that both deflection measurements and condition surveys be used to establish analysis sections. For major facilities, materials should be evaluated, particularly for the subgrade. In any event, designers should not sacrifice good engineering for the sake of simplicity.

CHAPTER ONE

INTRODUCTION

As the nation's highways age and are subjected to ever increasing loads and volume of traffic, they will inevitably deteriorate and eventually require some type of treatment to be able to provide a safe and serviceable facility for the user.

The types of treatments that are appropriate to maintain pavement serviceability can range from relatively simple maintenance to complete reconstruction, depending on the circumstances.

For pavements subjected to moderate and heavy traffic, the most prevalent treatment at the present time (1984) is to overlay the pavement with asphalt concrete. New treatments involving recycling of on-site materials and special treatments to minimize reflection cracking are finding increasing use in specific situations.

The primary purpose of this synthesis is to review methods for the design of resurfacing layers (overlays). Information pertinent to the use of alternative treatments, such as grinding and surface seals, will also be discussed and some reference will be made to the use of recycled materials as part of a rehabilitation strategy.

NEED FOR RESURFACING (GUIDELINES)

Resurfacing or rehabilitation of a pavement, including overlays, grinding, and recycling, may be required for one or more of the following reasons:

1. Inadequate ride quality
2. Excessive pavement distress
3. Reduced coefficient of friction between tire and pavement
4. Excessive maintenance requirements
5. Unacceptable user costs
6. Inadequate structural capacity for planned use

The application of a properly designed overlay can provide a cost-effective way of correcting each of the deficiencies enumerated above for a substantial period of time. Grinding and recycling (with or without overlay) provides an added dimension for rehabilitation and deserves attention by the designer when considering alternative rehabilitation procedures.

The guidelines that need to be considered in selecting a resurfacing or rehabilitation treatment should include the following factors as a minimum:

1. Present condition with regard to both smoothness and distress.
2. Traffic, both past and future in terms of both volume and weight of vehicles.
3. Structural evaluation of existing pavement.
4. Environment, usually as represented by rainfall and temperature (both high and low regimes).
5. Drainage of both surface and subsurface water.
6. Terrain with respect to general topography; i.e., mountainous, flat, cut or fill, and transitions.
7. Constraints imposed by contiguous structures; i.e., bridges, drainage structures, curbs and gutters, shoulders.
8. Design life required for treatment.
9. Materials used in original construction and planned for overlays.
10. Age of pavement.
11. Maintenance history (frequency, cost).

The specifics of the above factors as they apply to decisions relative to treatments may or may not be included in design procedures. The commonly used design factors are (a) present condition of pavement with regard to distress, (b) traffic in terms of equivalent 18-kip (80-kN) axle loads estimated for design period, and (c) structural capacity of existing pavement. Implicit in most overlay design procedures is a consideration for environment.

Environment is included in the overlay design procedure because most procedures have been developed empirically by state agencies; thus, methods for evaluating material properties and structural capacity include environmental considerations.

Overlay design procedures assume that adequate drainage will be provided; hence, the designer must evaluate drainage requirements as a separate design consideration in conjunction with any resurfacing or rehabilitation treatment. Considerations regarding terrain or constraints imposed by contiguous structures fall into the same category as drainage; i.e., they must be considered as a separate design consideration.

Design life is often inherent in the design procedure or may be specified by agency policy. Asphalt overlays are often designed for a 10-year life cycle; some agencies believe that longer life cycles (e.g., 20 years) are possible. An extended life cycle may require multiple overlays. Consideration of funds available and user inconvenience associated with frequent overlays can also affect the choice of a design life. Experience with performance of overlays will also be a consideration.

The materials used in the original construction can influence overlay requirements. For example, overlays over portland cement concrete (rigid) will involve different design considerations than those used for overlays over asphalt concrete (flexible) pavements.

Materials and procedures used for construction of the overlay can influence design recommendations. For example, the use of a cushion course between old pavement and new asphalt con-

crete surface or special treatment incorporating various stress relieving layers could influence the final design recommendation.

The type, amount, and severity of distress in the existing pavement are important considerations with regard to resurfacing and rehabilitation. NCHRP Synthesis Reports 76 (*1*) and 92 (*2*) discuss the collection and use of condition data and methods for minimizing reflection cracking of pavement overlays.

OVERLAY DESIGN PROCEDURES

The traditional approach used before 1960 for the design of asphalt overlays was based on (a) engineering judgment or (b) component analysis. Since 1960 the use of nondestructive (deflection-based) procedures has gained wide acceptance and in the mid 1970s several quasi-mechanistic procedures have been developed for overlay design. Mechanistic design procedures are based on the correlation of stress and strain or deformation using layered system analysis and observations or performance. No specific design procedures have, as yet, been developed that specifically incorporate structural or life-cycle considerations with recycled materials or stress-relieving interlayers.

The use of engineering judgment usually involves a decision by the person or persons responsible for the maintenance of the pavements within a specific geographical or political subdivision. Typically a district engineer or equivalent, with whatever staff assistance is available, will decide on the overlay requirements based on the pavement condition and a knowledge of performance at the site.

The use of component analysis as described in Chapter 3 involves the comparison of the existing pavement structure with a new pavement design for site-specific conditions and traffic. A considerable amount of judgment is required in this method to evaluate the structural coefficients for pavement layers; e.g., cracked or raveled asphalt concrete and degraded base and subbase. Also, this analysis often relies on laboratory procedures to predict the equilibrium conditions of materials; e.g., density and water content.

The use of deflection measurements as described in Chapter 3 makes use of nondestructive testing to estimate the in-situ structural capacity of a pavement at the time the measurements are made. Measurements are obtained by testing in-service pavements. It is generally believed that such testing will be more representative of in-situ conditions and will not rely on predictions of water content or density as is required by component-analysis procedures. Specific methods for selection of the design deflection are dependent on the design procedure and the observed condition of the pavement. Overlay thickness requirements from deflection measurements are based on empirical correlations with field performance; hence, it is essential that procedures for each method be strictly followed if reliable results are to be obtained. Intermixing criteria or procedures developed by different agencies could be misleading as regards design recommendations.

TABLE 1

MONTHS OF OVERLAY PERFORMANCE FOR THREE DESIGN METHODS (CALTRANS)

Design Method	Design Period	Performance	
		Average	Range
Engineering judgment	120	94	25-163
Gravel equivalency	120	100	27-173
Deflection	120	129	88-170

The California Department of Transportation (*3*) has compared the performance of overlays designed by engineering judgment, gravel equivalency (component analysis), and deflection; the results are summarized in Table 1.

Of particular interest in this comparison is the low side of the range of service provided by each method. The deflection method provided a minimum service life of 88 months (7.3 years), compared to 25 months (2.1 years) by engineering judgment. Overall the deflection method was judged best of the three procedures compared.

Since 1975 there has been an increasing interest in the use of mechanistic (analytical) procedures for overlay design. Some of these methods can be considered as more comprehensive than others, but they are all quasi-mechanistic and all rely on empirical correlations as a back-up for establishing specific design requirements. Several of these methods are summarized in Chapter 4.

The advantage of the mechanistic procedures is their adaptability to a variety of materials, environments, and pavement conditions. These procedures should provide an improved method for modeling the pavement, which in turn should provide more reliable correlations between design and performance.

The disadvantage of the mechanistic methods is the requirement to evaluate the in-situ elastic or viscoelastic properties of the various materials incorporated in the structure, including any effects of damage in the pavement layers. Also, damage criteria for rutting and fatigue cracking for overlays has not been verified by field correlations. Nevertheless, it is the general consensus of engineers worldwide that procedures that incorporate some mechanistic concepts together with nondestructive testing should emerge as an improvement over current deflection measurements.

CHAPTER TWO

PAVEMENT PERFORMANCE

Before proceeding with a discussion of overlay design procedures it will be useful to discuss those factors pertinent to the performance of pavements that contribute to the need for an overlay. Specifically, performance can be divided into five types; (a) functional, (b) structural, (c) structural capacity, (d) safety, and (e) appearance. Considerations applicable to appearance are generally not as compelling as the first four enumerated herein; however, depending on land use, property values, etc., an unpleasing appearance may be cause for an overlay.

FUNCTIONAL PERFORMANCE

Functional performance refers to the ability of a pavement to satisfy its primary function; i.e., to provide a serviceable pavement in terms of ride quality. Serviceability can be evaluated subjectively or by physical measurements correlated with subjective evaluations. In either procedure pavement roughness has been found to be the primary factor influencing serviceability. Literature dealing with pavement serviceability will include two definitions; one for present serviceability rating (PSR) and one for present serviceability index (PSI).

PSR is an indication of a pavement's ability to provide a smooth, comfortable ride at the specific time when the pavement is being evaluated. The PSR is an average value obtained by having a group of raters ride over selected sections of roadway for which a rating is required. The number of raters necessary to obtain a representative PSR has not been rigorously defined. Panels of 12 or more are usually recommended; however, as few as two experienced raters have been used in special cases. Further information pertinent to PSR can be found in references *4* and *5*.

In many situations the need for an overlay because of unacceptable serviceability can be identified by subjective evaluations. However, most agencies would prefer some type of measurement in lieu of a panel and, hence, a number of devices have been developed for measuring serviceability by measuring roughness. It is important to remember that measurements of roughness, per se, can have very little meaning unless correlated with some type of PSR scale. Thus, roughness measurements are estimates of the average serviceability obtained by subjective evaluations by a group of raters.

Techniques and equipment for measuring roughness are described in references *5* and *6*. New devices are being developed and inquiries should be made before purchasing any such equipment as to latest developments.

The most prevalent type of road roughness measurement equipment in use in the United States and Canada at the present time measures a vehicle's response to the longitudinal profile or roughness. This type of equipment is classified as response-type road roughness measuring systems (RTRRM systems). Alternative methods for measuring the longitudinal profile or its characteristics are also available and referred to as surface dynamic profilometers (SDP). Although the latter type of equipment is more reliable and provides more information, it is also considerably more expensive.

The main advantages of the RTRRM systems are their relatively low cost and simplicity. The equipment can be operated at various speeds but is usually operated at 50 mph (80 km/h). The disadvantages are in correlating measurements made with different types of equipment and the susceptibility to changes in operating characteristics (response) over time. Typical equipment in this class includes the Mays meter, PCA road meter, and Cox road meter.

If this type of equipment is to be used for roughness measurements, it will be important to establish methods of calibration and interpretation consistent with known characteristics of the equipment. References *6–8* provide background information useful for the calibration of RTRRM systems.

In addition to providing a measure of serviceability, roughness is often associated with user costs. User costs are related to (a) vehicle operating costs and (b) speed profiles, both of which can be affected by roughness. Reference *5* provides some indication of user costs as a function of roughness. For example, the difference in vehicle operating costs for smooth and rough rural two-lane roads could be 1.1¢ per vehicle mile (0.7¢/km) (*5*). Translated into the cost of gasoline, this would be equivalent to increasing the cost from $1.30 per gallon to $1.52 per gallon ($0.34 to 0.40/L). If the cost for lost time owing to traffic delays and speed reduction is factored in, the excess user costs could amount to 2.7¢ per vehicle mile (1.7¢/km) or about $324 per year per vehicle based on 12,000 miles (19,000 km) of driving per year.

STRUCTURAL PERFORMANCE

Structural performance refers to the ability of a pavement to maintain its structural integrity without exhibiting distress. Engineers associate structural performance with preservation of investments; i.e., when is the best time to intervene with the performance of a pavement to minimize the life-cycle cost of the pavement over a long period of time.

Pavement distress can be classified into three categories: (a) fracture, (b) distortion, and (c) disintegration. Table 2 summarizes these three modes of distress with specific causative factors. Distress, as used in Table 2, is concerned with defects resulting from traffic and environment. Loss of skid resistance

TABLE 2

CATEGORIES OF DISTRESS FOR PAVEMENTS[a]

Distress Mode	General Cause	Specific Causative Factor
Fracture or cracking	Traffic-load associated	Repeated loading (fatigue) (includes few number of load applications as well as many repetitions)
		Slippage (resulting from braking stresses)
		Reflection cracking (may be accelerated by traffic loading)
	Nontraffic associated	Thermal changes
		Moisture changes
		Shrinkage of underlying materials (reflection cracking - see above)
Distortion	Traffic-load associated	Rutting (from repetitive loading)
		Plastic flow or creep (from single or comparatively few excessive loads)
	Nontraffic associated	Heave (resulting from swelling clays or frost)
		Consolidation settlements
Disintegration		This mechanism is associated more with material than with structural design considerations

[a]Although not intended to be a complete listing, it is believed that the major causes are included.

can be considered a form of distress but is not included in this section. It will be discussed in a subsequent section on safety.

A knowledge of the types, severity, and extent of various types of distress should influence the need for special treatments associated with overlays. For example, as a function of the structural performance it may be necessary to use special stress-relieving layers to minimize reflection cracking in the overlay or it may be necessary to do special testing as a consideration for deciding on a rehabilitation procedure. Deflection testing is primarily related to traffic-associated fatigue cracking of a pavement. If the pavement is exhibiting deformation, it will be necessary to sample and test component layers before deciding on an overlay. Removal of some layers (materials) may be necessary to prevent further deformation or an increased thickness of overlay may be necessary to reduce stresses in the weak layer to a level that will eliminate further deformation. The latter approach could lead to the use of a mechanistic (analytical) analysis approach to develop overlay design requirements.

Procedures for the conduct of pavement condition surveys have not as yet been standardized. References *1, 9–16* provide useful background information with regard to condition surveys. An important consideration in establishing procedures for the conduct of condition surveys is to be sure the information is relevant to pavement performance and preservation of investment and provides a systematic way to categorize pavement condition. Avoid collecting information that will not be useful for prioritizing overlays or that is not relevant to the design of the overlay or rehabilitation of the pavement. The three types of distress considered most relevant are (a) fatigue (alligator) cracking, (b) rutting, and (c) raveling. Other forms of cracking (longitudinal, transverse, block, shrinkage, etc.) should be considered as they occur and as they would affect the action to be taken.

STRUCTURAL CAPACITY

Structural capacity is evaluated in terms of a component analysis or by means of nondestructive testing, such as deflection testing. Methods for evaluating structural capacity are part of the overlay design procedure; i.e., how much overlay or pavement strengthening is required to accommodate traffic and environment. Because Chapter 3 is devoted to evaluating existing and future structural requirements in terms of structural capacity, it is not necessary to expand on this item in this section.

SAFETY

From a safety standpoint, the pavement elements of concern for scheduling an overlay are the skid resistance and hydroplaning potential of the pavement.

The potential for hydroplaning exists whenever the depth of water on the pavement exceeds about 0.2 in. (5 mm) and the velocity of the vehicle is equal to or greater than that determined from the expression:

$$V = 10\sqrt{\sigma_0} \tag{1}$$

where

V = velocity in mph ($\times 1.6$ km/hr) and
σ_0 = tire pressure in psi ($\times 6.895$ kPa).

Rutting can contribute to hydroplaning. If the rut depth exceeds 0.4 inches (10 mm) to 0.5 inches (13 mm) on pavements with a 2 percent crossfall, there is the potential for trapping a sufficient depth of water to create the potential for hydroplaning.

Skid resistance is the force developed when a tire that is prevented from rotating slides along the pavement surface (*17*). Skid resistance is usually expressed in terms of a measured skid number although a visual assessment of surface smoothness (texture), bleeding asphalt, or polished aggregate is sometimes used as an indication of skid resistance. The skid number is a function of the coefficient of friction of the pavement measured under a very specific set of conditions by one of several different types of equipment.

No specific criteria are available for requiring an overlay based on surface texture or skid number. NCHRP Report 37 (*18*) gives recommendations with regard to skid number values for main rural highways.

If a pavement is judged to have unacceptable surface friction properties, it will be necessary either to overlay the pavement or to introduce some type of surface treatment to correct this deficiency.

THRESHOLD VALUES

The previous portions of this chapter have described pavement condition considerations that can be used to establish a need for an overlay. The specific conditions (thresholds) that could be used to establish a "need" have not been discussed.

Such criteria tend to be agency dependent and to a significant degree may be influenced by the road classification and availability of funds.

In terms of functional performance, PSR/PSI, a scale ranging from 0–5 is used to categorize ride quality in the United States. In Canada a ride comfort index (RCI) ranging from 0–10 is used for the same general purpose (*19*). For Interstate highways or primary routes with high traffic volumes, a threshold value of 2.5 is often used to indicate the need for an overlay; for all other roads a value of 2.0 is preferred. These values are not considered as absolutes and can vary from agency to agency; in fact, there is a general trend to increase the threshold value for Interstate and other high-volume primary highways to 3.0 and to 2.5 for other roads.

The best time to intervene in the performance of a pavement with some type of maintenance or rehabilitation is subject to local interpretation; there are no specific guidelines at the present time. The development of a comprehensive pavement management system will include life-cycle cost evaluations, performance prediction models, and establishment of performance standards. When properly combined and documented, a pavement management system can answer the question as to what type of rehabilitation, including overlays, is appropriate and the best time (condition) to initiate maintenance or rehabilitation.

CHAPTER THREE

OVERLAY DESIGN PROCEDURES

As previously indicated, overlay design procedures can be categorized into three types: (a) component analysis, (b) deflection based, and (c) analytically based (mechanistic). There can, of course, be variations on these categories; however, it is believed that these are sufficiently general to include all current procedures.

COMPONENT-ANALYSIS PROCEDURES

Figure 1 outlines the procedure that is used for component analysis. Virtually any pavement design procedure can be adapted to this method because it involves comparing in-place construction to design requiements for new construction. References *20* and *21* specifically describe the approach to overlay design by the component-analysis technique.

Sampling and Testing

To use a component-analysis procedure, it is necessary to obtain samples of materials from the existing pavement. A sampling program, which can require six to eight (or more) sampling locations depending on length, terrain, observed conditions, and prior knowledge of subsoil types, is necessary to evaluate in-place materials. In-place density and water content profiles are developed to a depth of 3 to 6 ft (0.9 to 1.8 m) below subgrade elevation as input information for laboratory testing and interpretation.

Standard laboratory tests appropriate to design method (e.g., CBR, R-value, Group Index) are performed on each pavement component, including the foundation soils, to develop design parameters. The testing program should be designed in such a way as to provide estimates for in-situ conditions. Most standard procedures provide for some interpretation based on in-situ information; otherwise it may be necessary to interpret results based on standard laboratory conditions (e.g., optimum water content, maximum density, soaked conditions, or specified exudation pressures).

Pavement Design Parameters

Overlay design procedures require sets of information or parameters to develop specific overlay requirements. Design parameters usually include (a) some measure of strength of the subgrade or foundation soils, (b) characteristics of paving materials (e.g., treated and untreated materials), (c) traffic, usually

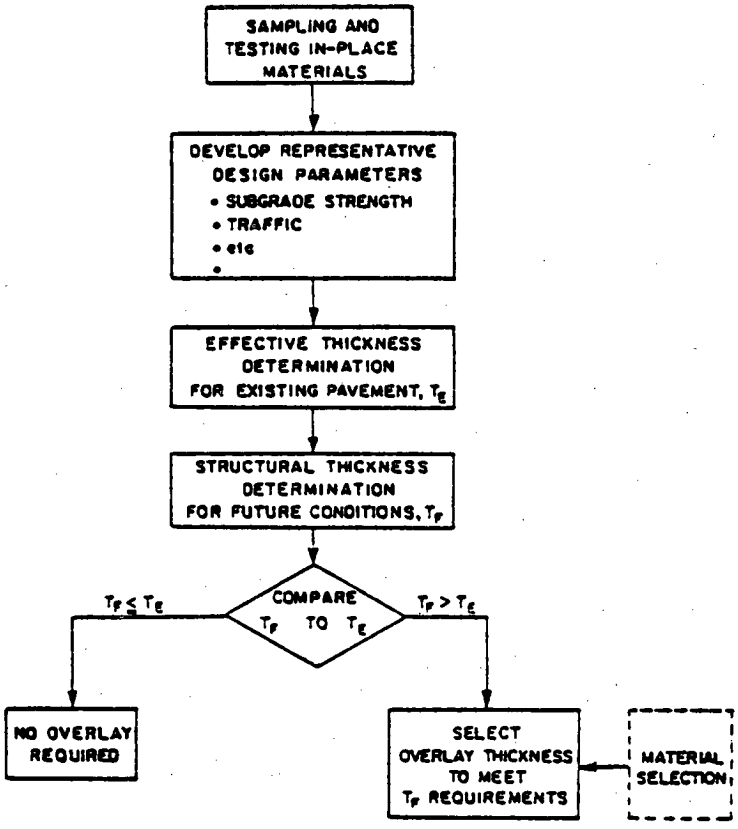

FIGURE 1 Overlay design by component analysis.

in terms of equivalent 18-kip (80-kN) single-axle loads, and, possibly (d) regional factors as represented by rainfall or frost penetration. Traffic will be discussed further under deflection-based overlay design procedures.

It is important in developing sets of design inputs to be consistent with the design procedure. For example, strength measurements, traffic identification, and environmental factors should be used exactly as they are specified by the design method. Interchanging design factors may not produce the desired results inherent in the design procedure.

Evaluation of In-Place Pavement Structure

The in-place pavement must be evaluated to determine its structural equivalency. A convenient procedure to satisfy this requirement is to convert pavement layers into an equivalent structural number using procedures described in the AASHTO Interim Design Guide (*22*). The structural number is obtained by means of the following relationship.

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3 \qquad (2)$$

where

a_1, a_2, a_3 = layer coefficients representative of surface, base, and subbase, respectively, and
D_1, D_2, D_3 = thickness, in inches, of surface, base, and subbase courses, respectively.

A major disadvantage of this system is in the selection of the layer coefficients for in-service pavements.

The Asphalt Institute (*20*) provides recommended conversion factors for converting the thickness of existing pavement components (layers) to an effective thickness of asphalt concrete. The conversion values that can be assigned to uncracked asphalt concrete range from 0.9 to 1.0. Asphalt concrete that exhibits some distress will be assigned coefficients ranging from 0.5 to 0.8. The remaining components are assigned lesser values depending on their condition; e.g., granular subbase or base 0.2 to 0.5 depending on grading, plasticity, and general compliance with standards for the respective materials. Similar methods can be applied to the AASHTO design procedure (*22*) or any other pavement design procedure. A considerable amount of judgment and experience is required to assign realistic layer coefficients.

Pavement Structural Requirements

Based on the information obtained from the laboratory testing program, traffic analysis, and regional factors (if applicable), it is possible to determine thickness requirements, structural numbers, or equivalent thicknesses for new construction.

If the existing pavement is equivalent or exceeds the requirements for new construction, it should not be necessary to add additional layers of asphalt concrete for structural enhancement. Depending on the condition of the pavement it may, of course, be necessary to repair distressed areas and add an overlay to restore the pavement to its original condition. Overlay thicknesses of 1½ to 2 in. (38 to 50 mm) should be adequate to correct

most nonstructural deficiencies, such as raveling or roughness. Allowance must be made for materials required to level pavement irregularities and, if the pavement is very rough, a thin leveling course may be necessary. Recycling is also appropriate for restoring a pavement in this category.

It is pertinent to note that pavements have a finite life, usually in the range of 10 to 25 years. Thus, a pavement that is exhibiting distress may not be structurally deficient; age and traffic will take their toll, and the pavement simply requires a resurfacing to duplicate its past performance. Cold milling and filling 1 to 2 in. (25 to 50 mm) with new or recycled material is also appropriate in this case. If calculations show that the existing pavement requires an increase in structural capacity, an overlay will be necessary to meet the design requirements for new construction. This relationship can be illustrated by use of Eq. (3) as follows:

$$D_i = \frac{SN_1 - SN_0}{a_1} \tag{3}$$

where

D_i = thickness of overlay, inches,
SN_1 = structural number required for new construction,
SN_0 = structural number of in-place pavement, and
a_1 = layer coefficient for asphalt concrete.

Asphalt concrete overlays on portland cement concrete for airfields can be designed by using structural equivalencies to relate the thickness of asphalt concrete to portland cement concrete by means of the following empirical formula developed by the U. S. Corps of Engineers (*21*).

$$t = 2.5\,(Fh_d - h) \tag{4}$$

where

t = thickness of the overlay, inches (multiply t, inches by 25.4 to obtain millimeters),
F = a factor related to the expected future condition of existing pavement,
h_d = design thickness determined from the regular portland cement concrete pavement design procedures, in., and
h = existing pavement thickness, in.

The minimum recommended thickness of the overlay required by this procedure is 4 in. (100 mm).

The value of F is a function of the modulus of subgrade reaction and the type of loading expected, decreasing as the soil support value increases.

In addition to reference *21*, information pertinent to overlays of highways and airfields can be found in reference *23*, proceedings of a national workshop on pavement rehabilitation.

The Asphalt Institute (*20*) overlay design procedures include a component-analysis method (effective thickness) for use with portland cement concrete pavements. The effective thickness (T_e) approach used by the Asphalt Institute is simply a procedure whereby the in-situ thickness and condition of portland cement concrete is converted into an equivalent thickness of asphalt concrete. To determine T_e each layer of the existing portland cement concrete pavement must be converted to an equivalent thickness of asphalt concrete using recommended conversion factors. The conversion factor for portland cement concrete will range from 0.3 to 1.0; i.e., effective thickness of 1 in. (25 mm) of portland cement concrete would range from 0.3 in. (7.5 mm) of asphalt concrete to 1 in. of asphalt concrete depending on the condition of the portland cement concrete. If the portland cement concrete is broken and seated in 2-ft (0.6-m) pieces or less before placing the overlay, a conversion factor of 0.3 to 0.5 is recommended by the Asphalt Institute.

The procedure recommended by the Asphalt Institute is to determine the full-depth asphalt concrete thickness requirements for a project based on subgrade strength properties and traffic (equivalent 18-kip single-axle loads). The effective thickness of the in-place pavement, surface (including overlay if present), and subbase is determined by use of conversion factors. The difference between the full-depth requirements and the effective thickness is the recommended asphalt concrete overlay design.

Analysis Sections

Although not specifically noted on Figure 1, some consideration should also be given to design lengths as a function of pavement condition. For example, observations of conditions along a proposed project may be systematically different, suggesting the possibility of alternative designs. To make such an evaluation, it will be necessary to subdivide the project into analysis sections or sections with homogeneous characteristics, e.g., age, traffic, and design. Procedures for selecting analysis sections are discussed under the section for deflection-based overlay design procedures.

In summary, the component-analysis procedure provides one means of designing asphalt concrete overlays over existing pavements surfaced with asphalt concrete or portland cement concrete.

The method can be used to estimate the remaining life of the pavement for the comparison when no overlay is required. To make such an estimate, it is necessary to obtain the difference between the accumulated loadings to date and the total allowable traffic loadings for the in-place pavement. The difference can be converted to time based on the expected daily traffic loadings. It is important to recognize that such estimates are indicators only and should not be considered a precise estimate of remaining life. The uncertainty of such estimates can be large, depending on inherent variabilities and, primarily, on the reliability or unreliability of the basic empirical relationships and traffic information.

The major disadvantages of the component analysis method are: (a) the need to obtain samples for laboratory testing (expense), (b) the limited areal coverage that can be obtained by a limited testing program, (c) the uncertainty of laboratory testing programs to reproduce long-term field conditions, and (d) the selection of layer coefficients for each pavement component.

The major advantages of this method are: (a) the opportunity to evaluate individual components of the pavement structure, (b) the opportunity to explore potential groundwater and drainage problems, and (c) the opportunity to measure thicknesses of pavement layers.

It is pertinent to note that structural enhancement of a pavement, using the component analysis method, can be achieved

by replacement of untreated materials with treated materials without the need for an overlay. For example, if the thickness of asphalt concrete is increased by replacing untreated aggregate base with a stabilized material, the structural number is increased and the need for an overlay can be eliminated. Replacement procedures would be applicable primarily when there are elevation constraints or problems with drainage if an overlay is required.

DEFLECTION-BASED OVERLAY DESIGN PROCEDURES

Pavement deflection measurements as currently used for overlay design can be credited to the early studies by such agencies as the California Department of Transportation (*24*), Kentucky Department of Transportation (*25*), U. S. Corps of Engineers (*26*), The Asphalt Institute (*27*), the Transportation Research Board (*4, 28*), and the Canadian Good Roads Association (*29*). Since the mid-1960s, a great deal of research has been published in the technical literature regarding deflection measurements and their correlation with performance and use for overlay design.

Figure 2 outlines the general approach used in most nondestructive overlay design procedures based on deflection measurements. The three basic design elements of such design procedures are (a) deflection measurements, (b) pavement condition, and (c) traffic. Examples of deflection-based methods of overlay design are included in Appendix A. Specific procedures described include those of the Asphalt Institute, California Department of Transportation, U. S. Corps of Engineers, Transport and Road Research Laboratory (TRRL) of Great Britain, and Virginia Department of Highways and Transportation. Each agency has its own unique design procedure; however, the main components are similar. The following paragraphs attempt to briefly summarize the procedure outlined in Figure 2.

Deflection Testing

At the present time the objective of deflection testing is to measure the structural properties of the pavement. This is done by imposing a known load on the pavement and measuring its response. Thus, an overall or effective strength is measured that combines all influencing factors including material properties, thickness of pavement layers, subgrade material strength, and

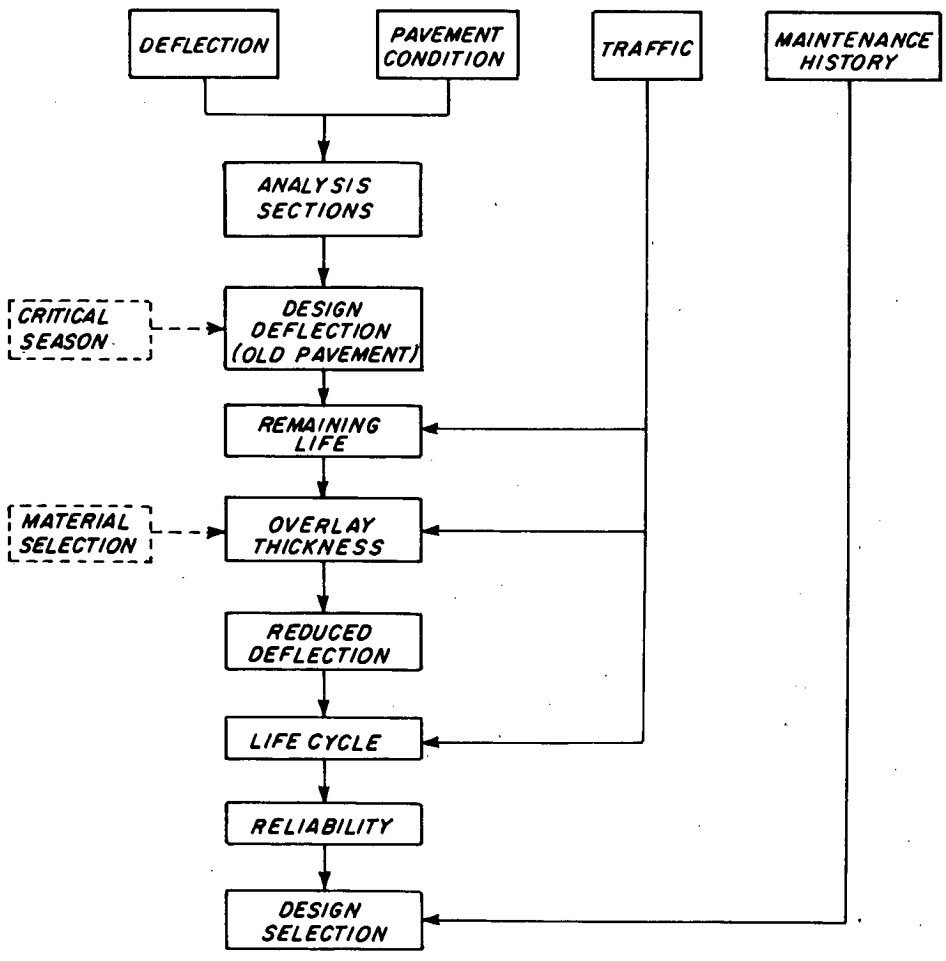

FIGURE 2 Methodology applicable to overlay design using deflection measurements.

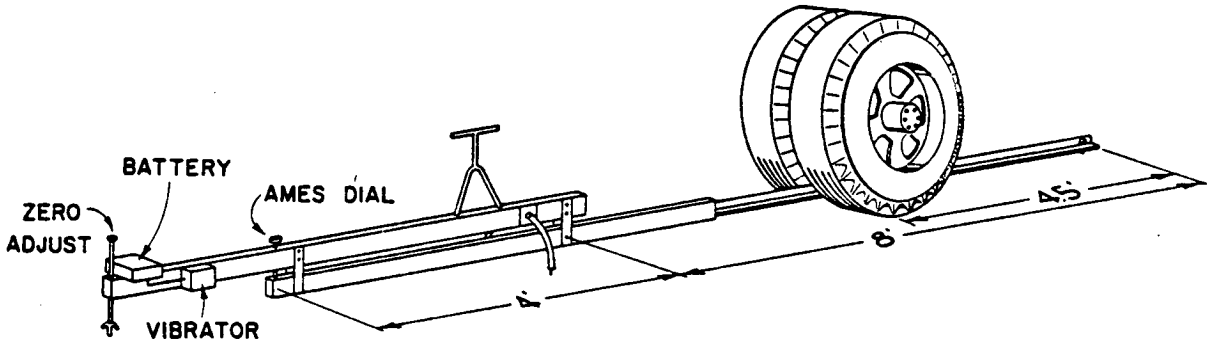

FIGURE 3 Benkelman beam as used in the WASHO method.

environmental effects. Most deflection-based design procedures do not routinely attempt to isolate material properties of individual pavement layers.

It is worth noting that extensive research and development is underway that will make it possible to estimate in-situ material properties from multi-sensor deflection measurements. These methods will make it possible to do a component analysis or a mechanistic analysis based on nondestructive deflection testing.

The dominant type of measurement used for nondestructive overlay design procedures is surface deflection obtained with known loading conditions (i.e., contact pressure, force, and time of loading). Each of these three factors can influence the pavement's response to loading. Some agencies have used a shape factor to evaluate structural capability. Vaswani (*30*) introduced the spreadability concept in the 1970s and Utah (*31*) has used similar procedures to estimate the base curvature index (BCI) and surface curvature index (SCI). These measurements are obtained by using combinations of different sensors associated with the Dynaflect deflection measuring equipment. The BCI and SCI are used primarily to estimate potential deficiencies in the base and surface layers, respectively.

Despite the considerable interest in the use of shape factors, maximum deflection measured directly under the load (force) remains the dominant procedure used for design of overlays from deflection measurements.

There are a number of different pieces of equipment that can be used to measure deflection, for example, Benkelman Beam, Dynaflect, deflectometer, Road Rater, and falling weight deflectometer. Design procedures will normally specify the type of equipment and test procedure. Correlations between devices are sometimes provided; for example, California uses the deflectometer as the basic piece of equipment but the referenced test method includes correlations for other methods of measurement.

Reference *32* contains a detailed summary of the various devices available for measuring deflection.

Engineers should familiarize themselves with equipment to be used for deflection measurements because the various overlay design procedures are keyed to the method of measurement. It is particularly significant in this regard to mention that two procedures are identified with Benkelman beam deflection measurements; one referred to as the WASHO method, developed at the WASHO Road Test, and a second referred to as the rebound or Canadian method.

The WASHO method is used by the California Department of Transportation. In this procedure the probe point is placed approximately 4.5 ft (1.37 m) in front of the dual tires as shown in Figure 3. The maximum deflection is recorded as the wheels slowly pass over the probe of the beam.

The Canadian method is used by the Asphalt Institute. In this procedure the probe point is placed directly under or only slightly ahead of the dual wheels and a maximum reading is obtained when conditions become static; i.e., a standing load with no significant vertical movement of pavement. The final reading is obtained after the wheels have been moved forward of the probe point and beyond any sphere of influence to the probe.

Because of the increased time of loading for the rebound method, a larger deflection may be recorded by this method than would be obtained by the WASHO method. Figure 4 illustrates one correlation found between the Canadian rebound procedure and the California deflectometer or WASHO performance.

In summary, there are a variety of ways by which pavement deflection can be measured. It is important that the method of measurement be compatible with the method used to design the overlay.

Pavement Condition

A pavement condition survey will serve several purposes including (a) establishing need for maintenance or rehabilitation, (b) identifying homogeneous segments, (c) indicating design requirements of procedures, and (d) pointing out special conditions influencing overlay design; e.g., drainage conditions.

Analysis Sections

This subject will be discussed further in connection with specific design procedures described in Appendix A.

By way of definition, analysis sections or homogeneous sections refer to sections of pavement that can be considered nearly alike in terms of performance, age, traffic, structural capacity, etc., and for which a single treatment is appropriate.

Identification of homogeneous sections is important since the treatment or overlay could be changed as a function of section

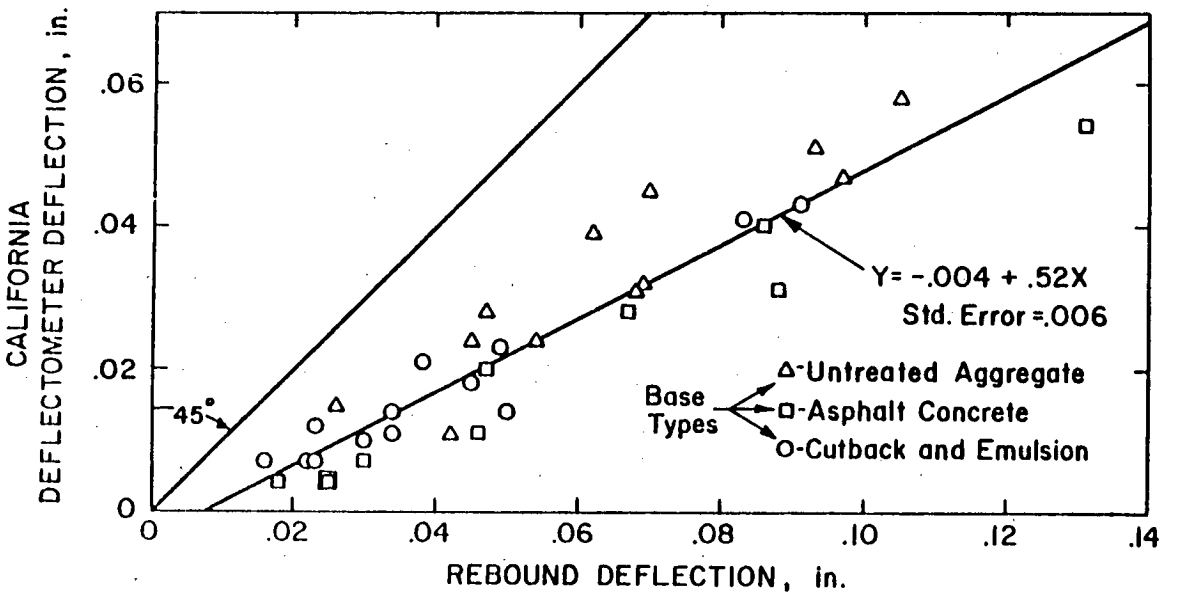

FIGURE 4 Comparison of CGRA and California Benkelman beam measurements, March 1969 (*33*).

homogeneity. For example, a 5-mile (8-km) project could exhibit differences in condition or deflection that would require different overlay thicknesses, cost of repairs, or special treatments in conjunction with the overall project rehabilitation. The designer is ultimately charged with the responsibility for a final decision: whether to subdivide the project into a set of rehabilitation treatments or to select one single treatment for the entire length. The choice will depend on the amount of variability, costs and benefits of multiple treatments, and lengths of individual segments that are considered eligible for different types of rehabilitation.

Most methods for classifying pavements will categorize pavement condition, functional or structural, into discrete intervals. For example, the PSR/PSI scale of 0 to 5 is divided into five intervals described as 1) very good, 2) good, 3) fair, 4) poor, and 5) very poor. One definition of a homogeneous project in terms of functional performance would be any combination of segments, within the limits of the project, that are rated or evaluated within one of the five intervals described. The number of intervals could easily be doubled by using intervals of 0.5 on the PSR/PSI scale. The Canadians have achieved some advantages in this respect by going to 10 intervals on the PSI scale previously mentioned. The new scale is referred to as the ride comfort index (RCI).

Structural performance (physical distress) can and usually is categorized by intervals. For example, if condition scores are used, intervals can be assigned that indicate the condition is acceptable, tolerable, or unacceptable.

The U.S. Corps of Engineers procedure (*12*) provides for a condition rating using a pavement condition index (PCI), which is a numerical indicator based on a scale from 0 to 100. The PCI measures the pavement's structural integrity and surface condition, including slipperiness (bleeding). The PCI values have been subdivided into seven intervals: 1) 86–100, excellent; 2) 71–85, very good; 3) 56–70, good; 4) 41–55, fair; 5) 26–40, poor; 6) 11–25, very poor; or 7) 0–10, failed. The intervals are a matter of judgment and experience and could be modified; however, pavements categorized within any of the seven intervals could be considered to be similar or homogeneous. A similar approach could be used for any method used to score the pavement's condition.

One disadvantage of a single numerical indicator for pavement condition is that information concerning the actual distress is lost; e.g., it would be impossible to tell whether a pavement had cracking or rutting if only the pavement condition index were known.

Some agencies prefer to use a distress state to describe a pavement's structural performance. In this system various distress types are subdivided into a matrix of severity and extent. Each cell within the matrix will represent a discrete interval for both severity and extent. Thus, pavements that fall within a particular cell are said to be in the same distress state and would be considered homogeneous with regard to structural condition. The level of performance can also be described by distress states; i.e., acceptable, tolerable, and unacceptable conditions or combinations.

When using nondestructive deflection test data, a highway can be divided into various analysis sections based on consistent differences in deflection measurements.

Using this technique, deflection data are plotted in the form of profiles of deflection. The analysis sections can initially be estimated by examination of the data. Trial designs can be obtained for each section and combined or segregated into separate design sections based on engineering judgment. When applying this technique, the designer should combine condition survey data and deflection data for the selection of analysis sections. The minimum length of an analysis section is also a matter of engineering judgment and economic considerations. In cities, alternative designs are viable for relatively short lengths; e.g., 500 ft (150 m). For state routes and large projects it may not be reasonable to change designs for lengths less than 1000 feet (300 m).

Two criteria can be used for selection of analysis sections based on deflection data and pavement condition information:

1. Based on condition (i.e., condition score or distress state), divide the project into analysis sections. Use deflection data to estimate overlay requirements for each section, or
2. Based on deflection data, divide project into analysis sections. Use deflection criteria and condition to estimate overlay requirements for each section.

The engineer must decide how best to "package" the designs to produce an economical and yet practical construction project.

Statistical methods for selecting analysis sections have been proposed by ARE, Inc. (*34*). Excerpts from reference (*34*) are included in Appendix C to illustrate the technique and statistical approach.

Figure 5 illustrates one approach based on engineering judgment. In this case the designer simply divides the project into two sections based on a visual evaluation of the deflection values.

If the overlay requirements between two analysis sections differs by less than 0.5 in. (1.25 cm) the designers should consider combining sections into a single project regardless of the results of statistical tests.

In summary, the analysis section should be representative of homogeneous conditions within a specific project. The objective is to develop designs that will correct deficiencies and will provide an economical overlay commensurate with structural conditions.

Design Deflection

The design deflection is the specific deflection value required for use with a specific overlay design procedure. The design deflection is usually a function of the mean deflection, the variation in deflection values for the analysis section, and the reliability level selected for design. The design deflection can be represented by the following relationship:

$$d = \bar{d} + zS \tag{5}$$

where

d = design deflection,
$\bar{d}$ = mean deflection for analysis section,
S = standard deviation of deflection measurements in analysis section, and
z = normal deviate expressed in terms of the standard deviation.

The z value is thus used to control the reliability or confidence that the design deflection value will only be exceeded by some predictable amount. Table 3 is a list of z values corresponding to various levels of confidence or reliability.

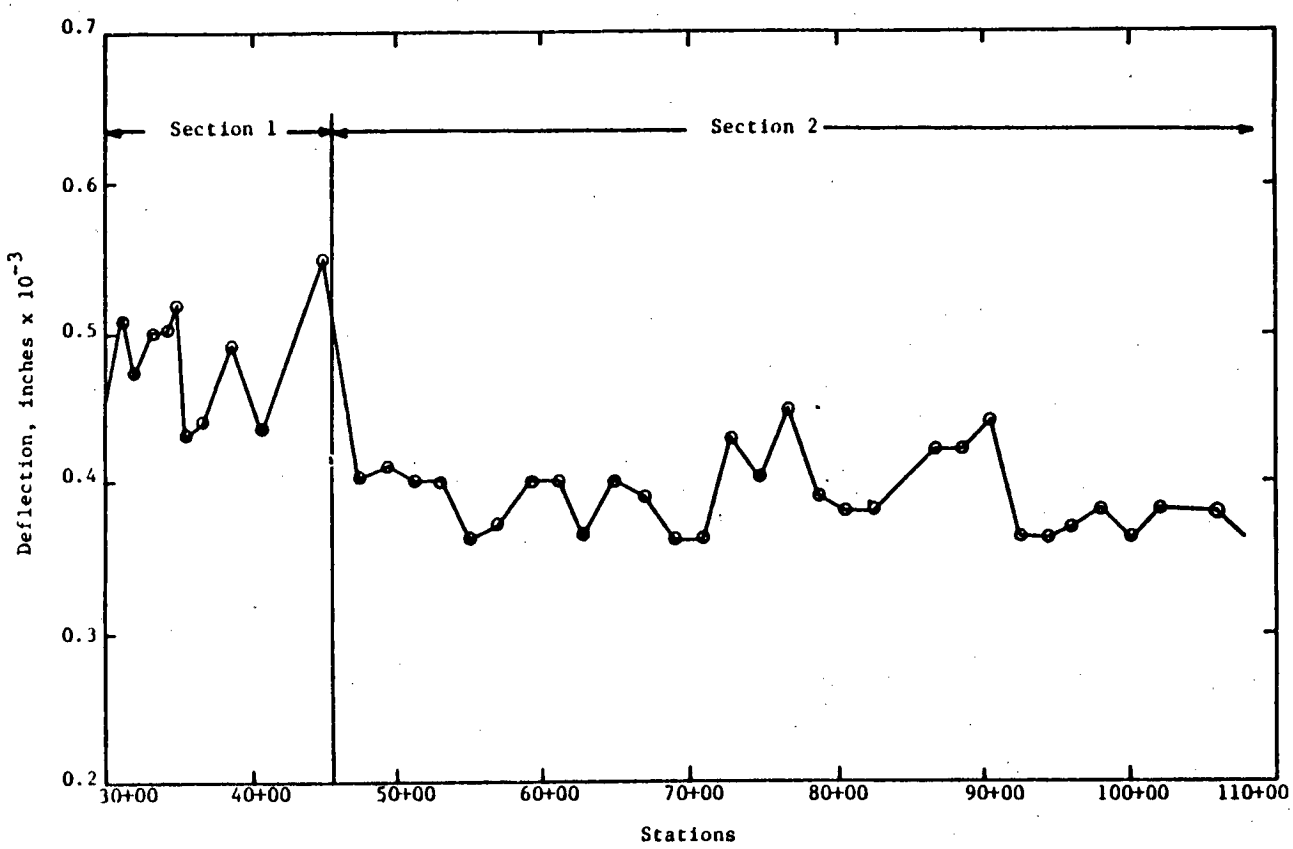

FIGURE 5 Sample deflection profile (*34*).

TABLE 3

VALUES OF z FOR VARIOUS CONFIDENCE LEVELS

Design Confidence Level	Reliability (1 - confidence level)	z Value
50	50	0
75	25	0.674
90	10	1.282
95	5	1.645
97.5	2.5	1.960
99	1	2.330

The Asphalt Institute uses a z value of 2.0. Thus, this method is based on a 97.5 percent confidence that the design deflection would only be exceeded 2.5 percent of the time if an infinite number of deflection measurements were made. Caltrans uses a value of 0.84 corresponding to the 80th percentile. This would suggest the Asphalt Institute design is more conservative; however, this is not necessarily true. The actual degree of conservatism will depend on how the correlations were made between the design deflection and pavement performance.

Pavement deflection measurements may vary depending on the season of the year or more specifically, environmental conditions. Deflections will usually be larger during or just following the rainy season or during the spring thaw. Most deflection-based overlay design procedures will recommend making measurements during the critical season when deflection measurements are expected to be at their peak. Realistically, this is not always possible and, therefore, some adjustments may be necessary based on local experience.

Very little specific guidance is provided with regard to correcting for the critical season, although several states have studied this problem.

The Asphalt Institute (*20*) does make some suggestions for obtaining adjustment factors; however, these recommendations entail an investigation of some considerable magnitude. In the final analysis, needed adjustments are based on engineering judgment. The usual procedure is to increase (never decrease) the z factor in Eq. (5).

Temperature adjustments are also required by some overlay design procedures. The Asphalt Institute attempts to normalize temperatures to 70° F (21° C) as a function of the thickness of the asphalt concrete. Caltrans has concluded that temperature normalization is not necessary. The rule should be to follow the selected procedure without modification. Thus, if the method does not require a temperature adjustment it should not be made. In selecting an overlay design procedure to be followed, the designer or agency should give some thought to the advantages and disadvantages of making temperature corrections.

It is worth noting that the design deflection can be adjusted (reduced) somewhat based on preparations (repairs) made in advance of the planned overlay. For example, if cracked areas are to be replaced with full-depth asphalt concrete patches, eliminate deflection measurements made in badly cracked areas.

Some consideration should also be given to areas with unusually high deflections. These areas should be reinforced by deep patching, improved drainage, etc., to minimize adverse effects of localized conditions.

Traffic

Some consideration of traffic is incorporated in all deflection-based overlay design procedures. This information is necessary to estimate the *remaining life* of the existing pavement, to determine the *overlay requirements,* and to predict the *life cycle* of the overlayed pavement.

Traffic is often summarized in terms of equivalent 18-kip (80-kN) single-axle loads. Field studies have shown that the effect on pavement performance of an axle load of any size can be represented by an equivalent number of 18-kip (80-kN) single-axle load (EAL) applications. For example, a 2000-lb (8.9-kN) single axle is equivalent to 0.0003 EALs, a 12,000-lb (53.4-kN) single axle can be converted to 0.189 EALs. By summing all load combinations and the number of each combination, it is possible to convert mixed traffic into a single design factor.

Procedures for combining truck loadings are given in references *20* and *22.* It is pertinent to note that traffic summarization should be accomplished using procedures appropriate to the design method. For example, procedures in reference *22* include consideration of the terminal PSI and the structural number. Reference *20* has no such requirement.

If some information is available regarding the type and volume of traffic already imposed on a pavement, it is possible to use deflection measurements on that pavement to estimate remaining life. Deflection measurements used for this purpose should be representative of undamaged sections.

It is important to recognize that the error in estimating remaining life can be rather large depending on reliability of historical traffic information and future traffic estimates and on the reliability of the design procedure used.

A number of different procedures can be used to convert mixed traffic to equivalent 18-kip single-axle loads. The most accurate would be a combination of loadometer studies and truck counts for each overlay project. In most cases this is not possible. Portable weigh-in-motion systems are being used in some states and improved technology will make this a more useful system in the future. In lieu of such procedures, agencies have developed representative values (multipliers) to use for various truck types, i.e., axle configurations. If this information is not available, a simple truck count may be used as a last resort. In this case a rather gross estimate of traffic is obtained.

EALs are calculated by multiplying the number of vehicles in each weight class extended over the design period, by the appropriate truck factor. The sum for all vehicles represents the design traffic.

The Asphalt Institute (*35*) provides much useful information concerning truck factors in the United States. Examination of these factors will indicate that the total EALs could vary by a factor of three or more depending on the class of highway and truck factor selected. This difference in traffic could result in an increase (or decrease) in overlay design thickness of 1 to 2 in. (25 to 50 mm) of asphalt concrete. In either case, the results would be representative of poor design; i.e., too expensive, or would result in less than desired service life. Designers should make an effort to identify truck factors to produce designs that reasonably meet design expectations.

Overlay Thickness—Reduced Deflection

Overlay thicknesses are determined in accordance with the specific procedure selected. The basic criterion for the overlay is to reduce the deflection to a tolerable level based on increasing the overall thickness of the pavement. As previously indicated, if the tolerable deflection is satisfied without structural enhancement, the overlay will be of nominal thickness required to correct surface conditions or ride quality. Tolerable deflections for different overlay design procedures are given in Appendix A of this report. Tolerable deflections have been empirically established and should not be compared as a means for evaluating alternative methods.

Materials used for overlay can influence the overlay thickness, i.e., dense-graded asphalt concrete, open-graded asphalt concrete, nonstabilized base with asphalt concrete wearing surface, or various types of portland cement concrete pavements. Most overlay design procedures are keyed to specific types of materials. Substitutions can be made based on concepts used in component analysis procedures; however, substitutions should only be made after considerable review of local experience, or based on local practices that have proved to be satisfactory.

Life Cycle

The life cycle of the overlay will depend on the accuracy of the traffic estimates and the ability of deflections to measure structural capacity in terms of allowable traffic loadings. Table 1 summarized overlay performance information reported by Caltrans. The life cycle (service life) ranged from 73 percent to 142 percent of the design life. Thus, in evaluating the performance of specific overlay projects, there is some chance or probability that it will vary around the expected service life but on the average should approximate the design life.

The life cycle of the pavement will be influenced by the durability of materials used in the overlay. Depending on climate, traffic, construction, and uncontrolled variables, the probable maximum life cycle may range from 15 to 30 years. The designer should recognize that structural considerations alone will not be the only factor influencing the life cycle.

Reliability

Reliability has been discussed in the section under design deflection. Another approach to reliability could be to use the mean deflection value and increase the traffic by a factor similar to the normal deviate (z) used for deflection. It can be expected that some future design methods may resort to this alternative approach.

Regardless of the approach used, the objective is to reduce the risk to some acceptable level. The risk should recognize the relative importance of the facility being considered; least for the more important or heavily trafficked facilities.

No specific recommendations are available as to specific levels of reliability to be used for various functional classes. The designer should be aware that adjustments can be made but that there are consequences; i.e., cost and expected performance.

Design Selection

In the final analysis, the designer must "package" the analysis sections into a manageable project that will provide the required performance at a minimum cost. A number of final checks are appropriate:

1. Have all adjustments been made to deflection measurements?
2. Have provisions been made to accommodate information from pavement condition survey?
3. Have laboratory tests been made to identify localized problems?
4. Have the analysis sections been selected to identify *significantly* different sections based on deflection data and pavement conditions?
5. Has maintenance history been reviewed to help identify localized problems requiring special treatment?
6. Is traffic input sufficiently reliable to justify determination of overlay?
7. Have alternative rehabilitation treatments been considered?
8. Have drainage considerations been evaluated?

If the designer is satisfied that the above considerations have been met, it should be possible to produce plans and specifications appropriate to the project.

It is important to note that although this report emphasizes the method to be used for the determination of overlay requirements, this is only one part of the total task of project design. The designer must give equal attention to material requirements and construction specifications. The general guideline is to follow the recommendations appropriate to the overlay design procedure; i.e., when using AASHTO methods use AASHTO guide specifications and tests.

ANALYTICALLY BASED OVERLAY DESIGN PROCEDURES

Results of implementable research developed in recent years can be effectively used in overlay design. One such framework within which this can be accomplished is shown in Figure 6.

As in the deflection-based procedures, nondestructive pavement evaluation, condition surveys, and traffic are required as input to this type of design methodology. In addition, some measure of the stiffness (modulus of elasticity) properties and distress characteristics of the various materials comprising the specific pavement structure are required. Distress refers to plastic deformation or fatigue cracking potential of asphalt concrete overlays. Stiffness characteristics of the various pavement components can either be defined by tests on undisturbed or representative specimens of the pavement components or inferred (estimated) from the nondestructive measurements. The following paragraphs briefly summarize the general approach shown in Figure 6.

Nondestructive Evaluation

Equipment in current use to measure structural response generally provides a measure of the surface deflection to slow mov-

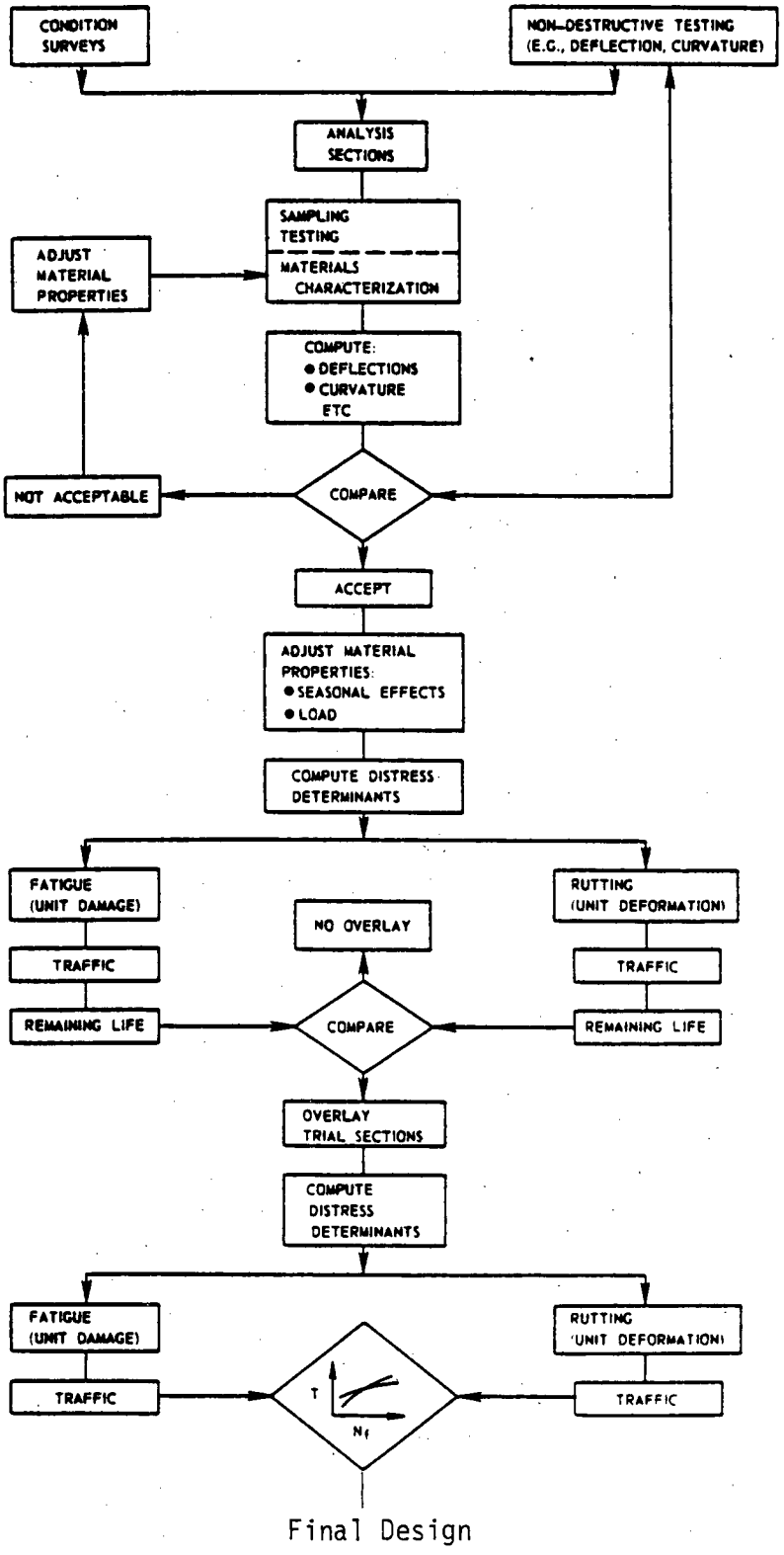

FIGURE 6 Overlay design based on analytical (mechanistic) analysis.

ing, vibratory, or falling loads. Reference *32* contains a detailed summary of the various devices available.

Reliance should not be placed solely on nondestructive testing for structural pavement evaluation, particularly for major projects, because small test specimens of representative layers can be quickly obtained and used within the framework illustrated in Figure 6.

Establishment of Analysis Sections

The condition of the existing pavements, including the nature and extent of distress, is carefully ascertained. Information can be stored in a data bank, where it is readily accessible. This information is useful not only in the establishment of analysis sections but also can be of assistance in establishing performance criteria for related distress.

Nondestructive measurements, such as deflections, should be obtained at reasonable intervals throughout the project(s) to establish some measure of the pavement response to load.

Table 4 provides some general guidelines for these intervals based on recommendations included in the methods summarized in this synthesis. For highway pavements the usual spacing is in the range of 100 to 200 ft (30 to 60 m) whereas for airfield pavements the spacing is in the 200- to 300-ft (60- to 100-m) range for heavily trafficked runways, taxiways, and aprons.

The deflection data within each section can be treated statistically, and it is assumed that the variation in deflection is normally distributed. Comparisons should be made between adjacent sections to ensure that the deflections are statistically different; for these comparisons, use can be made of the student's t-test at a significance level of five percent. Appendix C and reference *34* describe the procedure in more detail.

When the sections have been established, it is then necessary to establish a representative or "design" deflection for that section. It is recommended that the value be set somewhere in the 80 to 90 percentile range, i.e., 80 to 90 percent of the deflections in the section will be equal to or less than the value chosen.

For additional discussion on analysis section see previous explanations for overlay design by deflection measurements.

TABLE 4

SUGGESTED SPACING FOR NONDESTRUCTIVE MEASUREMENTS

Organization	Highways	Airfields
Asphalt Institute	20/mile (min.) 10/analysis section (min.)	
Transport and Road Research Laboratory	12 to 25 m	
U.S. Army Corps of Engineers Waterways Experiment Station		250 ft - runways and primary taxiways 250 to 500 ft grid patterns - aprons
Shell Research	25 to 50 m	25 to 100 m
Austin Research Engineers - FHWA	100 to 250 ft depending on terrain and on material uniformity	

Establishment of Representative Material Characteristics

In this phase, pavement cores, layer samples and thicknesses, and undisturbed subgrade samples to varying depths depending on the facility are obtained. This process need not cause much delay to traffic since the sampling holes are small.

Laboratory testing consists primarily of determining representative stiffness moduli by some form of dynamic or repeated load testing and establishing, where appropriate, distress (failure) criteria.

Utilizing the laboratory-determined stiffnesses, deflections under known loadings can be estimated using a suitable analysis procedure (e.g., multilayer elastic analysis). By comparing these values to the measured ones, adjustments in laboratory determined stiffness values are made until predicted and measured deflections are in reasonable agreement. In this phase of the analysis, the stress sensitivity of untreated paving materials must be recognized. For example, if relatively light equipment (small surface load) is used to determine deflections and, in turn, to estimate stiffness values, these stiffness values must be adjusted when they are used to estimate response under actual traffic loading.

Alternatively, the stiffness characteristics of the various layers can be estimated from surface deflection measurements (*36*). Figure 7 illustrates the deflected surface of a multilayered elastic system subjected to a surface load such as that which might be applied by the Dynaflect, Road Rater, and falling weight deflectometer, for example. The shape of the deflected surface is defined by deflections measured directly under the load and at a number of radii. By use of a computer program for selection of stresses and deformations in a multilayered elastic system, a set of modulus values (E_1, E_2, etc.) is determined that provides the best fit between the measured and computed deflected shapes off the pavement surface. Normally, the procedure involves assuming a "reasonable" set of modulus values and then iterating with the computer and modifying the moduli until the measured and computed deflections are in "reasonable" agreement.

It is recommended, however, if this procedure is used that some laboratory testing be performed to verify the results. AASHTO T 274 is one method for testing for resilient modulus of subgrade soils that can also be used for granular materials.

Estimate of Current Conditions (and Performance Parameters)

Traffic using the facility should be "reasonably" defined. The equivalency concept, as discussed in the previous section, may be useful in this analysis. For multilane highway pavements, the distribution of traffic across lanes and the concentration of truck traffic in the outer lane(s) should be recognized.

With the traffic information and stiffness properties suitably adjusted for load and environment (e.g., for water influences), critical performance parameters can be determined by analysis (e.g., using layered elastic analysis). These parameters can, in

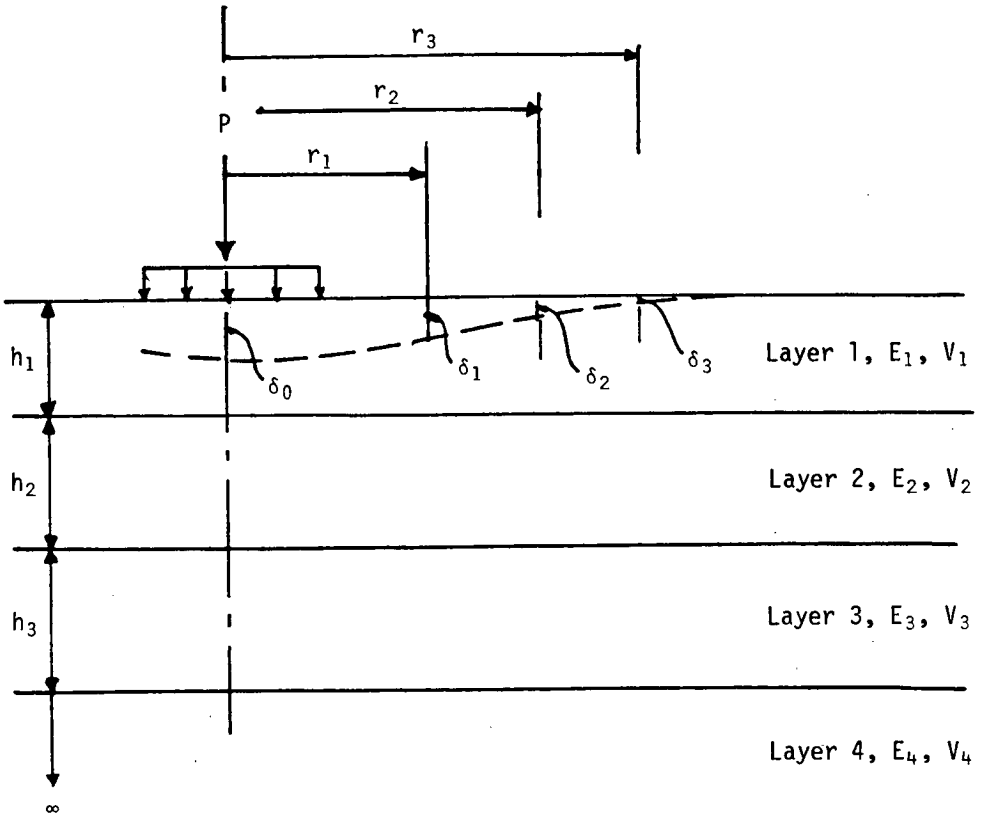

FIGURE 7 Idealized pavement representation and deflected shape under surface load.

turn, be related to "acceptable" and "not acceptable" performance areas observed in the condition survey as well as to laboratory-determined distress criteria.

Remaining life in the existing pavement relative to a particular distress mode can be estimated using some form of a cumulative damage hypothesis. For asphalt-surfaced pavements, for example, fatigue in the asphalt-bound layer is an important distress mode to be considered. The linear summation of cycle ratios cumulative damage hypothesis permits an estimate of fatigue damage to be made, i.e.,

$$\sum_{i=1}^{n} \frac{n_i}{N_i} \leq 1 \tag{6}$$

where

n_i = actual number of applications at strain level i, and
N_i = number of applications to failure at strain level i.

Eq. 6 indicates that the fatigue life estimation for the range of loads and temperatures anticipated becomes a determination of the total number of strain applications at which the sum equals unity. The concept is referred to as the linear summation of cycle ratios cumulative damage hypothesis or Miner's hypothesis.

As an example, using a fatigue relationship based on strain, the general form of the equation is:

$$N = A\left(\frac{1}{\epsilon_t}\right)^b \left(\frac{1}{S_{mix}}\right)^c \tag{7}$$

where

N = number of applications to failure,
ϵ_t = tensile strain in asphalt concrete, in./in., and
S_{mix} = stiffness modulus of asphalt concrete, psi.

A, b, and c are representative of the specific asphalt mix and the coefficient A has also been adjusted to reflect some proportion of pavement cracking. It is possible to estimate whether or not damage exists and to compare the estimated results with those obtained from the condition surveys. This analysis requires that the pavement structure be represented in a manner, e.g., as a multilayer elastic system, that permits estimation of critical stresses or strains for specific loads and representative material properties. It should also be noted that such an analysis permits the establishment of specific distress criteria (e.g., fatigue) if none are available. Examples of this approach are described in Refs. *37, 38, 39,* and *40.*

The cumulative damage hypothesis can also be used to estimate *remaining life.* A simple form of the expression is:

$$\frac{N_r}{N_{D1}} = 1 - \frac{N_{A1}}{N_{D1}} \tag{8}$$

With remaining life defined as N_r/N_{D1} and

where

N_{A1} = number of equivalent single-axle load (e.g., 18 k) applications to date,

N_{D1} = allowable number of equivalent single-axle load applications according to fatigue relationships, and
N_r = additional equivalent single-axle load applications that can be applied to existing pavement.

Overlay Thickness Determination

If an overlay is required it must then be designed to resist potential distress modes that are considered to be significant; in Figure 6 fatigue and rutting are shown. In some instances it may be necessary to consider other modes as well; e.g., thermal cracking.

Another procedure that has been proposed is based on the hypothesis that there is a relationship between cumulative peak deflections imparted to a pavement system and the condition of that system (*41*). Thus performance trends can be predicted from knowledge of cumulative total peak deflections.

For *fatigue cracking* there are a number of possibilities depending on the condition of the existing surface. The following situations are presented to illustrate the process (*40*):

1. The existing surface is intact (has remaining life) with less than five percent *Class 2 cracking,* (AASHO Road Test definition); in this case, the overlay and the existing pavement may be considered to be a single layer, and the additional number of load applications will be dependent on the initiation of cracking in the existing layer.
2. The existing pavement has a moderate amount of cracking with the amount of *Class 2 cracking* greater than five percent but the amount of *Class 3 cracking* less than five percent; it is possible to assign a reasonably high value for stiffness to the existing asphalt layer; e.g., 70,000 psi.
3. The existing pavement is severely cracked with the amount of *Class 3 cracking* greater than five percent, the pavement should be assigned the same modulus as the untreated base on which its rests, or 20,000 psi, whichever is the larger value.

At the AASHO Road Test (*4*) three classes of cracking were established as follows; *Class 1* is the earliest type of load associated cracking observed and consists of fine, disconnected, hairline cracks. As distress increases the Class 1 cracks lengthen and widen until cells are formed into what is termed alligator cracking; a small amount of surface spalling at the cracks is usually evident; this is termed *Class 2* cracking. *Class 3* cracking is associated with severe spalling of the edges of the segments together with loosening of some of the cells and their rocking under traffic.

For a specific thickness of overlay to minimize fatigue, the tensile strain is determined on the underside of the existing layer using an analysis of the type noted above. The allowable number of repetitions can be estimated from the fatigue expression [Eq. (7)], modified by the remaining-life ratio N_r/N_{D1}. From a series of analyses for a range of overlay thicknesses, it is possible to define a relationship between overlay thickness and additional load applications.

Permanent deformation or rutting will normally be of concern at the surface of the overlay. It can be assumed that the existing rut, if any, will be filled and that the development of rutting will only be a function of the additional traffic to be applied on the "new" pavement structure with the overlay. As with fatigue, a relationship between overlay thickness and load applications can be determined.

If other modes of distress are considered, similar relationships between thicknesses and load applications can be developed. The design overlay thickness will be the maximum value required to satisfy the various conditions.

Summary

This approach, which makes use of recent developments in new design methodology, permits the extension of overlay design to both heavier traffic conditions and the use of new and different materials. Its use will depend on the size of the project since some additional engineering effort is required. It is also possible that "catalogues" of designs eventually might be developed for specific areas based on the improved methodology.

Four procedures representative of this approach, including those developed by:

1. Shell Research,
2. Austin Research Engineers (ARE) for the Federal Highway Administration,
3. Resource International, Inc. (RII) for the Federal Highway Administration, and
4. The State of Kentucky

are summarized in Appendix B of this synthesis.

CHAPTER FOUR

CURRENT METHODS FOR OVERLAY DESIGN BY DEFLECTION AND ANALYTICAL PROCEDURES

Current methods for overlay design include, as noted in Chapter 2, both deflection-based and analytically based procedures. In this chapter a number of these procedures will be briefly summarized. Detailed descriptions and references for each of the procedures following the format of Figures 2 and 6 are included in Appendixes A and B. The deflection-based procedures include those developed by The Asphalt Institute, the California Department of Transportation (Caltrans), Roads and Transportation Association of Canada (RTAC), the Virginia Department of Highways and Transportation, and Transport and Road Research Laboratory of Great Britain (TRRL) for highway pavements and the U.S. Army Corps of Engineers Waterways Experiment Station (WES) for airfield pavements. Analytically based procedures include those developed by Shell Research (Amsterdam), Austin Research Engineers (ARE) and Resource International Incorporated (RII) for the Federal Highway Administration, and the Kentucky Department of Highways.

DEFLECTION-BASED PROCEDURES

Table 5 provides an overview of the methods briefly summarized in this section. This table follows the format of Figure 2 and provides an indication of how each of the factors are considered in the methods that have been evaluated. This summary also provides the reader a basis for some of the recommendations contained in Chapter 6.

The Asphalt Institute Procedure

Deflections are measured using the Benkelman beam (rebound procedure). The deflection data together with condition survey data are used to establish analysis sections. For each analysis section the design deflection is taken to be the mean deflection plus two standard deviations with the mean based on a minimum of 10 deflection measurements. This value must be adjusted to that obtained in the critical season and to a temperature of 70° F (21° C). Remaining life is estimated for a relationship between allowable deflection and number of 18-kip (80-kN) single-axle load applications (EALs). Overlay thickness is determined from a relationship between EALs, design deflection, and thickness of asphalt concrete based in part on the assumption that the overlaid pavement behaves as a two-layer elastic system. Provision is included for overlays on portland cement concrete pavements. These overlay thicknesses are a function of different deflections at the joint, joint spacing, and temperature differential between the hottest and coldest months at the site.

California Department of Transportation Procedure

Overlay designs are based on deflection measured by the traveling deflectometer. A number of testing devices have been correlated to the deflectometer including the Dynaflect, Benkelman beam, Road Rater, Dehlen curvature meter, and Cox device. Because of its speed of operation and simplicity, the Dynaflect has become the primary testing device (*42*). The deflection data together with condition survey data are used to establish analysis sections and the design deflection for each section is taken as the 80th percentile deflection (actually the mean plus 0.84 standard deviation). The design deflection is checked against relationships between asphalt concrete thickness, tolerable deflection, and traffic (expressed by the Traffic Index).

If an overlay is required, sufficient thickness of overlay is determined that will reduce the deflection to a tolerable level associated with the appropriate asphalt concrete layer thickness in the overlaid pavement. Thickness of overlay required to reduce the deflection by a specific amount is expressed in terms of thickness of gravel and for overlay design purposes asphalt concrete is assigned a gravel equivalent of 1.9. Although no specific design procedure is available to minimize reflection cracking, guideline thicknesses and treatments are available based on strategies that have been proved effective on California highways.

Roads and Transportation Association of Canada Procedure

Deflections are measured using the Benkelman beam (rebound procedure). The data together with condition survey data are used to establish analysis sections and the design deflection is taken as the mean deflection plus two standard deviations for the spring condition. Overlay thickness selection is similar in format to that described above for the Asphalt Institute procedure.

Transport and Road Research Laboratory Procedure

Deflections for design purposes are based on the Benkelman beam although the TRRL deflectograph (TRRL modification of the LaCroix deflectograph) is extensively used. The deflectograph readings must be converted to equivalent Benkelman beam deflections and adjusted to a standard temperature, the adjustment factors are dependent on thickness of asphalt-bound

TABLE 5

DEFLECTION-BASED OVERLAY DESIGN PROCEDURES

Method	Deflection Measurement	Condition Survey	Establishment of Analysis Sections	Design Deflection	Provision for Remaining Life Estimate	Overlay Thickness Determination
Asphalt Institute	Benkelman beam rebound deflection	Yes	Yes	δ + 2S adjusted for temperature and critical season	Yes	Based on response of overlayed pavement as two-layer elastic system and relationship between allowable deflection and repetitions of 18-kip EAL.
California Department of Transp.	Dynaflect; Traveling deflectometer	Yes	Yes	δ + 0.845	No	Based on relation between permissible deflection as a function of asphalt layer thickness and repetitions of 18-kip EAL and reduction in deflection achieved by different thicknesses of overlay materials.
Transport and Road Research Laboratory	LaCroix deflectograph	Yes	Yes	85th percentile	Yes[a]	Observed damping effect on deflection under 18-kip EAL for various overlay thicknesses used to develop design charts as a function of repetitions of 18-kip EAL.
Roads and Transp. Association of Canada	Benkelman beam rebound deflection	Yes	Yes	δ + 2S	Yes	Overlay thickness selection procedure similar in format to Asphalt Institute procedure.
U.S. Army[b] Corps of Engineers Waterways Experiment Station	WES heavy[c] (16-kip) vibrator	Yes	Yes	Mean DSM	Yes	With parameters developed from nondestructive testing, the CBR of the subgrade is ascertained. Using the ESWL procedure, a pavement thickness is selected according to the current CE procedure. Overlay thickness is the difference between existing pavement thickness and the new thickness.

[a] A series of relationships developed between deflection change and traffic, depending on type of base course. Includes provision for different probabilities of achieving desired design life. Overlay material is hot-rolled asphalt.

[b] For airfield pavements; all others for highways.

[c] A dynamic stiffness module (DSM), defined as force/displacement, is used as the measure of pavement response rather than deflection.

layer. As with the other procedures, analysis sections are established using the deflection data and condition survey data obtained at the time of the deflection survey. A suggested design deflection for each section is the 85th percentile value. Remaining life is estimated from a knowledge of the design deflection, the amount of traffic applied to the time of the investigation (expressed in terms of 18 kip EALs, and observed relationships between change in deflection with traffic applications to failure. Relationships between change in deflection and traffic have been established for four base types including dense rolled macadam, lean concrete, and two types of untreated granular base. Probabilities of achieving a specific service life (in terms of EAL applications) have been included in the procedure. Overlay thicknesses of hot rolled asphalt are selected from charts similar in form to The Asphalt Institute and RTAC relationships. However, two design relationships are provided for overlays for each of the four base types; one based on a 50 percent probability of achieving the design life and the other based on a 90 percent probability.

U.S. Army Corps of Engineers Waterways Experimentation Procedure

In this procedure the pavement is subjected to vibratory loading by the WES 16-kip vibrator. Pavement response to load is defined by a dynamic stiffness modulus (DSM) defined as force/displacement. The DSM data together with condition survey data (based on the pavement condition index) are used to establish analysis sections and the mean DSM for a particular section is selected for evaluation and design. The DSM permits determination of the allowable gross aircraft load for specific gear configurations and number of aircraft repetitions (coverages). Overlay thicknesses are determined using the current WES procedure for pavement design since the DSM value has been related to the CBR of the subgrade through extensive field tests. The procedure requires converting the existing pavement structure to an equivalent pavement structure consisting of 3 in. (75 mm) of asphalt concrete, 6 in. (150 mm) of untreated base, and the remainder untreated subbase. For the specific

aircraft and number of coverages, a pavement thickness can be determined using the CBR procedure. The difference between the thickness of the new pavement structure and the existing equivalent pavement structure provides a measure of the overlay thickness required. This thickness difference, when divided by an equivalency factor of 1.7, provides the thickness of asphalt concrete overlay.

ANALYTICALLY BASED PROCEDURES

Table 6 provides an overview of the analytically based procedures briefly summarized in this section. The table follows the format of Figure 6 for easy reference. All of the procedures make use of the assumption that the pavement structure responds to traffic loading as a multilayer elastic solid.

Shell (Amsterdam) Research Procedure

Pavement deflections are measured with a falling weight deflectometer (FWD). Two sets of equipment are used, one for highway pavements and another, with larger loading capabilities, for airfield pavements. The deflection data together with condition surveys permit establishment of the analysis sections. Two parameters obtained from the deflection measurements are used for analysis and design, the center (directly under load) deflection and a measure of the shape of the deflected area obtained by dividing the deflection at some distance from the load by the center deflection and termed Q_r. For design purposes the 85th percentile deflection and the 15th percentile of Q_r are recommended for use. With these parameters it is possible to estimate the effective thickness of the asphalt layer in the existing pavement and the subgrade stiffness modulus by using the BISAR computer program and ascertaining the values for these parameters that provide reasonable correspondence between the measured and computed deflections and deflection ratios. With the properties of the existing pavement ascertained, remaining life both with respect to fatigue in the asphalt-bound layer and surface rutting as controlled by subgrade strain can be estimated.

If a decision to overlay is made, three different overlay thicknesses are determined: (a) a thickness to control fatigue in the asphalt-bound layer; (b) a thickness to limit surface rutting (i.e., a thickness to limit the subgrade strain to a prescribed level); and (c) a thickness assuming the existing asphalt layer is cracked and has the same stiffness characteristics as the granular base. If the last thickness is less than the first two it is used; otherwise the overlay thickness is the larger of the two values obtained to satisfy conditions (a) and (b). For highway pavements the design charts presented in Reference (*43*) simplify the computation process, particularly for asphalt pavements with granular bases.

TABLE 6

ANALYTICALLY BASED OVERLAY DESIGN PROCEDURES[a]

Procedure	Nondestructive Pavement Evaluation	Stiffness Determinations		Analysis Procedure	Distress Mechanisms		Provision for Existing Pavement	Overlay Thickness Determination
		In-Situ Measurement	Lab Testing		Fatigue	Rutting		
Shell Research	Falling weight deflectometer	Yes	No	BISAR computer program	Yes	Yes	Yes	Overlay thickness selected to (a) limit fatigue and (b) limit rutting for anticipated traffic; thickness also selected assuming existing pavement is cracked.
FHWA-ARE	Dynaflect; Benkelman beam	No	Yes	ELSYM computer program	Yes	Yes	Yes	Overlay thickness selected to (a) limit fatigue and (b) limit rutting for anticipated traffic; asphalt concrete assigned different stiffness values depending on conditions.
FHWA-RII	Dynaflect and others	Yes	Opt.	ELSYM computer program	Yes	No	Yes	Overlay thickness selected to limit fatigue for anticipated traffic; asphalt concrete assigned different stiffness values depending on conditions.
Kentucky	Road Rater	Yes	No	Graphic solution[b]	Yes	No	Yes	Overlay thickness selected as difference between pavement thickness required to accommodate all traffic (both applied and anticipated) and effective thickness of existing pavement as determined by nondestructive evaluation of existing pavement.

[a] All procedures require a condition survey, represent the pavement as a multilayer elastic solid, and provide an estimate of remaining life.

[b] Based on Chevron computer solution for multilayer elastic solid.

Federal Highway Administration—Austin Research Engineers Procedure

Deflection measurements can be made with any device that will produce satisfactory measurements. As a part of the condition survey, cracking is defined in terms of the AASHO Road Test definition (i.e., Class 2, Class 3). Also, it is recommended that rut depths be measured. Like the other procedures, these data are used to establish analysis sections. To ascertain the properties of the various layers of the existing pavement, it is recommended, that samples be taken from each of the analysis sections for laboratory stiffness modulus measurements. The measured values are used to calculate the design deflection (85th percentile suggested) using the ELSYM computer program. If required to obtain correspondence between measured and computed deflections, the material stiffnesses, particularly those of the subgrade, will be adjusted. Remaining life in the existing pavement can be estimated relative to fatigue in the asphalt concrete and to surface rutting based on an estimate of applied traffic to date and using the ELSYM computer program for determination of critical stresses and strains.

If a decision to overlay is made, for the fatigue analysis different moduli are assigned to the asphalt layer depending on the amount of cracking. For example, if the layer is badly cracked (Class 3 cracking 5 percent) it is assigned a stiffness modulus equal to that of the untreated base. Usually a range in thicknesses to satisfy fatigue and rutting considerations are determined. For the anticipated additional traffic, the larger of the two values is selected.

Federal Highway Administration—Resource International Incorporated Procedure

The procedure has been developed to accommodate deflection measurements obtained with the Dynaflect, the Road Rater, or a falling weight deflectometer. If other devices are used, the computer program developed to analyze the data must be modified. Because fatigue is the governing distress determinant, the extent of cracking (AASHO Road Test—Class 2 and Class 3 Cracking) is determined during the condition survey.

To define the material characteristics, the ELSYM computer program is utilized to match computed and measured deflections by an iterative procedure. In this analysis, the nonlinear (stress dependent) stiffness characteristics of both untreated granular materials and fine-grained soils are recognized. With the stiffness characteristics so determined, the analyses for remaining life and overlay design—based on considerations of fatigue—are accomplished in a manner similar to the ARE procedure.

It should be noticed that the methods identified herein as FHWA-ARE and FHWA-RII were sponsored by the Federal Highway Administration but have not been adopted by that agency as official, approved procedures. Studies are currently under way to establish the credibility of these methods for general application.

Kentucky Department of Highways Procedure

Deflections are measured with the Road Rater and are obtained directly under the load and at 1- and 2-ft (0.3- and 0.6-m) intervals from the load to permit definition of the shape of the deflection basin. The pavement is assumed to behave as a multilayer elastic system. In-situ subgrade moduli and effective asphalt concrete layer thicknesses are determined graphically by ensuring correspondence between the measured and computed shapes of deflected pavement surfaces.

Overlay thicknesses, if required, are determined as follows. With estimates of the total traffic (both already applied and anticipated) and the subgrade stiffnesses (converted to CBRs), thicknesses are determined using the Kentucky Pavement Design Procedure. It should be noted that the design procedure for "new" pavements also is based on the assumption that the pavement responds to traffic loads as a multilayer elastic system, and thicknesses of the components are selected to control fatigue in the asphalt concrete (tensile strain) and pavement rutting (subgrade strain). Because the evaluation procedure had provided an indication of the effective thickness of the existing pavement, the overlay thickness is the difference between this thickness and the thickness developed for total traffic.

CHAPTER FIVE

REFLECTION CRACKING

In 1982 the NCHRP published a synthesis concerned with minimizing reflection cracking in overlay pavements (*2*). This synthesis provides an excellent summary of design considerations for both asphalt concrete and portland cement concrete overlays. In this chapter a brief summary of workable solutions for asphalt overlays to minimize reflection cracking is discussed and is based on the synthesis as well as other recently published information.

EFFECTIVE MATERIALS AND TREATMENTS

Sherman (*2*) reviewed the variations in performance of available systems used to minimize reflection cracking. He concluded that systems with a demonstrated capability to retard reflection cracking of asphalt concrete overlays on existing asphalt concrete pavements in specific circumstances include: (a) low-viscosity asphalt cement (200-300 pen.) used in asphalt concrete as an overlay or as an interlayer; (b) heater-scarifier remix of the existing surface covered with a new layer of asphalt concrete; (c) stress-absorbing membrane interlayer (SAMI) constructed with asphalt-rubber; (d) certain fabric interlayers that retard reflection cracks other than those thermally induced; and (e) thick overlays, which are less likely than overlays with a thickness of 2 in. (50 mm) or less to reflect cracks over a period of time.

Further, he has noted that systems with a demonstrated capability to retard reflection cracking of asphalt concrete overlays on existing portland cement concrete pavements in specific circumstances include: (a) thick asphalt concrete overlays (6 in.—150 mm), which are more effective than thin (2 in. or 4 in.—50 or 100 mm) asphalt concrete overlays where vertical movement is not excessive; (b) prefabricated membrane strips; and (c) open-graded asphalt concrete base course approximately 3.5 in. (90 mm) in thickness with a dense graded asphalt concrete wearing course also approximately 3.5 in. (90 mm) in thickness. In some locations asphalt-rubber interlayers (SAMIs) have been proven effective when used with at least 2 in. (50 mm) of asphalt concrete (*44*).

Recently Caltrans (*45*), as well as other organizations (*46, 47*), has used the technique of breaking the existing portland cement concrete slabs into sections 3 to 5 ft (0.9 to 1.5 m) on a side and seating the broken slabs with pneumatic or vibratory rollers before placing an asphalt concrete overlay. Caltrans has used as the overlay 0.35 ft (107 mm) of asphalt concrete with an asphalt-filled fabric, the fabric being placed on an asphalt concrete leveling course 0.1 ft (30 mm) in thickness with 0.25 ft (75 mm) of asphalt concrete placed on top of the membrane. It should be noted that an optimum procedure and equipment for breaking and seating the concrete slabs has not yet been fully developed.

When either the SAMI or fabric interlayer is to be used, it is recommended that cracks or joints greater than 1/4 in. (6 mm) wide be filled before placing the overlay (*44*).

Information concerning recycling of bituminous materials containing fabric is given in reference (*48*). The report concludes that pavements containing fabric can be recycled with no significant problems.

Another treatment for reflection cracking consists of sawing and sealing joints in the overlay above the existing joints in the portland cement concrete pavement (*46*). This method assumes that reflection cracks are inevitable and seeks to create straight, smooth-sided cracks that can be easily resealed. Research in New York found that sawing and sealing was successful in controlling deterioration of the overlay (*46*).

THICKNESS DESIGN

Thickness design to minimize reflection cracking has been based largely on experience and is reflected in standard designs. An example of such an approach is that developed by The Asphalt Institute and included in the summary of this procedure contained in Appendix A.

POTENTIAL THICKNESS SELECTION PROCEDURES

Currently there are no documented analytical (mechanistic) procedures that can be used *routinely* to select overlay designs that incorporate the range of available materials necessary to minimize reflection cracking. A number of investigators have, however, been examining the problem and some guidelines for a design methodology can be inferred from these studies.

McCullough and Seeds (*49*) have presented an analytical procedure incorporated into a computer program termed RFLCR, which had been developed for the Federal Highway Administration (*50*) to analyze the reflection cracking problem. Essentially the analysis consists of procedures to define:

1. shear strains in the overlay resulting from differential vertical movements at the joint from traffic loading; and
2. tensile strains in the overlay resulting from horizontal movements at the joint caused by a drop in temperature.

The analysis considers many of the factors associated with reflection cracking including the following: (a) creep modulus of asphalt concrete overlay, (b) dynamic modulus of asphalt

concrete overlay, (c) thickness of existing portland cement concrete layer, (d) temperature changes in pavement materials, (e) coefficients of volumetric expansion and contraction for pavement materials, (f) forces acting between pavement materials (e.g., those caused by continuous reinforcement), (g) differential deflection at crack or joint; (h) width of bond breaker over joint or crack if any, (i) horizontal movement at joint or crack, and (j) crack or joint spacing.

A limitation of the procedure at this time is the definition of appropriate limits for the values of shear and tensile strains as a function of traffic and environment. Nevertheless, the procedure does consider many of the important variables that contribute to reflection cracking and, therefore, provides guidance to designers to assist them in preparing designs for their specific situations.

A promising analytical procedure to permit realistic examination of the reflection cracking problem is that which makes use of finite-element methodology. Such an approach has been used by a number of investigators including: Majidzadeh and Suchariah (*51*), Coetzee and Monismith (*52*), Chen et al. (*53*), and Yüce et al. (*54*). In these studies, the investigators demonstrate how the method can be used to evaluate specific design and material parameters for particular situations.

Although such an approach is not readily usable on a routine basis at this time because of the need for a computer with relatively large capacity, developments in the computer hardware and programming areas should make such an approach much more usable in the not-to-distant future; e.g., developments like those described in reference (*54*) involving the use of the microcomputer.

CHAPTER SIX

RECOMMENDATIONS

In this synthesis three general methods have been presented for the assessment of the structural capacity of existing pavements and for the design of overlays. Included are the *component-analysis, deflection-based,* and *analytically (mechanistically) based* procedures. Each method has certain advantages and disadvantages. These have been summarized in Table 7.

If use is made of the deflection-based procedure and other than the Benkelman beam is available for deflection measurements, the design requirements should be based on the equipment being used rather than correlations of the results with the Benkelman beam deflections. In addition, when using the deflection-based procedure, rutting may have to be evaluated by a component-analysis procedure because the deflection-based methodology usually does not include provision to consider rutting problems.

Regardless of the method selected, it is strongly recommended that both *deflection measurements* and condition surveys be used to establish the analysis sections. In addition, efforts such as those of the TRRL to include *uncertainty* in the design process should be made.

For major facilities it is important that materials evaluation, particularly for the subgrade, be accomplished. Moreover, designers are urged, regardless of the method selected, not to sacrifice good engineering for the sake of simplicity.

It is recommended that the analytically based type of procedure be developed further because it provides the most comprehensive approach to both evaluation of the existing structural capabilities and the design of overlay pavements. Among other things, seasonal variations can better be addressed by this method as compared to the deflection-based and component-analysis procedures.

For small projects it may be possible to develop design catalogues based on the analytical methodology.

New materials and procedures are available; e.g., asphalt rubber and recycling. The analytically based approach permits inclusion of such materials and procedures as design alternatives.

A number of materials and systems have been devised that are effective in reducing reflection cracking. To use these treatments effectively under a wide variety of loading and environmental conditions requires that effort be devoted to the development of analytically based methodologies. The use of finite-element procedures appears to be a promising approach.

Finally, although no discussion has been included on drainage aspects, as with the design of new pavements, any overlay design must include provisions to minimize the effects of water on the performance of the pavement structure and must include corrective treatments, if required, to the existing pavement structure as well. NCHRP Synthesis 96 (*55*) provides useful guidelines to assist engineers in this regard.

TABLE 7

ADVANTAGES AND DISADVANTAGES OF OVERLAY DESIGN PROCEDURES

Procedure	Advantages	Disadvantages
Component Analysis	Assesses individual layers as they exist in the pavement. Related to existing conventional design procedures that have large amount of background information.	Limited amount of sampling and testing (to minimize cost). Conditions at the time of sampling may not represent general state of materials. Time required for sampling and testing. Oriented to distress mode for which associated design procedure was developed; e.g., CBR procedure associated with plastic deformation. Not applicable to new materials.
Deflection-Based	Areal coverage. Measurements representative of in-situ conditions. Relatively inexpensive. Relatively fast. Relatively high degree of reliability possible.	Does not measure material properties. Limited to materials and constructions for which correlations are established. Related to one mode of distress; e.g., fatigue cracking.
Analytically Based (mechanistic)	Appropriate distress modes can be considered individually; e.g., fatigue, rutting, low-temperature cracking. Capable of considering: o changed loading and tire pressure effects, o new materials, o environmental influences, o aging effects, and o influence of changed suburface drainage conditions.	Unfamiliar to most current designers. Requires new and different equipment. Limited experience to date. May require the use of a computer.

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APPENDIX A

DEFLECTION-BASED PROCEDURES

The procedures described in this Appendix are representative of the methods as practiced by the respective agencies as of the date of preparation of this report.

Agencies may modify their procedures as research is completed and implemented. For latest developments and complete descriptions of procedures, it is recommended that the agency be contacted directly.

THE ASPHALT INSTITUTE METHOD (MS-17)

In the methodology of the Asphalt Institute (*20*), which is applicable to *highway* pavements, two facets are considered:

1. Evaluation of the structural adequacy of the existing pavement (e.g., determination of remaining life) and
2. Determination of required overlay thickness.

Both structural adequacy and overlay thickness can be determined either by *component analysis* or from *deflection measurements.* Only the deflection-based procedure is discussed in this section.

Deflection

Pavement deflections are measured with the Benkelman beam using a rebound test procedure. At least ten measurements should be made for a specific test section or a minimum of twenty measurements per mile are recommended. Pavement temperatures are measured at the time of the deflection measurements so that deflections can be adjusted to a standard temperature 70°F (21°C), Figure A-1. A random sampling technique is recommended for the selection of locations for deflection measurements.

Structural Condition

In addition to establishing surface condition (to determine if ride quality requires improvement), a condition survey is performed to establish structural adequacy. This survey includes recording each kind of observed distress and the degree to which it has developed, along with locations or the number of times each occurs.

Homogeneous (Analysis) Sections

Condition survey data [plus information on the existing structural pavement section(s)] are used to establish the analysis section. Actually, the test sections referred to earlier generally become the analysis sections.

Design Deflection (Old Pavement)

The design deflection for each section is termed the *Representative Rebound Deflections,* δ_{rrd}, and is determined as follows:

$$\delta_{rrd} = (\bar{\delta} + 2S) \cdot f \cdot c \qquad \text{(A-1)}$$

where

$\bar{\delta}$ = mean deflection,
f = temperature adjustment factor (Fig. A-1), and
c = critical period adjustment factor (c = 1 if tests are made during the most critical period).

Locations within sections with deflections greater than δ_{rrd} are recommended for special treatment. It is suggested that additional deflections be measured to determine the extent of such weak areas. These locations may require digout and replacement, etc., before overlay.

Remaining Life

With δ_{rrd}, an estimation of remaining life can be determined as follows. From Figure A-2 the permissible traffic in terms of 18-kip single-axle loads can be estimated. By comparing this number to the amount of traffic that has already been applied to the pavement, its remaining life (in terms of 18-kip axle loads) can be ascertained.

Overlay Design

If an overlay thickness is required, Figure A-3 permits determination of its thickness based on δ_{rrd} and the anticipated traffic expressed in terms of repetitions of an 18-kip axle load, r_{18}. These curves are based on two relationships. The first, Eq. (A-2), was developed by Kirk (*56*) and relates deflection to moduli and layer thickness for a two-layer elastic system.

$$\delta_d = \frac{1.5\,\sigma_0 a}{E_s}\left(\left\{1 - \left[1 + 0.8\left(\frac{h_1}{a}\right)^2\right]^{-1/2}\right\}\frac{E_s}{E_1} + \left\{1 + \left[0.8\,\frac{h_1}{a}\left(\frac{E_1}{E_s}\right)^{1/3}\right]^2\right\}^{-1/2}\right) \qquad \text{(A-2)}$$

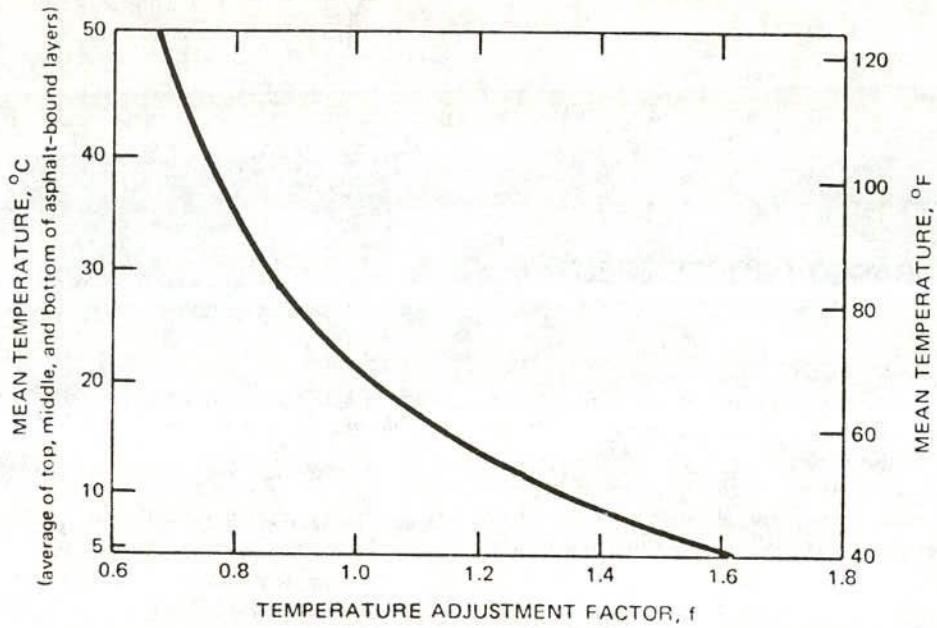


FIGURE A-1 Temperature adjustment factors for Benkelman beam deflections (*20*).

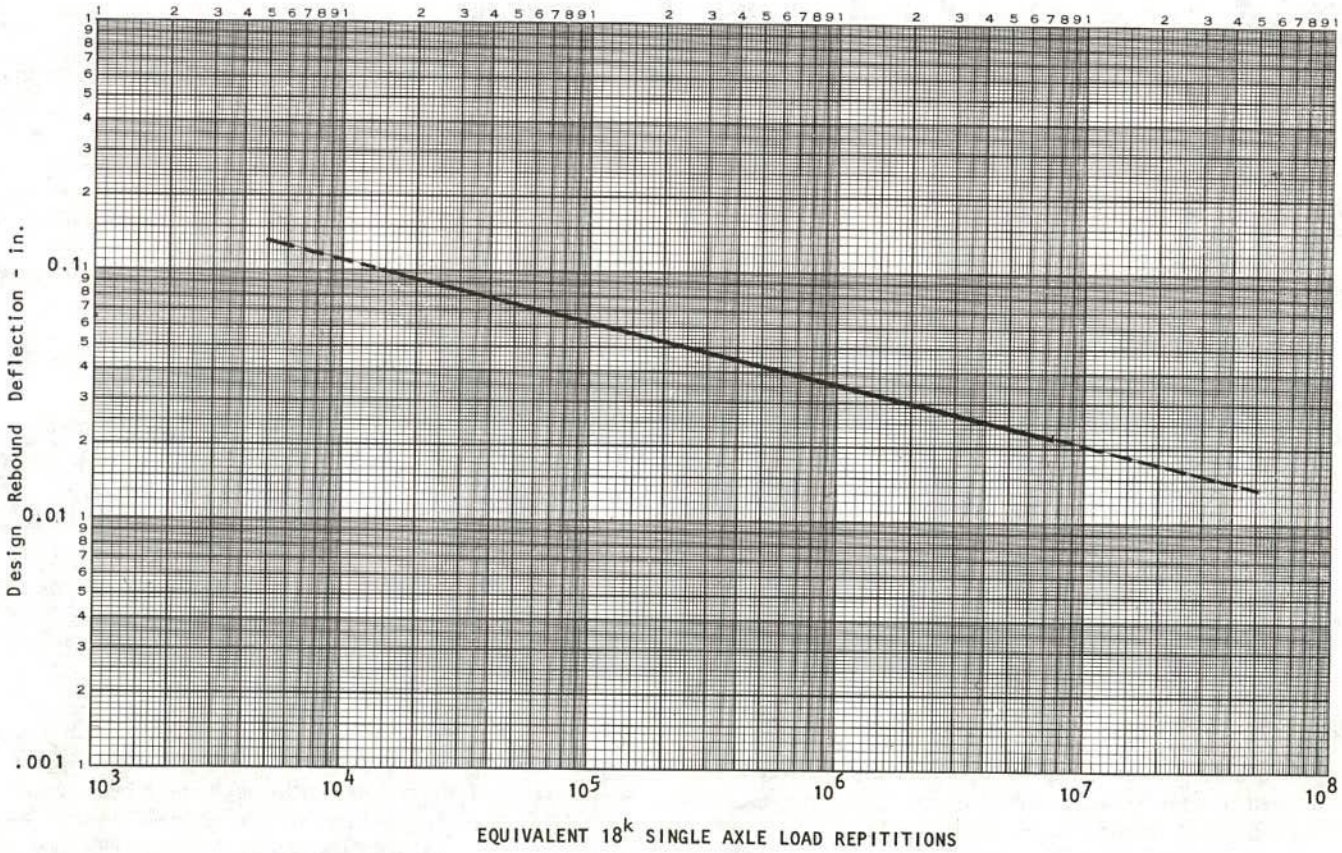


FIGURE A-2 Design rebound deflecting chart (*20*).

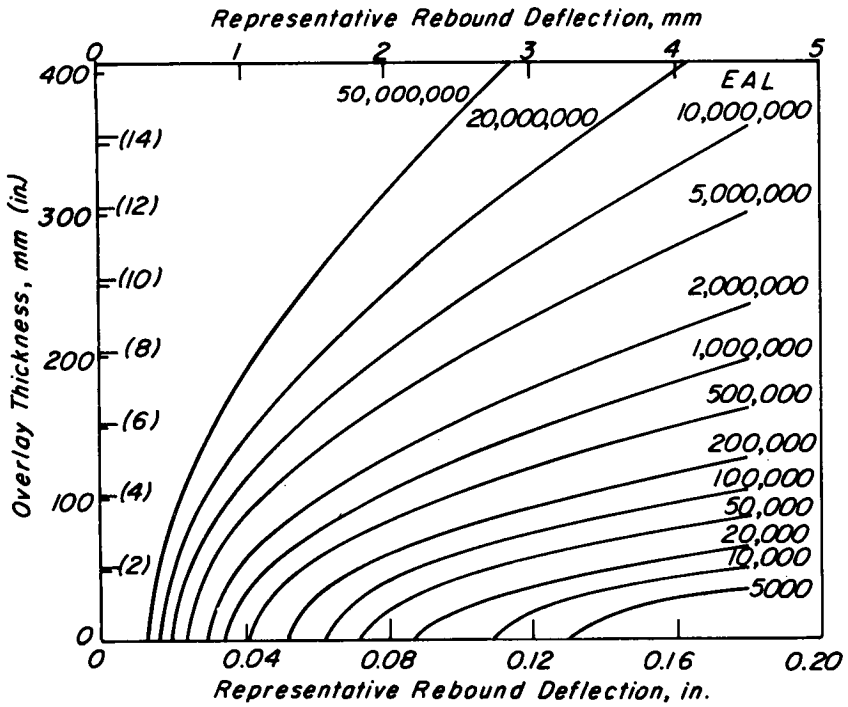


FIGURE A-3 Asphalt concrete overlay thickness required to reduce pavement deflection from a measured to a design deflection value (rebound test) (*27*).

where

δ_d = design deflection,
σ_0 = contact pressure,
a = radius of equivalent single loaded area to represent load on dual tires (assumed equal to 6.4 in.),
h_1 = pavement thickness,
E_1 = asphalt concrete overlay modulus (assumed equal to 500,000 psi), and
E_s = subgrade modulus and equal to $1.5\sigma_0\, a/\delta_{rrd}$.

The second relationship is as follows:

$$\delta_d = 1.0363\,(r_{18})^{-0.2438} \qquad \text{(A-3)}$$

A deflection-based procedure has also been developed for asphalt concrete overlays on jointed concrete pavements and on continuously reinforced concrete pavement (*57*). Deflections are measured at the pavement edge and on both sides of joints or cracks to determine load-transfer; the latter being important to indicate whether or not undersealing may be required.

Figure A-4 provides an indication of the overlay thickness required, depending on slab length and temperature change. Thicknesses obtained from this table should also be checked against their ability to reduce deflections, the assumption being that 1 in. of asphalt concrete is capable of reducing the deflection by 5 percent. A tolerable differential deflection at a joint or crack before the overlay of 0.002 in. is recommended. If the differential deflection exceeds this value, undersealing is recommended.

CALIFORNIA (CALTRANS) PROCEDURE

The State of California method is a deflection-based design procedure applicable to *highway* pavements. Overlay thickness requirements are controlled primarily by considerations of structural adequacy and the mitigation of reflection cracking (*42, 58*).

Deflection

Pavement deflections can be measured by the California (traveling) deflectometer, Benkelman beam (WASHO procedure), Dynaflect, Road Rater, Dehlen curvature meter, and Cox devices. Deflections of the various devices have been correlated with those measured by the deflectometer, since overlay designs are based on deflectometer values. The Dynaflect is the primary testing device used by Caltrans because of its simplicity and speed of operation.

Test sections are established for deflection measurements. If the project is less than one mile in length, the entire project is considered to be the test section. For projects greater than one mile in length, 1,000-ft sections are selected to be representative of each mile. For multilane highways, if the project is less than one mile in length, then the entire project is the test section; pavement deflections are measured in both outside lanes, and, if possible, in 1,000-ft sections in the inner lanes as well.

TEMPERATURE DIFFERENTIAL* (°F)

Slab Length (Ft)	30	40	50	60	70	80	Slab Length (m)
10 or Less	10cm (4 in.)	10cm (4 in.)	10cm (4 in.)	10cm (4 in.)	10cm (4 in.)	10cm (4 in.)	3
15	10cm (4 in.)	10cm (4 in.)	10cm (4 in.)	10cm (4 in.)	10cm (4 in.)	10cm (4 in.)	4.5
20	10cm (4 in.)	10cm (4 in.)	10cm (4 in.)	10cm (4 in.)	13cm (5 in.)	14cm (5.5 in.)	6
25	10cm (4 in.)	10cm (4 in.)	10cm (4 in.)	13cm (5 in.)	15cm (6 in.)	18cm (7 in.)	7.5
30	10cm (4 in.)	10cm (4 in.)	13cm (5 in.)	15cm (6 in.)	18cm (7 in.)	20cm (8 in.)	9
35	10cm (4 in.)	11.5cm (4.5 in.)	15cm (6 in.)	18cm (7 in.)	21.5cm (8.5 in.)	Use Alternative 2 or 3	10.5
40	10cm (4 in.)	14cm (5.5 in.)	18cm (7 in.)	20cm (8 in.)	Use Alternative 2 or 3	Use Alternative 2 or 3	12
45	11.5cm (4.5 in.)	15cm (6 in.)	19cm (7.5 in.)	23cm (9 in.)	Use Alternative 2 or 3	Use Alternative 2 or 3	13.5
50	13cm (5 in.)	18cm (7 in.)	21.5cm (8.5 in.)	Use Alternative 2 or 3	Use Alternative 2 or 3	Use Alternative 2 or 3	15
60	15cm (6 in.)	20cm (8 in.)	Use Alternative 2 or 3	Use Alternative 2 or 3	Use Alternative 2 or 3	Use Alternative 2 or 3	18
	17	22	28	33	39	44	

TEMPERATURE DIFFERENTIAL* (°C)

*Temperature differential (Δt) is the difference between the highest normal daily maximum temperature and the lowest normal daily minimum temperature for the hottest and coldest months, based on a 30-year average. See Tables 1 and 2 for maximum and minimum daily temperature at locations throughout the United States.

NOTE: There are several alternatives to the thicknesses in Sections B and C of this chart. See Art 3.04.

FIGURE A-4 Chart for selecting asphalt concrete structural overlay thickness for p.c.c. pavement (*57*).

Structural Condition

General conditions of the roadway in terms of visual appearances are noted. Limits and extent of rutting, ravelling, patching, and localized failures are recorded. Limits, extent, size, and type of surface cracking are obtained. Cracking is categorized into the following: hairline, up to ⅛ in. wide, ⅛ to ¼ in. wide, and greater than ¼ in. wide. Notation is made whether cracking is isolated, intermittent, or continuous. It is recommended that photographs of representative sections be taken as well.

Homogeneous (Analysis) Sections

With information from condition surveys, existing structural section data, and deflection data, analysis sections can be established. For each test section, a representative deflection is computed that is taken to be the 80th percentile deflection, δ_{80}, and is calculated as follows:

$$\delta_{80} = \bar{\delta} + 0.84 \cdot S \qquad \text{(A-4)}$$

where

$\bar{\delta}$ = mean deflection and
S = standard deviation.

It is recommended that a "deflection map" be drawn of the section of pavement under investigation, e.g., Figure A-5. From the 80th percentile deflection levels, together with the condition surveys, etc., analysis sections can then be established. In selecting areas of similar deflection levels, it is recommended that deflections should be within a 0.010-in. range.

Design Deflection (Old Pavement)

For each analysis section, the design deflection is taken to be the *average* of the 80th percentile values for the test sections encompassed.

Remaining Life and Overlay Design

There is no procedure for estimating remaining life in this methodology. Overlay design is accomplished as follows.

With the 80th percentile deflection representative of a particular analysis section, and with an estimate of the traffic to be applied, this representative deflection is compared with the tolerable deflection, δ_T, obtained from Figure A-6 for the existing pavement section. If the tolerable deflection exceeds the 80th percentile deflection, no *structural* overlay is required—at least for the near term.

If the representative deflection exceeds the tolerable value, a structural overlay is required. To determine the thickness of overlay, Figure A-7 is used. The required percent reduction in deflection is calculated as follows:

$$\text{Percent reduction in deflection} = \frac{\delta_{80} - \delta_T}{\delta_{80}} \times 100\% \quad \text{(A-5)}$$

Asphalt concrete used in the overlay is assigned a gravel equivalent value of 1.9. Thicknesses determined by this procedure are rounded to the nearest 0.05 ft. The curves shown in Figure A-6, with the exception of the one labeled 0.50 ft CTB or more, are applicable to existing pavement sections containing untreated aggregate underlying the asphalt concrete.

It should be emphasized that the selection of a tolerable deflection level requires the exercise of judgment relative to the condition of the existing surface. For example, if the pavement is continuously "alligator" cracked, one could assume that it is acting as an aggregate base and the overlay thickness alone would establish the tolerable deflection. If the existing pavement is largely intact with occasional transverse and longitudinal cracking, the combined thickness of overlay and existing pavement would establish the tolerable level.

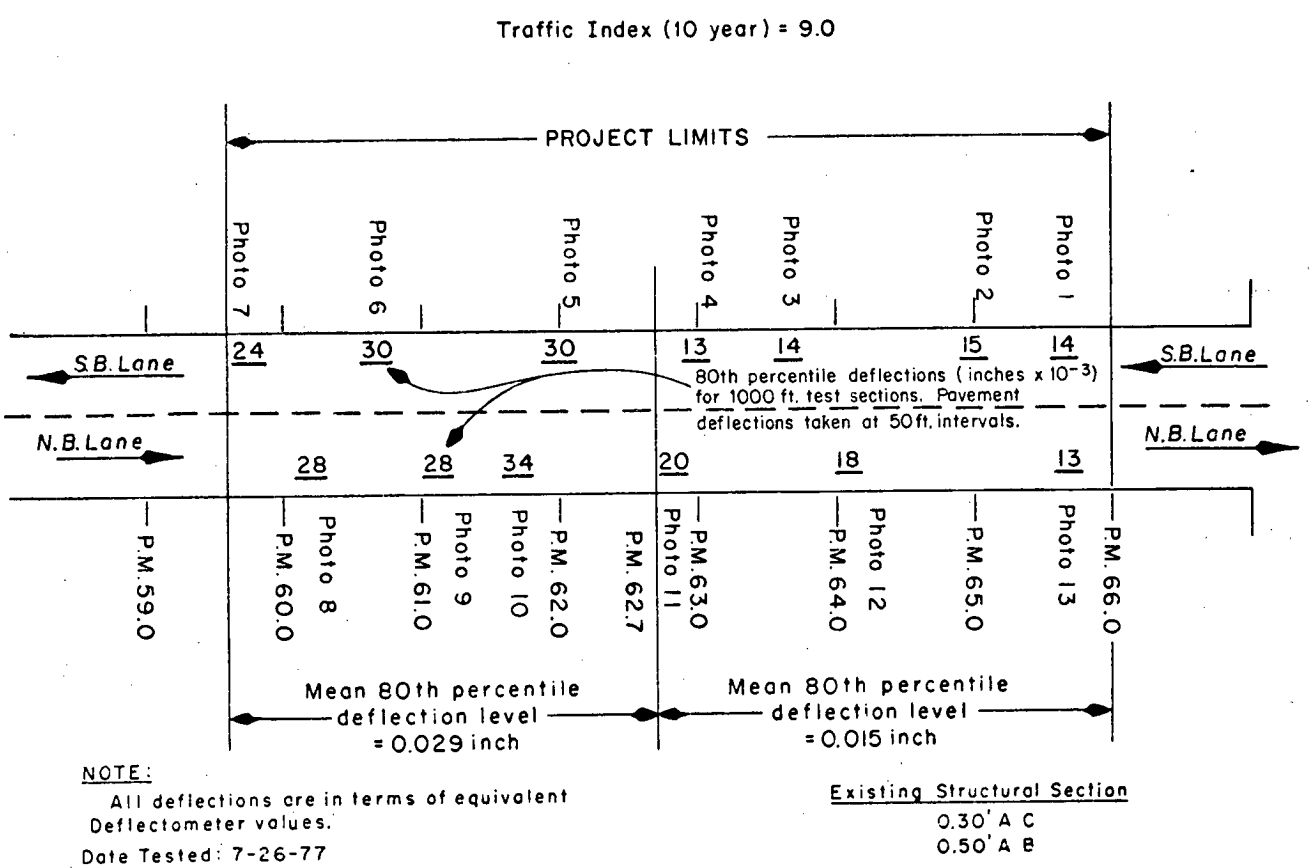


FIGURE A-5 Example of establishment of analysis sections (*42*).

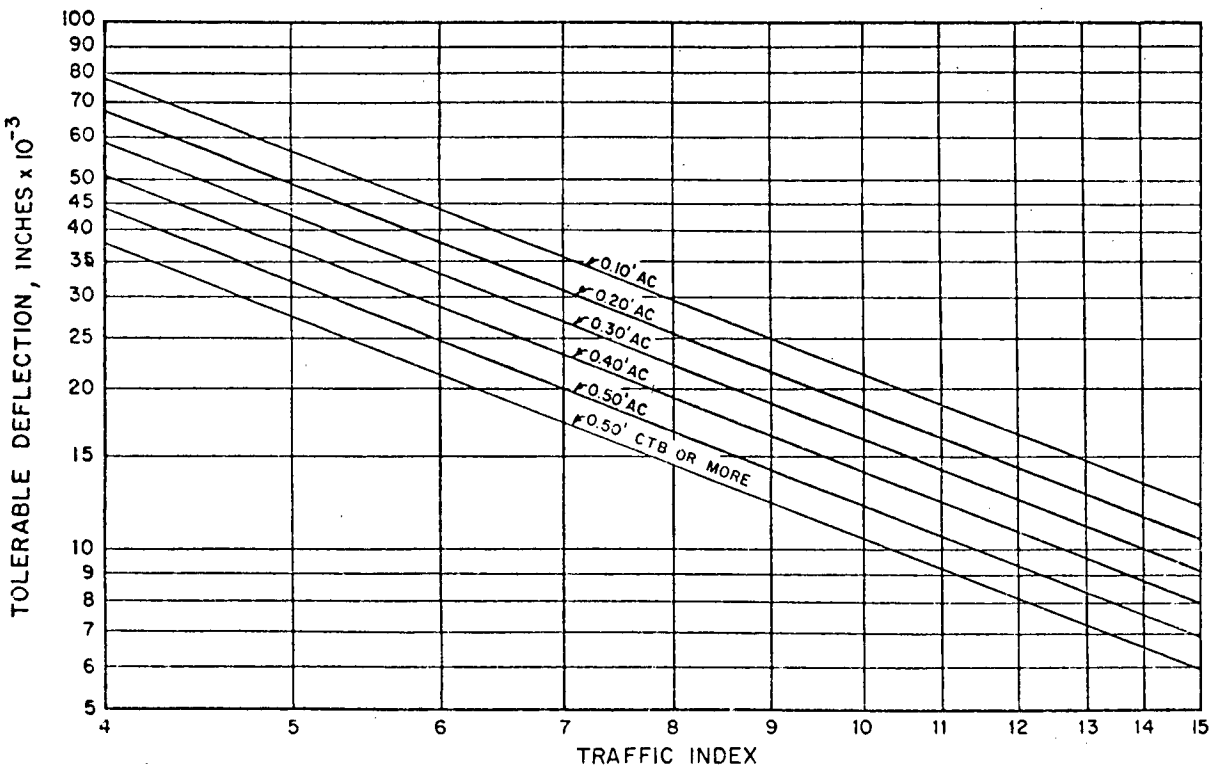


FIGURE A-6 Tolerable deflection chart (*42*).

No set procedure is available to mitigate the effects of reflection cracking. The selection of a tolerable deflection is dependent on the type, size, and amount of surface cracks, extent of localized failures, existing structural section, thickness and performance of previous overlays, ride quality, environmental factors, and anticipated traffic loadings.

It should be noted that the overlay should be at least half of the thickness of the existing asphalt concrete over untreated bases. This is extended to a maximum of 0.3 ft of asphalt concrete plus fabric, which appears to be sufficient to arrest reflection cracking over any thickness of surfacing if it has been stabilized (broken and seated in the case of portland cement concrete pavement).

In addition to the factors of structural adequacy and reflection cracking mitigation considered for overlay thickness design, Caltrans recommends that the following factors also be considered (*42*).

1. How have previous overlays on the roadway performed? What was the condition of the roadway before the overlay? How many years of service did the overlay thickness provide?
2. Do vertical grade controls limit the overlay thickness to less than that needed for a ten-year service life extension?
3. Can overlay requirements be reduced by digging out and repairing localized failures prior to the overlay?
4. Is a fabric interlayer or SAMI appropriate for the severity of cracking encountered?
5. Is recycling an acceptable alternative solution?
6. Is it more cost-effective to place a thick overlay or totally reconstruct the roadway?

Finally, it should also be noted that the minimum overlay thickness currently recommended is 0.15 ft owing to the difficulty in obtaining adequate compaction of thinner lifts.

ROADS AND TRANSPORTATION ASSOCIATION OF CANADA (RTAC) PROCEDURE

The RTAC (formerly Canadian Good Roads Association) procedure is a deflection-based procedure and has been in use since 1965 (*19, 59*). Although it is noted that overlays may be applied to improve ride quality (*19*) and some general guidelines are provided, the procedure is designed to increase the strength of the pavement.

Deflection

Pavement deflections are measured with the Benkelman beam using a rebound test procedure (*59*). Ten measurements are recommended for a specific analysis section. Deflections should be measured in the spring when they are at their peak values. Deflections can be measured at other times during the year; however, they must be converted to the maximum spring values by a suitable procedure. Pavement temperatures are measured at the time of the deflection measurements to permit values to be corrected to a standard temperature taken as 10°C (50°F) for seasonal strength loss normalization or 21°C (70°F) for summer pavement evaluation (*19*).

Structural Condition

Condition surveys are conducted and usually include not only the type of distress but also its extent, severity, and location (*19*).

Homogeneous (Analysis) Sections

The selection of the homogeneous sections is based on considerations of subgrade soil type, deflections, pavement condition, existing structural section thickness, and traffic; however, a minimum practical length of 300 m (1000 ft) is suggested for a specific section (*19*).

Design Deflection (Old Pavement)

The design deflection, δ_d, is taken as the peak spring Benkeleman beam rebound deflection and is determined as follows:

$$\delta_d = (\bar{\delta} + 2S) \tag{A-6}$$

where

$\bar{\delta}$ = mean of ten measurements and
S = standard deviation.

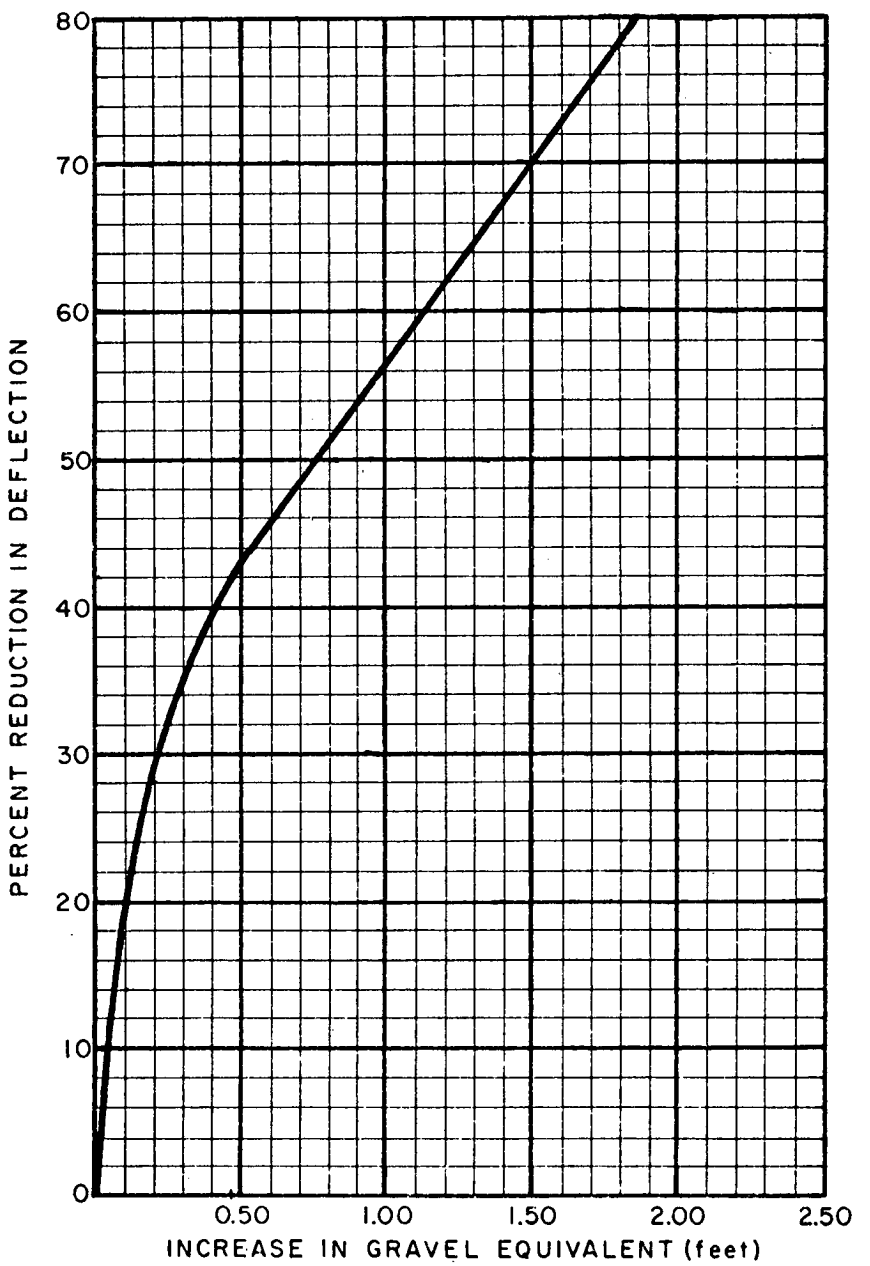


FIGURE A-7 Reduction in deflection resulting from pavement overlays (*42*).

Remaining Life

Although not indicated, remaining life presumably can be estimated from Figure A-8, which is a plot of deflection versus cumulative standard axles (in terms of 18-kip axle loads). With the design deflection, δ_d, the allowable number of load repetitions can be assessed. If an estimate of the traffic that has been applied to the pavement is available, remaining life (in terms of 18-kip EAL repetitions) can then be estimated.

Overlay Design

If an overlay is required, Figure A-9 is used. A thickness of granular material is selected that will reduce the measured deflection, δ_d, to a tolerable value associated with the anticipated traffic and obtained from Figure A-9. It is also noted that other relationships between deflection and traffic may be used based on local experience, and a number of relationships are available as seen in Figure A-10.

The granular material thickness can be converted to an equivalent thickness of asphalt concrete or combination of asphalt concrete and base using layer equivalencies shown in Table A-1. If a granular base is used, a minimum of 100 mm (4 in.) is recommended. Minimum thicknesses are recommended for the asphalt concrete layer ranging from 1.5 in. (38 mm) for minor roads to 5 in. (125 mm) for primary highways and freeways.

In conjunction with the methodology, it is recommended that the following procedure be used to assist in the assessment of the proper overlay thickness.

1. Estimate the traffic in equivalent 80-kN (18,000-lb) single-axle loads that the "failed" pavement has carried during its life.
2. Using the known structural thickness of the existing pavement, and the traffic volume carried, determine what the subgrade soil strength has been to carry these loads.
3. Estimate future traffic for which the overlay is being designed.
4. Using the existing structure, consider what additional thickness of asphalt concrete would be required to carry the traffic from step 3 if this existing structure were perfectly sound. This should give a minimum overlay thickness.
5. Using the existing structure, consider what additional thickness of asphalt concrete would be required to carry the traffic from step 3 with existing structural components downgraded because of their deteriorated state. A common practice is to assume existing asphalt layers have deteriorated to a point where they are structurally equal to a similar thickness of unbound granular base course. This should give the maximum thickness of asphalt concrete overlay required to carry the future traffic.
6. Steps 5 and 4 should act as boundary conditions; the asphalt concrete overlay thickness determined by deflection should fall between these two thicknesses. If this is not the case, the design should be reassessed to determine why the discrepancy exists and the design thickness modified based on this reassessment.

This procedure is recommended as a valuable tool for two reasons: (a) it acts as a crude check on the overlay design and (b) it acts as a continuous monitor of the adequacy or accuracy of the structural design procedure.

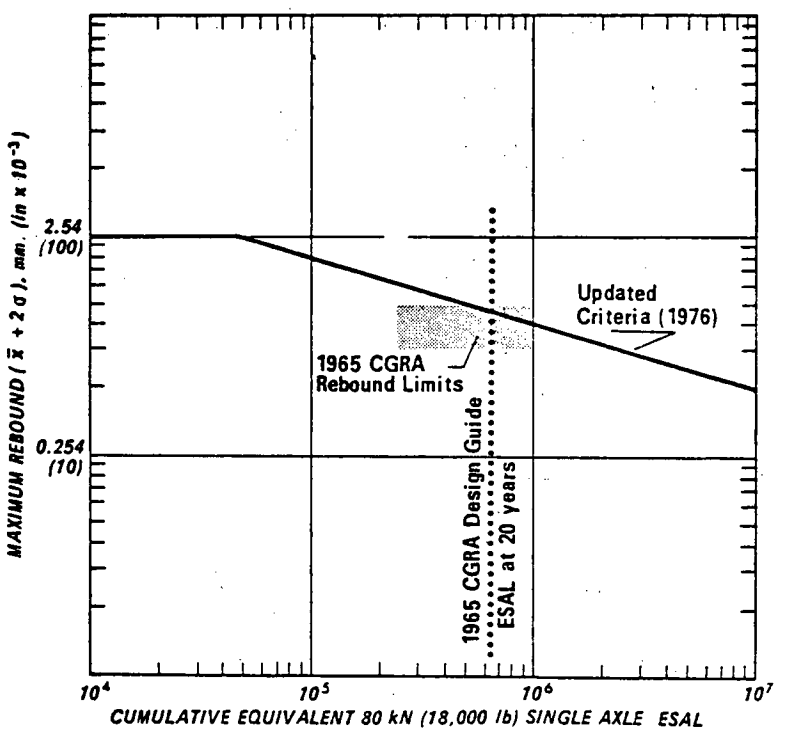


FIGURE A-8 Recommended criteria for Benkelman beam rebound versus cumulative axle loads.

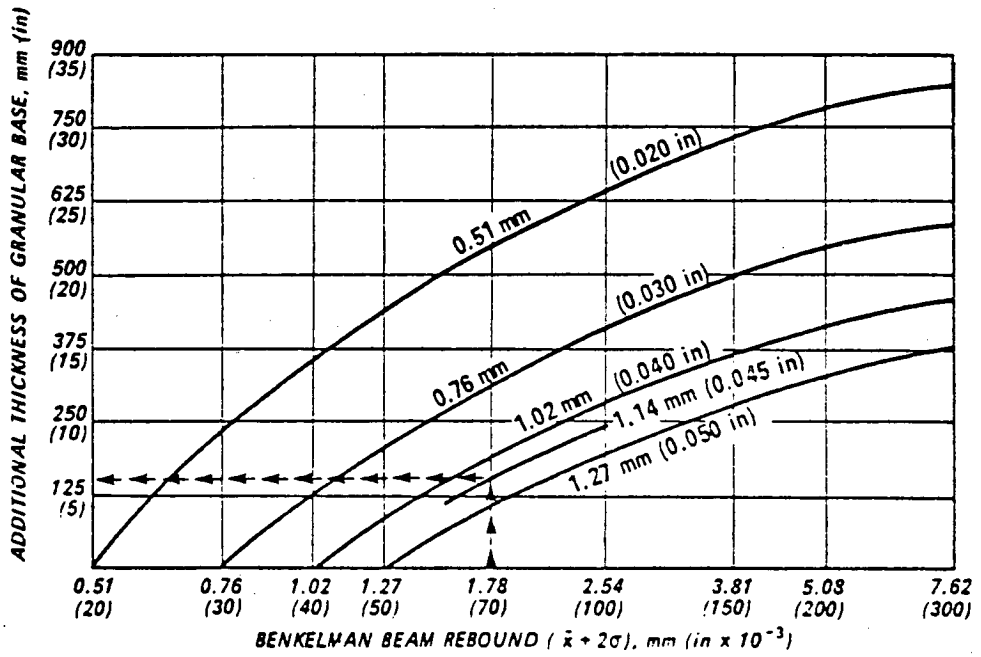


FIGURE A-9 Additional thickness of granular base required to reduce a Benkelman beam rebound on an existing surface to a designated or design rebound.

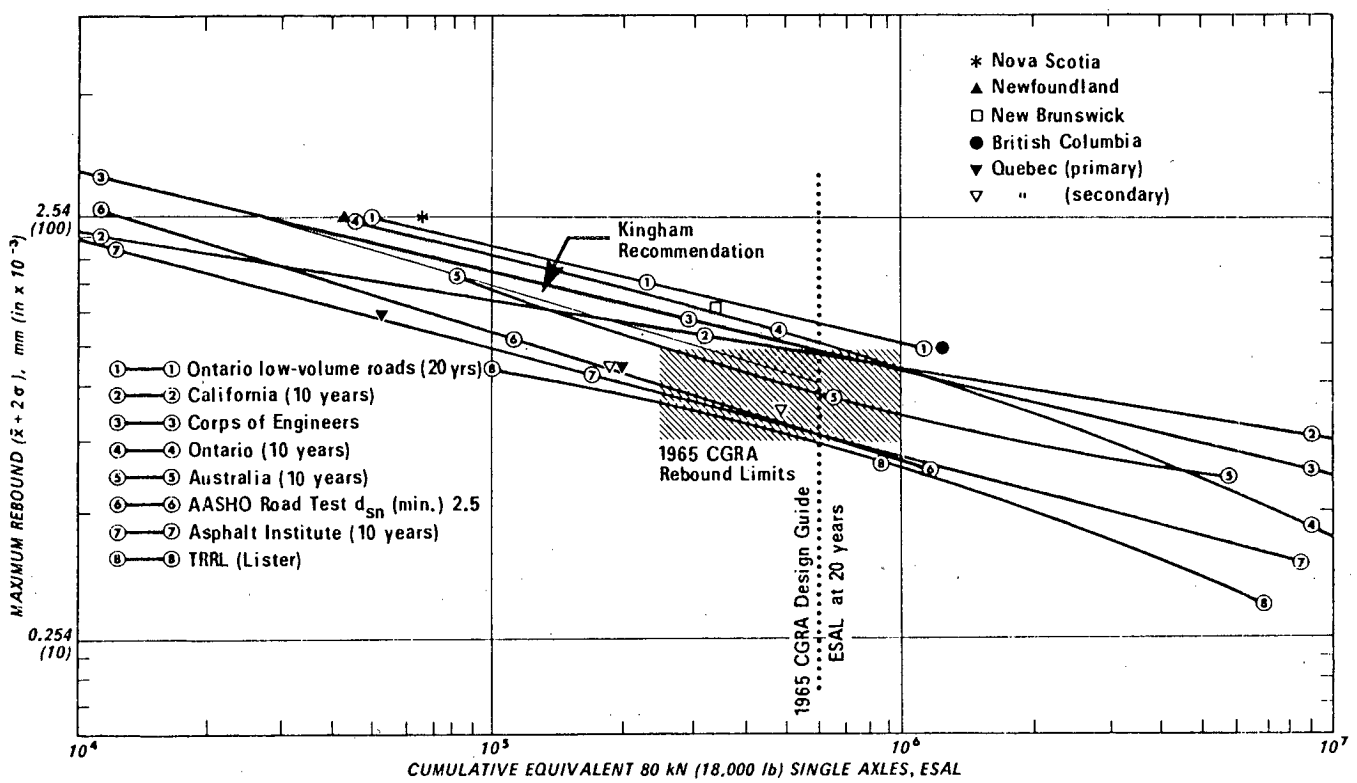


FIGURE A-10 Maximum Benkelman beam rebound ($\bar{x} + 2\sigma$) versus cumulative equivalent 80 kN (18,000 lb) single-axis loads.

TABLE A-1

APPROXIMATE LAYER EQUIVALENCIES USED BY PROVINCIAL AGENCIES (*19*)

Provincial Agency	Equivalencies	
British Columbia	1 mm Asphalt Conc.	= 2 mm gravel base (min.)
		= 2.5 mm sandy gravel subbase
Alberta	1 mm Asphalt Conc.	= 2.25 mm crushed gravel
		= 1.75 mm soil cement
		= 1.25 mm asphalt treated gravel base
Saskatchewan	Considered as a variable and therefore not used.	
Manitoba	1 mm Asphalt Conc.	= 2 mm gravel base
		= 1.5 mm sand asphalt or soil cement
		= 2 mm lime treated clay
Ontario	1 mm Asphalt Conc.	= 1 mm treated base (asphalt or cement)
		= 2 mm granular 'A' base
		= 3 mm granular (B, C, D) subbase
	1 mm Full Depth A.C.	= 2.7 mm granular 'A' base (tentative)
Quebec	1 mm Asphalt Conc.	= 2 mm crushed rock base
		= 2.5 mm gravel base or subbase
		= 5 mm sand subbase
		= 1.25 mm soil cement (150 mm thick or less)
		= 2 mm soil cement (more than 150 mm)
		= 3.3 mm lime stabilized clay
		= 1.8 mm asphalt stabilized base
Newfoundland	1 mm Asphalt Conc.	= 2.5 mm graded crushed rock
		= 2.5 mm graded crushed gravel
		= 2 mm soil cement stabilized
		= 3 mm gravel subbase
		= 4 mm sandy gravel
New Brunswick	1 mm Asphalt Conc.	= 2 mm crushed rock
		= 2 mm soil cement (150 mm thickness and over)
		= 2 mm (or less) asphalt stabilized base
		= 3 mm gravel subbase

TRANSPORT AND ROAD RESEARCH LABORATORY (TRRL) OF GREAT BRITAIN METHOD

This procedure, a deflection-based methodology for highway pavements is described in Reference *60*; supporting information is contained in References *61–63*.

Deflection

Deflections can be measured either with the Benkelman beam (termed deflection beam) or with a modified version of the Lacroix deflectograph. With either the Benkelman beam or the deflectograph, an axle load of 14,000 lb (62 kN) is used; load is applied through dual tires with an inflation pressure of about 85 psi (590 kN/m^2) for the truck used in Benkelman beam testing and 100 psi (690 kN/m^2) for the deflectograph.

Using the TRRL procedure for the Benkelman beam, that is, deflection measurements at intervals of 12 to 25 m (40 to 80 ft), depending on the condition of the existing surface, about 1 km (0.6 mile) of pavement can be surveyed in a working day. The deflectograph, on the other hand, automatically measures deflections at about 4.0-m (13-ft) intervals at a velocity of 2 km per hour (1.2 mph), permitting 10 to 12 km (6 to 7 miles) of pavement to be surveyed in a working day.

Measured deflections are adjusted to reflect the influence of temperature; Figure A-11 illustrates one such chart used to make these adjustments. The standard deflection corresponds to that measured when the pavement temperature is 20°C (68°F) at a point 40 mm (1.6 in.) below the surface. If the deflectograph is used these deflections are corrected to equivalent Benkelman beam deflections using the relationship shown in Figure A-12. Relations between deflection and temperature, like that of Figure A-11, should not be extrapolated beyond the limits shown. For thick bituminous layers (> 175 mm − 6.9 in.) the recommended rate of temperature change at the 40 mm depth should be less than 2 ½°C per hour (4.5°F/h) during deflection measurements.

Structural Condition

The condition of the existing road surface is classified as shown in Table A-2.

Homogeneous (Analysis) Sections

Deflection profiles can be plotted in different ways. Results from deflectograph measurements, adjusted to Benkelman beam deflections, are shown in Figure A-13. The upper diagram represents the mean of three deflections over approximately 12-m (40-ft) intervals. The lower table illustrates the interpretive procedure to be followed. With these data, as well as the condition surveys and other appropriate information including existing pavement sections and maintenance history, analysis sections are selected.

Design Deflection

In selecting design (representative) deflections, the TRRL considers the proportion of the length of strengthened road that may reach a critical condition during the design life, and the earliest point in the design life at which this length may reach a critical condition. This often results in the use of the 85th percentile deflection.

Remaining Life

Assessment of remaining life is obtained from charts like the one shown in Figure A-14. Use of these charts requires a representative (design) deflection and an estimate of traffic expressed by the number of repetitions of an 80-kN (18-kip) axle (termed number of standard axles).

Life expectancy of the pavement is determined by following the deflection trend line, from a point defined by the present level of deflection and total applied traffic at the time of measurement, to the envelope of the selected probability level. Four charts of this kind have been developed and include, in addition to that shown in Figure A-14, relations for pavements with noncementing granular bases, bituminous bases (Fig. A-15) and cement-bound bases. Figure A-16 (*64*) illustrates the type of data from which the charts represented by Figures A-14 and A-15 were developed.

Figure A-14 illustrates specific examples for remaining life estimates for a pavement that has carried 3×10^6 standard axles and has deflections of 25, 45, and 80×10^{-2} mm (0.010, 0.018, and 0.031 in.). For a probability of 50 percent in achieving the design life, the remaining life for a deflection of 25×10^{-2} mm (Point A) is 19×10^6 standard axles; for a deflection of 45×10^{-2} mm (Point B), the remaining life is only 3×10^6

TABLE A-2

CLASSIFICATION OF THE CONDITION OF ROAD SURFACES (*60*)

Classification	Code	Visible Evidence
Sound	1	No cracking; rutting under a 2 m straightedge less than 5 mm.
	2	No cracking; rutting from 5 to 9 mm
Critical	3	No cracking; rutting from 10 to 19 mm.
	4	Cracking confined to a single crack or extending over less than half of the width of the wheel path; rutting 19 mm or less.
Failed	5	Interconnected multiple cracking extending over the greater part of the width of the wheel path; rutting 19 mm or less.
	6	No cracking; rutting 20 mm or greater.
	7	Cracking confined to a single crack or extending over less than half the width of the wheel path; rutting 20 mm or greater.
	8	Interconnected multiple cracking extending over less than half the width of the wheel path; rutting 20 mm or greater.
	8	Interconnected multiple cracking extending over the greater part of the width of the wheel path; rutting 20 mm or greater.

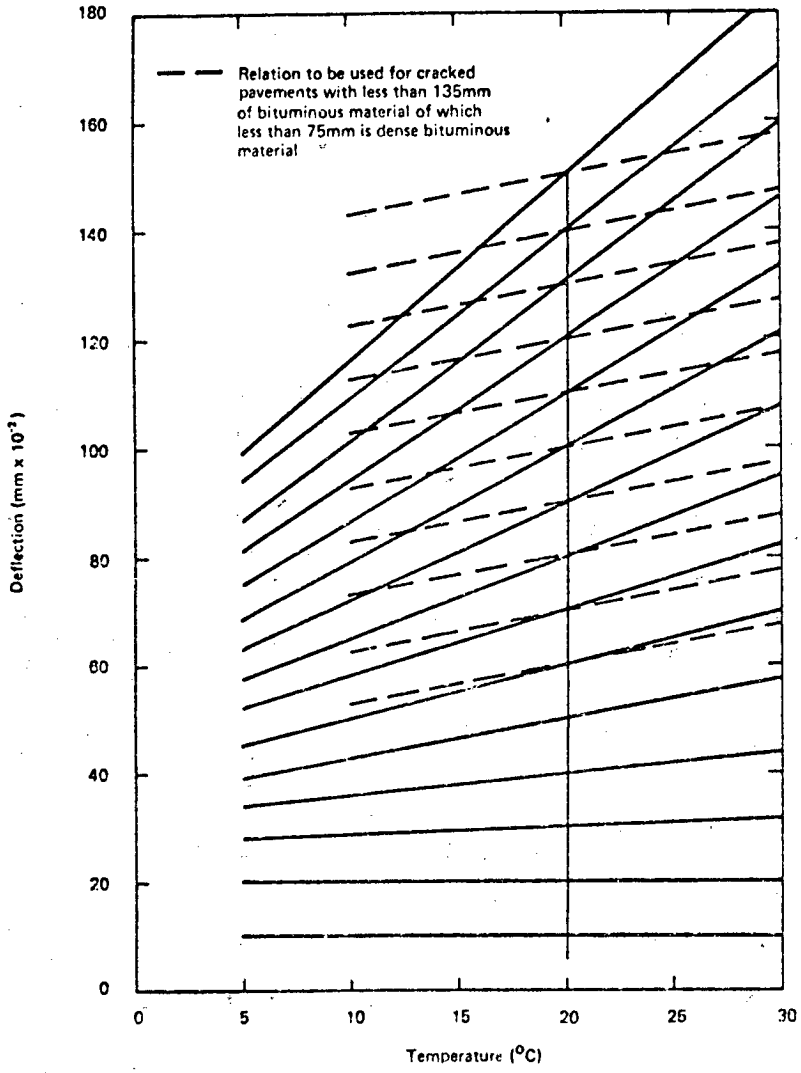


FIGURE A-11 Relation between deflection and temperature for pavements with less than 135 mm of bituminous material, of which less than 75 mm is dense bituminous material (*60*).

standard axles. A deflection of 80×10^{-2} mm (Point C) lies above the critical conditions curve; the pavement is structurally weak and strengthening is now required. Figure A-14 also shows the maximum deflection 29×10^{-2} mm (0.011 in.) that will give a probability of achieving an additional design life of 10×10^6 standard axles before overlaying (i.e., a total of 13×10^6 standard axles).

Overlay Design

Overlay thicknesses are selected from a series of charts like those shown in Figures A-17 and A-18 for probabilities of 0.5 and 0.9 of achieving a specific design life in terms of standard axles carried. Similar curves have been developed for the other base types noted earlier. From Figure A-17 and A-18 it will be noted that a minimum overlay thickness of 40 mm (1.6 in.) is recommended.

The design charts represented by Figure A-17 and A-18 were developed from observed data on reduction of deflection after overlaying like that shown in Figure A-19 (*64*).

Figure A-13 also illustrates strengthening requirements for the selected analysis sections. Overlay thicknesses are selected in 25-mm (1-in.) steps. Deflections associated with these thicknesses are then plotted, as shown in Figure A-13; designs are based on 10×10^6 and 7×10^6 repetitions. In this example, 15 percent of the road was allowed to become critical in the 7×10^6 to 10×10^6 applications period (85th percentile deflection having been selected). In actuality, because of the selection of 25-mm thickness increments, only about 10 percent of the pavements will reach this condition. It will also be noted that the 10 percent is reasonably distributed throughout the length of the project.

Finally, it should be noted that the design charts, like those shown in Figure A-17 and A-18, are for overlays using *hot rolled asphalt* (BS594 type asphalt mix). If other asphalt mixtures are

used [e.g., asphalt concrete with a dense (well) graded aggregate], adjustment factors must be applied to the thicknesses obtained.

U.S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION (WES) PROCEDURE (DYNAMIC STIFFNESS BASED)

This procedure is a deflection-based evaluation and overlay design procedure for *airfield* pavements (*65, 66*). It utilizes a dynamic deflection device (WES heavy vibrator) and the methodology that personnel of the Corps of Engineers have developed over the years for the design of both asphalt concrete and portland cement concrete airfield pavements. Although a dynamic stiffness modulus (DSM) rather than a deflection is determined, the procedure follows the format discussed in Chapter 2 for overlay design using deflection measurements.

Dynamic Stiffness Modulus (DSM)

Response of the pavement to load is measured in a specific steady-state vibratory loading. The loading device (termed the WES heavy vibrator) exerts a static load of 16 kip on the pavement and 0 to 15 kip vibratory loads at a frequency of 15 Hz; a steel plate 18 in. in diameter is used to apply the load.

It is recommended that DSM tests be made at:

1. 250-ft intervals along main gear paths on alternate sides of the facility centerline on runways and primary and high-speed taxiways.
2. 500-ft intervals on alternate sides of centerline for secondary taxiway systems and lesser used runways.
3. 250 to 500-ft grid pattern on apron areas.

At each test site a deflection versus load relationship is established. The DSM is the inverse of the slope of this plot, e.g., Figure A-20.

For existing portland cement concrete pavements, it is necessary to determine the deflection basin to permit an estimate of the radius of relative stiffness, ℓ.

The strength of the asphalt concrete pavements greater than 3 in. thick will vary with temperature. Because of this, the measured DSM values are adjusted to a common mean pavement temperature of 70°F (21.1°C). This adjustment allows DSM measurements taken at one temperature to be compared to

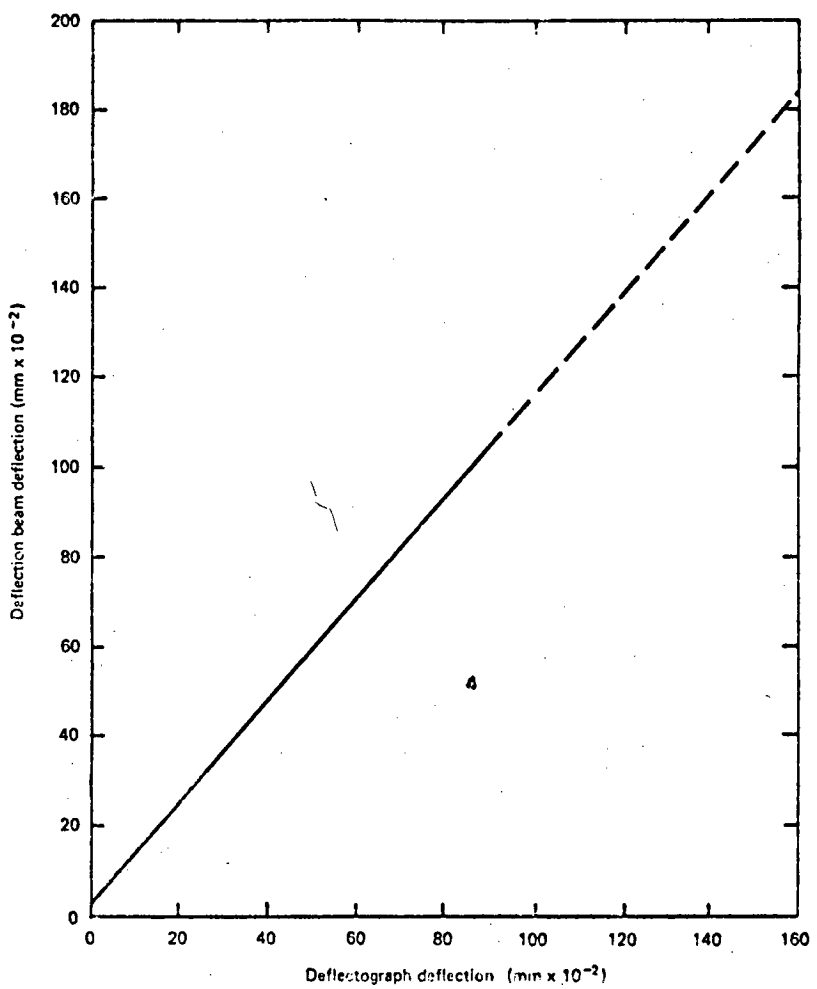
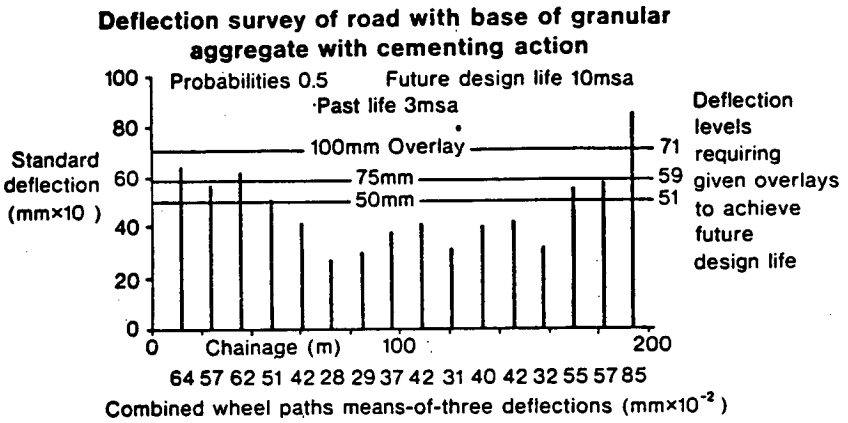


FIGURE A-12 Correlation between deflection beam and deflectograph (*60*).



Section	0 to 100m	100 to 200m
Characteristic deflection	62mm×10^{-2}	57mm×10^{-2}
Initial overlay design for 10msa future life	100mm	75mm
Maximum deflection to give 75% of 10msa future life with initial overlay thickness	82mm×10^{-2}	69mm×10^{-2}
Any deflections above this level?	No	Yes,at 192m
Any local reconstruction?	No	Yes,186 to 200m
Characteristic value of remaining deflections	—	55mm×10^{-2}
Revised overlay design	—	75mm
Maximum deflection to give minimum life requirement when overlaid	—	69mm×10^{-2}
Any deflections above this level?	—	No
Final detailed overlay design	100mm	100 to 186m — 75mm 186m to 200m–Local Reconstruction

FIGURE A-13 Example of detailed overlay design before application of practical restraints.

measurements taken at different temperatures. The correction factors used are determined from measurements of the pavement surface temperature during the test period combined with the previous five-day mean air temperature using relationships developed with the nondestructive evaluation procedures.

Structural Condition

The procedures has been implemented for evaluation of Army airfields. For these pavements, condition surveys using the pavement condition index (PCI) are routinely conducted along with the nondestructive evaluation. The PCI is used primarily to determine maintenance requirements.

Homogeneous (Analysis) Sections

Analysis sections are established on the basis of existing pavement types and cross sections (construction and maintenance history) and the DSM measurements.

Design Dynamic Stiffness Modulus

The design DSM to be assigned to a specific section is the statistical mean (average) of all of the DSM values for the section. It is noted that if an occasional DSM value is very much higher than other values in the group, it may be disregarded.

Remaining Life

The representative DSM value for a particular pavement section permits determination of the ability of that pavement to carry some number of repetitions of an aircraft with a specified gross load.

From both nondestructive and test pit evaluation at specific sites, a correlation has been established between 24,000 passes (4,633 coverages) of the allowable single wheel load (ASWL) and the DSM, shown in Figure A-21. The ASWL, as determined, has a contact area of 254 sq. in. (the same contact area as the 18 in. diameter load plate of the heavy vibrator). Additional factors for different contact areas and coverage levels are shown in Figure A-22. The load factor, F_k, is 0.0437 for the data shown in Figure A-21 for 24,000 passes of the ASWL with a contact area of 254 sq. in.

To determine the allowable gross aircraft load for multiple-wheel aircraft, use is made of the equivalent single wheel (ESWL) concept developed in conjunction with the CBR thickness selection procedure (*67*). The gross aircraft load, P_G, is determined from

$$P_G = \frac{F_k\,(\text{DSM})}{S\,(\%\text{ESWL})}\,\frac{N_m}{N_c}\,100 \qquad \text{(A-7)}$$

where

F_k = load factor, Figure A-22, for specific number of coverages;
S = factor representing proportion of load on main gear of aircraft, usually taken as 0.9;
N_m = number of main gear wheels;
N_c = number of wheels used to determine ESWL; and
%ESWL = ESWL expressed as a percentage and obtained from Figure A-23 for specific aircraft.

To determine the %ESWL from Figure A-23 requires that an equivalent pavement thickness, T_t, be established for the existing pavement. This equivalent pavement consists of an asphalt concrete surface, crushed stone base, and granular subbase. This is done by first converting the thickness of the existing pavement to a total equivalent subbase thickness, T_s, using equivalency factors shown in Table A-3. This total equivalent subbase thickness is then converted to the equivalent pavement with a thickness, T_t, as follows:

$$T_t = 3 \text{ in. AC} + 6 \text{ in. base} + (T_s - 13.5) \text{ in. subbase} \qquad \text{(A-8)}$$

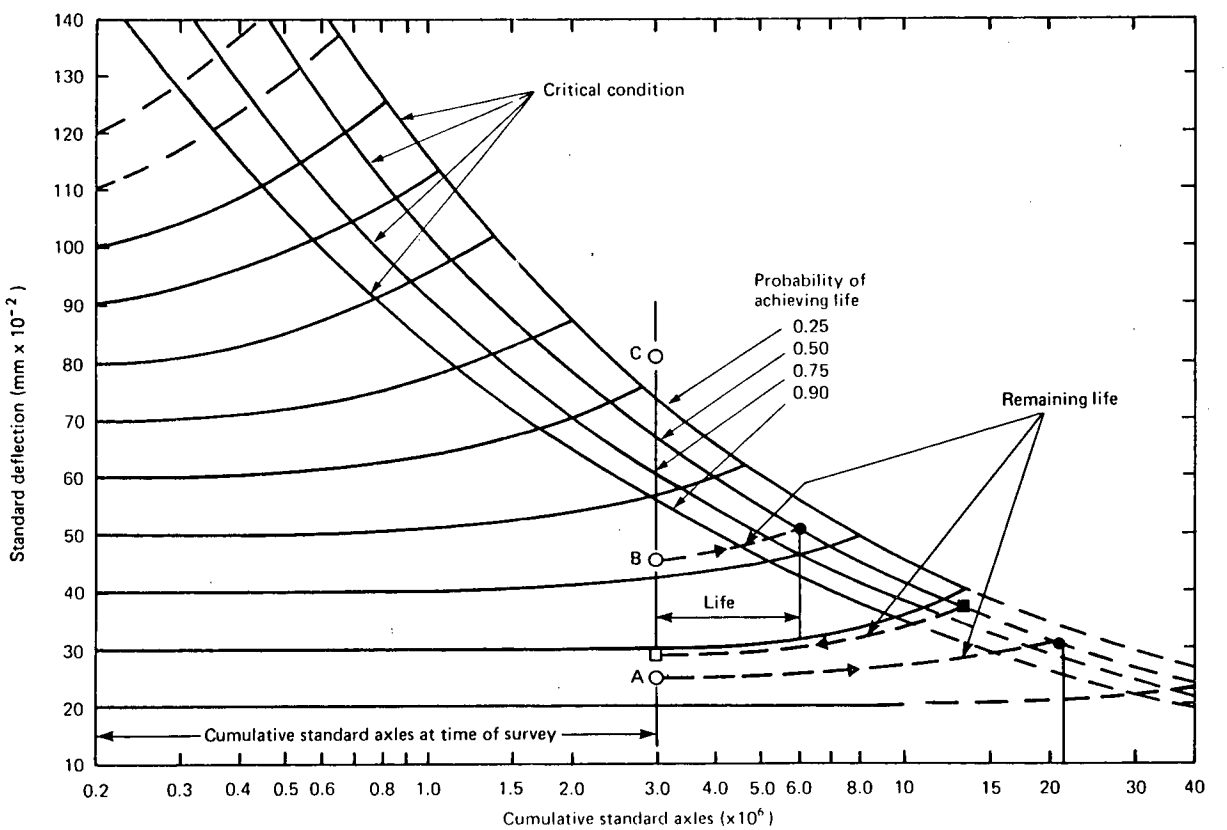


FIGURE A-14 Relation between standard deflection and life for pavements with granular road bases whose aggregates exhibit a natural cementing section—design example (*60*).

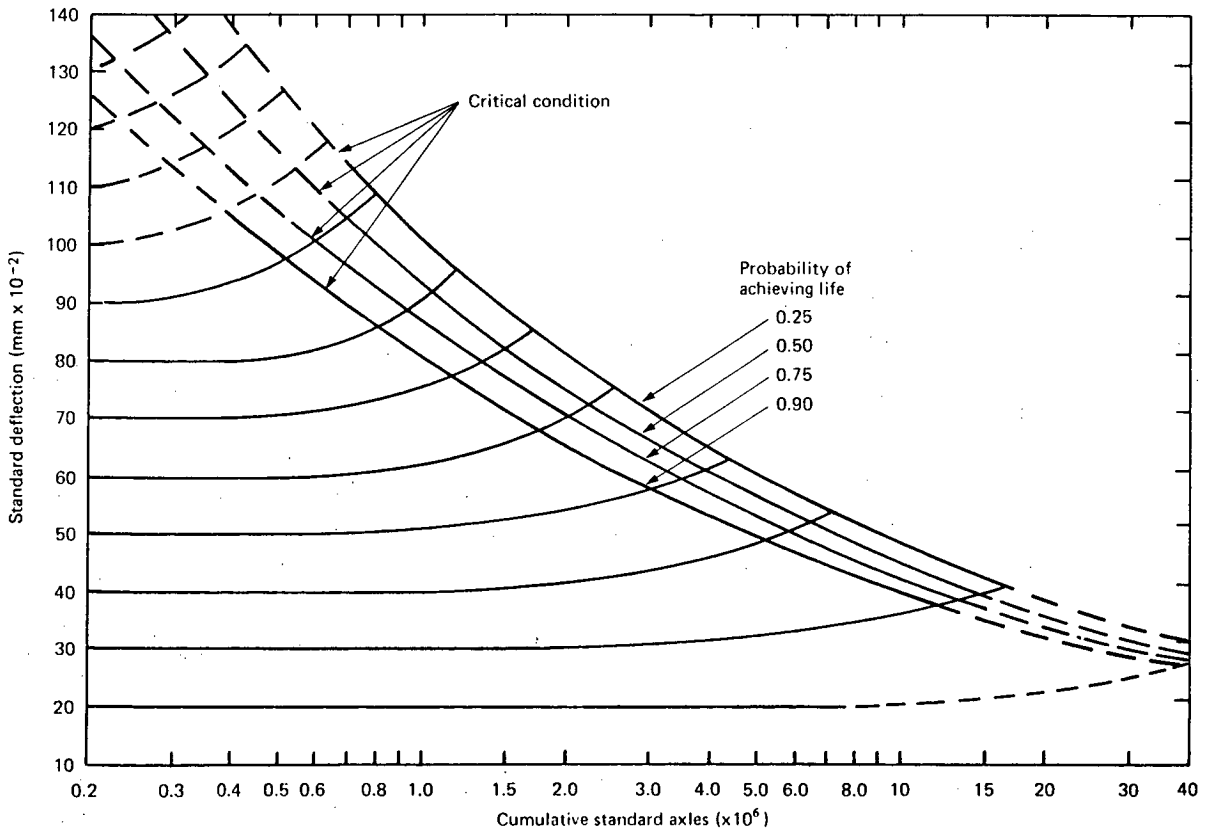


FIGURE A-15 Relation between standard deflection and life for pavements with bituminous road bases (*60*).

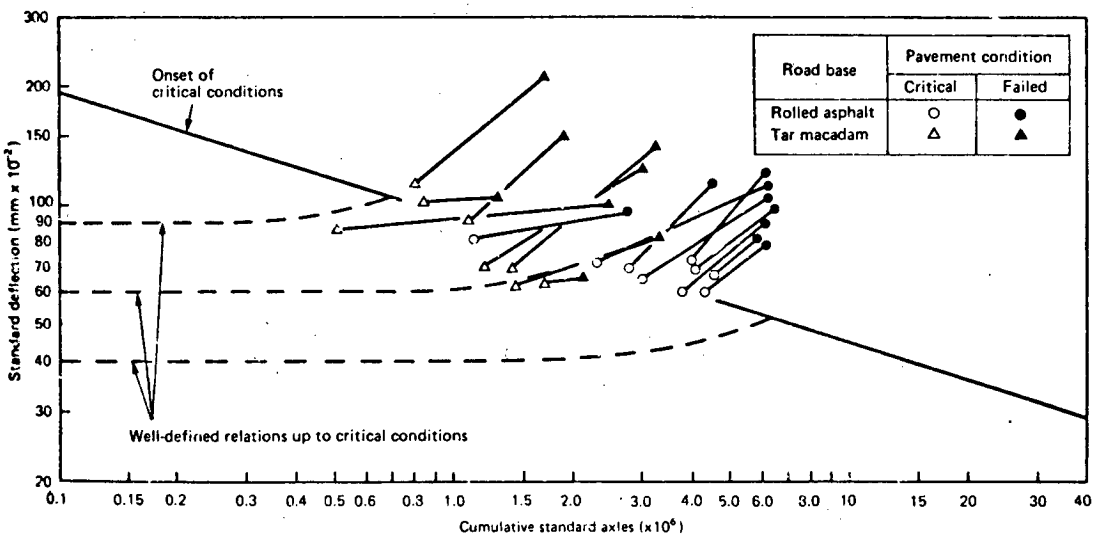


FIGURE A-16 Examples of the relation between critical and failure conditions for pavements with bituminous road bases (*64*).

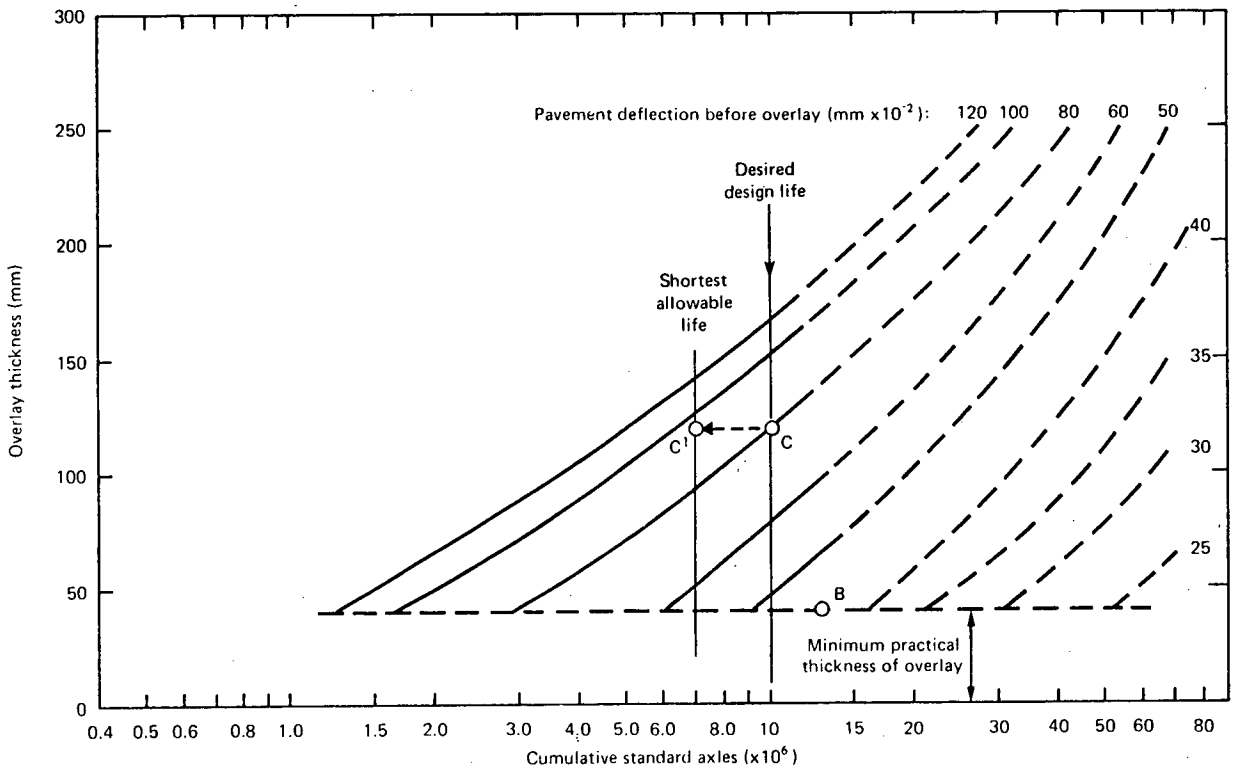


FIGURE A-17 Overlay design chart for pavements with granular road bases whose aggregates have a natural cementing action (0.50 probability) design example (*64*).

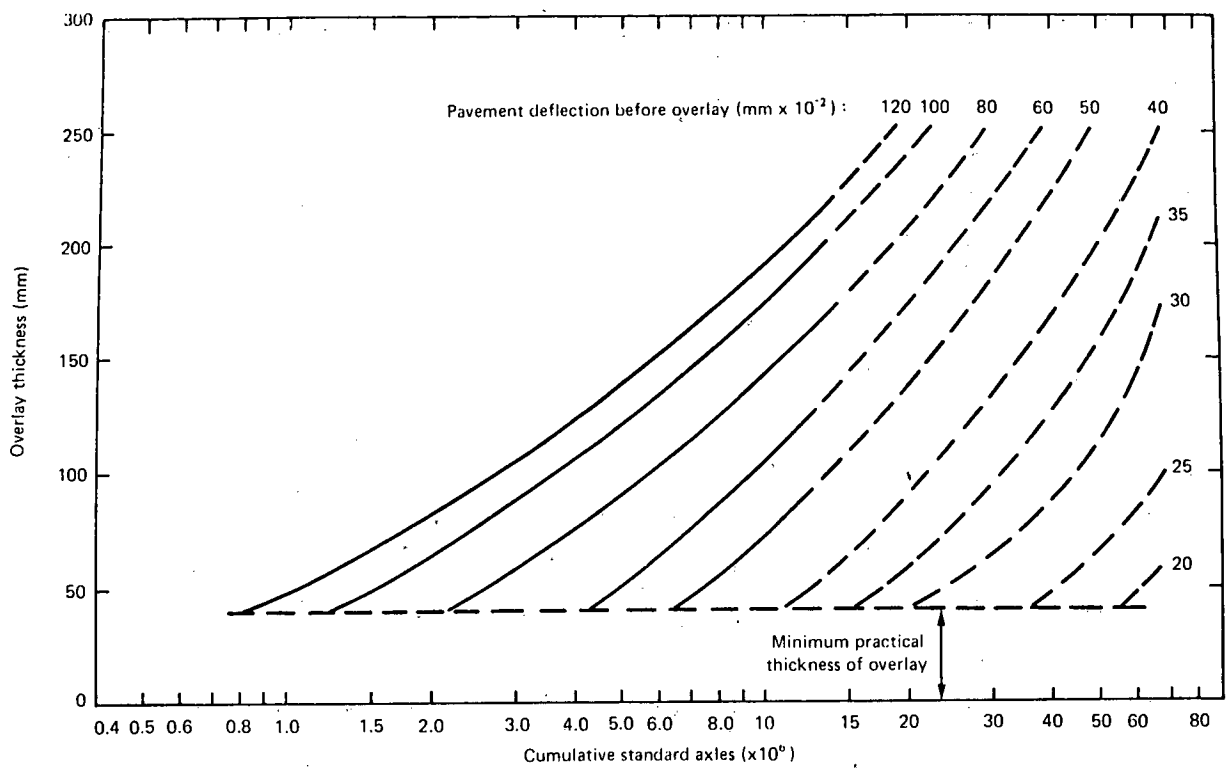


FIGURE A-18 Overlay design chart for pavements with granular road bases whose aggregates have a natural cementing action (0.90 probability) (*64*).

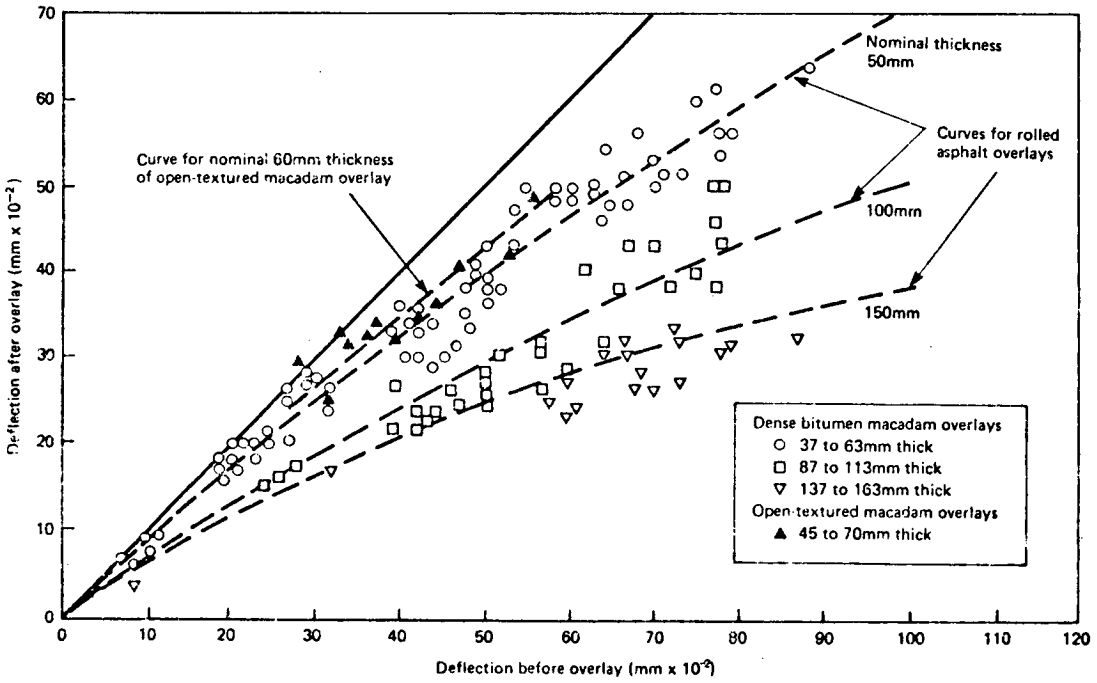


FIGURE A-19 The reduction in deflection achieved by macadam overlays of different thickness and type (*64*).

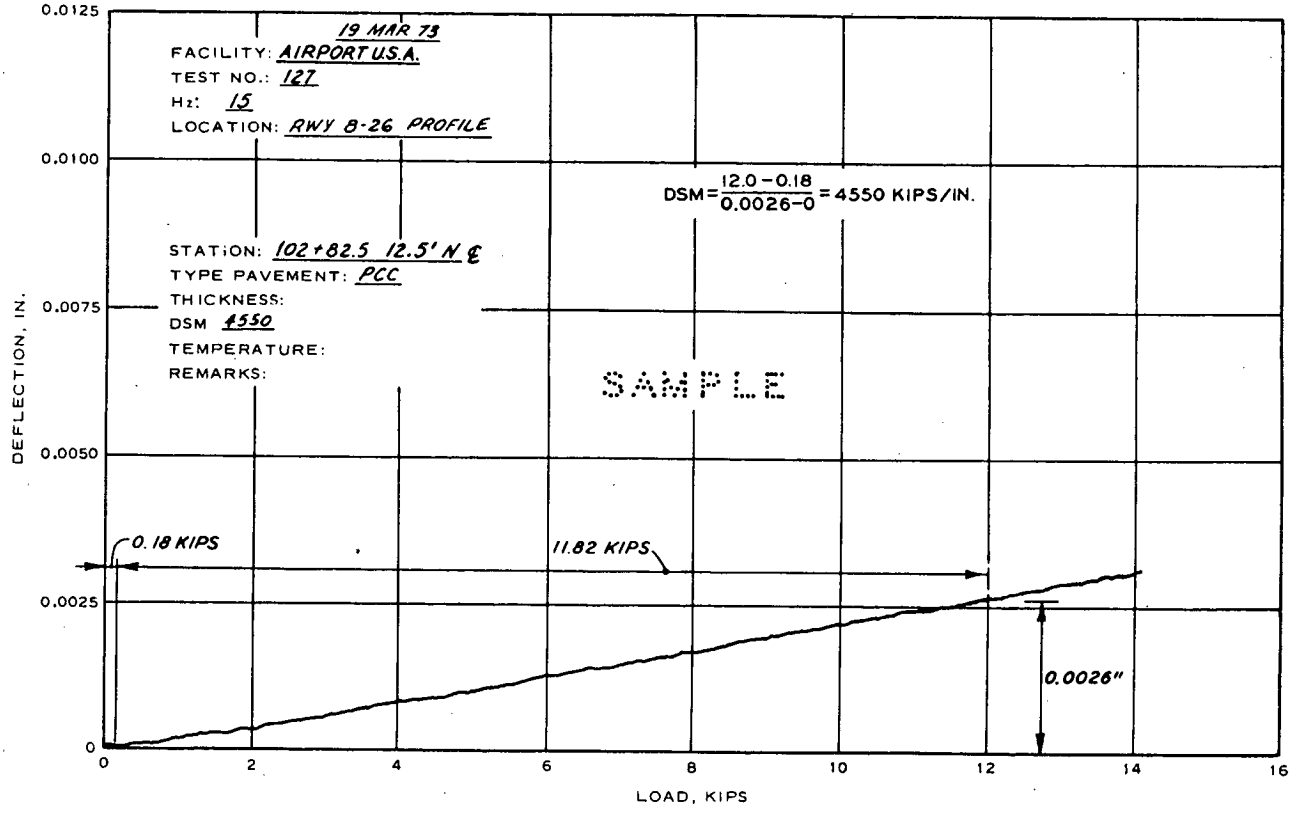


FIGURE A-20 Deflection versus load (*65*).

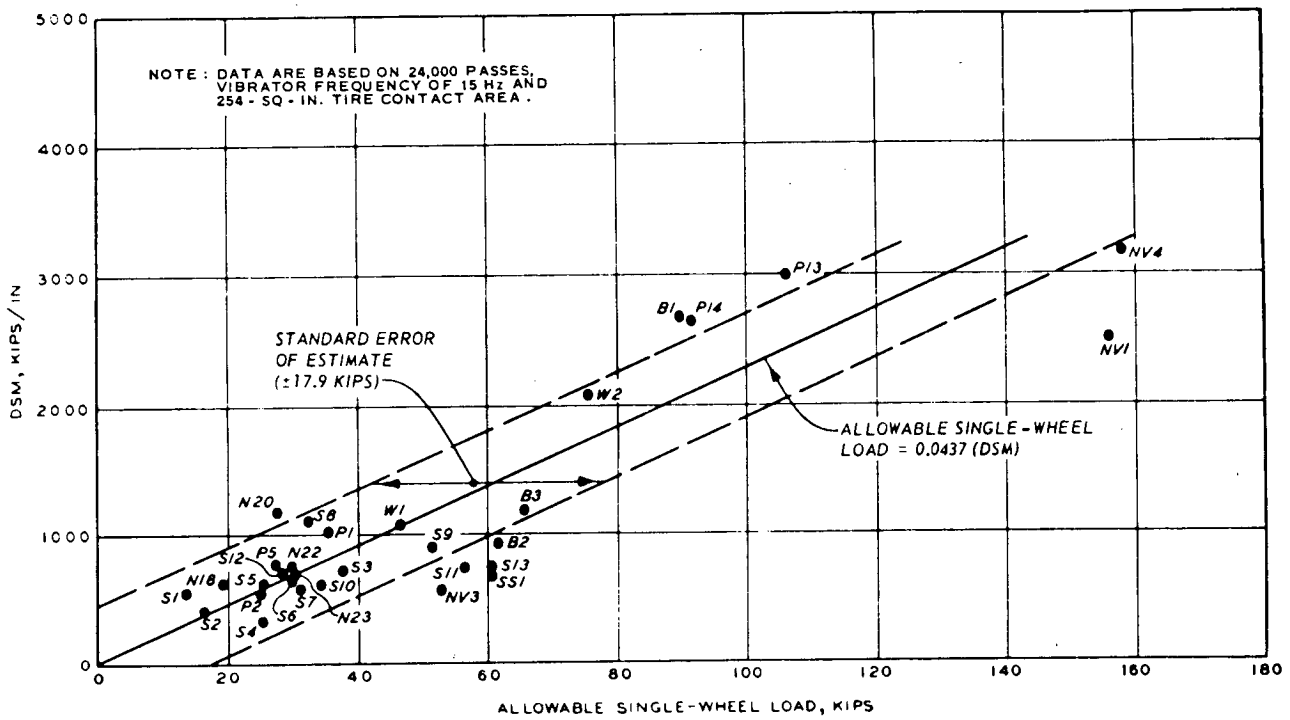


FIGURE A-21 The DSM versus allowable single-wheel load for flexible pavement (*65*).

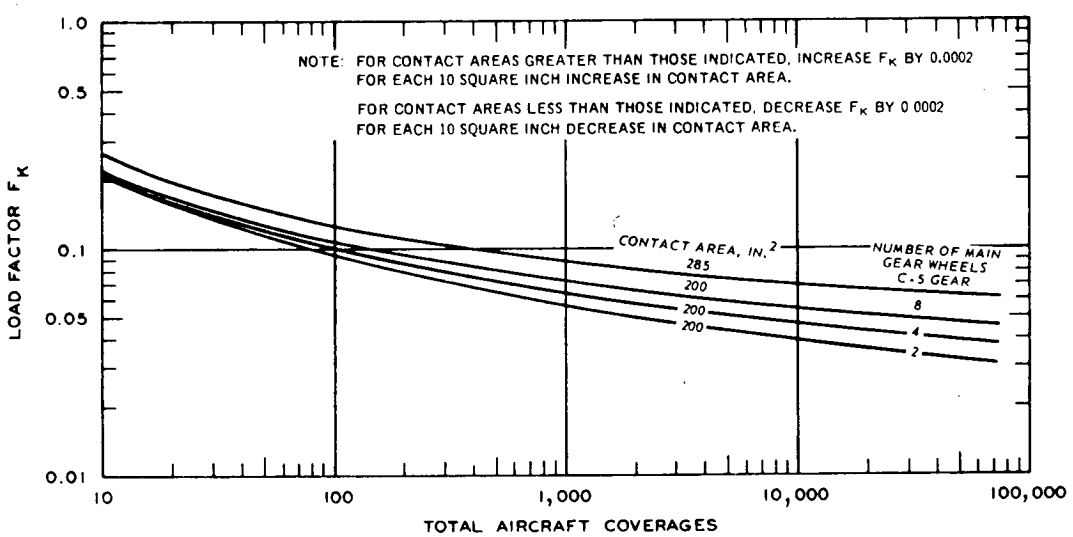
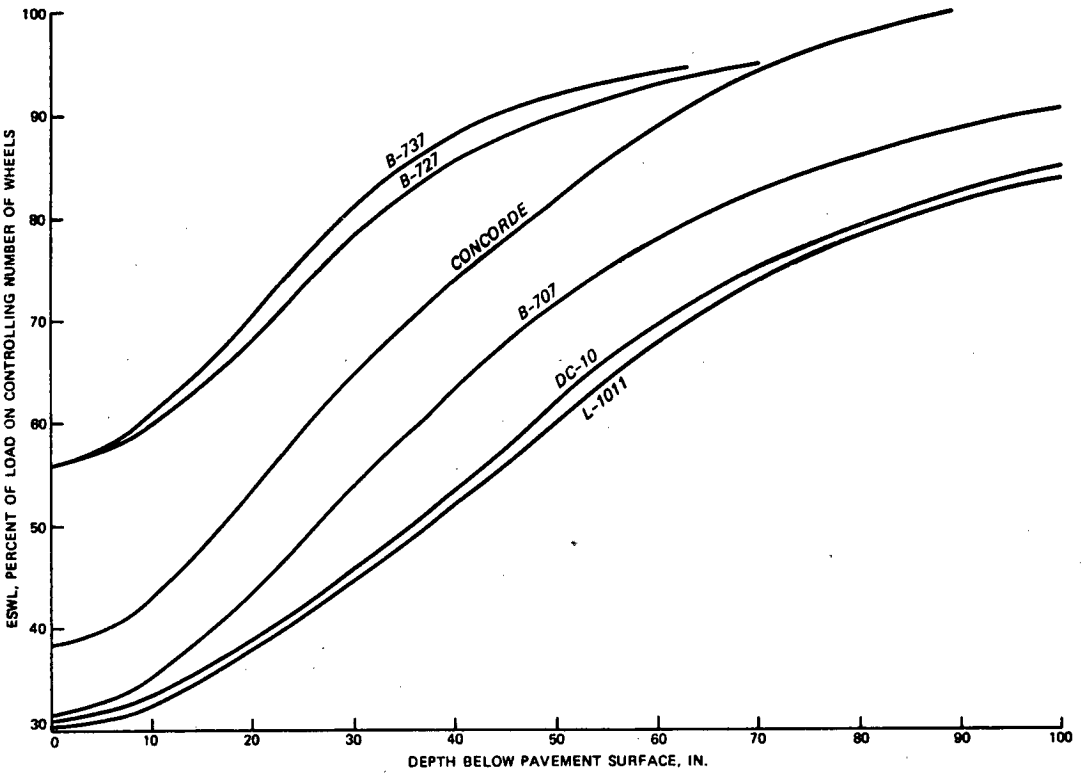


FIGURE A-22 Load factor versus aircraft coverages (*65*).



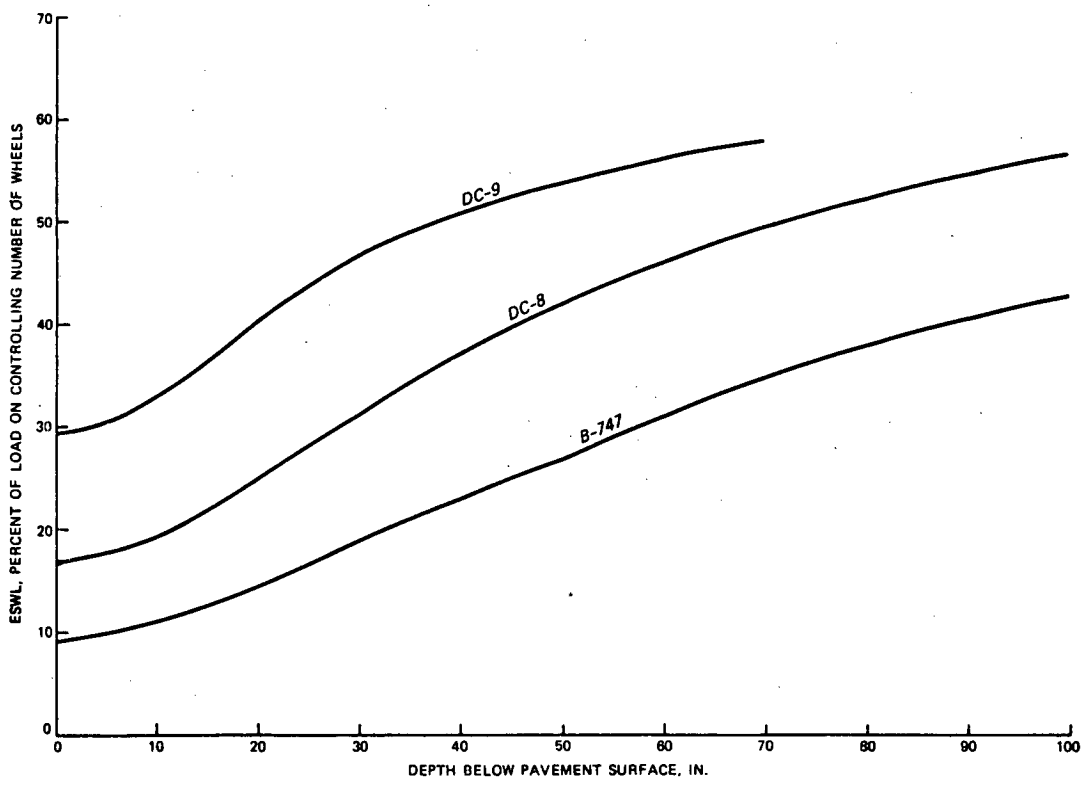


FIGURE A-23 Percent ESWL obtained from equivalent pavement thickness.

TABLE A-3

EQUIVALENCY FACTORS (*65*)

Material	Stabilizing Agent	Surface Course	Base Course	Subbase Course	Subgrade
Asphaltic Concrete	Asphalt	1.70	1.70	1.70	---
Unbound Crushed Stone	---	---	1.40	1.40	---
Sand-Gravel	Cement	---	1.60*	1.60**	---
Clay-Gravel	Cement	---	1.45*	1.45**	---
Fine-Grained Soil	Cement	---	1.25*	1.25**	---
Clay-Sand	Cement	---	1.15*	1.15**	---
Clay-Sand	Fly ash	---	---	1.15**	---
Sand-gravel or Clay-gravel†	Asphalt	---	1.50	1.50	---
Fine-Grained Soil	Lime	---	---	1.10††	1.10‡
Unbound Granular Material	---	---	---	1.00	---

*To use equivalency factor in evaluation, unconfined compressive strength of layer must be 1,000 psi.

**To use equivalency factor in evaluation, unconfined compressive strength of layer must be 700 psi.

†Bituminous.

††To use equivalency factor in evaluation, unconfined compressive strength of layer must be 2,000 psi.

‡To use equivalency factor in evaluation, unconfined compressive strength of layer must be 100 psi.

The 13.5 in. value results from the following conversions:

3 in. AC × 1.70	=	5.1 in equivalent thickness of subbase
6 in. Base × 1.40	=	8.4 in. equivalent thickness of subbase
Total		13.5 in.

It should be noted that in the FAA procedure for civil airport pavements this 3 in. thickness of asphalt concrete has been changed to 4 in. (*66*).

This total pavement thickness is than used to obtain the percent ESWL (e.g., from Fig. A-23) to permit the gross aircraft load to be obtained from Eq. (A-7).

When it is desired to determine the allowable number of coverages for a specified gross load, Eq. (A-7) for gross load is solved for F_k, permitting, in turn, the number of coverages to be selected from Figure A-22.

A similar procedure is used for evaluation of portland cement concrete (pcc) pavements; however, as noted earlier, deflections are also measured at distances of 18 in. and 60 in. from the vibrator centerline (Δ_{18}, Δ_{60}) and the ratio Δ_{60}/Δ_{18} is used estimate the radius of relative stiffness of the pavement.

Overlay Design

Overlay thickness determinations make use of the same thickness design relationships as developed for "new" pavement designs for both asphalt concrete and p.c.c. pavements. Provision is included for asphalt concrete (flexible) overlays on asphalt-surfaced (flexible) pavements, for asphalt concrete (flexible) overlays on p.c.c. (rigid) pavements, and p.c.c. (rigid) overlays on p.c.c. (rigid) pavements. In this summary, only the procedure for an asphalt concrete on an existing asphalt pavement is discussed.

From the DSM value, the allowable gross aircraft load for a single-wheel gear (P_{ASWL}) for 24,000 passes is determined for the existing pavement from:

$$P_{ASWL} = 0.097\ (DSM) \tag{A-9}$$

Using the procedure described above, T_t is calculated for the existing pavement. With T_t and P_{ASWL} the effective subgrade California bearing ratio (CBR) is obtained from Figure A-24. This subgrade CBR is then used with a curve similar to Figure A-24 for the appropriate aircraft and the required total pavement thickness, T_R, is determined. The overlay thickness, h_0, in terms of asphalt concrete is:

$$h_0 = \frac{T_R - T_t}{1.7} \tag{A-10}$$

where the 1.7 factor is the equivalency value obtained from Table A-3 for asphalt concrete.

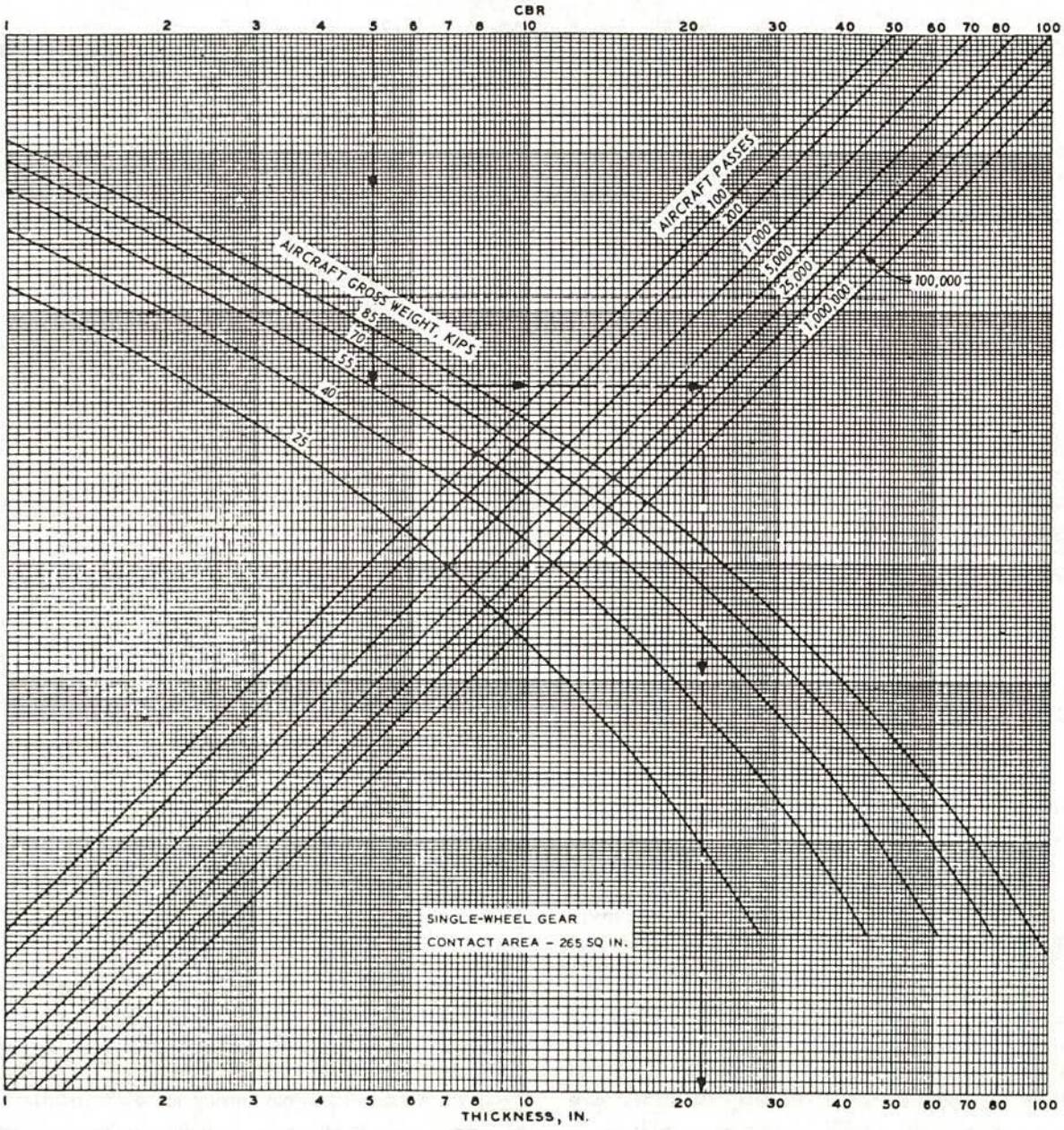
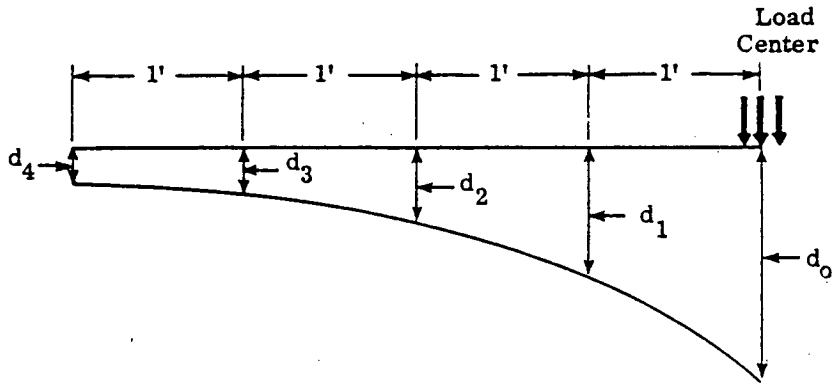


FIGURE A-24 Flexible pavement design curves for army airfields, type B traffic areas (single-wheel gear, contact area—265 sq. in.) (*65*).



$$\text{Spreadability} = \frac{d_o + d_1 + d_2 + d_3 + d_4}{5\,d_o} \times 100$$

FIGURE A-25 Dynaflect deflection basin.

VIRGINIA HIGHWAY AND TRANSPORTATION RESEARCH COUNCIL

The Virginia procedure (*68*) proposes to combine deflection measurements with a deflected shape factor to estimate the strength of the in-service pavement. The determination of the overlay requirement uses the component analysis procedure as discussed in Chapter 2.

Deflection and Spreadability

The Virginia procedure relies on nondestructive testing with the Dynaflect equipment making measurements at five locations as shown in Figure A-25.

The deflection value is measured directly under the load (d_0) as shown Figure A-25. The spreadability is the average deflection of the five sensors divided by the deflection under the load. Increasing values of spreadability are indicative of stiffer pavements with increasing load distribution characteristics.

Overlay Design Procedure

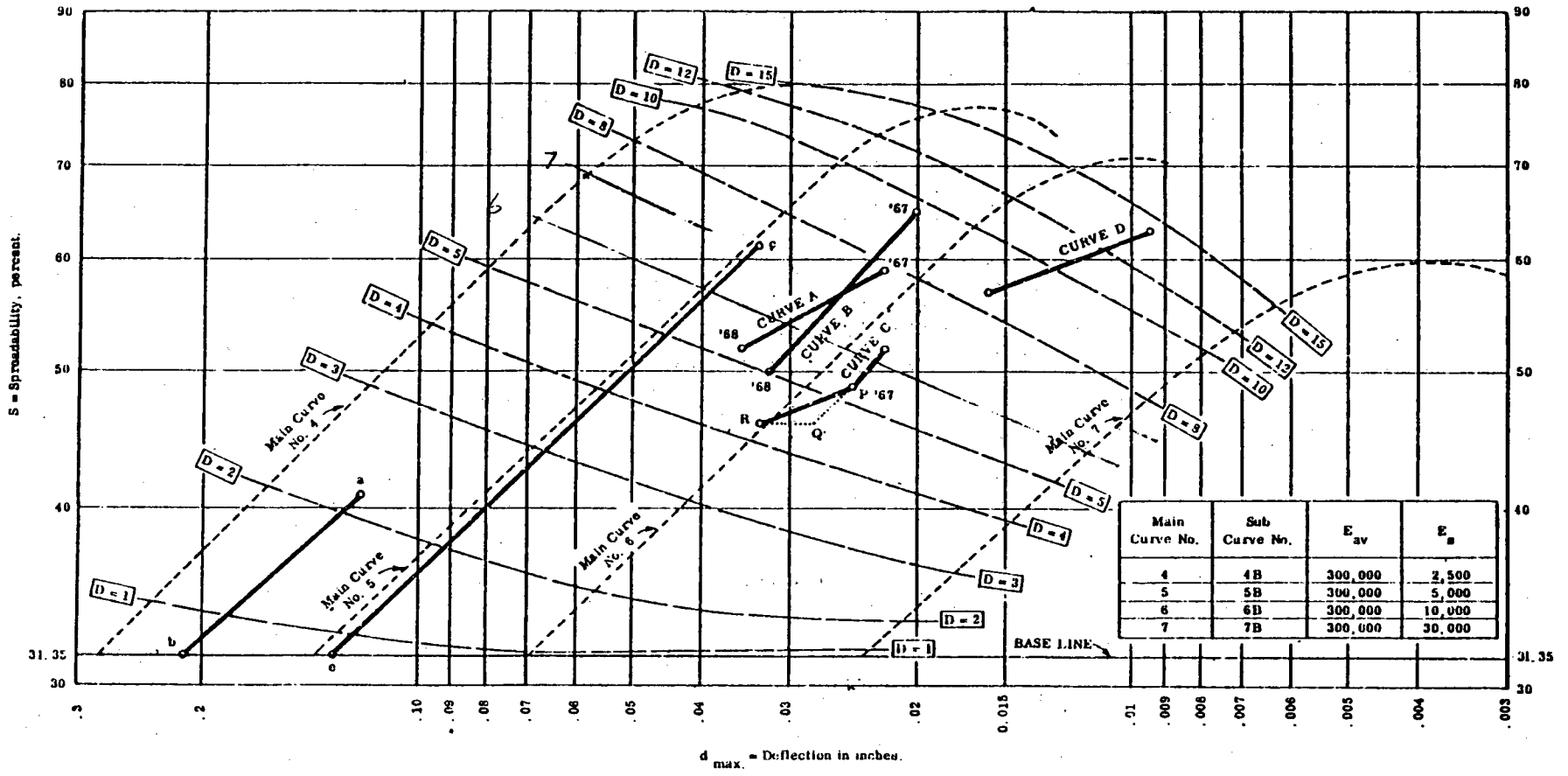
For purposes of rehabilitation, the objective usually concerns the thickness of asphalt concrete required to bring a pavement up to the required thickness index used by Virginia to design new pavements. Thus, the Virginia Council approach is to determine the thickness index (i.e., equivalent thickness of asphalt concrete) of the in-place pavement and to compare this assessment with the required thickness index of a new pavement.

The thickness index of a new pavement is a function of the soil support value, as defined in reference 68, and daily 18 kip equivalent loads.

Based on field correlations, the structural capacity (thickness index) of a pavement is related to deflection and spreadability (Fig. A-26). For example, a pavement with 0.030 in. deflection, using correlations to the Benkelman beam, and a spreadability of 50 will have a thickness index of approximately 5. If the deflection was 0.015 in., the thickness index would be 7.

To illustrate the procedure, it can be assumed that the thickness index of the existing pavement is evaluated at 7 and the required index is 9. The difference would be a 2-in. (5-cm) asphalt concrete overlay.

The method proposed by Virginia is based on extensive studies, both empirical and theoretical, by the Virginia Highway and Transportation Research Council. Agencies considering the use of this method should repeat, as a minimum, the empirical correlations developed by Virginia.



Main Curve No.	Sub Curve No.	E_{av}	E_s
4	4B	300,000	2,500
5	5B	300,000	5,000
6	6D	300,000	10,000
7	7B	300,000	30,000

FIGURE A-26 Example of evaluation of satellite projects in Virginia by means of general evaluation chart.

APPENDIX B

ANALYTICALLY BASED PROCEDURES

THE SHELL RESEARCH PROCEDURE

This method is an analytically based procedure and follows the format of Figure 6. It is applicable to both highway and airfield pavements (*69–71*). Structural response is measured by the falling weight deflectometer (FWD). Overlay thicknesses are selected to minimize excessive permanent deformation (rutting) in the pavement structure and fatigue cracking in the asphalt-bound layer. For highway pavements, design charts developed by the Shell researchers (*72*) can be used to assist in selecting overlay thicknesses.

Nondestructive Testing

Deflections, obtained with the FWD, are used to define the response of the pavement structure to load. For highway pavements, the peak force of the FWD can be varied in the range 15 to 48 kN (3.4 to 10.8 kip). A heavier device has been developed for airfield pavements with a full range of 40 to 125 kN (9.0 to 28.0 kip). Both FWD units exert a fixed time of loading, with a pulse width of 28 ms. Pavement response is defined by a maximum deflection, δ_0, and a measure of the shape of the deflected surface, Q_r—defined as the ratio of the deflection measured at a distance r from the center of the load application, δ_r, to the deflection measured directly under the load, δ_0. The distance r can be fixed, depending on the type of structure, and is preferably such that Q_r is about 0.5. For airfield pavements, particularly, at least two deflections are measured away from the load and two values of Q determined.

Velocity transducers (geophones) are used to measure the deflections, which are determined at prescribed intervals sufficient to provide a reasonable indication of response. For highway pavements, for example, the interval might be of the order of 50 m (160 ft) along the length between wheel tracks. It should also be noted that a check can be made by measuring deflections in the wheel tracks as well. If these values are significantly larger than those obtained outside the wheel tracks, the pavement is reaching its service life. Figure B-1 illustrates a typical pattern for deflection measurements recommended for a runway.

Pavement temperatures are also obtained to permit estimation of the stiffness of the asphalt-bound layer at the time of measurement.

Analysis Sections

Analysis sections are established from plots like those shown in Figure B-2 and using construction and maintenance data. In

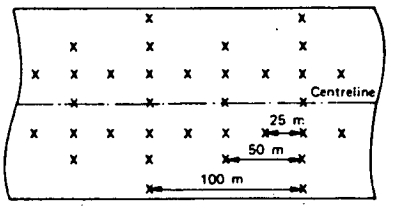


FIGURE B-1 Typical pattern of measurements of a runway.

Figure B-2, which is a highway pavement example, the plotted points for both δ_0 and Q_r are the results obtained from three-point moving averages of the measured data. Generally, the 85th percentile value for δ_0 and the 15th percentile value for Q_r are selected to be representative for each analysis section.

Materials Characterization

In this methodology the pavement is represented as a multilayer elastic structure in which the stiffness characteristics of each of the layers is characterized by a modulus, E, and a Poisson's ratio, ν.

From the representative measured parameters, δ and Q, certain characteristics of the existing pavement are usually computed using the BISAR computer program (*69*). For three-layer pavements consisting of asphalt concrete, untreated granular base and subbase, and untreated subgrade, the modulus of the asphalt-bound layer, E_1, is estimated using a nomographic procedure (*69*) and the thickness of the granular layer, h_2, either is estimated from construction reports or determined by coring in the field. With these parameters, the subgrade modulus, E_3,

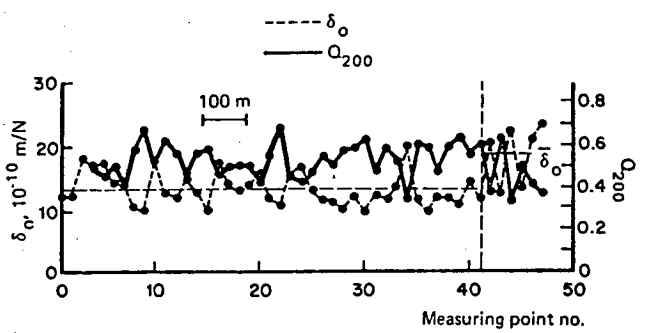


FIGURE B-2 Graphical representation of δ_0 and Q_r.

and the effective thickness[1] of the asphalt concrete layer, h_1, are determined by an iterative process in which the values of the parameters are adjusted until the measured and computed values for δ and/or Q are in reasonable agreement. (In the Shell method: $E_2 = kE_3$ where k is a function of h_2. Thus, it is not an independent variable in the computation process.)

For three-layered pavements using granular base (and subbase—all treated as one layer), the Shell researchers have developed relationships between E_1, δ_0, Q_r, and h_1 for predetermined values of E_2, h_2, E_3, and *a* (radius of loaded area representative of the plate of the FWD); e.g., Figure B-3. With such charts, having measured δ_0 and Q_r, two unknown parameters, E_3 and h_1, can be estimated if the other variables are known and/or can be estimated. Because of the predetermined dependency of E_2 on E_3, interpretation charts need only be prepared for one E_3 value, since structural pavement properties for the subgrade moduli can be derived by a simple shifting procedure (*69*).

Distress Determinants

In this method, excessive permanent deformation is controlled by limiting the vertical compressive strain at the subgrade surface to a value defined as follows:

$$(\epsilon_v)_3 = 2.8 \times 10^{-2} \times N^{-0.25} \qquad \text{(B-1)}$$

where N = number of load aplications.

For asphalt-bound layers, fatigue cracking is controlled by limiting the tensile strain (ϵ_t) in this layer. Specific limiting values have been established for two mix types (*69*) that are representative of a wide range of materials used in practice. The general form of these relationships is:

$$N = A\left(\frac{1}{\epsilon_t}\right)^a \left(\frac{1}{E_1}\right)^b \qquad \text{(B-2)}$$

where A, a, b = representative mixture parameters.

Remaining Life Estimation and Overlay Design

With the various layer moduli (and Poisson's ratios, which are assumed to be 0.35 for all layers), layer thicknesses, design values for δ_0 and Q_r, and a knowledge of the applied traffic, the design life of the existing section can be estimated. If this is inadequate to accommodate the anticipated traffic, an overlay thickness can be ascertained. Actually, three separate thickness determinations are made: (a) a thickness to satisfy the subgrade strain criterion; (b) a thickness to satisfy the asphalt concrete fatigue strain criterion; and (c) a thickness based on the assumption that the existing pavement is deteriorated to such an extent that it is regarded as an unbound granular layer (this thickness provides an upper bound to the required overlay thickness).

[1] This effective thickness is presumed to take into account differences between the actual and assumed asphalt concrete stiffness modulus, the presence of cracks, as well as other variables, and appears more suitable for design purposes than a derived asphalt concrete stiffness.

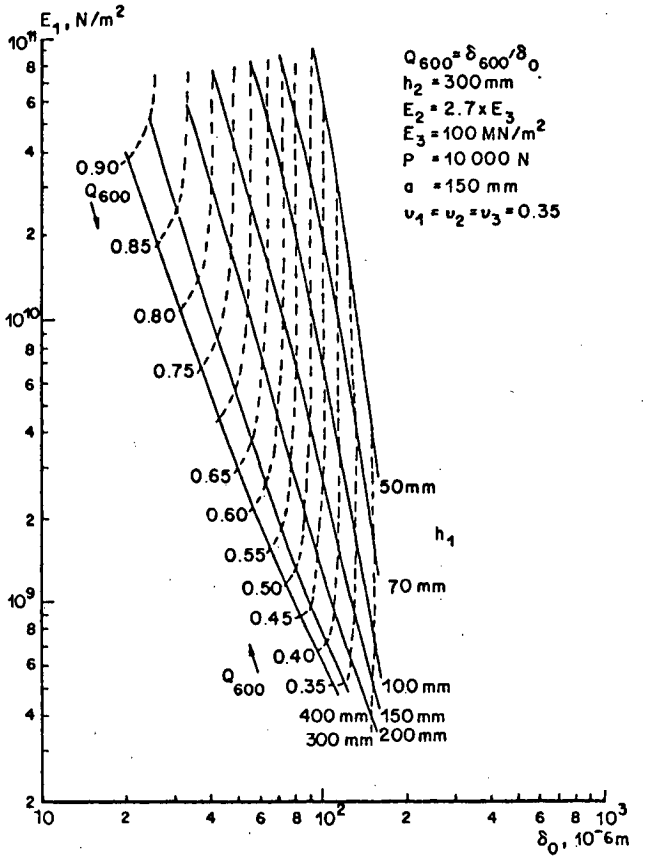


FIGURE B-3 Deflection interpretation chart.

To illustrate the method, consider the following example for highway loading conditions, the results of which are illustrated in Figure B-4 (a, b, and c). From cores the mix was identified according to the code that the Shell researchers have established (in this example, S1-F2-100) and the granular thickness, h_2, ascertained to be 200 mm (7.9 in.) in thickness. With these data, the field measurements, and the procedure described earlier the subgrade modulus, E_3, was determined to be 60 MPa (8.7 ksi) and the effective thickness of the asphalt bound layer, $h_{i_{eff}}$, was 250 mm (9.8 in.). The weighted mean annual air temperature for the site was established as 18°C (64°F). The Shell design charts (*72*) are shown in Figure B-4 (a–c). Figure B-4a is based on subgrade stain criteria, Figure B-4b is based on tensile strain criteria; and Figure B-4c is based on the assumption that the existing pavement was cracked (h_2 = 200 + 250 = 450 mm) (7.9 + 9.8 = 17.7 in.).

Plotting the data determined from the field measurements, Point A (E_3 = 60 MPa, h_1 = 250 mm) Figure B-4a, the original design life based on the subgrade strain is 18,000,000 80-kN (18-kip) axles. From traffic estimates the actual number of 80 kN axles is estimated to be 15,000,000. The residual life is thus 3,000,000 standard axles. If the pavement structure must accommodate an additional 30,000,000 standard axles (N_{A2} = 30,000,000), an overlay will be required. Point B of Figure B-4a indicates that a total thickness for h_1 of 290 mm (11.4 in.) will accommodate the applied traffic. Thus, to satisfy the subgrade strain criterion, an overlay of 40 mm (290-250) [1.6 in. (11.4-9.8)] is sufficient.

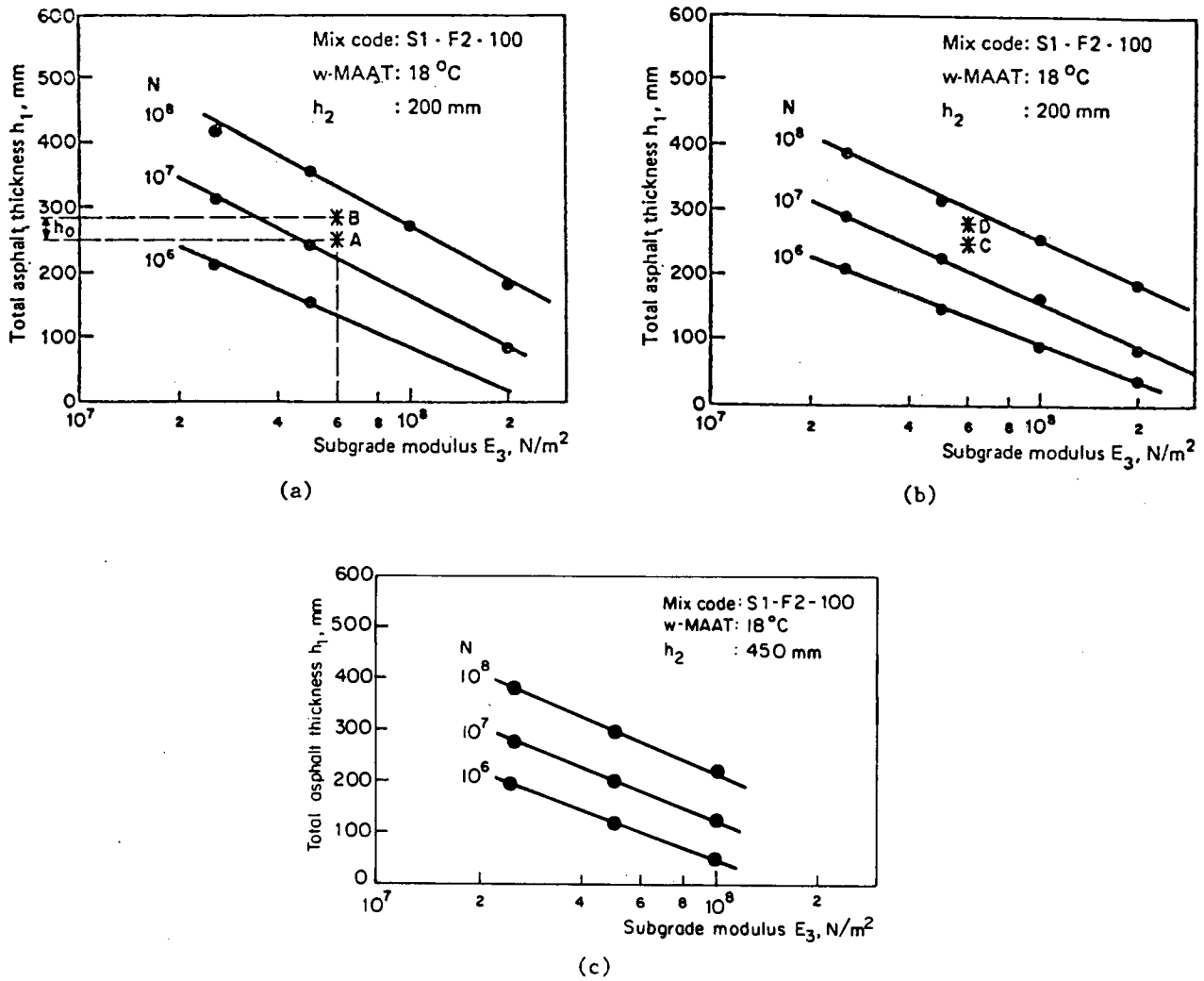


FIGURE B-4 Interpolated design charts: (a) based on subgrade strain criterion, (b) based on asphalt strain criterion, and (c) based on the assumption that the existing asphalt concrete behaves as granular base (*70*).

Considering the tensile strain criterion, Point C of Figure B-4B represents the initial conditions indicating a design life of 30,000,000 standard axles. The remaining life, 15,000,000 standard axles, is not sufficient to accommodate the additional traffic. An overlay thickness to satisfy the asphalt concrete fatigue strain criterion must take into account the "life" used by the pavement to date. This is done by adjusting the additional traffic as follows, recognizing the fact that the overlay will become a part of the existing structure and that the maximum tensile strain will occur on the underside of the existing pavement even though its magnitude will be less. This adjusted value (termed a design number N_{D2}) is determined from

$$N_{D2} = ND_1 \, N_{A2}/(N_{D1} - N_{A1}) \qquad \text{(B-3)}$$

where N_{A2} = number of additional standard axles anticipated.

Using this expression, N_{D2} is determined as follows:

$$N_{D2} = \frac{(30 \times 10^6)\,(30 \times 10^6)}{(30 \times 10^6) - (15 \times 10^6)} = 60 \times 10^6 \qquad \text{(B-4)}$$

The asphalt thickness required for 60,000,000 standard axles is shown by Point D in the chart and corresponds to h_1 = 280 mm (11.0 in.). Thus, the overlay to satisfy the tensile strain criterion is 30 mm (280-250) [1.2 in. (11.0-9.8)], which is less than that required to mitigate rutting. If the existing pavement were allowed to crack, the pavement thickness, h, required to accommodate the added traffic would be in excess of 200 mm (7.7 in.) as seen in Figure B-4c. Thus, the design for this example would be an overlay of 40 mm (1.6 in.), based on the rutting considerations.

The above example, developed from the Shell design charts, is representative of the procedure for highway loading conditions and for pavements with granular bases or full depth asphalt concrete. For airfield pavements, the compuer program BISAR would be used to make thickness determinations similar to those illustrated in Figure B-4 (a, b, and c).

Recent correspondence with R. Koole has indicated some differences with the information reported in this section.

1. Both falling weight deflectometers have been adapted to cover the load range 30 to 125 kN (7 to 28 kip) and a new device is available for loads up to 140 kN(31 kip). Four to seven geophones are used to define the deflection basin.
2. The majority of deflection measurements are made in the wheel tracks on both highway and airfield pavements.
3. The computer program BISAR is used almost exclusively for modulus, remaining life, and overlay thickness determinations; the approach is essentially the same for both highway and airfield pavements, the difference being in the nature of the loading associated with each of these types of pavements.

FEDERAL HIGHWAY ADMINISTRATION—ARE PROCEDURE

This method was developed for the Federal Highway Administration by Austin Research Engineers (ARE) as part of a comprehensive overlay design methodology for both asphalt concrete and portland cement concrete pavements (*34, 40*). For asphalt pavements, the portion of the procedure to be discussed herein, both fatigue cracking in the asphalt bound layer and rutting at the pavement surface are considered. The procedure follows closely the format of Figure 6.

Nondestructive Testing

Deflections are used to define the structural response of the pavement. Any type of deflection equipment that provides satisfactory deflection results can be used. Guidelines for deflection measurements are provided. For example, depending on terrain, the following spacings are suggested:

Type of Location	Spacing
Rolling terrain	100 ft
Numerous cut-to-fill transitions	100 ft
Level with uniform grading	250 ft

Measurements are recommended to be made in the outer wheel paths. For two-lane, two-way facilities, for example, it is recommended that deflection measurements be made at staggered intervals (e.g., if the measurements are made at 100-ft intervals, one set of measurements will be staggered 50 ft with respect to the other).

Condition Surveys

Condition surveys are recommended to be conducted at the same time that the deflection measurements are being made. Type, extent, and severity of cracking are recorded and cracking is defined according to the AASHO Road Test definition (*34*). Rutting is measured and it is recommended that rut depths be measured at 500-ft intervals in both wheel paths. In addition, general information, such as changes in soil type, drainage conditions, cut/fill transitions, etc., should be noted.

Analysis Sections

Deflection data are plotted in the form of profiles throughout the length of the roadway (e.g., Fig. B-5). This information together with construction details and condition survey data permit the establishment of analysis sections. Adjacent sections are compared using statistical procedures[2] to ensure that the sections are different. The design deflection for each section is computed from:

$$\delta_d = \bar{\delta} + Z \cdot S \qquad \text{(B-4)}$$

where Z = deviation from mean to selected significance level on a normal distribution curve. For a design confidence level of 90 percent, Z = 1.282.

Materials Characterization

The pavement structure is represented as a multilayer elastic system necessitating that a modulus (stiffness) value and Poisson's ratio be defined for each layer. It is recommended that at least one representative sample for laboratory testing be obtained for each of the pavement components from each analysis section.

For the asphalt-bound layer(s), stiffness moduli should be defined over a range in temperatures at a time of loading corresponding to moving traffic. For design purposes the stiffness at a temperature of 70°F is recommended for use. Poisson's ratio for the asphalt concrete is assumed to be 0.3.

If the asphalt concrete layer is cracked, it is assigned a modulus of 70,000 psi if the cracking is defined as class 2 (AASHO Road Test definition); the modulus should be equal to the base modulus or 20,000 psi, whichever is greater, if the cracking is defined as class 3.

Treated base course materials can be tested in the same way as the asphalt concrete. Unbound base course materials, on the other hand, should be tested at representative conditions of water content and dry density in repeated loading and a resilient modulus defined. AASHTO test procedures can be followed to obtain the resilient modulus.

Subgrade specimens should be tested over a range in deviator stresses to permit defining a relationship of the form:

$$M_R = A\sigma_d^b \qquad \text{(B-5)}$$

[2] A computer program incorporating the Student's t test, termed TVAL, has been developed to make these comparisons. The five percent level has been suggested as the level of significance at which to check the difference.

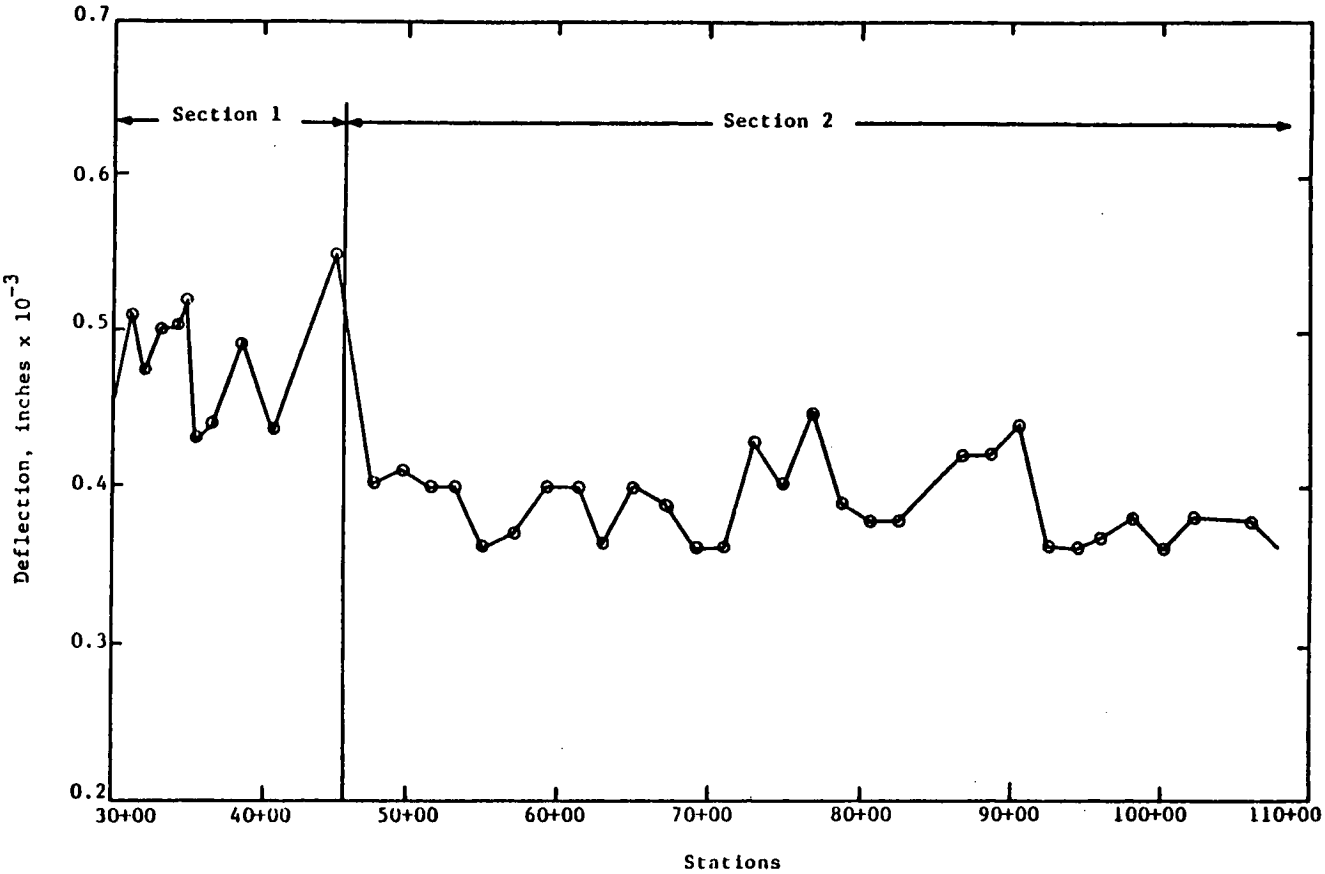


FIGURE B-5 Sample deflection profile.

where

M_R = resilient modulus,
σ_d = applied deviator stress, and
A,b = laboratory determined coefficients.

To select an appropriate subgrade modulus for analysis, it is necessary to use the design deflection selected for the particular analysis section and by an iterative procedure select a modulus value from Eq. (B-5) that provides the same value for the computed as the measured deflection. As seen in Figure B-6, some adjustments in the laboratory data may be required to achieve compatability.

Distress Determinants

Fatigue in the asphalt-bound layer is controlled by the magnitude of the tensile strain and defined by Eq. (B-6) developed from an analysis of AASHO Road Test data:

$$N = 9.73 \times 10^{-15} \left(\frac{1}{\epsilon_t}\right)^{5.16} \qquad \text{(B-6)}$$

where

N = allowable number of 18-kip single-axle load applications and
ϵ_t = horizontal tensile strain on the underside of asphalt-bound layer being analyzed.

Rutting is controlled by an equation developed from regression analyses of the general form:

$$\log N = f\left(R, \frac{\epsilon_{z1}}{\sigma_{z1}}, \sigma_{z2}, \sigma_{x2}, \sigma_{z3}, \epsilon_{z4}, \sigma_{z5}, \epsilon_{z5}, d_T\right) \qquad \text{(B-7)}$$

where

R = allowable rut depth;
ϵ_{z1}, ϵ_{z4} = vertical strain at bottom of layer 1 and 4, respectively;
σ_{z1}, σ_{z2}, σ_{z3} = vertical stress at bottom layer 1, 2, and 3, respectively;
ϵ_{z5}, σ_{z5} = vertical strain and stress at top of layer 5;
σ_{x2} = horizontal stress parallel to load axle in bottom of second layer; and
d_T = number of days per year when average daily temperature is equal to or greater than 64°F (five-year average).

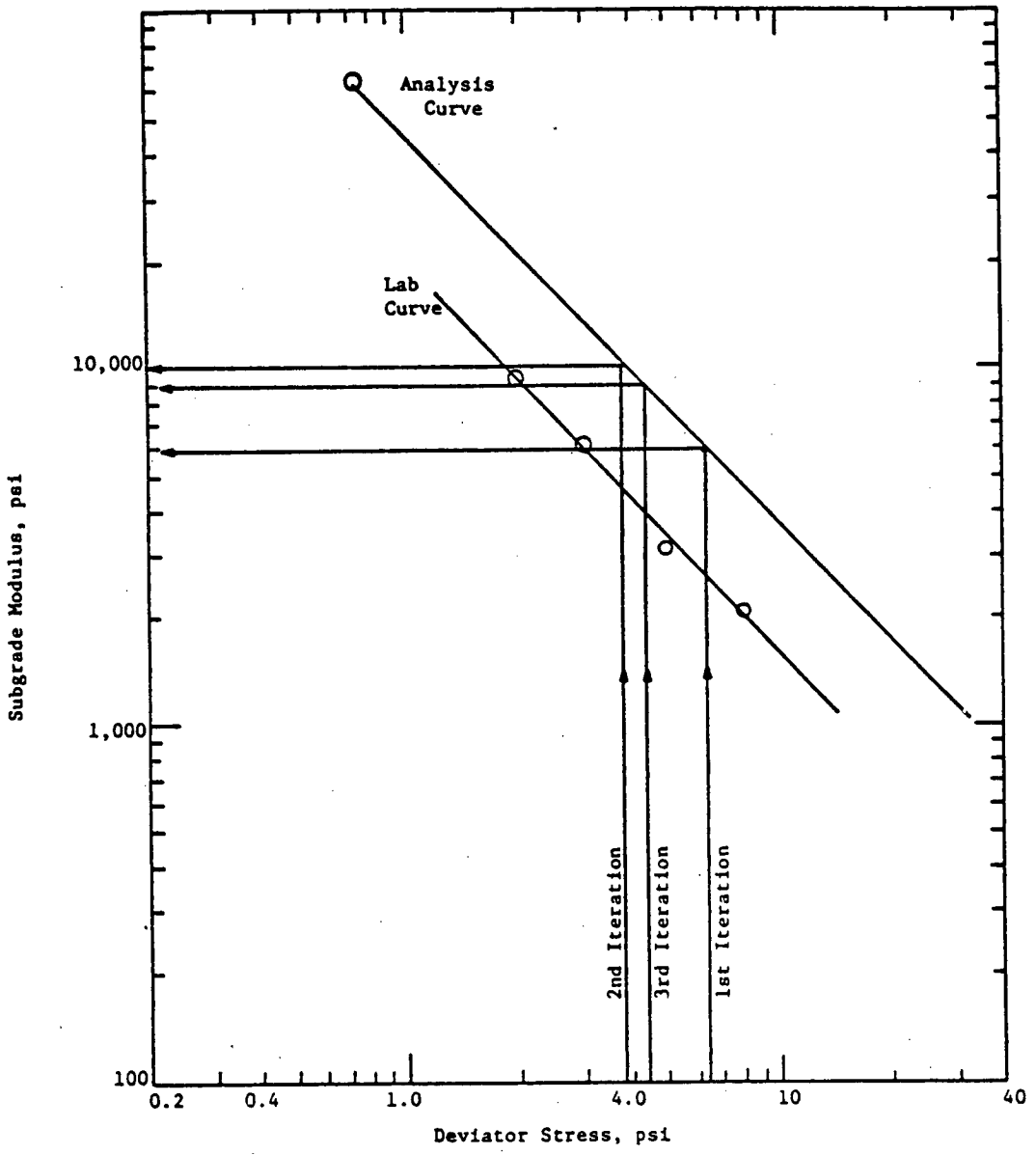


FIGURE B-6 Relationship of resilient modulus and repeated deviator stress.

Remaining Life Estimation and Overlay Design

If the pavement is in the uncracked condition, remaining life can be determined. This requires estimation of the amount of traffic, in terms of 18-kip single-axle loads applied to date and the tensile strain(s) on the underside of the asphalt-bound layer.

Mixed traffic is converted to repetitions of the 18-kip single-axle using AASHTO equivalency factors. In addition, the total traffic is modified by the Regional Factor according to the following:

$$n_D = (18\text{kip EAL}) \cdot (RF) \qquad \text{(B-8)}$$

where

n_D = design traffic volume and
RF = Regional Factor.

With the strain (or strains) determined from an analysis of the system using the ELSYM computer program and the modulus values and Poisson's ratios for the various layers, the total number of load repetitions, N, can be estimated from Eq. (B-6). The remaining life is then calculated using the linear numeration of cycle ratios cumulative damage hypothesis, i.e.,

$$\frac{n_r}{N_1} = 1 - \frac{n_D}{N_1} \qquad \text{(B-9)}$$

where n_r = number of additional 18-kip single-axle loads that can be carried at the computed strain level, if the value for n_r is less than the additional traffic to be carried, an overlay will be required.

Various thicknesses of overlay can be calculated to accommodate both the fatigue [Eq. (B-6)] and rutting [Eq. (B-7)] criteria.

To satisfy the fatigue requirement, the tensile strain is determined on the underside of existing pavement considering the thickness of this layer as the sum of the existing pavement thickness plus that of the overlay. The actual number of repetitions can be computed from Eq. (B-6) modified by the remaining life.

$$N_2 = 9.73 \times 10^{-15} \left(\frac{1}{\epsilon_t}\right)^{5.16} \frac{n_r}{N_1} \qquad \text{(B-10)}$$

where

ϵ_t = strain normal to direction of traffic and
N_2 = allowable number of 18 kip repetitions for the overlaid pavement.

Similarly, use of Eq. (B-7) permits estimation of the allowable amount of traffic associated with various overlay thicknesses to satisfy rutting considerations.

The resulting computations can be plotted as shown in Figure B-7 and the appropriate overlay thickness corresponding to the anticipated traffic selected.

If the amount of Class 2 cracking exceeds 5 percent and the Class 3 cracking is less than 5 percent, the asphalt concrete is assigned a modulus of 70,000 psi and overlay thicknesses to satisfy both the rutting and fatigue criteria are determined and plotted as in Figure B-7.

Similarly, if the amount of Class 3 cracking exceeds 5 percent, the asphalt concrete is assigned a modulus equal to that of the untreated base course but not less than 20,000 psi and overlay thickness is determined to satisfy the criteria.

The above procedures have been combined into two computer programs termed DEFANL and OVLANL

FEDERAL HIGHWAY ADMINISTRATION—RII PROCEDURE

This procedure was developed for the Federal Highway Administration (FHWA) for highway pavements by Resource International Inc. (RII) (*36*) and is based on a method developed by Austin Research Engineers for FHWA. This method, like that of the Shell researchers, follows, to some extent, the format of Figure 6. Unlike the Shell procedure, only fatigue cracking in the asphalt-bound layer is considered in the design process.

Nondestructive Testing

Deflections can be obtained with the Dynaflect, Road Rater, or a falling weight deflectometer. The computer program has

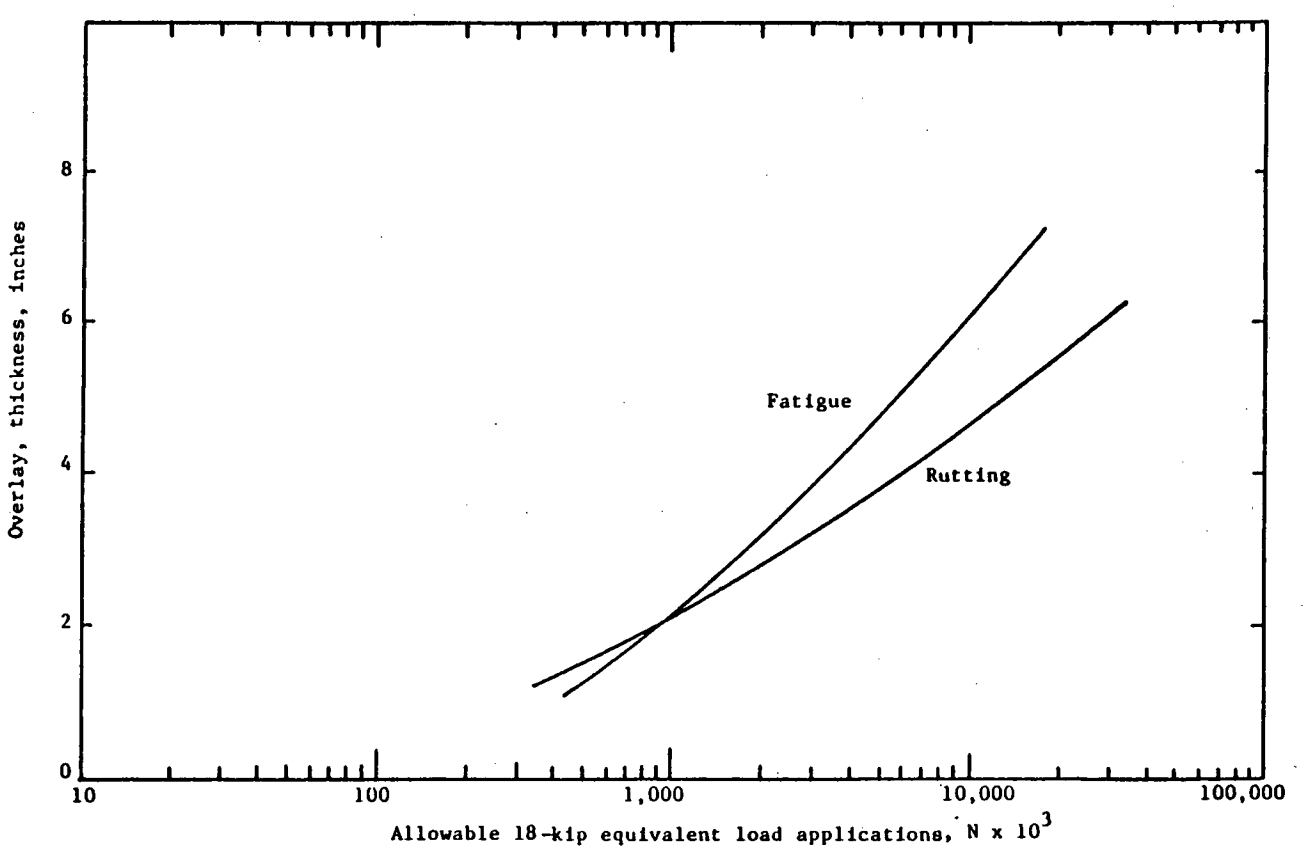


FIGURE B-7 Sample overlay thickness design curves.

been developed to accommodate the deflection results from one of these devices. It is noted (*36*) that the program can be modified to accommodate other deflection testing devices. As a part of the procedure, it is necessary to measure deflections at least four distances from the applied load to define the shape of the deflected pavement surface.

Deflection measurements are recommended to be made at 50- to 150-ft intervals along the roadway depending on the type of terrain, uniformity of soil conditions, and existing pavement condition. For two-lane, two-way highways it is recommended that deflections be measured in the outer wheel path for each direction of travel. Similarly, for divided highways deflection profiles should be obtained in the outer lanes of both roadways.

Condition of the existing pavement should be noted and the extent of Class 2 or Class 3 cracking (AASHO Road Test definition) delineated.

Pavement temperatures at the time of the deflection measurements can be obtained directly or estimated from air and pavement surface temperature measurements using the procedure developed by Southgate (*72*).

Analysis Sections

Rather than using analysis sections developed from deflection measurements and the condition evaluations, it is recommended that the deflection data at each test location be analyzed separately and an overlay thickness determined.

Materials Characterization

The pavement is represented as a multilayer elastic structure (either three or four layers) necessitating a representative modulus (stiffness) and Poisson's ratio for each of the layers. Nonlinear response characteristics of granular layers are represented by an equation of the form:

$$M_R = K_1 \sigma^n \tag{B-11}$$

where

σ = sum of principal stresses including algebraic sum of both load and gravity stresses, and
K_1 and n = material coefficients.

Moduli of fine-grained soils are considered to be a function of applied stress according to the relation:

$$M_R = K_2 \, \sigma_d^{\,m} \tag{B-12}$$

where

σ_d = deviator stress and
K_2, m = material coefficients.

For asphalt concrete, modular values are dependent on the environment as represented by a design temperature that is dependent on the mean annual air temperature at a specific site.

Layer moduli are determined for a specific site by inputing the following information into a computer program making use of the ELSYM solution for stresses and deformations in multilayer elastic systems:

1. Surface deflection measurements (and the load configuration, e.g., Dynaflect, by which they were obtained),
2. Base type,
3. Layer thicknesses,
4. Poisson's ratio for all layers,
5. Modulus of asphalt concrete at test temperature.

Essentially, the program solves for the moduli of the various layers (2 or 3) by attaining compatibility between measured and computed deflections at the locations for which deflection data were acquired in the field. Alternatively, moduli can be determined from laboratory tests on representative specimens of the various layers.

When the existing pavement is analyzed for actual traffic conditions, the moduli of the untreated materials reflect the influences of traffic loading since the use of Eq. (B-11) and Eq. (B-12) permit such effects to be considered.

If the existing pavement exhibits Class 2 or 3 cracking, the asphalt concrete is assigned a modulus of 70,000 psi.

Distress Determinant

Fatigue in the asphalt-bound layer is the only distress mode considered. As in other design methods, this is controlled by the magnitude of the tensile strain repeatedly applied. The relationship between traffic applications and strain is represented by Eq. (B-13), which was developed from an analysis of AASHO Road Test data:

$$N_f = 7.56 \times 10^{-12} \left(\frac{1}{\epsilon_t}\right)^{4.68} \tag{B-13}$$

where ϵ_t = maximum tensile strain parallel to direction of traffic.

Remaining Life Estimation and Overlay Design

Having determined the various layer moduli by an iterative solution matching measured and computed deflections, an estimate of remaining life for the uncracked pavement can then be made. To do this, the amount of traffic that has used the structure to date must be estimated in terms of repetitions of an 18,000 lb, single-axle load. This level must then be adjusted by a *regional factor* and by a *seasonal factor* reflecting the subgrade strength at the time at which the deflection measurements are made. The linear summation of cycle ratios cumulative damage hypothesis is used in the same manner as described in Chapter 2, i.e.,

$$\frac{N_r}{N_{D1}} = 1 - \frac{N_{A1}}{N_{D1}} \tag{B-14}$$

N_{D1} is obtained from Eq. (B-13) using the tensile strain computed in the underside of the asphalt concrete for the 18,000 lb. axle load and N_{A1}, the actual traffic applied to date, is determined as noted above.

If the decision to overlay is made, a particular thickness of asphalt concrete is added and the resulting strain is computed either on the underside of the combined layers if the existing pavement is intact or on the underside of the overlay with the existing asphalt layer assigned a modulus of 70,000 psi. The computations have been incorporated in a computer program termed OAF and shown schematically in Figure B-8.

As noted earlier, an overlay thickness is computed for each location at which deflection data were obtained. From an evaluation of the resulting thicknesses, the project recommendations are then made for the values to be used, usually based on some percentile (e.g., 95th percentile of thickness).

KENTUCKY DEPARTMENT OF HIGHWAYS PROCEDURE

The method developed by personnel of the research group associated with the Kentucky Highway Department (*73, 74*) can be considered a deflection-based procedure that makes use of some of the concepts of the analytically based methodology of Figure 6. Because this procedure utilizes the design charts developed by Kentucky for new pavement designs, fatigue in the asphalt concrete and rutting as determined by subgrade strain are the principal distress considerations in controlling overlay thickness.

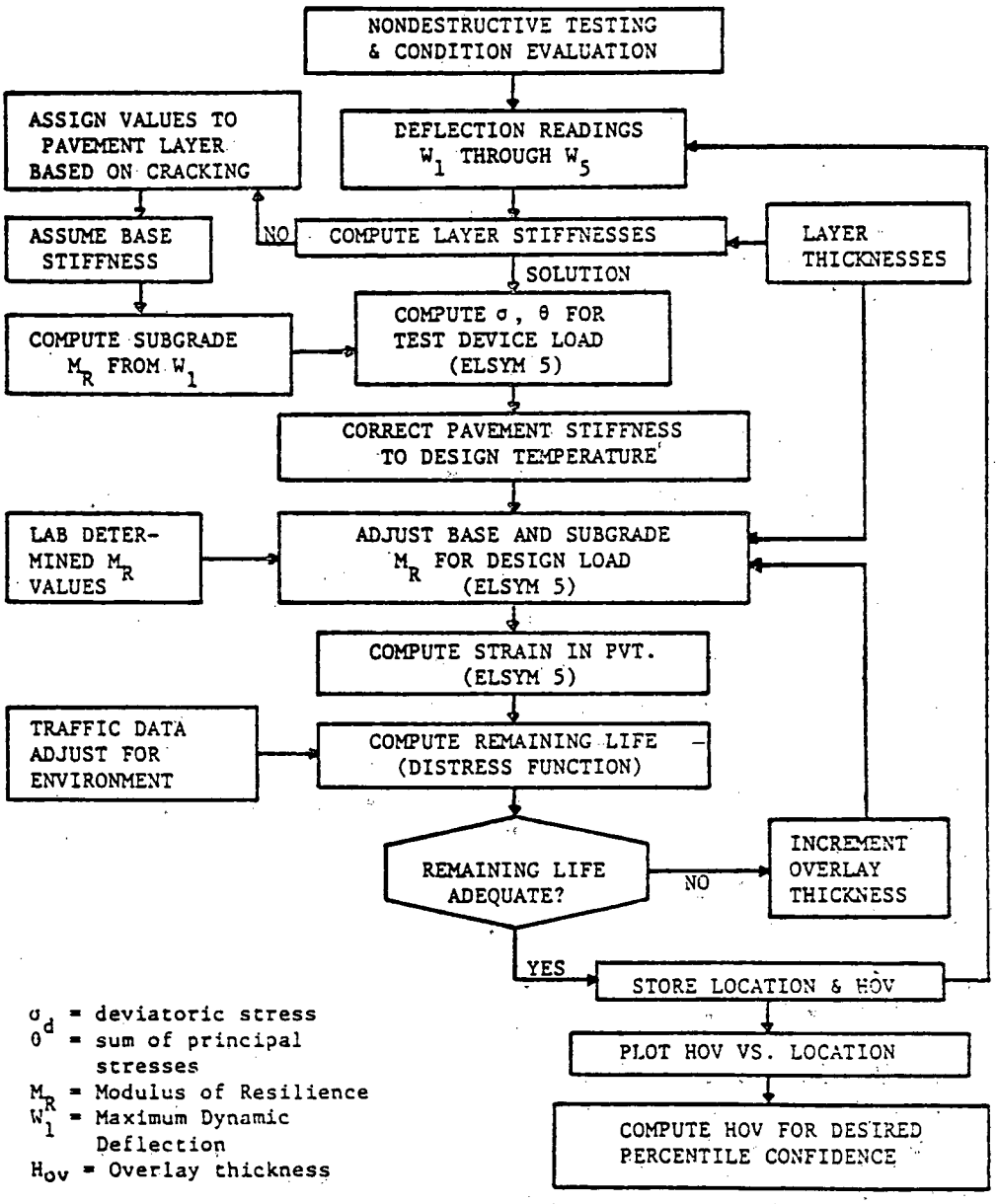


FIGURE B-8 Simplified flow chart of OAF program.

Nondestructive Testing

The Road Rater is used to test the pavement at a frequency of 25 Hz and with a peak-to-peak force of about 600 lb. Deflections are measured directly under the load and at *offsets* of 1, 2, and 3 ft by velocity transducers. Temperatures of the pavement are obtained at the time of the deflection measurements and the observed deflection values are adjusted to a standard temperature of 70°F (21°C). Table B-1 contains a summary of the factors used to adjust the deflections for sensors 1, 2, and 3 (0-, 1-, and 2-ft offsets).

Frequency of testing is recommended to be at 0.1 mile intervals for each direction or lane tested. If a problem area is encountered, a smaller interval between measurements may be necessary (e.g., 100-ft intervals).

Analysis Sections

Analysis sections are selected from strip charts like those shown in Figures B-9 and B-10, which are plots of subgrade modulus and effective thickness of asphalt concrete determined from the field deflection measurements for each location tested. It is recommended that the data for a particular section be treated statistically and that the parameters be selected to minimize premature failure in the overlay (e.g., the 90th percentile level).

Materials Characterization

Subgrade moduli and effective thicknesses of the asphalt concrete layer are determined from the deflection data. These

TABLE B-1

EQUATION AND CONSTANTS FOR DEFLECTION ADJUSTMENT FACTORS FOR THE KENTUCKY ROAD RATER

$$\log AF_j = [\log AC - (H_1 E_{AC}^3 + H_2 E_{AC}^2 + H_3 E_{AC} + H_4)] \; [M_1 E_{AC}^3 + M_2 E_{AC}^2 + M_3 E_{AC} + M_4]$$

in which AF = deflection adjustment factor,
AC = thickness of asphaltic concrete (inches),
E_{AC} = mean modulus of elasticity of asphaltic concrete (psi), and
j = Road Rater sensor number (1, 2, 3).

THREE LAYERED PAVEMENTS

Crushed-Stone Base less than 8 inches thick

j	H_1	H_2	H_3	H_4
1	-2.0312535E-19	7.1127654E-13	-8.4587020E-07	0.25466949
2	7.2614981E-20	-1.0302809E-13	-1.3874220E-07	0.46069097
3	1.6419243E-19	-3.3570986E-13	-4.2060526E-08	0.66081522

j	M_1	M_2	M_3	M_4
1	1.0225763E-19	-4.4990262E-13	7.0628626E-07	-0.37155742
2	4.3336498E-20	-2.2091077E-13	4.5988841E-07	-0.30474423
3	3.3403716E-20	-1.6395133E-13	3.4805071E-07	-0.23820381

Crushed-Stone Base equal to or greater than 8 inches thick

j	H_1	H_2	H_3	H_4
1	-2.3472762E-20	1.1931522E-13	-2.9552194E-07	0.15345469
2	8.6274124E-20	-1.3810588E-13	-8.8295169E-08	0.42052283
3	1.1280263E-19	1.7456748E-13	-1.3783142E-07	0.60022647

j	M_1	M_2	M_3	M_4
1	8.6110078E-20	-3.8725065E-13	6.2848481E-07	-0.34173343
2	3.5850121E-20	-1.8167083E-13	3.8532939E-07	-0.25976352
3	2.1116466E-20	-1.0783377E-13	2.5034102E-07	-0.18049747

TWO LAYERED PAVEMENTS

j	H_1	H_2	H_3	H_4
1	-1.1966613E-18	3.6419900E-12	-3.3712189E-06	0.40220812
2	8.4194518E-20	-1.0803309E-13	-2.9750845E-07	0.63921054
3	2.8337843E-19	-6.0413664E-13	6.2056443E-08	0.84294820

j	M_1	M_2	M_3	M_4
1	1.0486807E-19	-4.5399608E-13	6.6726565E-07	-0.32106577
2	1.0429773E-19	-4.7586726E-13	7.9423008E-07	-0.44438965
3	9.6133265E-20	-3.9709184E-13	6.8153597E-07	-0.41509883

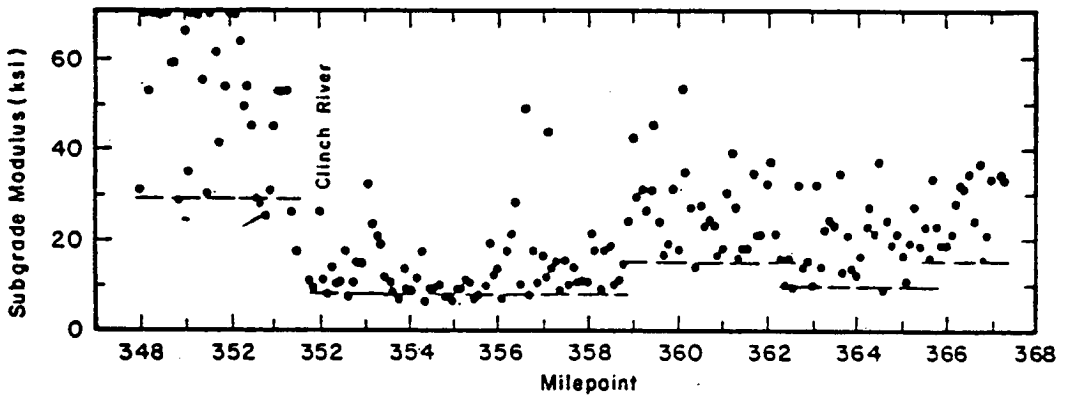


FIGURE B-9 Strip chart of subgrade modulus.

estimates are accomplished by ensuring that a realistic combination of layer thicknesses at reference conditions and subgrade modulus produce a correspondence between the measured and computed shape of the deflected surface (defined by sensors 1, 2, and 3). These parameters are determined for each measuring point and printed on the strip chart referred to above.

A graphical procedure is used to make the estimate of these two parameters and is illustrated in Figure B-11. The relationships between sensor deflection, subgrade modulus, and effective layer thickness were developed on the assumption that the pavement responds to the Road Rater as a multilayer elastic system. In determining the response, the modulus of the untreated granular layer is assumed to be a function of the modulus of the subgrade.

Overlay Design

For overlay design it is necessary to estimate the traffic expected to use the section in terms of 18-kip equivalent axle loads. With the traffic data, subgrade modulus (estimated from CBR value), and actual thickness of granular layer, a range of thicknesses are selected from design charts like that shown in Figure B-12. These design charts are based on the conclusion that the pavement responds as a multilayered elastic system. Thicknesses of the layers are controlled by tensile strain in the asphalt-bound layer (fatigue cracking) and vertical compressive strain at the subgrade surface. The resulting thicknesses are plotted as shown in Figure B-13 and labeled Curve A.

With the design EAL and in-place subgrade modulus, three

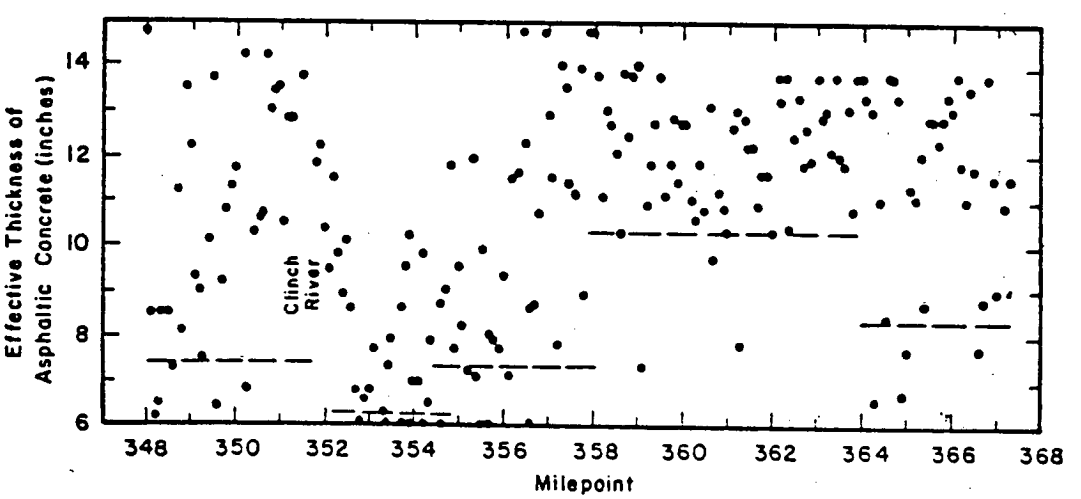


FIGURE B-10 Strip chart of effective thickness of asphaltic concrete.

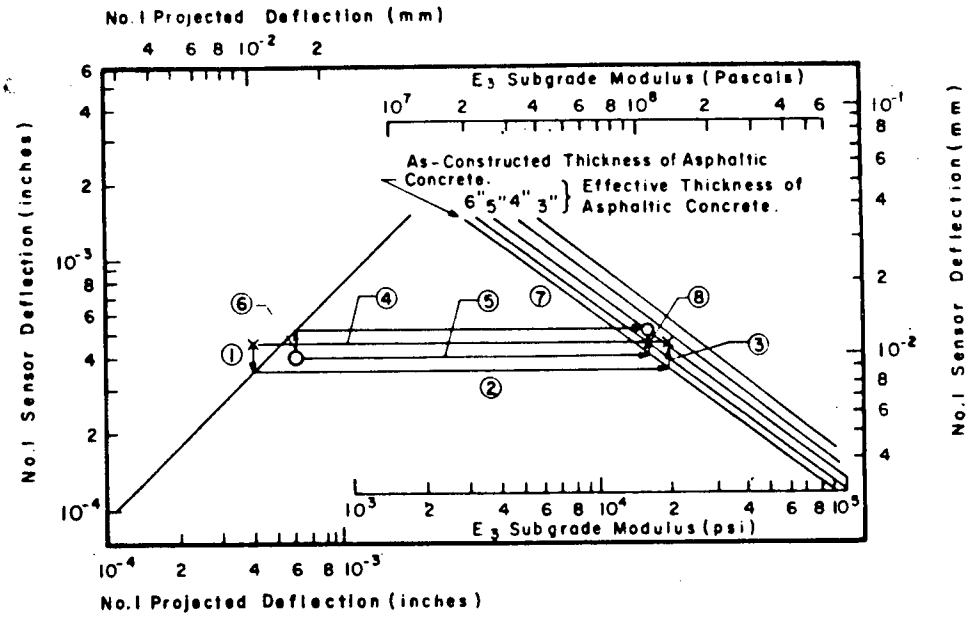


FIGURE B-11 Illustration of a method for estimating in-place subgrade modulus and effective thickness of asphaltic concrete.

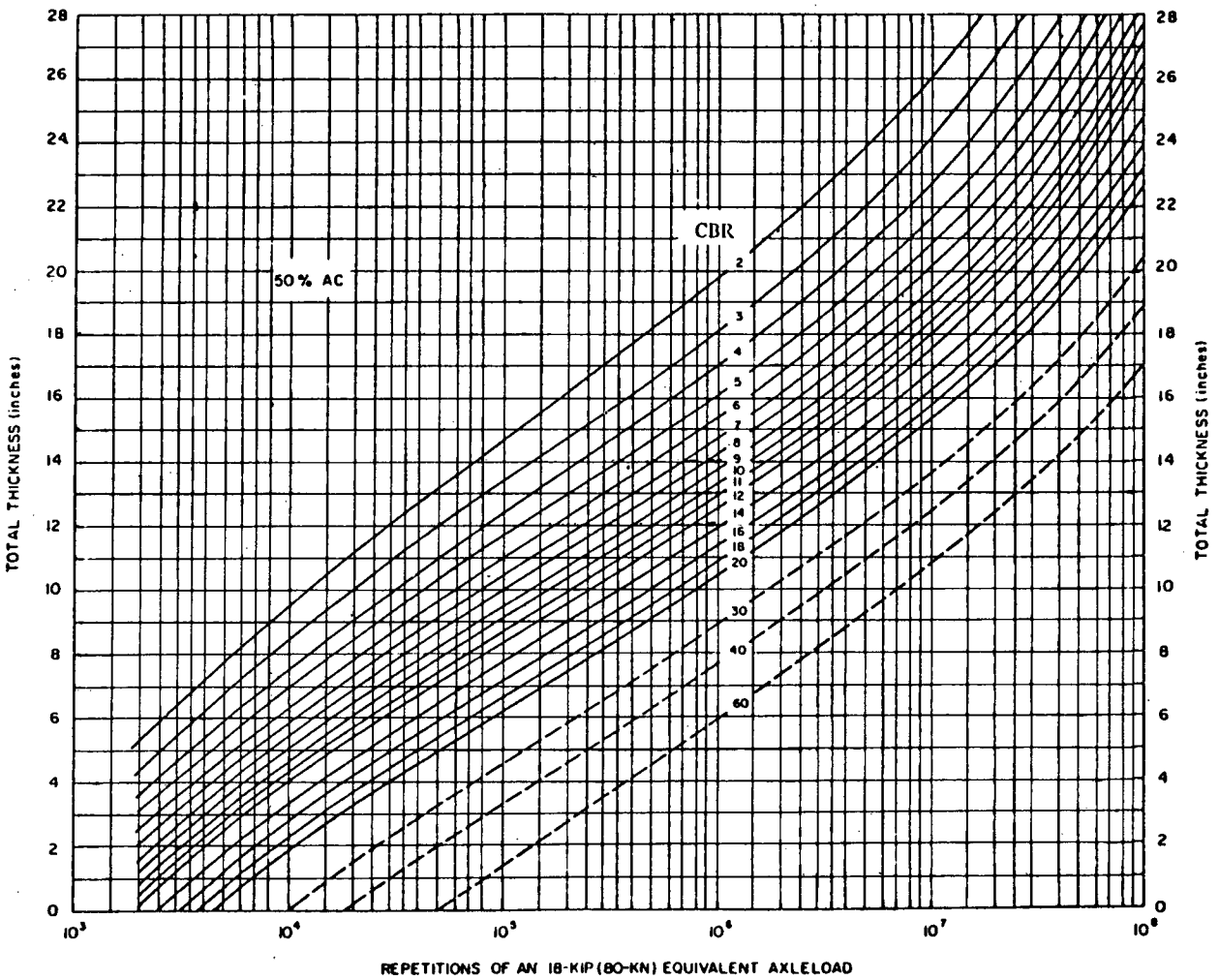


FIGURE B-12 Example of one design chart used to plot curve A in Figure B-13.

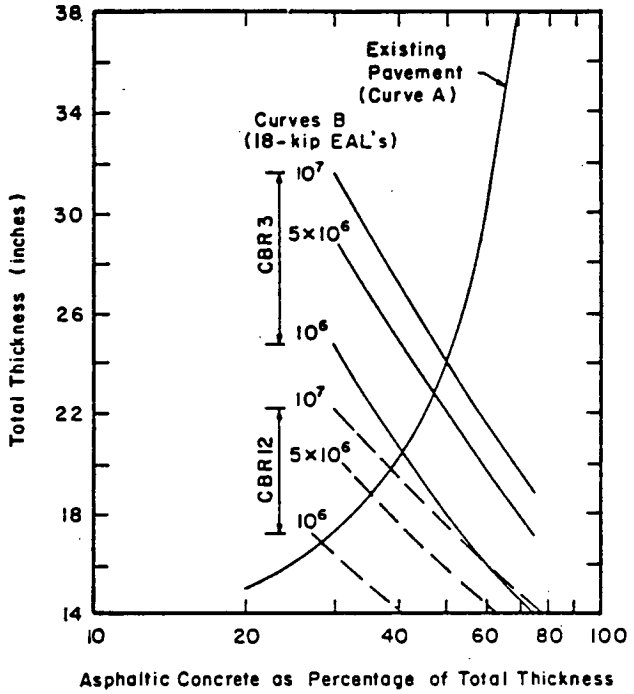


FIGURE B-13 Example of relationship between total design thickness and percentage of total thickness, owing to asphaltic concrete.

design thicknesses are determined and plotted as curves B of Figure B-13. The intersection of curves A and B for a particular subgrade modulus (CBR) and EAL is the required total thickness for the design conditions. Thickness of the overlay is simply the difference between the total thickness required for the "new" construction and the effective thickness of the existing pavement.

An alternative procedure is to prepare an overlay design for each test point and to plot the individual thicknesses on the strip chart. Analysis of these thicknesses would then permit selection of overlay designs for appropriate degrees of risk of premature failure (e.g., 10 percentile level).

APPENDIX C

USE OF STATISTICAL ANALYSIS TO SELECT ANALYSIS SECTION (*31*)

Using the nondestructive deflection test data, a candidate project can be divided into various design sections as illustrated in Figure 5 of this synthesis.

Adjacent design sections should be checked to see if they are significantly different or whether they are from the same population of data.

Statistical hypothesis techniques can be used to determine whether the deflection data can be considered to be from a single population or should be treated as representative of more than one population with regard to structural properties. Standard statistical methods for testing of significance between two samples, such as the hypothesis test for equal means, should be used for making this evaluation. The following steps and formulas are illustrated for making this test. This example is for illustration only and should not be used as a "prescription" for such testing. If such procedures are to be used, the reader should refer to appropriate textbooks on probabilities and statistics for better understanding of the concept and its application.

To illustrate the procedure, assume that the candidate project used in Figure 5 has been divided into two design sections based on engineering judgment. The step-by-step procedure is described as follows:

where

a, b = individual measurements or variates in sections designated 1 or 2, respectively;
$\bar{a}$, $\bar{b}$ = mean value of measurements of variates in Sections 1, 2; and
n_a, n_b = number of variates in Sections 1, 2.

Step 1—Calculate the mean ($\bar{a}$) from the section 1 data:

$$\bar{a} = \frac{\Sigma a}{n_a} \tag{C-1}$$

Step 2—Calculate the mean ($\bar{b}$) from the section 2 data:

$$\bar{b} = \frac{\Sigma b}{n_b} \tag{C-2}$$

Step 3—Calculate the "pooled estimate of the standard deviation" (S) from the two sections. This way the standard deviation determined is not affected by any difference that may exist between the means of each section.

$$S = \left[\frac{\Sigma(a - \bar{a})^2 + \Sigma(b - \bar{b})^2}{n_a + n_b - 1}\right]^{1/2} \tag{C-3}$$

Step 4—Determine the best estimate of the standard deviation of the mean of samples of n variates for Section 1, $S_{\bar{a}}$

$$S_{\bar{a}} = \frac{S}{(n_a - 1)^{1/2}} \tag{C-4}$$

Step 5—Do Step 4 for Section 2,

$$S_{\bar{b}} = \frac{S}{(n_b - 1)^{1/2}} \tag{C-5}$$

Step 6—Calculation from Step 4 and Step 5.

$$S_{(\bar{a} - \bar{b})} = (s_{\bar{a}}^2 + S_{\bar{b}}^2)^{1/2} \tag{C-5}$$

Step 7—Hypothesis: $M_1 - M_2 = 0$ where M_1 and M_2 are means of two normally and independently distributed sections. Calculate t-value for student t-distribution.

$$t = \frac{(\bar{a} - \bar{b}) - M_{\bar{a} - \bar{b}}}{S_{(\bar{a} - \bar{b})}} \tag{C-6}$$

Step 8—Obtain t value from Student's t-distribution in statistics tables to check hypothesis.

Step 9—Check hypothesis—compare computed t-value with Table value. If computed value is larger than table value, the two sections are significantly different.

If two adjacent sections are not significantly different, they should be combined into one and then checked against the next section. This procedure will establish the design sections, each of which then becomes a separate design problem.

The designer may select the percentile level of significance at which to check the difference in deflection. The five percent level is suggested, but the designer may select a value either larger or smaller relative to the specific problem. The statistical check may be made either by hand solution or with computer program.

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