State-Of-The-Art On Pavement Overlay Procedures

Vol. I, State-of-the-Art Review and Research Plan

WA-RD 65.1

Final Report
December 1983

Washington State Department of Transportation
Planning, Research and Public Transportation
In Cooperation with
United States Department of Transportation
Federal Highway Administration

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No. WA-RD 65.1	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle State-of-the-Art on Pavement Overlay Design Procedures Vol. 1 - State-of-the-Art Review and Research Plan Vol. 2 - Annotated Bibliography		5. Report Date December 15, 1983
		6. Performing Organization Code
7. Author(s) Newcomb, D.E., Bubushait, A.A., Mahoney, J.P. and Sharma, J.		8. Performing Organization Report No.
9. Performing Organization Name and Address University of Washington Department of Civil Engineering Seattle, Washington 98195		10. Work Unit No.
		11. Contract or Grant No. Agreement Y-2811 Task Order No. 3
12. Sponsoring Agency Name and Address Washington State Department of Transportation Highway Administration Building Olympia, Washington 98504		13. Type of Report and Period Covered Final Report Sept. 15 - Dec. 15, 1983
		14. Sponsoring Agency Code

15. Supplementary Notes

WSDOT Contract Manager - Mr. Art Peters

16. Abstract

The study is reported in two volumes. The first volume summarizes the state-of-the-art on pavement overlay design and presents a research plan to develop an overlay design procedure for the Washington State Department of Transportation (WSDOT). The second volume contains an annotated bibliography of significant pavement overlay literature published since 1968.

The state-of-the-art review presented in Volume I addresses pavement evaluation using nondestructive testing and subjective ratings. These are discussed along with the topics of traffic and seasonal variations. Pavement overlay design concepts are identified and discussed. A research plan is presented for the development of a pavement overlay design system for WSDOT.

17. Key Words Pavement, Overlay, Design Performance, Nondestructive Testing, Traffic, Seasonal Variations	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price

Form DOT F 1700.7 (8-69)
HWY 6/72

STATE-OF-THE-ART ON PAVEMENT OVERLAY DESIGN PROCEDURES

VOLUME I STATE-OF-THE-ART REVIEW AND RESEARCH PLAN

by

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Prepared by the
University of Washington
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Washington State Transportation Center
for the
Washington State Transportation Commission
Department of Transportation
and in cooperation with
U.S. Department of Transportation
Federal Highway Administration

WSDOT Contract Agreement Y-2811

December 1983

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ACKNOWLEDGMENTS

The authors gratefully acknowledge the assistance of Ms. Barbara Russo, WSDOT Librarian, and Ms. Beverly Green, UW Librarian, in conducting the literature search for this study. Appreciation is extended to Ms. Carol Sanders for locating and retrieving reference materials and Ms. Jo Lary for assisting with report editing. Ms. Beverly Odegaard greatly contributed to the effort by typing the manuscript.

A special thanks goes to Mr. Art Peters and Mr. Newton Jackson of the WSDOT Materials Laboratory for their valuable suggestions and encouragement. It is uncommonly motivating to work directly with the users of research products.

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Washington State Transportation Commission, Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

CHAPTER I
INTRODUCTION

OBJECTIVE

The primary objective of the reported study was to conduct a state-of-art review of pavement overlay design procedures. Further, this review will be used as the first step in the development of a mechanistically based overlay design approach which can use deflection basin data obtained with the Washington State Department of Transportation (WSDOT) Falling Weight Deflectometer.

SCOPE

The study was a cooperative effort between the University of Washington, Department of Civil Engineerng and the WSDOT Materials Laboratory. The principal tasks reported herein include a review of the state-of-the-art study, identification of the components to be included in the overlay design procedure, and development of a detailed work plan for future overlay design development activities.

BACKGROUND

The design of pavement structures can be traced to the Roman era - approximately 2 to 3,000 years ago. Since that time, these structures have evolved from massive quarried stone sealed with crude pozzolanic cements with thicknesses often in excess of three feet thick to the generally thinner more complicated pavement structures of today.

Current pavement design practice in the United States is based essentially on technology developed during the 1930's through the early 1960's. This fact in itself does not mean that current design practice is poor; however, the design procedures are based principally on empirical performance data and material characterizations which are a function of the available traffic and other conditions which existed 20 to 40 years ago. For example, truck traffic has increased in frequency, size, and weight (including increasing tire pressures). Further, the kinds of materials and layer configurations used in pavements are continuing to change. An example of this for flexible pavements is the gradual change from the essentially exclusive use of dense graded asphalt concrete wearing courses on the Interstate and Primary System to the partial use of open-graded friction courses with asphalt or rubber-asphalt binders. Thus, pavement structural design practice needs improvement to better use currently available materials including recycled materials as well as future new materials.

To improve structural design practice, material parameters which are used in the design process must be reexamined. Current laboratory material testing is based on empirical procedures which do not relate well to actual pavement performance. Newer testing procedures, such as resilient modulus characterization, can be related directly to expected pavement performance and be used to intergrate new materials into the design process.

The emphasis of pavement design practice has shifted from the analysis of soils and the design of new roadway sections to the analysis of existing pavements and the design of various rehabilitation or reconstruction strategies. WSDOT makes extensive use of asphalt concrete overlays for rehabilitation and maintenance. The selection of an overlay thickness is based primarily on design techniques developed 30 years ago for new pavements tempered with considerable judgment. With acquisition of the Falling Weight Deflectometer (FWD), in situ pavement deflections and ultimately material properties can be obtained. Such information along with performance criteria (or failure criteria) uniquely available in the WSDOT data base can be used to select the proper thickness of overlay which will achieve the most economical solution (although not always the least expensive option initially).

The proposed study as described in Chapter VII - Research Plan provides an avenue to develop an improved overlay design procedure, based on modern structural design methods and actual pavement performance in Washington State. This includes the ability to use new materials more effectively, the FWD for in situ material characterization, updated laboratory materials testing and the use of the WSDOT pavement management system for the development of the required performance criteria. Further, this activity will provide a basis for a completely updated structural design system if required by WSDOT.

REPORT AND OVERLAY DESIGN OVERVIEW

The report is organized into eight chapters. These are:

Chapter I: Introduction,

Chapter II: Pavement Models,

Chapter III: In Situ Nondestructive Testing,

Chapter IV: Traffic,

Chapter V: Failure Criteria,

Chapter VII: Research Plan, and

Chapter VIII: Conclusions and Recommendations.

As can be discerned from the above chapter headings, this report will be used to summarize the state-of-the-art of the critical components incorporated in pavement overlay design procedures. A flow chart showing how the analysis and design process might be visualized is shown in Figure I-1. The components are shown with the chapters of the report in which they are discussed. The process begins with deflection testing and condition surveys of the pavement section under consideration. Output from the deflection testing are adjusted for environmental conditions at the time of testing. The deflection and pavement survey data are compared to failure criteria for the type of road being investigated. If the pavement has not failed, the remaining life is estimated for the level of expected traffic. If it has failed, then decisions must be made about material selections for overlays, seasonal variations must be determined, and traffic must be estimated. These are used as inputs to the stress-strain analysis to determine the required overlay thickness for the desired design life.

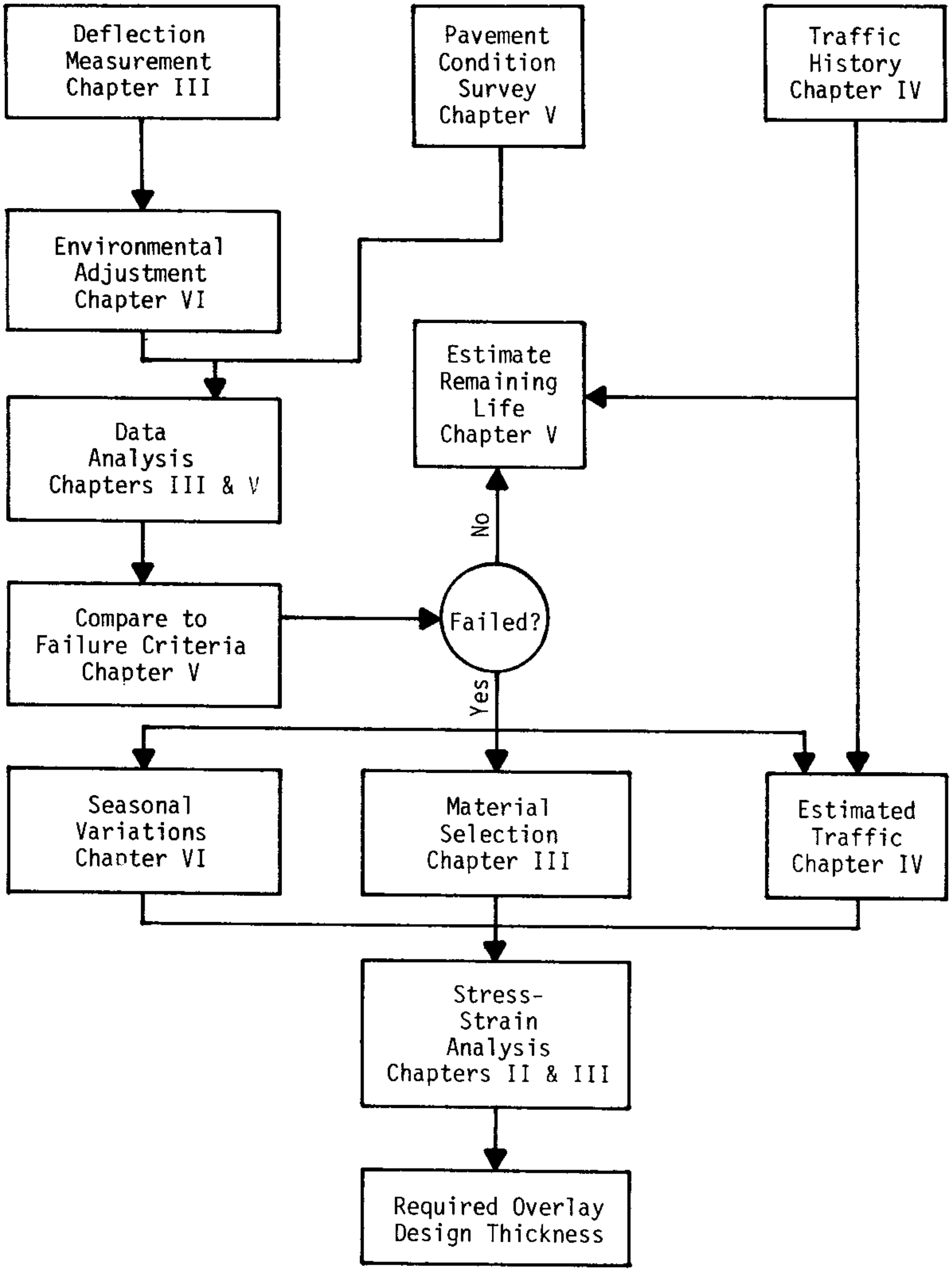


Figure I-1. Analysis Process for Pavement Overlay Design

CHAPTER II
PAVEMENT MODELS

INTRODUCTION

The sections contained in this chapter are used to briefly describe a number of analytical methods (or models) which can be used in an overlay design procedure. The primary emphasis is on computer codes (layered elastic or finite element) which provide for the estimation of stresses, strains, and/or deflections given material properties and the loading condition. To understand how such estimated stresses, strains, and deflections change before and after various rehabilitation strategies is fundamental to the design process.

ELASTIC LAYERED SYSTEMS

Westergaard is generally credited with the first use of elastic-layered theory to predict the response of rigid pavements to wheel loadings [II-1]. Burmister [II-2] later used the theory of elasticity as an approach to the solution of elastic-multilayered pavement structures. In the development of his solution, Burmister assumed that each layer could be represented as a homogeneous, isotropic, and linearly elastic material. Each layer was assumed to extend infinitely in the horizontal direction and the bottom layer was assumed to extend infinitely downward. The other layers were assumed to have a finite thickness.

One Layer System

According to Boussinesq [II-3], in a one layer system, the vertical stress at any depth below the earth's surface due to a point load at the surface is given by:

$$\sigma_z = kP/z^2$$

where $k = \frac{3}{2\pi} \frac{1}{[1 + (r/z)^2]^{5/2}}$

P = load

r = radial distance from point load,

z = depth.

Vertical stresses due to a concentrated load are distributed in a bell-shaped form on horizontal surfaces beneath the load. The maximum stress may be found on a vertical line to the point of loading. The pressure decreases with depth.

In actual pavement systems, loads generally occur over an elliptical area and not at a specific point. However, the decrease of stress with depth would maintain the same general pattern as in point loading.

Several influence charts and tables have been developed to determine the stresses, strains, and deflections at any point in a homogeneous mass for any value of Poisson's ratio. [II-3]. Stress influence charts developed by Newmark [II-4] for elastic soil masses have been extensively used in foundation design. Ahlvin and Ulery [II-5] presented a variety of solutions for one-layer elastic equations. Some of these equations contained specific functions which were expressed in terms of radial distance and depth.

Most pavements are not homogeneous and therefore require solutions which are more complex than those discussed above. This is certainly true for pavement structures which incorporate an overlay of an existing system.

Two Layer System

Typical pavements are composed of different layers such that material stiffness decreases with depth. The end result is the reduction of stresses, strains, and deflections in the subgrade compared to the one-layer case [II-3].

In two-layer systems, materials within a specific layer are assumed to be homogeneous, isotropic, and elastic. Furthermore, the layers are assumed to extend an infinite distance horizontally. The surface layer has a finite depth and the underlying layer is assumed semi-infinite in the vertical. For boundary and continuity considerations, there are no shearing and normal stresses outside the loaded area for the surface layer and the layers are assumed to be in continuous contact [II-3].

For a two-layered system, the total surface deflection, can be obtained as follows (flexible plate case):

$$\Delta = 1.5 \frac{pa}{E_2} F_2$$

where: Δ = total surface deflection,

p = unit load on circular plate,

a = radius of plate,

E_2 = modulus of elasticity of lower layer,

F_2 = dimensionless factor depending on the ratio of moduli of elasticity of the subgrade and pavement as well as the depth to radius ratio.

Three Layer System

For a three-layer pavement system, several charts and tables have been developed by Peattie, Jones and Fox [II-3] to determine the stresses, strains, and deflections. Peattie [II-6] developed graphical solutions for vertical stress in three-layer systems. An example of this is shown in Figure II-1. Jones [II-7] presented solutions for horizontal stresses in a tabular form. Both of these solutions were based upon a Poisson's ratio of 0.5 for all layers. The following parameters were used in both solutions:

$$K1 = \frac{E_1}{E_2}$$

$$K2 = \frac{E_2}{E_3}$$

$$A = \frac{a}{h_2}$$

$$H = \frac{h_1}{h_2}$$

where: E_i = modulus of elasticity for layer i,

a = radius of loaded area, and

h_i = layer thickness for layer i.

Once these parameters are defined, the appropriate factor may be chosen to calculate stress at a given point.

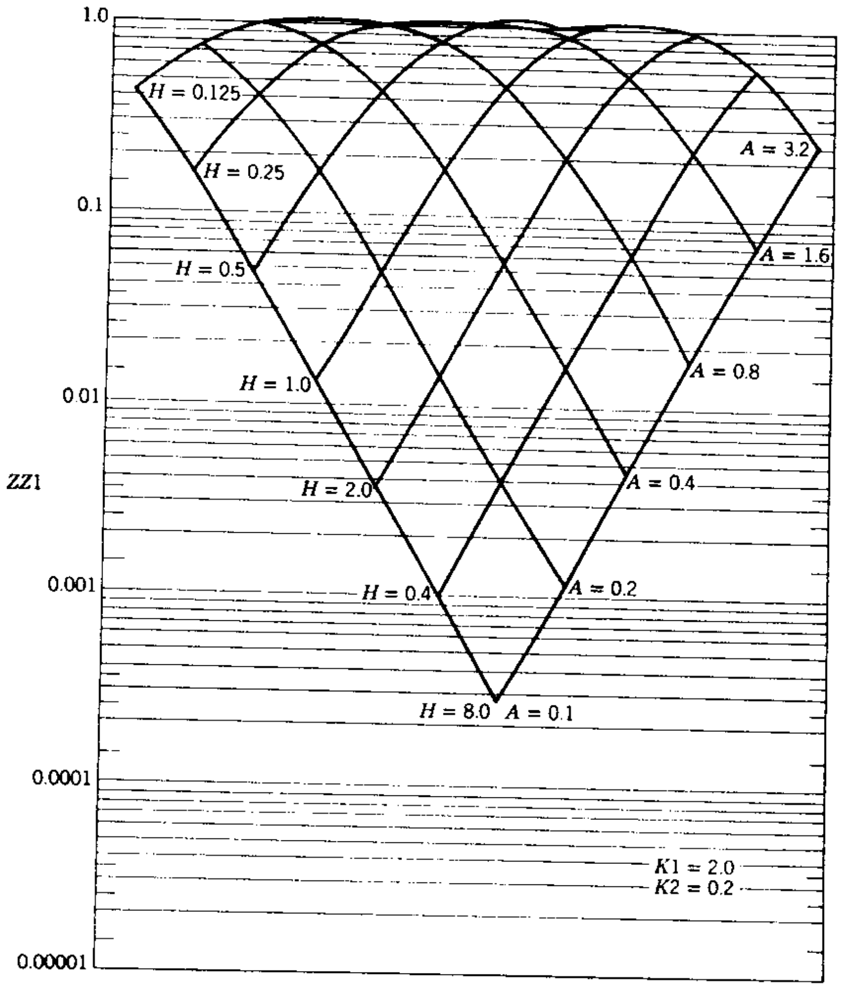


Figure II-1. Example Three-Layer Solution Developed by Peattie, [after Reference II-6].

Computer Codes

In recent years, several computerized solutions have been developed for the elastic analysis of multilayered systems. Some programs use the principle of superposition in order to consider the effects of multiple loads.

The input data required for use of a basic layered theory program are load magnitude and contact area or pressure, modulus of elasticity (E) and Poisson's ratio (μ) of each layer, and the thickness of each layer except the lowest.

CHEVNL.

This program presents solutions for multilayered elastic systems. It was developed by Chevron Research Company (formerly California Research Corporation) in the early 1960's [II-8]. It computes stresses, strains, and deflections as a result of a single uniform load applied vertically to the pavement as shown in Figure II-2. This sytem is capable of analyzing up to 15 layers. All layers are of finite thickness except the bottom which is of semi-infinite thickness. The horizontal dimension is infinite for all layers. The surface of the pavement is assumed to have no shear forces acting upon it.

Radial and vertical distances are expressed in cylindrical coordinates as R and Z, respectively. The Z-axis at R=O extends through the center of the load as shown in Figure II-2.

The vertical load and contact pressure are used to describe the problem loading conditions. Using these parameters, the program computes the load radius. Material properties of individual layers are expressed in terms of modulus of

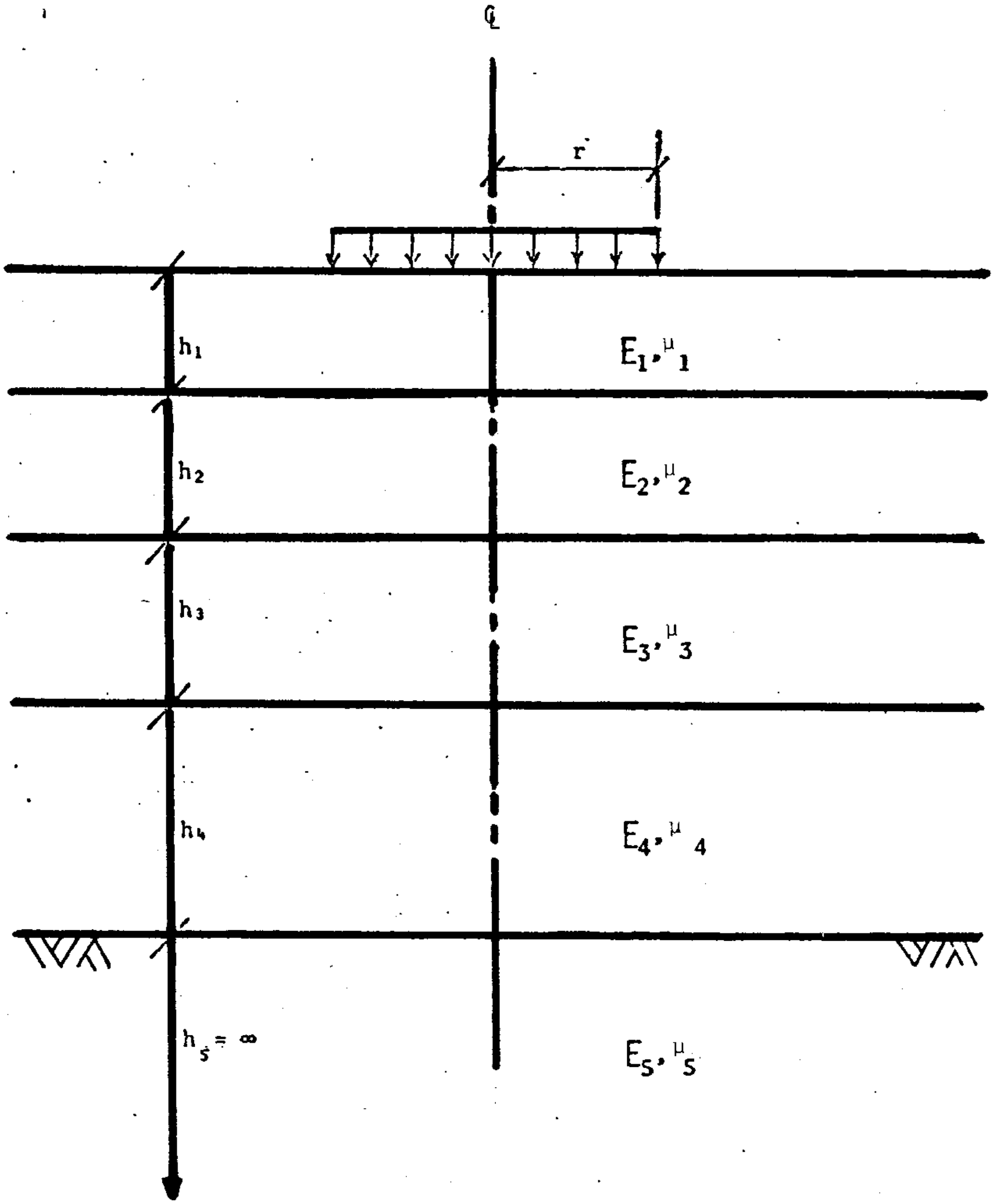


Figure II-2. Multilayered Elastic System [after Reference II-9].

elasticity, Poisson's ratio, and thickness. The theoretical approach of CHEVNL is outlined in Reference II-8 and the following characteristics have been attributed to the analysis:

1. Up to 15 layers may be incorporated in the program.
2. The materials may be assigned any values of moduli.
3. Poisson's ratio may be any value other than one.
4. The mathematics are relatively easy and self-contained.
5. The effects of multiple wheel loads must be computed outside of the program using superposition.

CHEV5L - ITERATION

The CHEV5L WITH ITERATION program is a multilayered elastic system which may be used to determine stresses and strains while allowing material moduli to vary with stress levels [II-9]. This program is an extension of an earlier version called CHEV5L (not CHEVNL). The major advantage of this program is the estimation of subgrade modulus from the modulus-deviator stress relationship which is used as input. For non-stress dependent materials, a horizontal relationship is used as input.

Another advantage of this program is that overburden pressures may be incorporated into the solution by superimposing load-induced and overburden stresses. This, of course, is dependent upon a knowledge of material densities and thicknesses. The iterative process of this program compares the stress state in the material with the initially assumed modulus. This is repeated until the stress state and modulus value are reconciled to specified accuracy limits.

Other characteristics of this program have been identified [II-9] as:

1. Five layers may be used in the analysis.
2. Output values may be obtained for 48 to 121 points in the pavement.
3. No negative data may be used for input.
4. Poisson's ratio may be any value except one.
5. Effects of multiple wheel loads must be computed outside of the program using superposition.

BISAR

This computer program uses elastic-layered theory to solve for stresses, strains, and displacements in pavement systems with one or more uniform circular loads applied vertically at the surface [II-9]. Typical component stresses are illustrated in Figure II-3. BISAR additionally has the capability of considering surface loads to be combinations of vertical normal and unidirectional horizontal forces. The usual elastic-layered assumptions apply in this program except for continuity. Layer interfaces are assumed to either be in full continuity or frictionless.

Stresses, strains, and displacements due to each load are calculated separately in a cylindrical coordinate system. In multiple load problems, the cylindrical coordinate system is transformed to Cartesian. The effects of multiple loads are computed by summing the effects of each individual load. Specific output parameters must be designated in the program for locations and components.

Some of the characteristics of BISAR include [II-9]:

1. A maximum of 10 layers may be used.

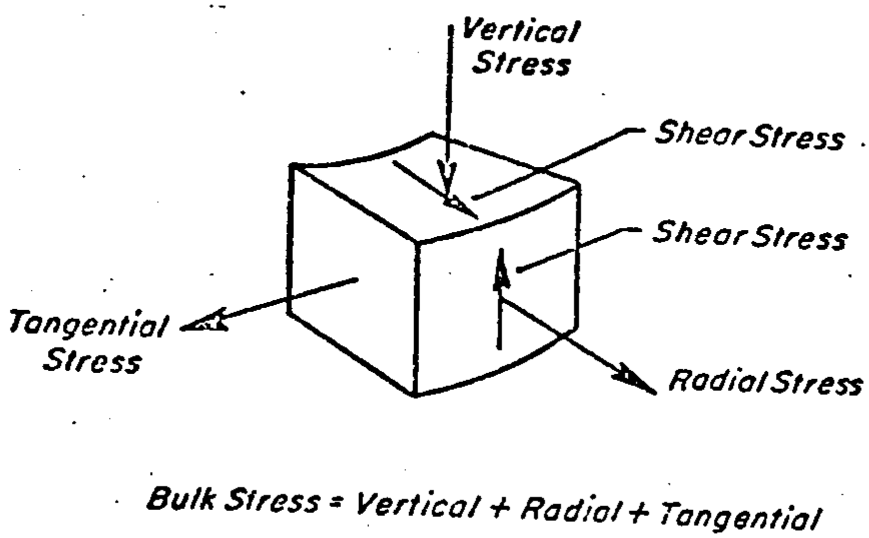


Figure II-3. Stresses on a Typical Element
[after Reference II-9].

2. Up to 99 systems may be evaluated in one run.
3. Up to 99 points within a system may be specified for evaluation.
4. No negative data may be used as input.
5. There are no provisions for non-linear behavior in the materials.

ELSYM5

This elastic layered system computer program was developed at the University of California, Berkeley [II-10] and can be used to analyze up to ten identical loads on a five layer system. It computes various components of stress, strains, and displacements along with principal values in a three-dimensional ideal elastic layered system.

The top surface of the pavement is assumed to have no shear. As with other elastic layer systems, the layers are assumed to have uniform thicknesses and to be infinite in horizontal distance. Layer interfaces are assumed to be continuous. A finite thickness may be used for the bottom layer or it may be assumed semi-infinite. If a finite thickness is used, the program assigns a rigid underlying layer to support it and a continuous or frictionless interlayer must be assumed [II-9].

Input data for ELSYM5 are any two of three load determinants (load in pounds, stress in pounds per square inch, radius of loaded area in inches), load position, elastic modulus, Poisson's ratio, location of analysis points, and thickness of each layer (except the lowest).

Loads are assumed to be uniform, static, and circular, and the principle of superposition is used for determining the effect of multiple loads. Hicks, et al. [II-9] identified the following program characteristics:

1. One to five systems may be evaluated in a single run.
2. One to five layers may be used in one system.
3. One to ten identical circular loads may be applied to the pavement.
4. One to 100 points may be specified for results.
5. No depth may be specified for results if it extends below the top of the rigid underlying layer.
6. No negative data are allowed except for horizontal distances.
7. Poisson's ratio must be any value except one. For a subgrade on rigid support, Poisson's ratio must not be within the range of 0.748 to 0.752.
8. Results are approximate at or near the pavement surface and at some horizontal distances from the load. This is due to a truncated series used in the integration process.

PSAD2A

This program is the same as CHEV5L WITH ITERATION [II-9]. The only difference is that PSAD2A calculates stresses and strains due to dual wheel configurations. This is done automatically by the program. Program characteristics which have been noted [II-9] include:

1. Three to 20 modulus-deviator stress relationships may be used as input.
2. The modulus-deviator stress curve may be negative or

flat.

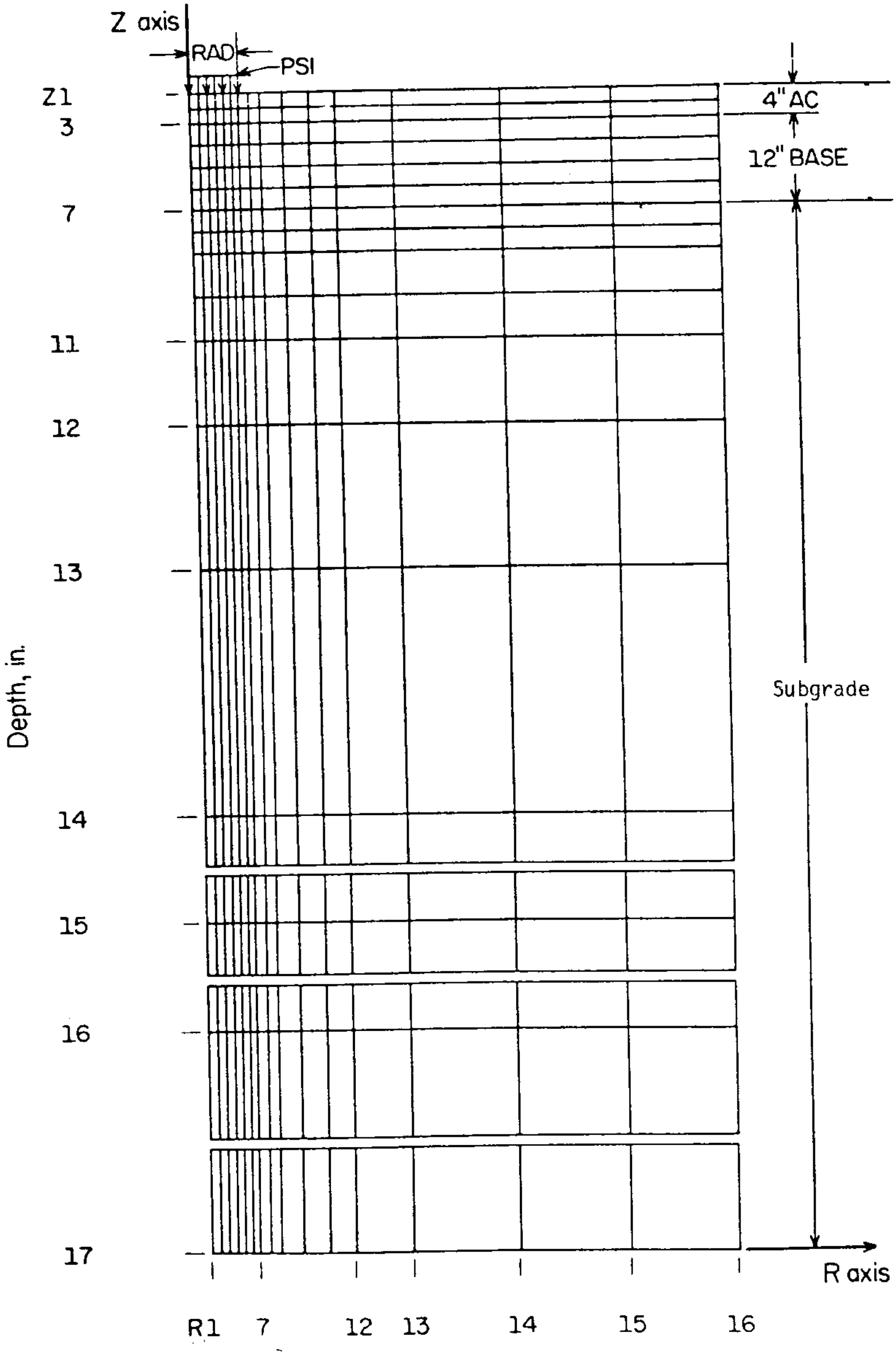
3. Five layers must be input.

4. Other characteristics of this program are listed under CHEV5L WITH ITERATION.

FINITE ELEMENT METHOD (4)

Finite element analysis [II-11, 12, 13] of a pavement system begins by defining the pavement in terms of elements which are connected by nodal points as shown in Figure II-4. The stiffness at each nodal point is calculated by means of assuming displacement variation within the element along with a knowledge of the stress-strain behavior of the element material. Equilibrium at each nodal point may be expressed by two equations. These equations use displacements and stiffnesses to define nodal forces. The equations are used to solve for the unknown displacements. Once the displacements at all of the nodal points have been calculated, the stresses and strains for each element may be computed [II-11].

The finite element method offers a means of solving practical problems (i.e. realistic problem geometry) by theory. The analytical procedure provides a very powerful tool for determining the mechanical behavior of a pavement structure because of the modeling flexibility; however, the two major drawbacks in using a finite element method are: 1) the storage capacity of the computer to handle large volumes of iterations and 2) the model requires an extensive amount of computer time.



Distance from Centerline - Inches

Figure II-4. Finite Element Configuration [after Reference II-12].

ILLI-PAVE

ILLI-PAVE is a finite element pavement analysis program developed at the University of Illinois [II-12]. Solutions are obtained for a two-dimensional half space of a finite solid. As shown in Figure II-5(a), the solutions may be applied to the three-dimensional model by rotating the section. The plane radial section is shown as a meshed rectangular half space in Figure II-5(b). The boundary conditions of this analysis are such that the inner and outer vertical boundaries can move only in the vertical. The lower boundary can move neither vertically nor horizontally.

The ILLI-PAVE finite element method of pavement analysis has the ability to incorporate both nonlinear and linear stress-strain behavior of the pavement materials. A loading condition is specified in terms of the surface contact pressure and radius of loaded area. The loading is of the "flexible plate" type and only one load can be accommodated.

Hoffman and Thompson [II-13] compared the use of the layered elastic theory (BISAR computer program) and the finite element method (ILLI-PAVE program) in calculating the pavement response under a 9,000 lb. wheel load (Figure II-6). They noticed that the two methods yielded an identical deflection basin but gave different stresses and strains at selected locations in the pavement structure. The table in Figure II-6 lists the differences between ILLI-PAVE and BISAR. It may be noted that the deflections obtained by both programs are very similar. However, the vertical stress at the top of the subgrade is approximately 5 psi lower as calculated by the finite element method.

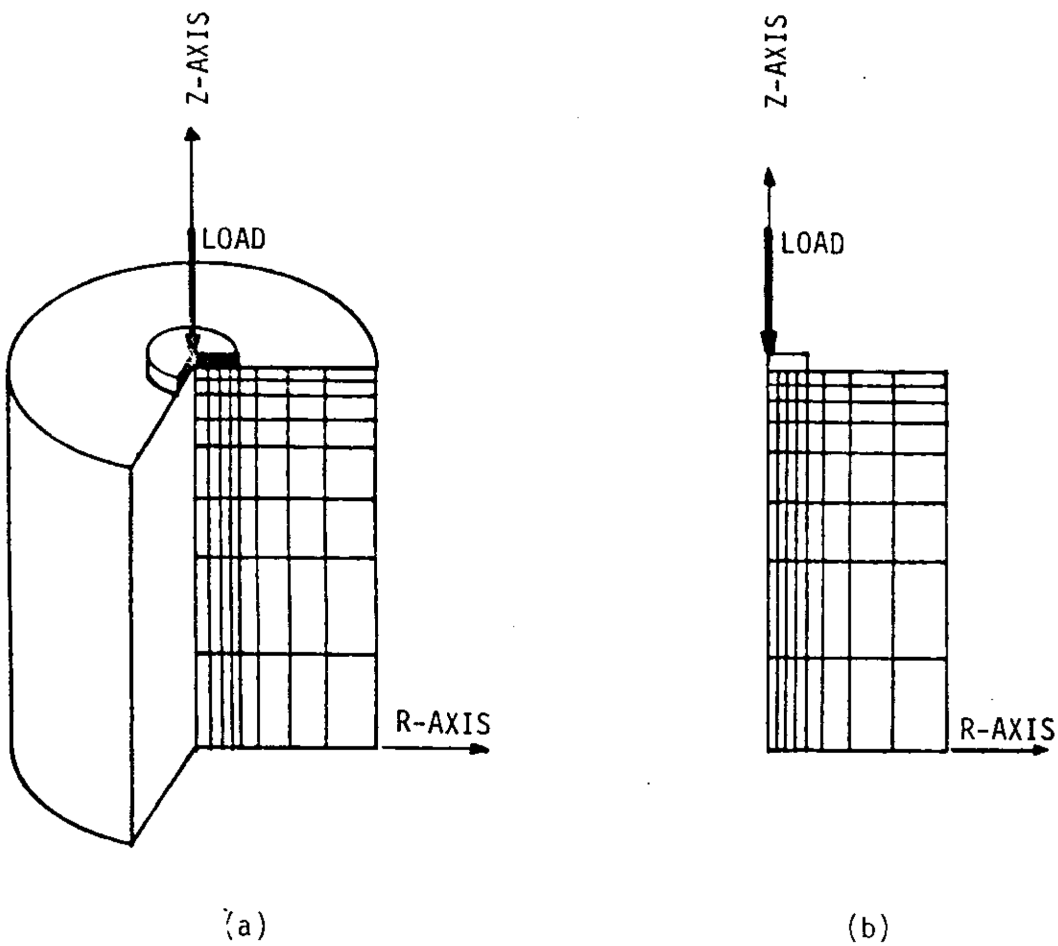
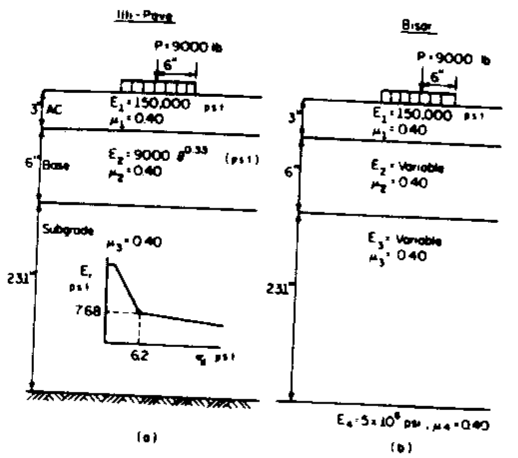


Figure II-5. ILLI-PAVE Model: a) Cylindircal Pavement Configuration and b) Rectangular Half Space of an Axisymmetric Solid [after Ref. II-5] [after Reference II-13].



Response Parameter	ILLI-PAVE	BISAR
D0, deflection at r = 0.0 in (mils)	37.86[a]	37.37
D1, deflection at r = 12.0 in (mils)	18.64	17.37
D2, deflection at r = 24.0 in (mils)	7.98	8.22
D3, deflection at r = 36.0 in (mils)	4.74	4.86
Deflection-basin area (in)	15.18[a]	15.00
Vertical stress, top of subgrade (psi)	15.80 (C)	20.90 (C)
Radial stress, bottom AC layer (psi)	40.00 (T)	112.40 (T)
Radial strain, bottom AC layer (0.0001 in/in)	4.90 (T)	5.90 (T)
Radial stress, bottom granular layer (psi)	3.80 (C)	14.10 (T)
Vertical strain, top of subgrade (0.001 in/in)	2.10 (C)	1.80 (C)
Deviator stress, top of subgrade (psi)	13.30	15.70
Deflection in AC layer (%)	1.00	1.50
Deflection in granular base (%)	20.00	25.50
Deflection in subgrade (%)	79.00	73.00

Note: (C) = compression; (T) = tension.

[a]Equal D0 and area are the basis of the "equivalent" pavement systems.

Figure II-6. Comparison Between BISAR and ILLI-PAVE Results for the same Pavement Cross-Section [after Reference II-13].

The radial stress at the bottom of the asphalt concrete surface was about three times greater in the BISAR solution. The radial stress at the bottom of the granular base was found to be 3.80 psi in compression for ILLI-PAVE and 14.10 psi in tension for BISAR.

ILLI-SLAB

This finite element program was also developed at the University of Illinois for cases involving portland cement concrete (PCC)[II-14]. It was specifically developed to provide analytical solutions for pavements having jointed or cracked slabs and various load transfer systems at these interfaces. The program is based upon classical theory of a medium-thick plate on a Winkler foundation. Cases which may be investigated with this system include:

1. Jointed concrete pavements with load transfer system.
2. Cracked, jointed concrete pavements.
3. Continuously reinforced concrete pavements.
4. Concrete shoulders with and without tie rods.
5. Concrete pavements with a stabilized base or an overlay.
6. Concrete slabs with differing thicknesses, moduli, and subgrade supports.

The ILLI-SLAB model and its components are shown in Figure II-7. The three displacement components are shown in Figure II-7a. These are vertical deflection (w) in the z-direction, a rotation about the x-axis (θ_x), and a rotation about the y-axis (θ_y). Dowel bars are modelled as shown in Figure II-7b. The displacement components at dowel bars are the vertical

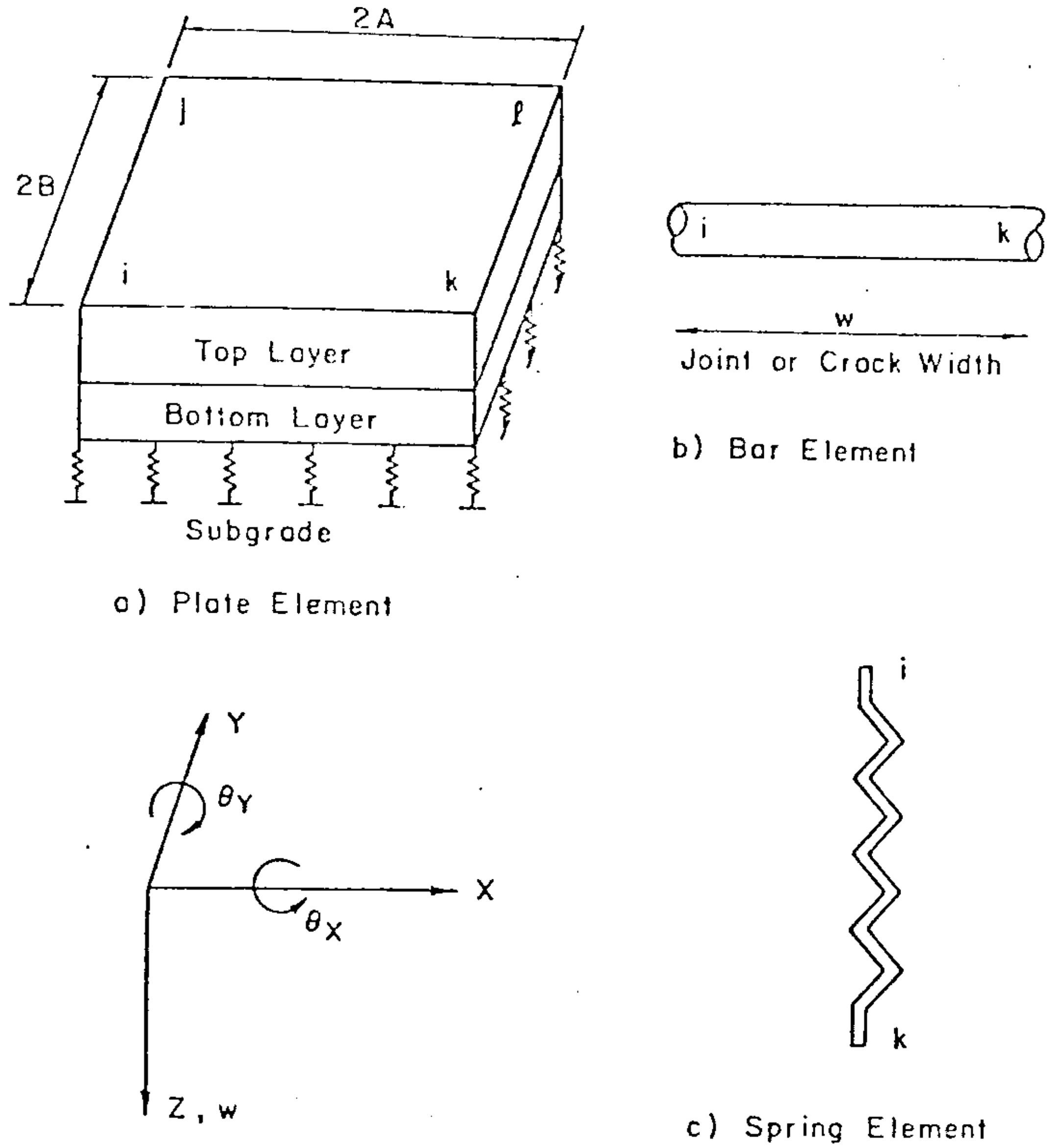


Figure II-7. ILLI-SLAB Model [after Reference II-14] deformation and rotation about the y-axis. A vertical spring element (Figure II-7c) is used to model the relative deformation of the dowel bar and surrounding concrete.

The pavement for analysis may consist of 1, 2, 3, 4 and 6 slabs with one longitudinal and two transverse joints. Slabs are divided into rectangular elements of various sizes and the joints are treated as rectangular elements of zero width. Wheel loads may be applied to any of the slabs. The thickness of the slabs, concrete modulus of elasticity, and the modulus of subgrade reaction may be varied at the nodal points. The output from the program includes stresses and deflections at all nodes in the slab, stresses in the stabilized base or overlay, vertical stresses in the subgrade and load transfer at the dowel bars.

LINEAR VISCOELASTICITY

Certain materials may exhibit combined solid-like and liquid-like characteristics even under small strain rates. If such material is subjected to a constant stress, it continues to deform slowly with time (creep). If it is constrained at a constant deformation, the required constraining stress diminishes gradually, or relaxes. When the applied stress on such a material oscillates sinusoidilly, the resulting oscillating strain is not in phase with the stress, but lags with an angle somewhere between zero and 90 degrees [II-15].

Linear viscoelastic models have undergone extensive development. An example of an available program which incorporates linear viscoelastic theory as the structural model is the one reported by Moavenzadeh and Elliot [II-15].

Temperature, humidity, and traffic are used as random variables in the model. The pavement system is a three-layer system in which a layer has properties which vary in a statistical manner. The primary material properties are considered to be creep compliance and Poisson's ratio. The creep compliance is the only property which is allowed to vary with environmental conditions. This viscoelastic model assesses pavement damage in terms of permanent deformation and fatigue cracking.

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CHAPTER III
NONDESTRUCTIVE TESTING

INTRODUCTION

As stated by Monismith and Finn [III-1], pavement overlay designs may be accomplished by the following methods:

1. engineering judgment,
2. component analysis,
3. nondestructive testing with a limiting deflection criteria, and
4. mechanistic interpretation of nondestructive testing data with a mechanistic failure criteria.

Nondestructive testing procedures have become increasingly popular since their introduction in the early 1960's. The concept of mechanistic overlay design procedures has recently been introduced with widespread support. However, none of these procedures have yet addressed the potential of newly developed paving materials and techniques.

Engineering judgment is usually employed by people within an agency who are responsible for pavement maintenance. In this process, the engineer bases the overlay design upon the existing pavement's performance and condition; particular site conditions, e.g., geometrics; and available funding [III-1].

In component analysis, the structural value of an existing pavement cross section is compared to that for a new pavement relative to traffic and other site specific variables. The AASHO pavement design method is an example of component analysis. A considerable amount of judgment is required in this method in order to evaluate the structural coefficients for pavement layers, e.g., cracked asphalt concrete, and degraded base and subbase. Additionally, component analysis often relies on laboratory procedures to predict the equilibrium conditions of materials, e.g., R-values, density, water content, etc.

The structural integrity of a pavement system may be evaluated through the interpretation of deflection data obtained by nondestructive means [III-1]. These data reflect material properties in existing systems under actual conditions. Limiting deflection criteria are chosen in accordance with the design procedure to be used as well as a visual survey of the pavement condition. Empirical relationships between deflection data and observed pavement performance are used to establish overlay thickness requirements. Due to the empirical nature of each method, good results are usually obtained only when there is strict adherence to the method being used. Combinations of criteria or procedures adopted by different agencies are to be avoided in order to prevent erroneous conclusions.

Recently interest has been developed in the use of mechanistic overlay design procedures. Monismith and Finn [III-1] have stated that these methods are quasi-mechanistic and emphasize that they are dependent upon empirical relationships in the establishment of specific design criteria. The greatest advantage of such methods is their versatility in evaluating different materials under various environments and pavement conditions. The mechanistic procedures provide a basis for rationally modeling pavement systems. As these models improve, better correlations should be forthcoming between design and performance parameters. The general consensus is that these procedures will replace the limiting deflection methods since the latter do not account for subsurface material properties [III-1].

LIMITING DEFLECTION-BASED OVERLAY DESIGN PROCEDURES

Overlay design procedures which are based on deflection measurements have been developed by agencies such as the California Department of Transportation, Kentucky Department of Transportation, U.S. Corps of Engineers, Asphalt Institute, Transportation Research Board, and Canadian Good Roads Association [III-1]. A considerable amount of research effort has been directed toward the development of correlations between deflection measurements and pavement performance. Table III-1 lists the agencies that have developed and used limiting deflection criteria for overlay design. Figure III-1 outlines the general approach used in most of the overlay design procedures based on deflection measurements. The three basic elements of such design procedures are: 1) deflection measurements, 2) pavement condition, and 3) traffic.

Deflection Testing

Currently, deflection data are used primarily to evaluate the overall pavement system response to load. The influence of variables such as layer thicknesses, material properties, and environmental effects are considered collectively [III-1]. Most deflection based design procedures do not routinely attempt to isolate material properties of individual pavement layers.

Table III-1. Limiting Deflection Criteria for Pavement Evaluation [after Ref. III-2]

Reference	Deflection Criteria	Remarks
WAASHO [III-2]	Spring Δ_{max} = 45 mils Fall Δ_{max} = 35 mils	Conventional flexible pavements. Deflections measured under 18 kip axle.
Hveem [III-3]	$\Delta_{all} \leq$ 50 mils (1) $\Delta_{all} \leq$ 17 mils (2)	(1) Surface treatment; (2) AC layer thickness = 4 in. Deflections measured under 15 kip axle. Δ_{all} = allowable maximum deflection
Carneiro [III-4]	20 mils $\leq \Delta_{max} \leq$ 35 mils	Conventional flexible pavements. Benkelman beam deflections under 18 kip. axle, 80 psi tire pressure.
Whiffin et al [III-6]	20 mils $\leq \Delta_{max} \leq$ 30 mils (1) 5 mils $\leq \Delta_{max} \leq$ 15 mils (2)	(1) Asphalt concrete over granular base. (2) Asphalt concrete over cement treated base. Traffic volume considered. Benkelman beam deflections under 14 kip axle, 85 psi tire pressure.
State of California [III-7]	Δ_{all} = f(Tac,N)	Δ_{all} = Allowable maximum deflection Tac = Thickness of AC layer N = Number of repetitions of a 5 kip EWL Examples: Δ_{all} = 60 mils for Tac = 1.5 in. and N = 10,000 Δ_{all} = 37 mils for Tac = 1.5 in. and N = 10^6 Δ_{all} = 46 mils for Tac = 6 in. and N = 10,000 Δ_{all} = 22 mils for Tac = 6 in. and N = 10^6
Asphalt Institute [III-8]	Δ_{all} = f(DTN, Temp)	DTN = Design traffic number = average daily 18 kip axle loads Δ_{all} = Allowable maximum deflection (plus two standard deviations) Examples: Δ_{all} = 22 mils for DTN = 1000 Δ_{all} = 100 mils for DTN = 2 Benkelman Beam Deflections
Lister [III-9]	N = f($\Delta_{in.}$, pavement type)	N = Cumulative number of 18 kip axle repetitions $\Delta_{in.}$ = Initial Benkelman beam deflection (14 kip axle) Graphical relations between N and $\Delta_{in.}$ for different pavement types. For AC pavement with granular base layer: $\Delta_{in.}$ = 20 mils; N = 4.5x10^6 $\Delta_{in.}$ = 40 mils; N = 0.5x10^6
Maguro et al [III-10]	$\log N = 0.179\Delta^2 - 1.117\Delta + 6.772$	N = Number of repetitions to failure of heavy loads (over 18 kip) Δ = Benkelman beam deflections
Joseph and Hall [III-11]	$\Delta = 1.1315/N^{0.233}$	Δ = Initial deflection (mils) under a given load N = Repetitions to failure of that load

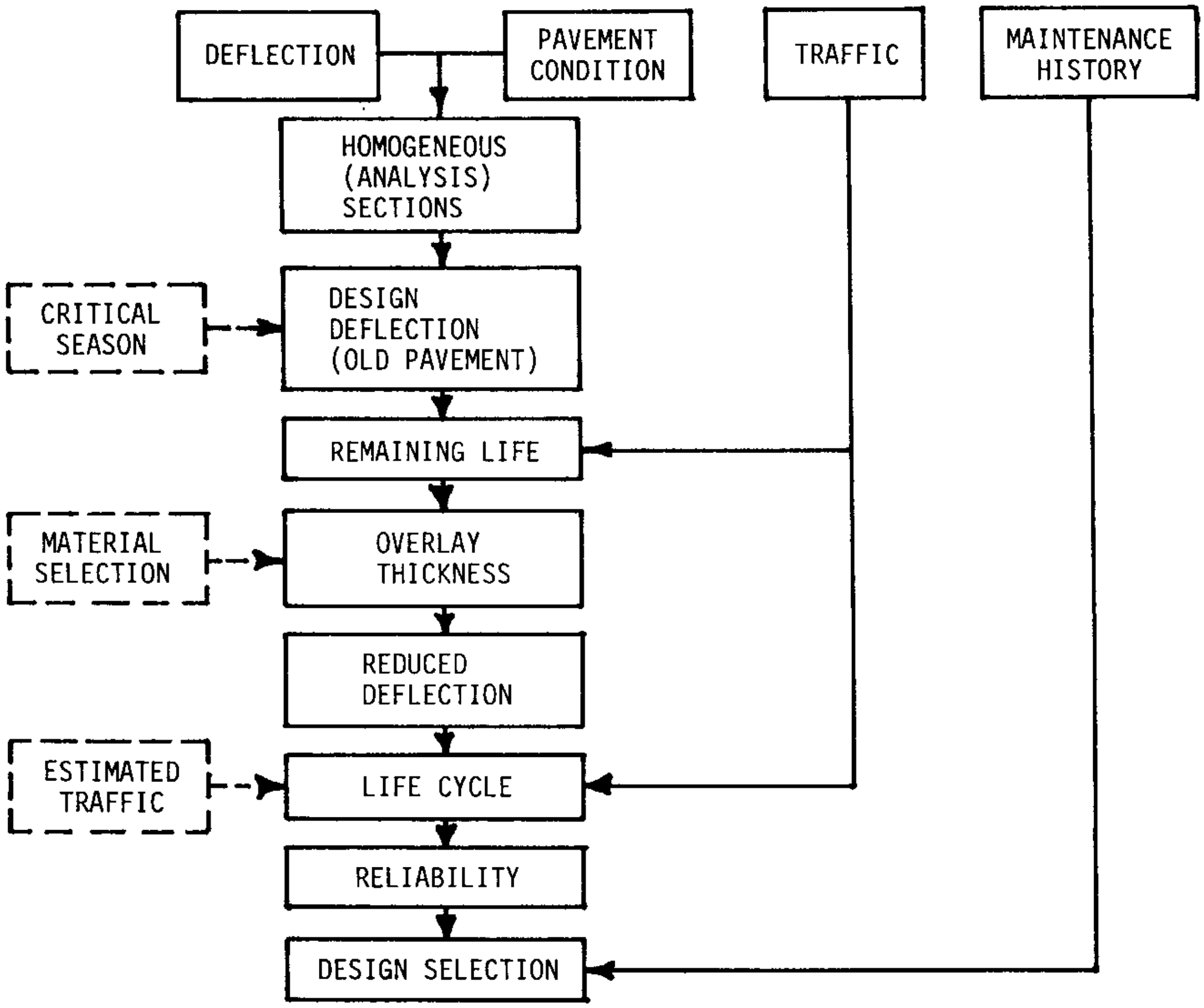


Figure III-1. Overlay Design with Deflection Measurements (Non-Destructive Testing) [after Ref. III-1]

Tables III-2 and 3 list a variety of pavement analysis procedures along with the appropriate deflection testing equipment for a given procedure. All of these procedures are based upon surface deflection measurements made under known loading conditions such as contact pressure, force, and loading time. Deflection measurement devices include the Benkelman Beam, Dynaflect, Deflectometer, Road Rater, and Falling Weight Deflectometer. Correlations have been developed for data gathered by different methods [III-1]. California Test Method 356 describes four of the deflection measuring devices. These descriptions are presented below to acquaint the reader with them.

Benkelman Beam. As shown in Figure III-2, this device based upon a simple lever arm principle. A probe at the end of an eight foot beam is placed between the dual tires of an 18,000 lb. single rear axle of a truck. As the truck moves toward the end of the beam, the pavement is depressed and the beam pivots about a reference point. Readings are taken from a dial guage measuring the difference between the reference and rotating beams. The primary disadvantage of this test is that the rate of loading is limited to creep. The advantages include simplicity versatility, and rapidity of measurements [III-1].

Dynaflect. The Dynaflect measures the deflection of a pavement surface produced by an oscillating load superimposed on a static load. Counter-rotating eccentric flywheels produce the dynamic force and a series of five geophones measure the resulting deflection. The pavement surface deflection is then read on instrumentation inside of the tow vehicle [III-1].

Table III-2. Flexible Pavement Analysis Methods Based on the Interpretation of Measured Deflection Basins [after Ref. III-2]

	Pavement Model	NDT Method Used	Required Input for Analysis	Method of Analysis	Output from Analysis
Scrivner et al [III-13]	Two-layer linear elastic; bottom layer of infinite depth. Point Load.	Dynaflect	Two deflections from geophones 1 & 3. Thickness of upper layer. $\mu_1 = \mu_2 = 0.5$	Computer Program	E_1, E_2
Swift [III-14]	Two-layer linear elastic; bottom layer of infinite depth. Point load.	Dynaflect	Two or more deflections. Thickness of upper layer. $\mu_1 = \mu_2 = 0.5$	Graphical fitting between measured and computed deflection basin.	E_1, E_2
Moore [III-15]	Two-layer linear elastic; bottom layer of infinite depth. Point Load.	Dynaflect	Five deflections. Thickness of upper layer. $\mu_1 = \mu_2 = 0.5$	Fitting of measured deflections to an approximate equation derived by Swift (41).	E_1, E_2
Cogill [III-16, 17]	Five-layer linear elastic; bottom layer of infinite depth. Point Load.	Dynaflect	Five deflections. Thickness of four upper layers, Poisson's ratios of the materials.	Iterative solution of 5 equations (deflections) with 5 unknown (E-values)	E_1 to E_5
Paterson & Van Vuuren [III-18]	Five-layer linear elastic; bottom layer of infinite depth; uniform load over a circular area.	Benkelman Beam on surface. 8 LVDT's at different depths.	Deflection basin and deflections with depth. Thickness of layers and Poisson's ratio.	Iterations of the horizontal and vertical deflections and comparison with computed values.	E_1 to E_5
Vaswani [III-19]	Linear elastic point load and uniform load on circular area with radius a = 6.4 in. and p = 70 psi.	Dynaflect or Benkelman Beam (Theoretical)	Five deflections at given distances.	Nomograph relating Δ_{max}, the spreadability, and theoretical thickness index of pavement.	E of subgrade and E of upper pavement.
Wiseman [III-20]	Hogg model; $h/\ell_0 = 10$; load of any shape with influence charts.	Benkelman Beam, Road Rater, Plate bearing test (theoretical)	Two deflections at given distances; $\mu_2 = 0.5$	Grapho-analytical solution relating measured values with model parameters.	E of subgrade and D-flexural stiffness of upper pavement.
Claessen et al [III-21]	Three-layer linear elastic. $E_2 = 0.206\ h_2^{0.45} E_3$ (h_2 in mm) Uniform load on circular area.	Falling-weight deflectometer, Benkelman Beam, Deflectograph (Lacroix)	Two deflections at given distances $\mu_1 = \mu_2 = \mu_3 = 0.35$	Grapho-analytical solution of three layer system. Relationships between E_1, E_3, h_1, & h_2	Two of four parameters: E_1, E_3, h_1, or h_2
Grant & Walker [III-22]	Three-layer linear elastic. Bottom layer of infinite thickness. Uniform load over circular area.	Benkelman Beam, Curvature meter, LVDT's thru depth.	Radius of curvature, Δ_{max}, and deflections with depth.	1) Relation between the curvature and Δ_{max} as a function of E_2/E_3. E_1 = constant. 2) Iterations of deflections thru depth. Fitting between measured and computed values.	E_1, E_2, E_3
Kooie [III-23]	Three-layer linear elastic. $E_2 = 0.2\ h_2^{0.45} E_3$ (h_2 in mm) Uniform load on circular area.	Falling-weight deflectometer	Two deflections at given distances $\mu_1 = \mu_2 = \mu_3 = 0.35$ E_1 and h_2	Nomographs relating E_1, Δ_{max}, h_1, and Δ_r/Δ_{max}. (r is normally 2 feet)	E_3 and h_1 "effective"
Treybig et al [III-24]	Linear elastic layered theory.	Dynaflect (recommended)	Moduli of elasticity of pavement layers, and layer thicknesses. Dynaflect deflection.	Nomograph	Modulus of elasticity of subgrade
Sharpe et al [III-25]	Three-layer linear elastic. $E_2 = f \times E_3$	Road Rater	E and thickness of AC layer. Thickness of granular layer. RR deflections at 0,1, and 2 feet.	Graphical	E of the subgrade
Wiseman et al [III-26]	Two-layer linear elastic and Hogg model. Influence charts. Bottom layer underlain by a rough rigid base.	Road Rater and Benkelman Beam	Two deflections in the deflection basin. Thickness of upper layer.	Nomograph	E_1 and E_2 or ℓ_0 and E_2 (Hogg)

Table III-3. Recent Pavement Analysis Methods Based on the Interpretation of Measured Deflection Basins

Reference	Pavement Model	NDT Method used	Required Input for analysis	Method of Analysis	Output from analysis
Bush [III-26]	Four layer linear elastic theory	FWD	Four Deflections layer thicknesses	Computer Program (BISDEF)	E_1, E_2, E_3, E_4
Kilareski Anani [III-27]	Four layer linear elastic theory	Road Rater	Four deflections layer thicknesses	Computer program	E_1, E_2, E_3, E_4
Thompson and Hoffman [III-1, 28]	Finite element-non-linear elastic	Road Rater	Five deflections layer thicknesses non-linear material constants	Computer program or chart form	E_1, E_2, E_3
Little [III-29]	Linear elastic theory	Dynaflect	Five deflections layer thicknesses	Computer program or chart form	$E_{subgrade}$
Ullidtz [III-30]	Non-linear elastic theory	FWD	Seven deflections layer thickness	Computer program (ELMOD)	E_1, E_2 and E_3

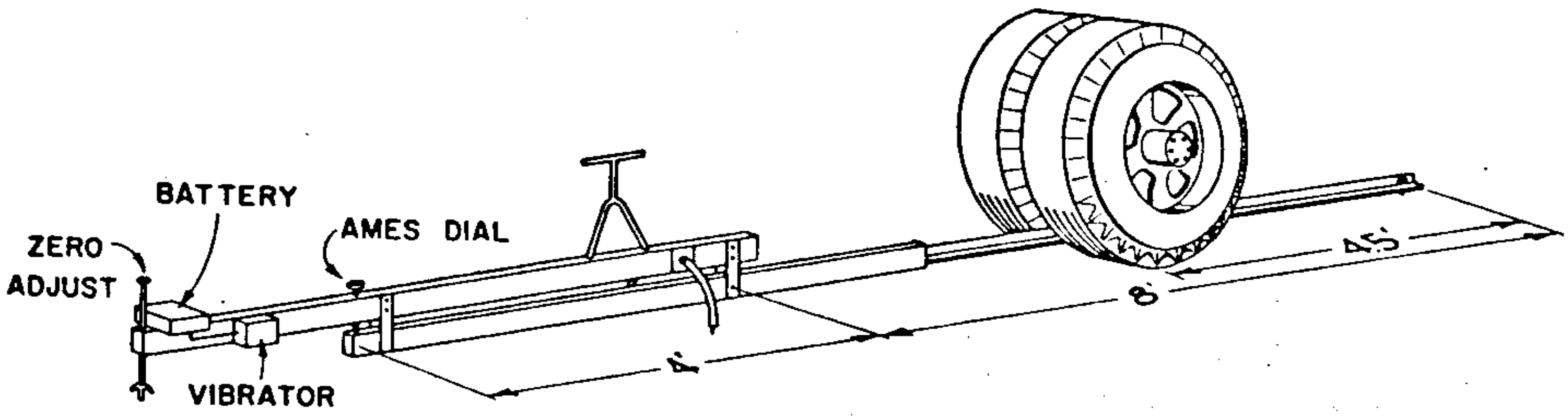


Figure III-2. Sketch of a Benkelman Beam [Ref. III-31]

Road Rater (Model 400). Like the Dynaflect, the Road Rater exerts an oscillating load on the pavement surface to cause deflections. The deflections are measured by two transducers located at the center of the load and at a distance of 12 in. from the load. Deflection signals are sent to a control panel in the tow vehicle [III-1].

Falling Weight Deflectometer (FWD). This device applies a pulse load to a pavement surface by means of a drop weight. The weight falls on a set of springs or rubber buffers which transmit the load. Five geophones are located at various distances from the point of load application to measure deflections. The greatest advantage to this device is that the weight, springs, and falling height may be varied to give a range of force levels and loading times [III-1].

MECHANISTIC INTERPRETATION OF NONDESTRUCTIVE TESTING

The results of nondestructive testing may be interpreted either empirically or mechanistically. Empirical approaches which have been established are the result of experience and observation of conditions for a specific locale. While these methods have proved useful, they lack the general applicability of mechanistic procedures.

Mechanistic procedures use NDT results to estimate in-situ material properties of the pavement system. Specifically, the stiffness properties may be inferred from test results. NDT results may be compared with the results of laboratory characterization to further refine estimates of material properties. An overlay design procedure using this approach is illustrated in Figure III-3.

Nondestructive Evaluation

The majority of NDT equipment in use is of the deflection measuring types discussed earlier. Recently, wave propagation devices have been developed for NDT purposes. Monismith and Finn [III-1] advise against the use of NDT results only in the structural evaluation of pavements. They recommend laboratory testing representative samples of pavement materials within the scope of the procedure shown in Figure III-3.

Establishment of Analysis Sections

The condition of the pavement to be overlayed should be carefully measured and documented. By identifying different types and levels of distress, analysis sections and performance criteria may be established. In order to ascertain the pavement response to load, NDT measurements shoulb be taken at selected intervals throughout the analysis sections. The NDT data should be treated statistically and an appropriate comparison test (e.g., Student t-test) should be conducted to judge whether deflection data from adjacent sections are significantly different [III-1].

Once the sections have been chosen and tested, it will be necessary to select a design value for deflections after overlay. A good method for doing this is to select a deflection value which will be equal to or greater than 80 to 90 percent of deflection measurements after overlay [III-1].

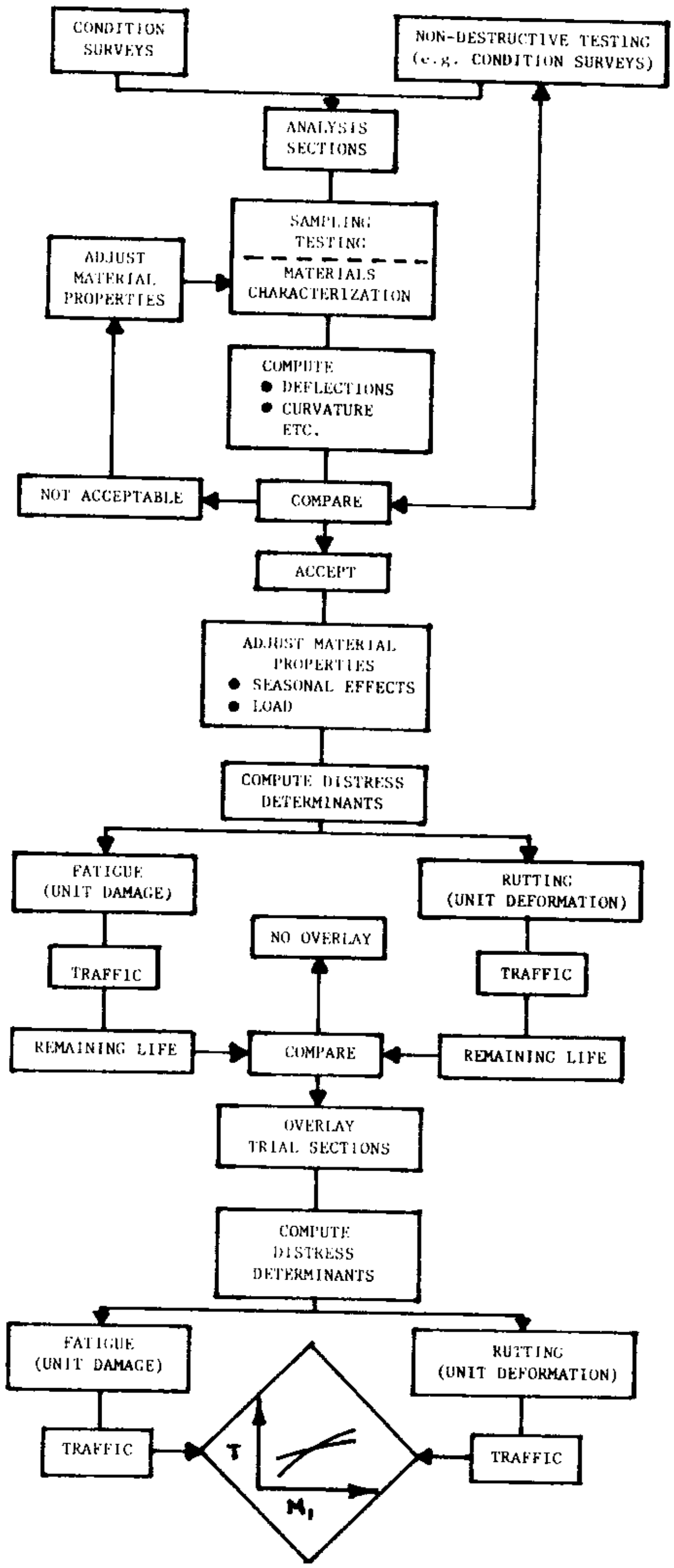


Figure III-3. Overlay Design Based on Mechanistic Analysis [after Ref. III-1]

Establishment of Laboratory Material Characteristics

In this portion of the overlay design procedure, thicknesses and properties of the various pavement layers are measured. The focus of laboratory testing should be directed toward the determination of elastic moduli. Once the elastic moduli of the layers have been estimated in the laboratory, they may be used in layered elastic analysis to ascertain their response to a known load. The deflection from the analysis may be compared to the NDT deflection. Layer stiffness values may be adjusted until the results of the laboratory and NDT show a reasonable agreement [III-1].

It must be recognized that some unbound materials will exhibit a stress sensitivity. This stress sensitivity can be measured in the laboratory and modulus values may be adjusted to reflect material behavior under actual traffic conditions [III-1].

Parameters to Describe the Shape of the Deflection Basin

The importance of deflection basin shape in the interpretation of measured surface deflections has long been recognized. Initially, only qualitative statements could be made about the relationship of the basin shape to the pavement structural condition. An early publication noted the following observations regarding deflection basin shape resulting from Dynaflect measurements (III-33):

1. A concave shape indicates the surface is the weakest layer in the total system.
2. A convex shape indicates the base is the weakest layer in the total system.

3. Steep slopes indicate weakness.

4. Flat slopes indicate strength.

5. Sensors closest to the force wheels provide knowledge of the surface layer.

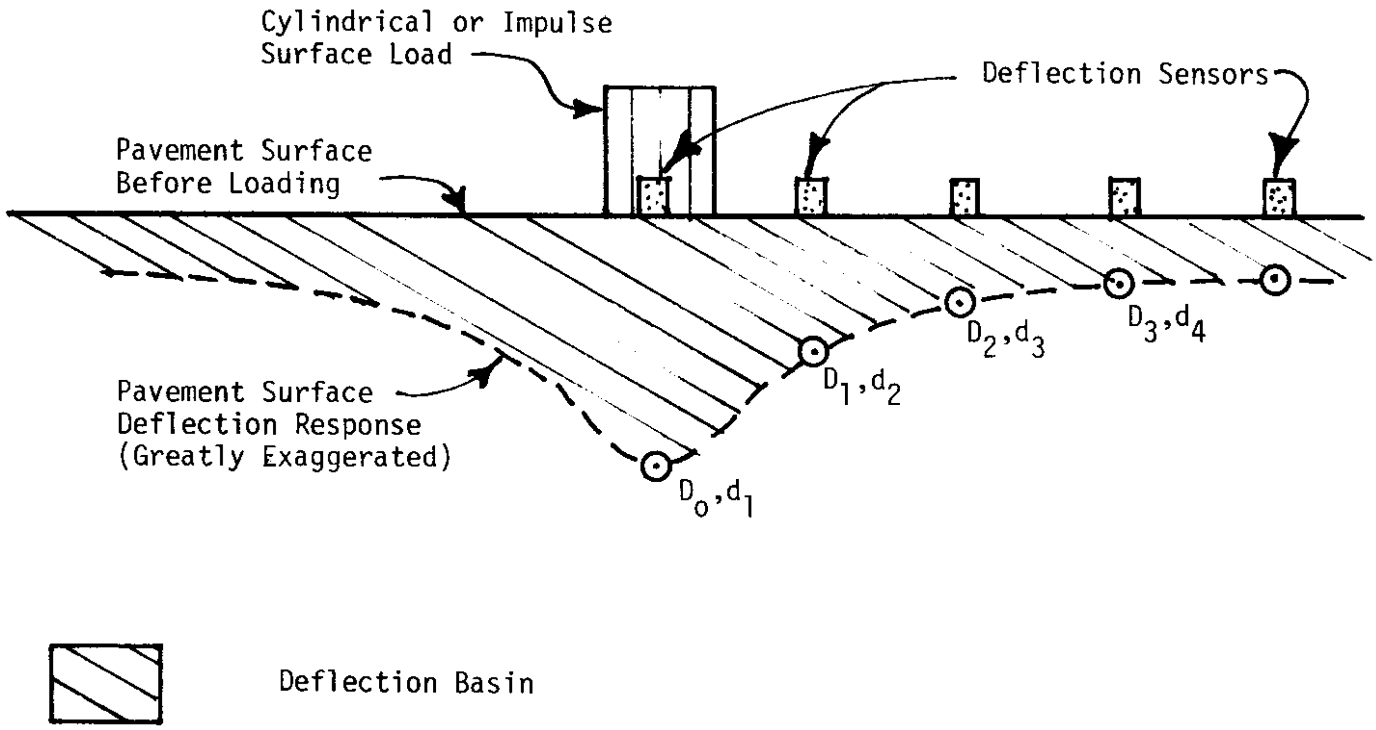
6. Sensors furthest from the force wheels indicate the strength feature of supporting layers.

Over the last decade, quantitative descriptions of the deflection basin shape have emerged in the form of calculated deflection basin parameters. When stressed by a static, dynamic, or impulse load, the pavement typically responds with a surface deflection basin which is bowl-shaped, as shown in Figure III-4. Maximum deflection typically occurs at the center of loading. Surface deflection is measured by sensors located in a straight line, parallel to the direction of traffic, usually at fixed distances from the center of loading.

Several parameters have been presented in the literature to quantitatively describe the shape of the basin, in addition to the limiting curvature criteria. These parameters include: Spreadability, Area, Shape Factors, Surface Curvature Index, Base Curvature Index, Qr, and Projected Deflection. Brief descriptions of these parameters follow.

Spreadability. The spreadability concept was introduced by Vaswani [III-18] as a parameter to describe the Dynaflect deflection basin. It is the average deflection, expressed as a percentage of the maximum deflection, and is calculated by:

$$S = \frac{d_{max} + d_1 + d_2 + d_3 + d_4}{5}$$

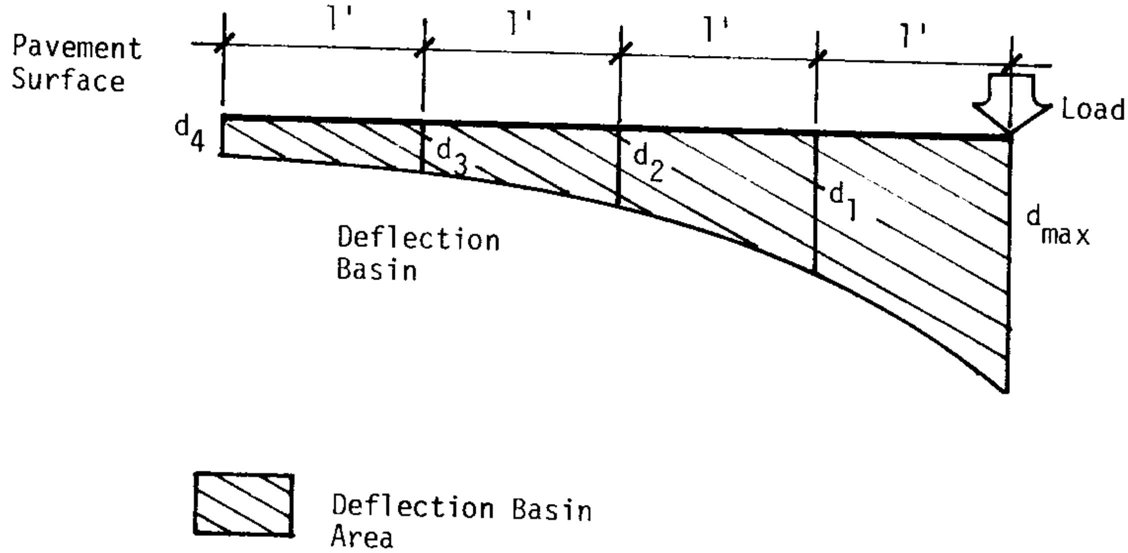


D_o, D_1, D_2, D_3 - Used by Hoffman and Thompson to denote FWD and IDOT Road Rater Deflections

d_1, d_2, d_3, d_4 - Used by Wang, et al. denoting Road Rater Deflections

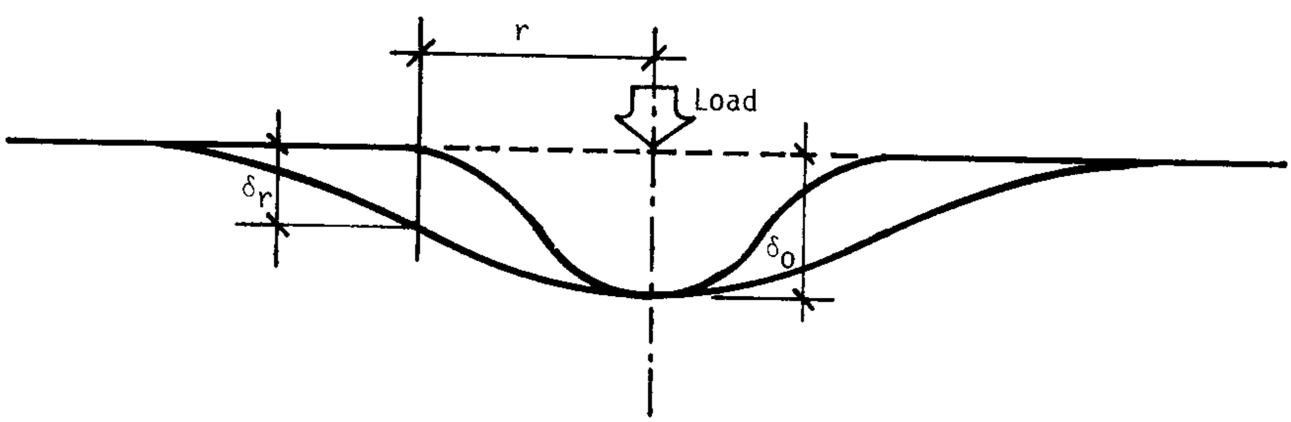
(a) Notation Used by Hoffman and Thompson [III-2] and Wang, et al. [III-35]

Figure III-4. Deflection Basin Measurement Notation [after Ref. III-34]



(b) Notation Used by Vaswani [III-36] and by Utah [III-37] and others to Describe Dynaflect Deflection Measurements

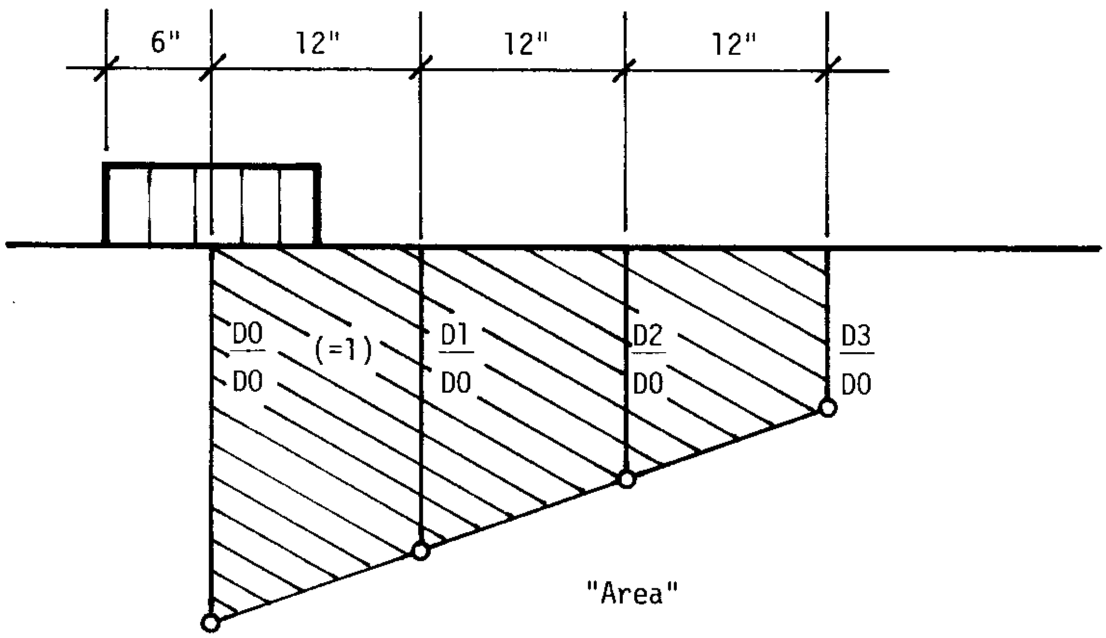
(Figure from Reference III-36)



(c) Notation Used by Claessen [III-20] and Koole [III-22] to Describe Falling Weight Deflectometer Deflections for Calculating Deflection

(Figure from Reference III-22)

Figure III-4.(Cont.).



$$\text{Area (in.)} = 6\left(1 + 2\frac{D1}{D_0} + 2\frac{D2}{D_0} + \frac{D3}{D_0}\right)$$

(d) Deflection Basin Area Parameter as Defined by Hoffman and Thompson [III-2]

Figure III-4.(Cont.).

where:

S = spreadability,

$d_{max}, d_1, d_2, d_3, d_4$ = deflections at Dynaflect sensors (see Figure III-4b)

Area. The area, bounded by the undeflected pavement surface on the top, the deflected basin curve on the bottom, and the deflections at each end of the basin, has also been used by Vaswani [III-36] to describe the Dynaflect deflection basin. The shaded portion of Figure III-4b illustrates the area parameter, which is calculated by:

$$A = 6(d_{max} + 2d_1 + 2d_2 + 2d_3 + d_4)$$

where:

A = deflection basin area (sq. in.),

$d_{max}, d_1, d_2, d_3, d_4$ = deflections at Dynaflect sensors

Based on the correlation study by Hughes [III-38], Vaswani [III-36] uses the relationship that the estimated area under a 9,000 lb. wheel load is 28.6 times that under Dynaflect loading.

Hoffman and Thompson [III-2] have expressed the area parameter in a slightly different fashion by normalizing the deflection basin curve with respect to D_o, as illustrated in Figure III-4d. The following equation is used to calculate the area parameter using Hoffman and Thompson's definition:

$$A = 6(1 + 2(D_1/D_o) + 2(D_2/D_o) + (D_3/D_o))$$

where:

A = deflection basin area (in.),

D_o, D_1, D_2, D_3 = deflections at FWD sensors (Figure III-5a)

The area expressed in this fashion has an upper and lower bound of 36 and 11.1, respectively. The upper bound is obtained when all deflection values are equal, while the lower bound is obtained when an elastic half-space model, the Boussinesq model, is assumed. The stiffer the pavement, the larger the area.

The area parameter introduced by Hoffman and Thompson is related to the Spreadability parameter by the following equation:

$$A = 0.24(S) + 6(D_1 + D_2)/D_o)$$

where:

A = area parameter (in.),

S = spreadability (%), and

D_o, D_1, D_2 = deflections at FWD sensors (Figure III-4a).

<u>Shape Factors</u>. Hoffman and Thompson [III-2] have defined two shape factors to describe deflected basin shape. Denoted F1 and F2, they are defined as follows:

$$F1 = (D_o - D_2)/D_1$$

$$F2 = (D_1 - D_3)/D_2$$

where: D_o, D_1, D_2, D_3 are as defined previously.

<u>Surface and Base Curvature Index</u>. These factors were developed during Dynaflect testing in Utah [III-37] and are denoted as SCI and BCI, respectively. The indices represent differences between successive deflection readings in the deflection basin, and are calculated as follows:

$$SCI = d_{max} - d_1$$

$$BCI = d_2 - d_3$$

where: d_{max}, d_1, d_2, d_3 are as defined previously.

Qr. This is a value developed by Claessen [III-20] and represents the ratio of the deflection at some distance "r" from the center of loading to the deflection under the center of the load (Figure III-4c)

$$Q_r = \delta_r / \delta_o \tag{9}$$

where: Q_r = deflection basin shape parameter,

δ_r = deflection at some distance "r" from the center of the load,

δ_o = deflection under the center of the load.

Projected Deflection. The concept of projected deflection has been introduced in research conducted by the Kentucky DOT [III-24, 39] to assist in describing the deflection basin. Figure III-5 illustrates the concept of projected deflection. Deflections from the three sensors closest to the loading are graphed on a semi-log plot in which arithmetic values of distance from loading are plotted against the log of deflection. A straight line is drawn through the points corresponding to the two sensors furthest from the load. The intersection of this line with the center of loading is the value of the No. 1 "Projected Deflection".

Comparing the difference between the "measured" and "projected" No. 1 deflection (both magnitude and direction) provides information regarding the shape of the deflection basin and the structural condition of the pavement. The following can be used for determining the No. 1 Projected Deflection.

No. 1 Projected

$$= 10^{(2 \log \text{No. 2 Deflection} - \log \text{No. 3 deflection})}$$

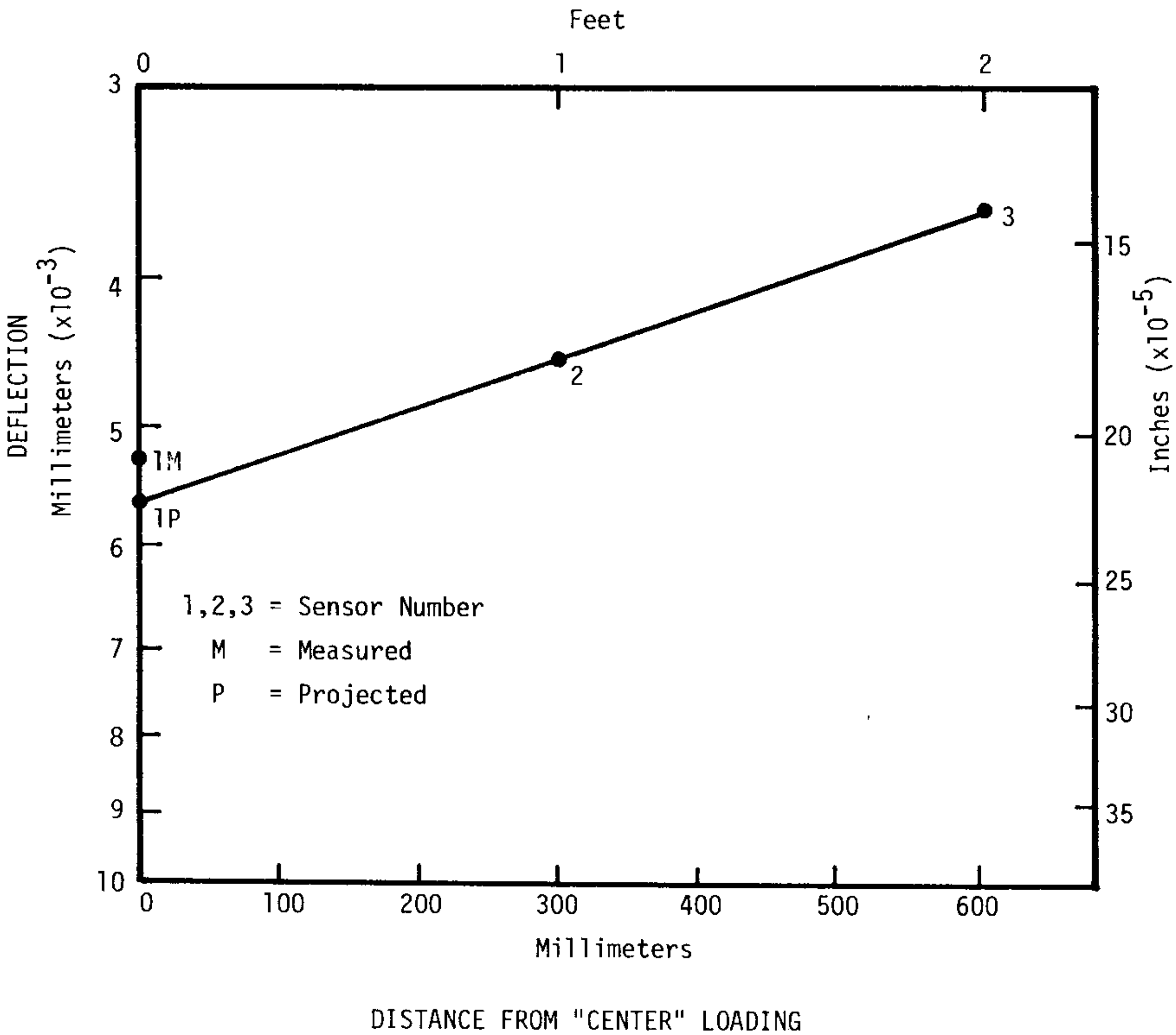


Figure III-5. Determination of No. 1 Projected Deflection [after Ref. III-24]

Techniques Incorporating Layered Elastic Theory to Analyze and Interpret Measured Surface Deflection Basins.

A considerable number of methods have emerged over the past fifteen years for analyzing measured surface deflection basins. A partial listing of methods is presented in Tables III-2 and 3. Review of these tables yields the following observations about common characteristics:

1. The methods use two to five deflection measurements to describe the deflection basin.
2. The linear elastic pavement models have two to five layers, thus, two to five moduli of elasticity are backcalculated.
3. Some of the methods are developed for a specific nondestructive testing device.
4. There are two basic methods of analysis, computerized iterative solutions and graphical fitting or nomographs.

Limitations to Layer Elastic Theory. Linear elastic layer theory is not without its limitations. Some of these limitations or unknowns include [III-40]:

1. inability to analyze effects of loads at discontinuities (e.g. cracks or edges),
2. validity of assumptions regarding interface conditions, and
3. each layer is assumed to be linear elastic.

Probably the most important limitation is the latter. It is generally accepted that most materials which typically comprise the pavement section, particularly untreated bases and subgrades, exhibit non-linear stress-strain behavior during laboratory testing [III-41, 42, 43, 44, 45]. Therefore, modulus is a function of the stress level. The implications of this fact are that the modulus will change with depth and lateral position.

Recent research at Illinois [III-2] has demonstrated that the relationship between the load applied at the surface of a pavement structure and the resulting vertical deformation is non-linear for Road Rater and Falling Weight Deflectometer loadings. Pavements in their study responded to increased loads with even greater increases in deflection. When loads increased from 2 kips to 10 kips, pavement stiffness (the ratio of load to deflection) decreased on the order of 20 to 60 percent for the Road Rater and 10 to 20 percent for the FWD.

Two approaches can be taken in trying to account for non-linearity in materials and in pavement response to loading. One is to use non-linear finite element theory [III-41, 46] (recall that the basis for finite element computations were presented in Chapter II). In this procedure, stresses and strains are calculated using assumed moduli. The calculated stresses are then used to estimate a new stress-dependent modulus from experimentally measured material properties. Additional stress states are calculated and the process is repeated by an iterative or incremental procedure. In both cases (iterative or incremental), the modulus is matched with the stress state in each element [III-40].

An alternate approach is the use of non-linear, iterative elastic layer solutions [III-41, 47, 48]. In this approach, the base and subbase are subdivided into several fictitious layers for better accuracy. A modulus in each layer is used which is dependent on the average stress state that exists in that layer beneath the wheel loadings.

In spite of the limitations and assumptions, the number of procedures given in Tables III-2 and III-3 would indicate that most researchers feel that layer elastic theory is sufficient as a pavement model to interpret measured basins.

Swift [III-13] has reported that measurements of deflections of pavement structures have been noted to bear a strong resemblance to the deflections computed for layered elastic system.

Brands and Cook [III-49] state that the structural parameters of a pavement are sufficiently linear over a broad enough range that the energy or force used in deflection or impulse testing need not be as great as in previously accepted methods. The authors state that other research supports this claim [III-50, 51]. Brands and Cook have assumed linearity over an extremely wide range, since their testing device for pavement evaluation supplied an impulse load.

<u>Stress Dependent Linear Elastic and Stress Dependent Finite Element Models</u>

Research performed in Florida [III-52] and Illinois [III-2, 27, 53] has criticized the use of non-stress dependent elastic layer theory to model pavement systems with sufficient accuracy for evaluation and design.

The stress-dependent linear elastic model reported by Sharma and Stubstad [III-51] represents a compromise between the more desirable stress-dependent finite element model and a linear elastic model. The authors make the following statement:

> "If a linear-elastic program, in which calculated versus measured deflections are matched by juggling E-values (a common procedure) is used to model the pavement system, gross errors will result even if the stress dependent nature of the materials (especially the semi-infinite subgrade) are comparatively minimal."

This led these researchers to the development of ISSEM4 (a reverse iterative version of ELSYM5). It was modified to backcalculate material properties based on stress levels, recalculate modulus values, and recompute stresses.

In their research, Hoffman and Thompson [III-2] stated that the non-linearity of materials and of pavement response poses a number of problems. Pavement parameters derived from low loading magnitudes are inadequate for the analysis of higher traffic loads and linear elastic parameters are insensitive to changing stresses effected by changes in geometry. When overlays are placed, the stress state, and hence modulus, are changed in the lower layers. This is not taken into account in linear elastic layered pavement models. Their research work has resulted in a series of algorithms and nomographs developed from a non-linear stress dependent finite element model.

Irwin [III-34] has commented that if resilient moduli are to be estimated from surface deflection testing, then the test procedure should be as close as practical to the design loading. The secant moduli determined from the deflection basin will have the greatest accuracy.

CHAPTER SUMMARY

This section has dealt in a general fashion with various analysis methods for interpreting measured pavement deflections.

The analysis methods most common for flexible highway pavements can be divided into two main groups: empirically based procedures using limiting deflection criteria for evaluation, and procedures incorporating linear elastic layered pavement models to determine material properties or effective thickness (deflection basins). The theoretical procedures either use computerized iterative solutions, graphical fitting or nomographs to analyze and evaluate pavement structural condition.

Recently, criticism has been directed against the almost exclusive use of layer elastic theory to model pavement behavior in analysis and evaluation techniques of pavement structural condition. Thompson and Irwin have presented a "better" method of analyzing pavement deflections, using a finite element model of the pavement structure.

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CHAPTER IV

TRAFFIC

INTRODUCTION

One of the most important parameters which is necessary to estimate for pavement design and rehabilitation purposes is the damaging effect of traffic. The parameter to characterize traffic is a function of vehicle type and volume, in a general sense, and is more specifically a function of axle configurations, wheel loads, tire pressures, tire/pavement contact areas and axle repetitions. Further, the traffic parameter (for design input) may be further modified to accommodate the effects of dynamic loads.

For highway design, historically the concept of defining traffic in terms of a "fixed standard vehicle" has been used (as opposed to a "fixed traffic level" as illustrated by the single wheel load concept) [IV-1]. The "fixed standard vehicle" concept requires that the damaging effect of all vehicle types be converted to an equivalent number of repetitions of the standard vehicle. The conversion of mixed traffic to standard vehicle repetitions is accomplished by the use of "equivalent wheel load factors" (EWLF).

Currently in the State of Washington, the traffic input for design is in terms of equivalent 5,000 lb. wheel loads (more specifically Traffic Index which is equal to the logarithm of the equivalent number of 5,000 lb. wheel loads) [IV-2, IV-3]. A number of states have adopted the 18,000 lb. single axle as the "standard vehicle". This concept was originally developed during the AASHO Road Test and at a time in the United States when the legal maximum axle load was 18,000 lb. for a single axle (and 32,000 lb. for a tandem axle).

To understand and evaluate the required traffic parameters for the development of a new pavement rehabilitation design procedure, first a review of equivalent wheel load factors should be made (which results in an overall characterization of the traffic parameter for design) followed by a discussion of the development of the current WSDOT traffic parameter.

EQUIVALENT WHEEL LOAD FACTORS

Measures of Traffic Induced Pavement Damage Developed at the AASHO Road Test

Numerous techniques exist which can be used to measure or estimate pavement damage caused by wheel loads. Such damage can be caused by a few extremely heavy loads or numerous lighter loads. In either case wheel load damage to paved highways has been shown to be essentially caused by truck traffic (at least during short term evaluation periods). Load repetitions due to autos have little or no effect on expected pavement life.

The overall effect of wheel load and environmental caused pavement damage (environmental effects are not addressed in this chapter) is to deteriorate the pavement structure hence shortening pavement life and increasing the required maintenance for both the pavement and the vehicles using the pavement. This pavement damage may be manifested as surface roughness, cracking, rutting, pot holes or various combinations of these and other observable types of distress. Such manifestations are not the cause but the result of pavement damage.

Pavement damage has been the subject of extensive road test research in the U.S. Highway officials have always been confronted with constructing and maintaining pavements to carry increasing numbers and sizes of loads. Various road tests have been planned, constructed and evaluated to establish relationships concerning the effect of axle loads of various magnitudes on pavement damage. During the 1940's, the Stockton Test Track (constructed by the U.S. Army Corps of Engineers) and the Brighton Test Track (constructed by the California Division of Highways) were evaluated. Road Test One-MD (Maryland) was conducted in 1950 under repeated applications of two single and two tandem axle loads [IV-4]. This was followed by the WASHO Road Test [IV-5], consisting of a number of specially built flexible pavements in Idaho tested in 1953-54 under the same loads used in the Maryland test. Finally, the road test conducted in 1958-60 by the American Association of State Highway Officials (AASHO) at a cost of more than $20 million remains the largest analytical effort conducted to date [IV-6]. The AASHO Road Test is of specific interest since this effort resulted in relationships which can be used to estimate pavement life for given wheel load and environmental conditions.

The work accomplished at the AASHO Road Test resulted in numberous pavement technology improvements and advancements. At the Road Test, trucks of different sizes having different wheel loads were used to traffic separate pavement loops with each loop containing a large number of sections of experimental pavement of both flexible and rigid construction. No surface treated or unsurfaced roads were considered. Only one untreated base and subbase type were used in the primary study with limited sections containing asphalt and cement treated bases. Single and tandem axles were studied separately.

At the road test, the deterioration of pavements under traffic was expressed in terms of the Present Serviceability Index [IV-7]. The serviceability of a pavement is its ability to provide a satisfactory ride for motor vehicles at a point in time. A panel of road users assessed the roads quality on a scale increasing in quality from 0 (worst) to 5 (best). The panel was allowed to inspect the road but the final assessment was primarily a measure of the ability of road to carry them comfortably. Their rating was correlated statistically with physical measurements including longitudinal slope variance (SV), rut depth (RD), and the percentage of the road surface which was cracked or patched (C + P). The relation obtained for flexible pavements is [IV-7]:

$$PSI = 5.03 - 1.91 \log (1 + SV) - 1.38(RD)^2 - 0.001(C + P) \quad \text{(IV-1)}$$

Equation IV-1 indicates that the principal factor contributing to loss in serviceability over time (performance) is slope variance, a measure of roughness. Since this original work, many agencies have modified this relationship slightly to allow road roughness to be measured with car ride meters in lieu of slope variance.

AASHO Load Equivalencies

Wheel load equivalency has been one of the most widely adopted results of the Road Test, i.e., to relate relative damage attributed to axles of different type (single vs. tandem) and weight. A variety of equivalency factors can be used depending on the pavement section (defined by a structural number (SN) and the terminal serviceability index (P_t)). For state maintained highways, a P_t of 2.5 is normally used.

For flexible pavement, the axle load equivalency is given by the following:

$$\frac{W_x}{W_{18}} = \left[\frac{18 + 1}{L_x + L_2}\right]^{4.79} \left[\frac{10^{G/\beta_x}}{10^{G/\beta_{18}}}\right] \left[L_2\right]^{4.33}$$

where:

W_x = axle load equivalency,

L_x = axle load,

L_2 = code for axle configuration

1 = single axle

2 = tandem axle,

$G = \log\left(\frac{4.2 - P_t}{4.2 - 1.5}\right)$ = a function of the ratio of loss in serviceability at time t to the potential loss taken to a point where P_t = 1.5

P_t = Terminal serviceability index

$$\beta = 0.4 + \frac{0.081\ (L_x + L_2)^{3.23}}{(\overline{SN} + 1)^{5.19} L_2^{\ 3.23}}$$

$\overline{SN}$ = Structural number

The standard vehicle selected is an 18,000 lb. single axle load ($L_1 = 18$ and $L_2 = 1$). A summary of the equivalent load factors for flexible pavement and $P_t = 2.5$ is given in Table IV-1.

Current highway design in most states is based on the number of 18,000 lb. axle load equivalents anticipated over a future 10 to 20 year period. When these equivalents are produced by 16,000 lb. axles, a large number of repetitions is required for the same effect. When they are produced by 20,000 lb. axles, a smaller number of repetitions is required.

The relationship between repetitions is not arithmetically proportional to the axle loading. Instead, a 10,000 lb. axle needs to be repeated many more than 1.8 times the number of repetitions of an 18,000 lb. axle to have the same effect - in fact, more than 12 times. Similarly, a 22,000 lb. axle needs to be repeated less than half the number of times of an 18,000 lb. axle to have an equivalent effect.

General Development of Load Equivalencies

A reasonable definition for equivalent wheel load factors (EWLF) was provided by Yoder and Witczak [IV-1]:

> "An...EWLF defines the damage per pass caused to a specific pavement system by the vehicle in question relative to the damage per pass of an arbitrarily selected standard vehicle moving on the same pavement system."

By this definition, an EWLF can be defined as follows:

$$F_i = \frac{N_{fs}}{N_{fi}} \qquad \text{(IV-3)}$$

Table IV-1. AASHO Equivalence Factors for Flexible Pavement [Ref. IV-7]

Single Axles, $p_t = 2.5$

Axle Load, Kips	Structural Number, SN					
	1	2	3	4	5	6
2	0.0004	0.0004	0.0003	0.0002	0.0002	0.0002
4	0.003	0.004	0.004	0.003	0.003	0.002
6	0.01	0.02	0.02	0.01	0.01	0.01
8	0.03	0.05	0.05	0.04	0.03	0.03
10	0.08	0.10	0.12	0.10	0.09	0.08
12	0.17	0.20	0.23	0.21	0.19	0.18
14	0.33	0.36	0.40	0.39	0.36	0.34
16	0.59	0.61	0.65	0.65	0.62	0.61
18	1.00	1.00	1.00	1.00	1.00	1.00
20	2.61	1.57	1.49	1.47	1.51	1.55
22	2.48	2.38	2.17	2.09	2.18	2.30
24	3.69	3.49	3.09	2.89	3.03	3.27
26	5.33	4.99	4.31	3.91	4.09	4.48
28	7.49	6.98	5.90	5.21	5.39	5.98
30	10.31	9.55	7.94	6.83	6.97	7.79
32	13.90	12.82	10.52	8.85	8.88	9.95
34	18.41	16.94	13.74	11.34	11.18	12.51
36	24.02	22.04	17.73	14.38	13.93	15.50
38	30.90	28.30	22.61	18.06	17.20	18.98
40	39.26	35.89	28.51	22.50	21.08	23.04

Tandem Axles, $p_t = 2.5$

Axle Load, Kips	Structural Number, SN					
	1	2	3	4	5	6
10	0.01	0.01	0.01	0.01	0.01	0.01
12	0.02	0.02	0.02	0.02	0.01	0.01
14	0.03	0.04	0.04	0.03	0.03	0.02
16	0.04	0.07	0.07	0.06	0.05	0.04
18	0.07	0.10	0.11	0.09	0.08	0.07
20	0.11	0.14	0.16	0.14	0.12	0.11
22	0.16	0.20	0.23	0.21	0.18	0.17
24	0.23	0.27	0.31	0.29	0.26	0.24
26	0.33	0.37	0.42	0.40	0.36	0.34
28	0.45	0.49	0.55	0.53	0.50	0.47
30	0.61	0.65	0.70	0.70	0.66	0.63
32	0.81	0.84	0.89	0.89	0.86	0.83
34	1.06	1.08	1.11	1.11	1.09	1.08
36	1.38	1.38	1.38	1.38	1.38	1.38
38	1.75	1.73	1.69	1.68	1.70	1.73
40	2.21	2.16	2.06	2.03	2.08	2.14
42	2.76	2.67	2.49	2.43	2.51	2.61
44	3.41	3.27	2.99	2.88	3.00	3.16
46	4.18	3.98	3.58	3.40	3.55	3.79
48	5.08	4.80	4.25	3.98	4.17	4.49

where:

F_i = equivalent wheel load factor for vehicle or load i,

N_{fs} = number of repetitions to failure for the standard vehicle or load,

N_{fi} = number of repetitions to failure for vehicle or load i.

Equation IV-3 is a direct function of the failure criterion used to obtain the repetitions to failure (N_f). Such criteria commonly have been a function of the following:

1. Flexible pavements:

   (a) Tensile strains at the bottom of an asphalt-bound layer (related to fatigue cracking).

   (b) Vertical compressive strains at the top of the subgrade (related to rutting).

2. Rigid pavements:

   (a) Tensile stress in the portland cement concrete layer (related to fatigue cracking).

For load equivalency factors based on fatigue cracking (flexible pavements), repetitions to failure are commonly defined by:

$$N_f = K_1 \left[\frac{1}{\varepsilon}\right]^{K_2} \qquad \text{(IV-4)}$$

where:

N_f = number of load repetitions to failure,

ε = maximum principal tensile strain,

K_1, K_2 = constants.

Since F_i was defined as (from Equation IV-3):

$$F_i = \frac{N_{fs}}{N_{fi}} \text{ ; then}$$

$$[illegible]^{K_2} \qquad \text{(IV-5)}$$

where:

ε_i = tensile strain induced by load i,

ε_s = tensile strain induced by the standard load,

K_2 = 5.5, a common value used for asphalt concrete surfaced pavements).

For load equivalency factors based on portland cement concrete fatigue cracking, an example is the fatigue relationship developed by Vesic and Saxena [IV-8].

$$N_{2.5} = 225{,}000 \left(\frac{f_c}{\sigma}\right)^4 \qquad \text{(IV-6)}$$

where:

$N_{2.5}$ = load repetions to a serviceability index of 2.5,

f_c = tensile strength of concrete (psi),

σ = tensile stress (psi).

The load equivalency factor (after Equation IV-3) then could be calculated after the N_f for the standard and load i cases are obtained.

<u>Development of the WSDOT Truck Factors</u>

Current WSDOT practice provides for the use of Traffic Index as the necessary traffic parameter for <u>new</u> design. For the design of asphalt concrete overlays and depending on the design method used, WSDOT may use either Traffic Index or 18 kip equivalent axle loads (18 KEAL).

Of primary interest in this section is a brief review of how

Traffic Index has evolved. To do this provides insight into its continued usefulness (or lack of) for the planned pavement rehabilitation design system.

As illustrated by LeClerc and Sandahl [IV-3], the calculation of Traffic Index (TI) starts with multiplying the percent of 2 through 6 axle trucks in the traffic stream by appropriate truck constants for each of the truck axle groupings. These values are summed and result in the Annual Equivalent 5,000 lb. Wheel Loads (EWL) divided by the Average Daily Traffic (ADT). The complete calculation process for TI is illustrated in Figure IV-1.

Hveem and Carmany [IV-9] reported the development of the "Hveem" pavement design procedure in 1948 (adopted by California and in 1951 by Washington). They found that the following general relationship can be used to obtain a pavement thickness:

$$T = \frac{KD(90 - R)}{S} \qquad \text{(IV-7)}$$

where:

T = thickness of all layers above the subgrade (includes pavement surfacing and base),

K = a constant which is a function of units and a factor of safety,

D = deforming effect of wheel loads,

R = resistance value of the soil (0 to 100),

S = tensile strength of pavement or base or both.

1. Traffic Data

Year	ADT
1970	11,400
1980	14,250

2. Truck Classification

2 Axle	4.0%	. 350*	=	14.00
3 Axle	1.9%	. 1050*	=	9.45
4 Axle	0.4%	. 1650*	=	6.60
5 Axle	0.7%	. 3300*	=	23.10
6 Axle	-	. 2320*	=	0.00

Annual EWL/ADT = 53.15 (X)

3. ADT Calculations

No. of Lanes 4 Design Period 10 yrs. (Y)

ADT for current yr. = 11,400

ADT for current yr. + (Y) = 14,250

Average ADT = 12,825

Average one-way ADT = 12,825 2 = 6,412; Average one-way ADT heaviest lane = 6,412 x 0.85 = 5451 (Z)

Design EWL = X · Y · Z = 53.15 EWL/ADT · 10 yrs · 5451 ADT/yr. = 2,897,207 EWL

Traffic Index = Log (Design EWL) = 6.46

*WSDOT truck constants

Figure IV-1. Calculation of Traffic Index (after Ref. IV-3).

After further analysis of field and laboratory data, Hveem and Carmany finalized Equation IV-7 with the following result:

$$T = \frac{(KP\sqrt{a}\ \log r)^{(P_h/P_v\ -\ 0.10)}}{\sqrt[5]{c}} \tag{IV-8}$$

where:

T = thickness of cover of surfacing and base (in.),

K = 0.02 for design purposes (factor of safety included),

P_h = transmitted horizonal pressure in the stabilometer (psi),

P_v = applied vertical pressure in the stabilometer (typically 160 psi),

P = effective tire pressure (psi),

a = effective tire area (sq. in.),

r = number of load repetitions,

c = tensile strength of the cover material measured by the cohesiometer (approximately the modulus of rupture multiplied by 45.4), and

$P_h/P_v - 0.10 = 90 - R$

where:

R = resistance value of material tested = $(1 - P_h/P_v)100$

Thus, the evidence compiled by Hveem and Carmany indicated that the cover material necessary to protect the subgrade from plastic failure was directly proportional to:

1. average tire pressure,
2. square root of the effective tire imprint area,

3. logarithm of the load repetitions,

4. characterization of subgrade strength (a function of P_h/P_v - 0.1),

5. inversely proportional to the 5th root of the tensile strength of the base and surface (or cover material).

Out of the five basic inputs into Equation IV-8, three are a function of traffic.

The expression "$P\sqrt{a}$ log r" indicates the destructive effect of traffic. This relationship was modified to accommodate mixed traffic in terms of equivalent wheel loads. To convert mixed traffic wheel loads to 5,000 lb. EWL, Grumm [IV-10, IV-11] used work by Bradbury [IV-12] to develop wheel load equivalency factors (based on a portland cement concrete fatigue concept - analogous to the current Portland Cement Association highway and airfield pavement design procedures). Tables IV-2 and IV-3 are provided to illustrate how these initial EWL determinations were made. Specifically, Table IV-3 illustrates the destructive effects of increasing wheel loads (5,000 to 10,000 lbs.) which were initially selected for use in California. Table IV-4 shows how these data were modified for convenience so that normal truck count data (2 through 6 axle trucks) could be multiplied by EWL constants to achieve an overall characterization of the destructive traffic effect for design.

CHAPTER SUMMARY

Table IV-5 provides an overview of selected EWLF's developed in California (Col. 3) and further modified for use in Washington State (Col. 4). For comparison, the EWLF's developed from the AASHO Road Test (Cols. 1 and 2) are shown for terminal

Table IV-2. Bradbury's Calculations of Equivalent Load Repetitions for Various Wheel Loads [after Ref. IV-12]

Assumptions: (1) 8 in. thick slab (2) modulus of rupture = 750 psi
(3) k = 100 psi (4) E = 4,000,000 psi

Wheel Load (lb)	Applied Stress* (psi)			Load Repetitions to Cause Failure**	Actual Repetitions Per Year	Equivalent Repetitions Per Year for 396 psi Stress Level
	Load	Warping	Total			
5,000	146	250	396	360,000	600	600
6,000	166	250	416	135,000	550	$550\ (\frac{360,000}{135,000}) = 1,467$
7,000	185	250	435	55,000	500	$500\ (\frac{360,000}{55,000}) = 3,273$
8,000	204	250	454	23,000	450	$450\ (\frac{360,000}{23,000}) = 7,044$
9,000	222	250	472	11,000	400	$400\ (\frac{360,000}{11,000}) = 13,091$
10,000	239	250	489	5,000	350	$350\ (\frac{360,000}{5,000}) = 25,200$

*Load and warping stresses calculated for exterior edge of slab. Warping stress calculated for spring-summer conditions.

**Based on concrete fatigue data from Illinois Division of Highways, Engineering Report 34-1.

Table IV-3. Calculation of Initial 5,000 lb. Equivalent Wheel Load Factors Developed for California [after Refs. IV-11 and IV-12]

Wheel Load (lb)	Equivalent Repetitions Based on 396 psi Stress Level (from Table IV-2)	Equivalent Repetitions Divided by Actual Repetitions	Multiplier for each Wheel Load Increment	EWL Factors Actually Adopted by California
5,000	600	1.000		1
			2.667	
6,000	1,467	2.667		2
			2.454	
7,000	3,273	6.546		4
			2.391	
8,000	7,044	15.653		8
			2.091	
9,000	13,091	32.728		16
			2.200	
10,000	25,200	72.000		32

Table IV-4. Initial EWL Constants Developed for California [after Refs. IV-9, IV-12 and IV-13]

Number of Axles/Truck	5,000 lb. EWL/Truck	EWL Constants 5,000 lb. EWL/Truck x 365
2	0.82	300
3	1.92	700
4	3.84	1,400
5	5.75	2,100
6	4.38	1,600

To achieve an overall characterization of the destructive traffic effect for design.

Table IV-5. Comparison of Selected Equivalent Wheel Load Factors

Wheel Load* (lbs)	Equivalent Wheel Load Factors						
	AASHTO (SN = 4)		(3) Original California $EWLF = r(2)^{WL-5}$ [Ref. IV-11]	(4) WSDOT $EWLF = \frac{r(1.585)^{WL}}{10}$ [Ref. IV-15]	(5) Fatigue - Dual Tires (SN≈4) [Ref. IV-14]	(6) Fatigue - Single Tires** (SN≈4) [Ref. IV-14]	
	(1) $P_t = 2.0$ [Ref. IV-6]	(2) $P_t = 2.5$ [Ref. IV-6]				12" Tire	16" Tire
3,000	0.18	0.21	0.25	0.40	0.34	-	-
4,000	0.61	0.65	0.50	0.63	0.73	-	-
5,000	1.55	1.47	1.00	1.00	1.33	0.48	0.30
6,000	3.33	2.89	2.00	1.59	2.16	0.78	0.48
7,000	6.42	5.21	4.00	2.51	3.27	1.18	0.73
8,000	11.46	8.85	8.00	3.98	4.68	1.69	1.05
9,000	19.28	14.38	16.00	6.31	6.41	2.32	1.44
10,000	30.92	22.50	32.00	10.00	8.51	3.07	1.90

*Based on a single axle with dual tires (e.g. if wheel load = 5,000 lb., then total single axle load = 20,000 lb.)

**Based on a single axle with single tires (e.g. if wheel load = 5,000 lb., then total single axle load = 10,000 lb.)

serviceability indices of 2.0 and 2.5, respectively. Recent work in the State of Washington [IV-14] based on layered elastic predictions of pavement response and appropriate fatigue based failure criteria (Col. 5) compare well with the currently used WSDOT EWLF's. EWLF's for single tires developed in Reference IV-14 are also provided.

Overall, calculated EWLF's vary significantly; however, the continued use of current WSDOT equivalencies appear reasonable. Thus, use of TI or 18 KEAL can be accommodated in the planned WSDOT overlay design procedure.

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CHAPTER V
FAILURE CRITERIA

INTRODUCTION

Pavement failure is not well defined in the literature. One reason for this inconsistency in the definition, as reported by Hudson, et al. [V-1], is the divergent viewpoints from which the engineers and users view pavement failure. Pavement failure is perceived differently by the design engineer, the construction engineer, and the maintenance engineer. Where failure is based on subjectively derived criteria, differences in opinion may vary significantly.

A survey of literature reveals that stress, strain, and deformation have been used as indications of pavement failure. Pavement cracking and distress may represent material failure but it does not imply functional failure of pavement. In this regard, two types of failure are defined, functional and structural failure. Functional failure occurs when the pavement can no longer carry the traffic safely and comfortably at design speed, while structural failure indicates a breakdown of one or more of the pavement components [V-2]. Failure of a pavement system generally does not occur suddenly. Failure is a condition that develops gradually over a span of time. Even though a pavement is designated as having "failed", it may still be able to carry traffic at a reduced service level.

From the ongoing discussion, failure of the pavement structural system may be defined as a condition where the distress in the system has exceeded an acceptable level based on design criteria. Pavement failure may be defined in terms of:

pavement condition ratings, deflection measurements, or a combination of pavement condition ratings and deflection measurements.

The intent of this chapter is to describe the distress mechanism; especially fatigue cracking, since it is the most prevalent mode of distress in flexible pavements in the United States. In addition, a brief discussion of deflection criteria and pavement rating is included in this chapter.

Distress Development

As mentioned earier, pavement failure develops gradually over a span of time. This span of time is defined as pavement life cycle. Most fairly sound pavement sections deteriorate with time in an orderly progression of defects, i.e. as traffic loads, environment, and other factors act upon the pavement system, the pavement responds with stress, strain, deformation and other types of behavior. When this behavior reaches a limiting response value during the pavement life cycle, distress results. The serviceability loss can occur as a result of the accumulation of a single type of distress (mainly fatigue cracking in Washington State) or as a combination of several types. Certain types of distress, such as a single crack, do not in themselves cause pavement failure. However, as a particular distress accumulates with time, it may combine with other types of distress such as distortion and disintegration to cause an unacceptable serviceability level.

As explained by Hudson, et al. [V-1] the relationship between distress and performance is not a clear relation, having at least three components: primary, secondary, and time dependent.

The primary distress types include those which have a direct and immediate effect on serviceability. The secondary distress types are those which occur as a corollary of certain types of primary distress. For instance, premature fatigue cracking due to traffic loadings is a primary distress. The crack itself may not reduce serviceability. However, water infiltration may weaken the underlying pavement materials which may cause permanent deformations that reduce serviceability. In this case, the deformations are a secondary distress.

The time dependent distress component may have no immediate effect on serviceability but given adequate time will lead to a reduction in the serviceability. For instance if a pavement section is rated with severe longitudinal cracking, with time during pavement life cycle, these cracks evolve into alligator cracking and potholes which cause the level of serviceability to drop below an acceptable level.

Since fatigue cracking is the predominant type of distress in flexible pavement in the State of Washington, most of this chapter will deal with this distress mode.

FATIGUE

Fatigue can be defined as the phenomena of load-induced cracking due to a repeated stress or strain level below the ultimate strength of the material [V-2]. In asphalt pavements fatigue is caused by the repetitive application of traffic and environment which induces stresses and strains sufficient to cause damage and thus loss of serviceability. The classical type of fatigue failure is commonly described as a "chickenwire" or "alligator" cracking due to the pattern of cracks which appear on the surface. Fatigue cracking of flexible pavement is the result of cracks which have extended through the asphalt blanket by the continued application of traffic loads. These cracks are the result of tensile stresses which reflect failure of the lower pavement layers to support the upper pavement layers.

To better understand the phenomenon of fatigue cracking a discussion of types of fatigue tests and the effect of mixture variables on the fatigue life are discussed below.

<u>Types of Fatigue Tests</u>

Fatigue failure, in the laboratory, may be defined arbitrarily as a point related to the ability of the test specimen to continue to perform as a load carrying medium under repetitive loading. As shown in Table V-1, the behavior of the test specimen depends on many variables. Type of loading plays a major role in determining the fatigue life. The most common form of load application is referred to as simple loading. The two modes of simple loading are referred to as:

1. Controlled stress mode - when the loading is an alternating stress of a constant amplitude or
2. Controlled strain mode - when the loading is in the form of an applied alternating strain or deformation of constant amplitude (Figures V-1 and V-2).

In the controlled stress mode, because of the progressive damage to the specimen, a decrease in stiffness results. This, in turn,

Table V-1. Variables Affecting Fatigue Behavior in Laboratory Testing [after Ref. V-3]

I. Load Variables
   A. Pattern of Stressing
      1. Types of Stresses
      2. Geometrical Stress Distribution
   B. Time Distribution of Loading
      1. Distribution of Time Between Successive Load Applications
      2. Mean Rate of Loading
      3. Shape of Load Curve
      4. Duration of Loading
   C. Testing Method
      1. Load History
         a. Simple Loading
         b. Compound Loading
      2. Mode of Loading
         a. Controlled Stress
         b. Controlled Strain
         c. Intermediate
II. Environmental Variables
   A. Temperature
   B. Moisture
   C. Alteration of Material Properties During Service Life
III. Mixture and Specimen Variables
   A. Aggregate
      1. Type
      2. Gradation
   B. Binder
      1. Type
      2. Hardness
   C. Specimens
      1. Bitumen Content
      2. Surface Texture
      3. Air Void Content
      4. Anisotropy
      5. Shape
      6. Size
      7. Stiffness

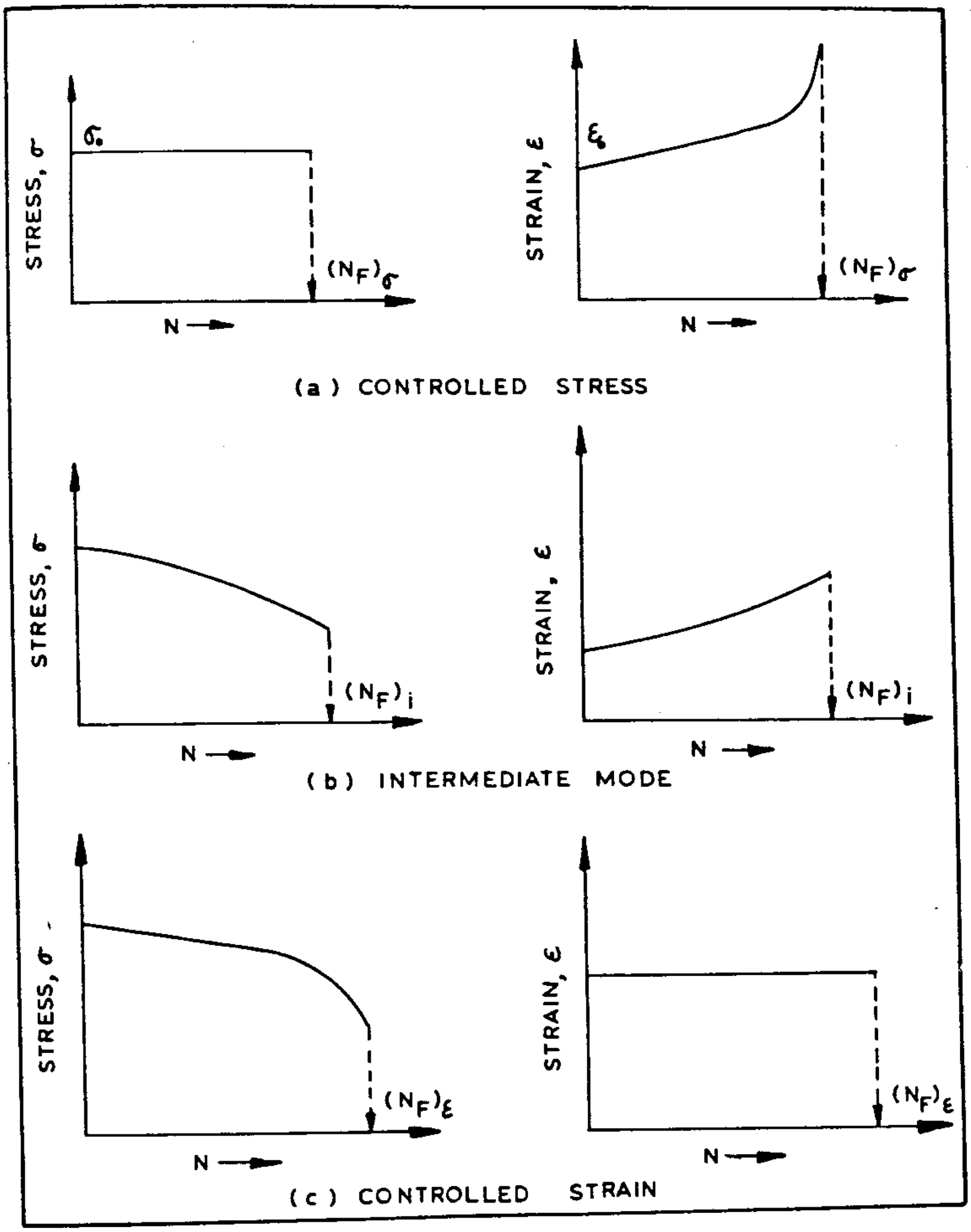

Figure V-1. Fatigue Tests Mode of Loading
[after Ref. V-5]

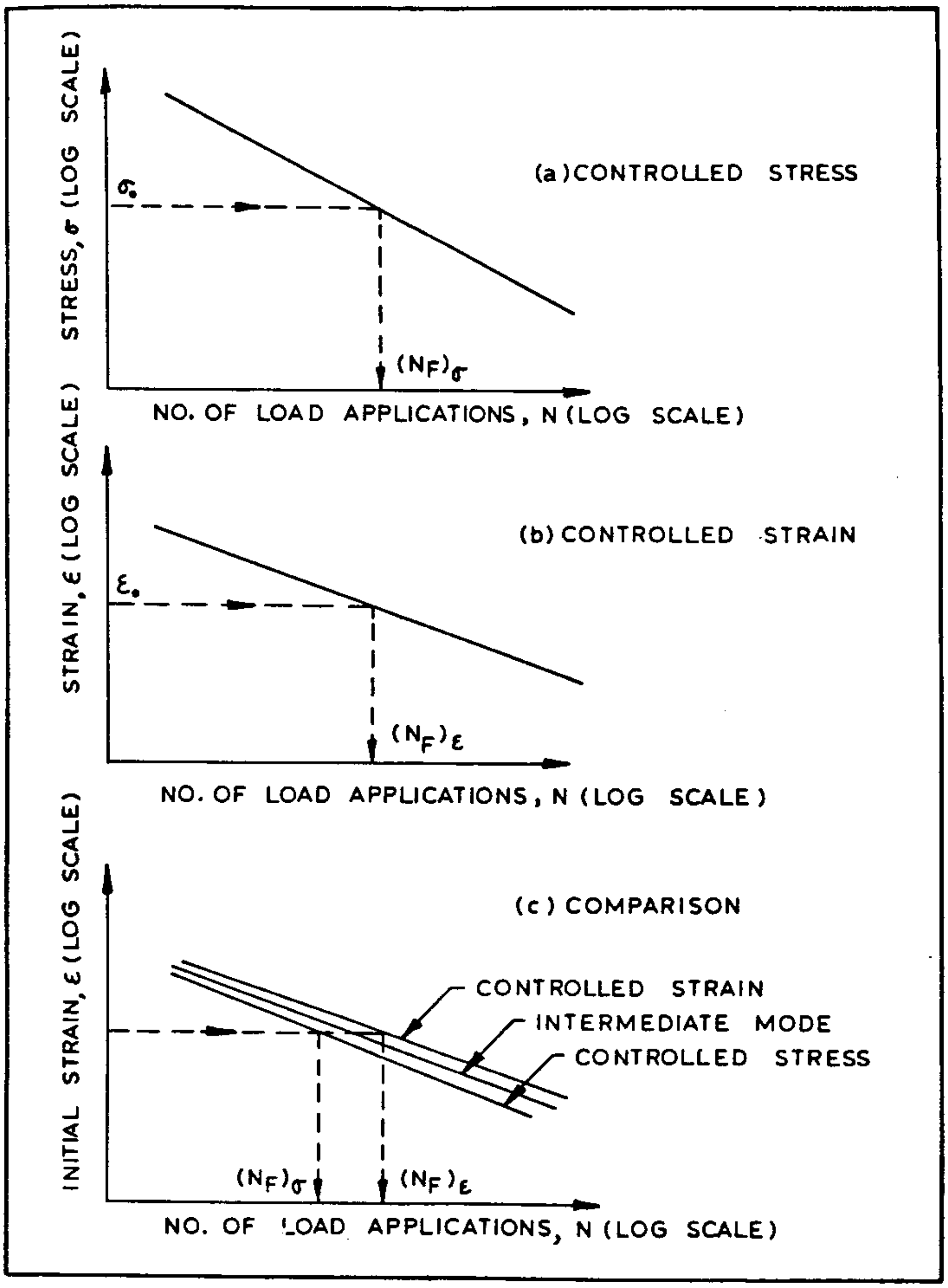

Figure V-2. Results of Fatigue Testing for Controlled Stress and Strain [Ref. V-8] causes an increase of the actual flexural strain with load applications. In the controlled strain test, the resulting stress on the specimen will decrease with decreasing stiffness. The service life of the specimens greatly depends on the type of test used.

Monismith and Deacon [V-4] have proposed a more quantitative method for differentiation between the two test modes. This is expressed by the mode factor:

$$\text{Mode Factor (MF)} = \frac{|A| - |B|}{|A| + |B|}$$

where: A = percentage changes in stress

B = percentage changes in strain

The mode factor is -1 for controlled stress loading and +1 for controlled strain testing. Where both stress and strain are changing during the test, the mode factor values lies between these two limits. In practice, controlled stress tests are considered to be applicable to thick asphalt layers ($\geq$ 6 in) while controlled strain conditions are considered applicable to thin asphalt layers ($\leq$ 2 in.).

For the intermediate thickness, some form of testing between those two extreme modes would be appropriate. Pell [V-6] suggested the use of a controlled stress test for the intermediate thickness, since it will give conservative estimates of fatigue life.

The difference between the two tests can be seen in Figure V-3. Figure V-3a shows the relationship between load applications and stress in a controlled stress test. For an

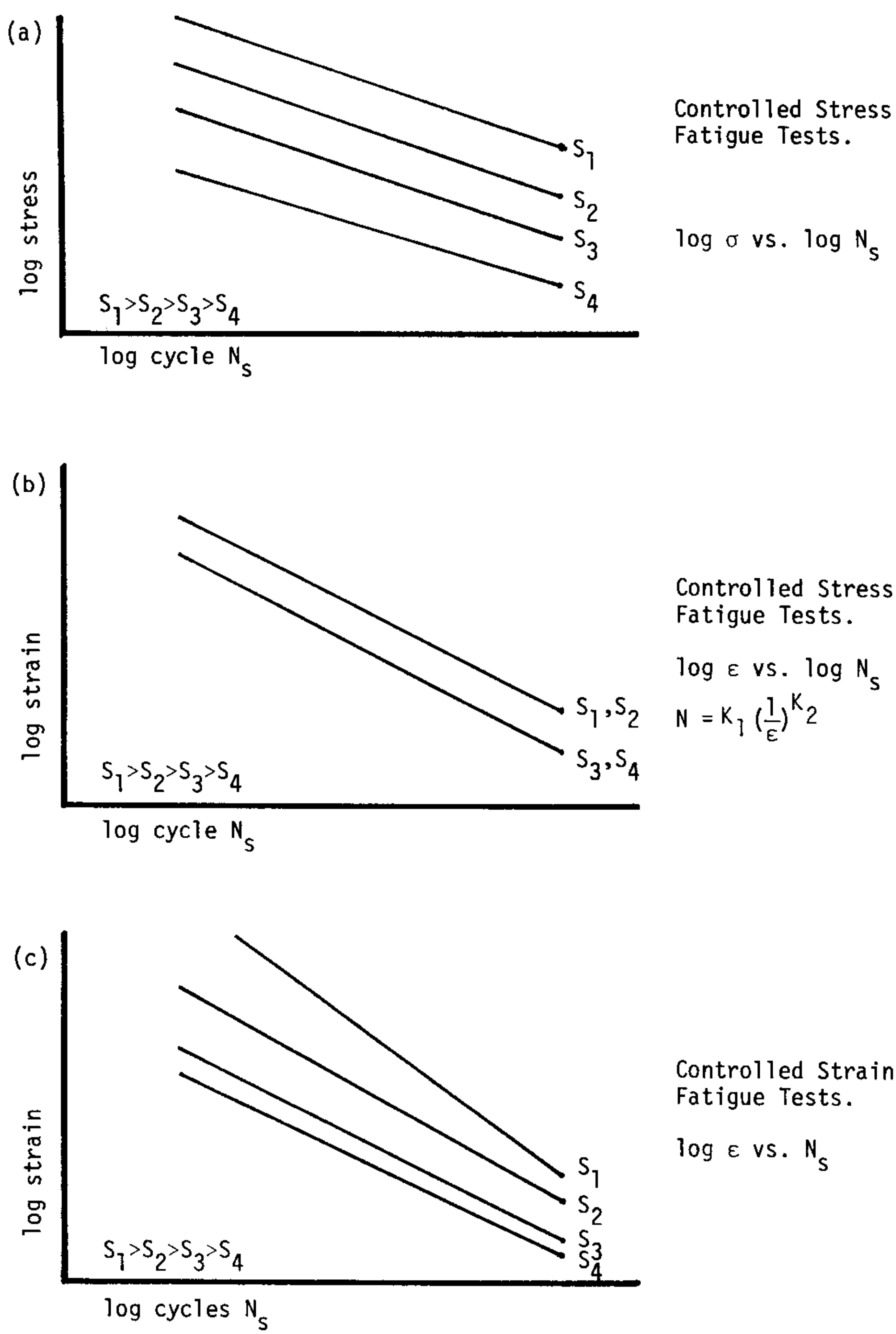

Figure V-3. Effect of Stiffness on Fatigue Life Under Controlled Stress and Controlled Strain [after Ref. V-6] identical specimen, Figure V-3b shows the relationship between load applications and strain. Figure V-3c combines those relationships along with that of the intermediate mode for comparison purposes.

Even though researchers have reported the use of both modes of loading, most of the development in fatigue testing is based on the controlled stress mode. Barksdale and Miller [V-6] suggested three practical reasons for the emphasis on controlled stress testing:

1. The controlled stress mode is most appropriate for investigating the fatigue characteristics of mixes to be used in at least primary and interstate highway construction, since the total asphalt concrete thickness is generally greater than 6 in.
2. In this type of testing, fatigue failure can be defined as the complete fracture of the specimen since crack propagation is rapid.
3. The controlled stress test gives conservative fatigue test results.

For both controlled stress and controlled strain tests, several research studies suggested that the logarithm of the initial strain is approximately proportional to failure. This can be expressed by the following general equation:

$$N_f = K_1 \left(\frac{1}{\varepsilon_i}\right)^{K_2}$$

where:

N_f = load repetitions to failure.

ε_i = initial tensile strain,

K_1, K_2 = fatigue parameters.

Pell [V-6] reported that the value of K_2 ranges from 5.5 to 6.5. Other values outside this range have been reported.

As mentioned eariler, fatigue life cycle depends on many variables. Some of these important variables will be discussed below.

<u>Mixture Variables</u>

It is important to understand the effects of changes in material characteristics in the fatigue life and failure condition. These design mix characterics include stiffness, air void content, asphalt content, binder type, and aggregate characteristics. Other variables such as rate of loading and temperature will also be discussed.

<u>Stiffness</u>. By definition, stiffness is the ratio of stress amplitude to strain amplitude. It is a function of temperature and time of loading:

$$S(t,T) = \frac{\sigma}{\varepsilon}$$

where:

$S(t,T)$ = mixture stiffness at a particular time and temperature, (psi)

σ, ε = axial stress and strain, respectively.

At short loading times or low temperatures or both, mixture stiffness approaches a constant value and is analogous to a modulus of elasicity. As the time of loading and temperature increase, stiffness decreases. Pavement stiffness may vary from approximately 4 x 10^6 psi (at low temperature and short loading time) to about 1 x 10^3 psi (at high temperature and long loading time).

A review of the literature reveals that the stiffness of an asphalt mix plays a major role in determining the fatigue behavior. Generally, in a controlled stress test, as the mixture stiffness is increased the fatigue life increases (Figure V-3a). On the other hand, in controlled strain conditions, as the stiffness of the mixture is increased the slope of the fatigue curve increases and fatigue life decreases (Figure V-3c). Pell [V-6] has explained the reason for this dichotomy. He reported that the mode of failure in the two tests is different. In the controlled stress test, the formation of a crack results in an increase in actual stress at the tip of the crack due to the stress concentration effect, and that leads to rapid propagation and failure. By the same token, in the constant strain test, cracking results in a decrease in stress and, hence, slow rate of crack propagation. Therefore, at low stiffnesses, a crack needs longer time to propagate through the specimen.

From his study, Pell [V-6] recommended the use of strain as a major criterion of failure. He found that when the results of the controlled stress tests were replotted in terms of strain, all the results from different stiffnesses coincide (Figure V-3b). The effects of rate of loading have been reported by Deacon and Monismith [V-3]. They indicated that the frequency of load application in the range of 30 to 100 repetitions per minute significantly decreased fatigue life (approximately 20 percent).

Monismith, et al. [V-9] have reported that frequency of load applications in the range of 3 to 30 repetitions per minute had no effect on specimen fatigue behavior.

Table V-2, after Monismith [V-11], summarized the effects of some of the more important mix design variables on stiffness and fatigue life.

Air Void Content. In the literature, it has been found that mixes containing higher air void contents exhibit shorter fatigue lives. Figure V-4, originally after Saal and Pell [V-12], illustrates a change in air void content from ten percent to four percent results in change in fatigue life of about one order of magnitude. The effect of voids is more pronounced in controlled stress testing or controlled strain testing at low temperatures [V-6, 13]. These effects can be stated as:

1. As void contents increase the stiffness decreases.
2. As void contents increase the stress concentration increases.

Asphalt Content. In addition to air void content, the asphalt content is one of the most critical factors that regulates asphalt concrete properties. Epps and Monismith [V-13], Pell [V-6] and other researchers have found that there is an optimum asphalt content for fatigue life. This optimum asphalt content corresponds to the optimum asphalt content required for maximum mixture stiffness and is in excess of that determined for stability requirements.

Binder Type. Several types of asphalt binders are available for use in asphalt concrete mixes. Asphalt is classified

Table V-2. Factors Affecting the Stiffness and Fatigue Behavior of Asphalt Concrete Mixtures [after Ref. V-10].

Factor	Change in Factor	Effect of Change in Factor		
		On Stiffness	On Fatigue Life	
			In Controlled Stress Mode	In Controlled Strain Mode
Asphalt Penetration	Decrease	Increase	Increase	Decrease
Asphalt Content	Increase	Increase[1]	Increase[1]	Increase[2]
Aggregate Type	Increase Roughness and Angularity	Increase	Increase	Decrease
Aggregate Gradation	Open to Dense Gradation	Increase	Increase	Decrease[4]
Air Void Content	Decrease	Increase	Increase	Increase[4]
Temperature	Decrease	Increase[3]	Increase	Decrease

[1] Reaches optimum at level above that required by stability considerations.

[2] No significant amount of data; conflicting conditions of increase in stiffness and reduction of strain in asphalt make this speculative.

[3] Approaches upper limit at temperature below freezing.

[4] No significant amount of data.

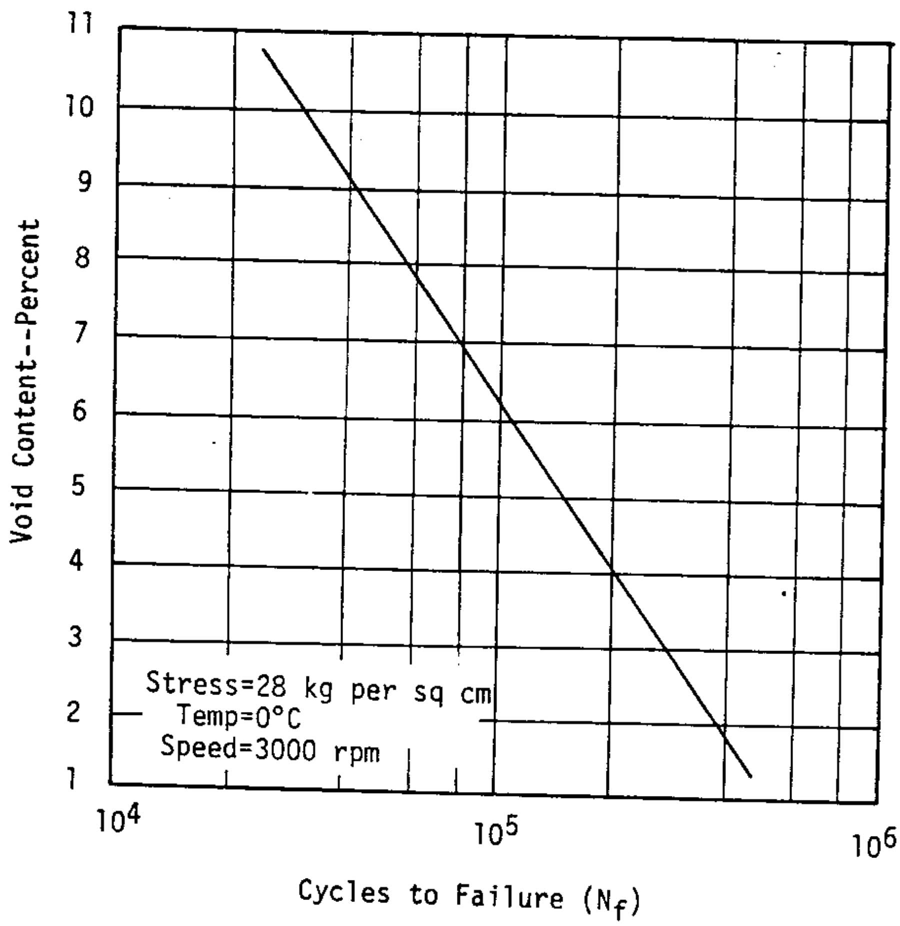

Figure V-4. The Effect of Voids Content on Fatigue Life [after Ref. V-11].

according to the viscosity, the relative hardness, or penetration characteristics. Generally, since increasing the hardness of asphalt results in increases in mixture stiffness, it will result in longer fatigue life in the controlled stress mode of loading and a shorter fatigue life in the controlled strain testing.

Aggregate Characteristics. One of the important classifications of aggregates for uses in asphalt concrete pavements is that based on size distribution or gradation. The proper selection of gradation criteria is a critical element of the design process. In addition to the gradation, aggregate type (surface texture shape and petrographic classification of aggregate) is important since there should be a firm bond between the aggregate and asphalt binder.

The literature review reveals that no clear effects of aggregate characteristics on fatigue have been found. Monismith [V-8] concluded that in controlled stress tests rough textured aggregates tended to perform better than smooth textured aggregate for equal amounts of asphalt.

Pell and Taylor [V-10] have presented results of controlled stress tests. In these tests, it was found that the geometric characteristics of the tested aggregate had a negligible effect on the fatigue behavior of the mixtures for identical asphalt contents. The same conclusion was reached by Kirk [V-14]. Hwang and Grisham [V-15] have concluded from their controlled stress tests that the fatigue response was not significantly affected by the geometric characteristics of the coarse aggregate, nor was it affected by the gradation of the aggregate in the mixtures.

The effect of adding filler to a mix is to increase the dynamic stiffness. There is no appreciable improvement in fatigue performance on the basis of applied dynamic strain if the relative binder volume content remains constant [V-16].

Temperature. All investigators seem to be in agreement that the lower the temperature, the longer the fracture life. Pell and Taylor [V-10] have examined the effect of temperature on fatigue life on controlled stress test. They found that as tempeature decreases, the stiffness of an asphalt mixture increases. Thus, for a specific stress the deformation (or strain) decreases, resulting in a longer life. The same conclusion have been reached by Jimenez [V-58].

Effect of Rest Periods

In practice, pavement is subjected to a random number of load applications and magnitude. The intervals between load applications may vary from short to long depending on the traffic flow. Since asphalt is a viscoelastic material, the long interval between load applications be considered as having a healing effect.

Recently, Bonnaure, et al. [V-17] in their study about the influence of rest periods on the fatigue characteristics of bituminous mixes, reached the following conclusions:

1. Rest periods have a beneficial effect on fatigue life. The benefit seems to reach a maximum around a rest period equal to 25 times the loading cycle.
2. Increasing the test temperature increases the beneficial effect.
3. Softer binders increase the beneficial effect.

Cumulative Damage

In practice, asphalt pavements are subjected to a form of compound laoding, and changes take place in the loading conditions during the life. Usually Miner's rule is used for evaluating cumulative damage. The rule states that the condition at failure is given by:

$$\sum_{i=1}^{r} \frac{n_i}{N_i} = 1$$

where: n_i = number of cycles of stress applied to the test specimen,

N_i = number of cycles to failure at constant stress amplitude from simple loading, and

r = number of loading conditions considered.

In the application of Miner's rule, no allowance is made for rest periods.

To account for the variations of both the traffic loads and the physical state of the pavement, let d_{ij} be the damage induced in the pavement by one application of the ith load while the pavement is in the jth physical state. If that particular load is repetively applied to the pavement in that state until the pavement fails and if N_{ij} represents the number of applications before failure, then d_{ij} can be estimated as follows [V-5]:

$$d_{ij} = 1/N_{ij}$$

The total cumulative damage D predicted during the design life is then:

$$D = \sum_i \sum_j d_{ij} n_{ij} = \sum_i \sum_j \frac{n_{ij}}{N_{ij}}$$

Failure occurs if D equals or exceeds 1.0.

<u>Fatigue Relationships</u>

The log ε - log N relation can be established in at least four different ways [V-5]: 1) from theroetical analysis of existing design curves, 2) from an analysis of the performance of in-service pavements, 3) from laboratory fatigue testing, and 4) a combination of the procedures given above.

<u>From Existing Design Curves</u>. Design curves, in which the traffic is represented by equivalent axle loads, can be used for establishing the log ε-log N relation. The design curves are examined to determine one or more pavement sections that are considered adequate for each of several equivalent axle loads. The tensile strains imposed by the base load in those structures are then estimated by layered elastic theory. Average environmental conditions in the region for which the design curves are applicable are used to establish representative material parameters for the sections. The computed tensile strains are finally related to the number of load applications to estimate the log ε-log N relation. Advantages of this method include:

1. Simplicity, and

2. Compatibility with an established history of design and performance experience.

Disadvantages of this method include:

1. The relation can be used with confidence only in geographical areas that have climatic conditions similar to those of the region for which the original curves were developed.
2. A one physical-state representation of the structure must be used in the design process (i.e., the average).
3. This method requires that the original design curves be based solely on the control of failure by fatigue. Most existing design curves consider all distress mechanisms simultaneously.
4. Designs that would incorporate new and different types of bituminous mixtures cannot be examined with such a criteria.

<u>From In-service Pavements</u>. Pavement sections with known traffic records and performance history offer a good way of establishing the failure criterion (log ε-log N relation). Accuracy can be improved by laboratory or field testing to determine the material parameters. Analysis can be limited to those test sections known to have failed by fatigue.

If an average annual characterization of physical state of the pavement is to be used, the analysis proceeds much as before. For a single pavement state, Miner's rule requires that, at failure:

$$\sum_i \frac{n_i}{N_i} = 1$$

and

$$N_i = K_1 \varepsilon_i^{K_2}$$

Combining the two equations results in the following:

$$\sum_i \frac{n_i}{K_1 \varepsilon_i^{K_2}} = 1$$

For each test section of the road test, n_i is known, ε_i can be calculated by layered elastic theory. Using 18,000 lb. equivalent wheel loads, the equation becomes $n = K_1 \varepsilon_i^{K_2}$. An equation similar to this is available for each test section. That set of equations can be solved for the unknowns, K_1 and K_2.

From Fatigue Testing. The ε-N relation can be also derived from the laboratory fatigue test. Test specimens are subjected to repeated flexing until failure, and the ε-N relation is determined directly. Laboratory testing is the simplest way in which the behavior of new material can be evaluated. However, there are several obstacles to the use of laboratory-derived ε-N relationships. One of the major difficulties is that of defining failure in the laboratory in such a way as to be compatible with failure in actual pavements.

Brown and Pell [V-18] suggested that in-service pavement life is of the order of 20 times the life of laboratory specimens. Recently, Ullidtz, et al. [V-19] reported that the disagreement in estimating the in-service pavement life from laboratory specimens is wider. He reported use of factors ranging from 100 to 5,000.

As previously discussed, laboratory fatigue specimens generally are subjected to either controlled-stress or controlled-strain modes with the result that the number of load applications to failure is dependent on the type of test. In-service pavements are subjected to a type of loading which is intermediate between those two modes.

In laboratory testing, the specimens are subjected to a higher load frequency than in-service pavement. These as well as other laboratory loading variables significantly effect the ε-N relation [V-5].

Rauhut, et al. [V-20] have suggested the following reasons why fatigue estimates differ between laboratory specimens and in-service pavements.

1. Failure in laboratory tests is relatively sudden, soon after cracks initiate, where some acceptable amount of surface cracks may occur in the field long after initial cracking at the bottom of the layer.
2. Actual stress and strain responses at the bottom of a cracked pavement are considerably different from those estimated with layered elastic theory.

Other reasons for the conservative fatigue behavior of in situ pavement includes the following:

1. Rest time between stress applications.
2. Healing of the pavement between stress applications.
3. Variability in the position of the load within the wheel path.

4. The use of an average condition of base and subbase saturation in the determination of the properties of each layer.

Van Dijk [V-21] has illustrated the progression of fatigue cracking from crack initiation to final stage (failure). Figure V-5 has been developed from wheel tracking test and shows several stages of crack development. Cracks appeared at the surface only near the last stage, designated as "failure", which represent several times as many loads repetition as required to initiate cracking.

Pickett, et al. [V-22] reported that the fatigue lives measured in the laboratory describe the number of cycles for initial crack formation (N_1 in Figure V-5) while the fatigue life which should be used as the field fatigue life occurs when major cracks form and initiate a loss in Pavement Serviceability (N_2 in Figure V-5).

Several methods have been proposed for transforming laboratory fatigue data into field fatigue data. The most common method makes use of a horizontal shift factor. Several shift factors have been found in the literature, some of these factors are summarized in Table V-3 along with the appropriate reference.

FATIGUE MODELS

During the last 20 years extensive research studies have been carried-out to establish fatigue models. Most of the reported models were developed on laboratory testing, with very little attempt to correlate results with occurrence of actual pavement distress.

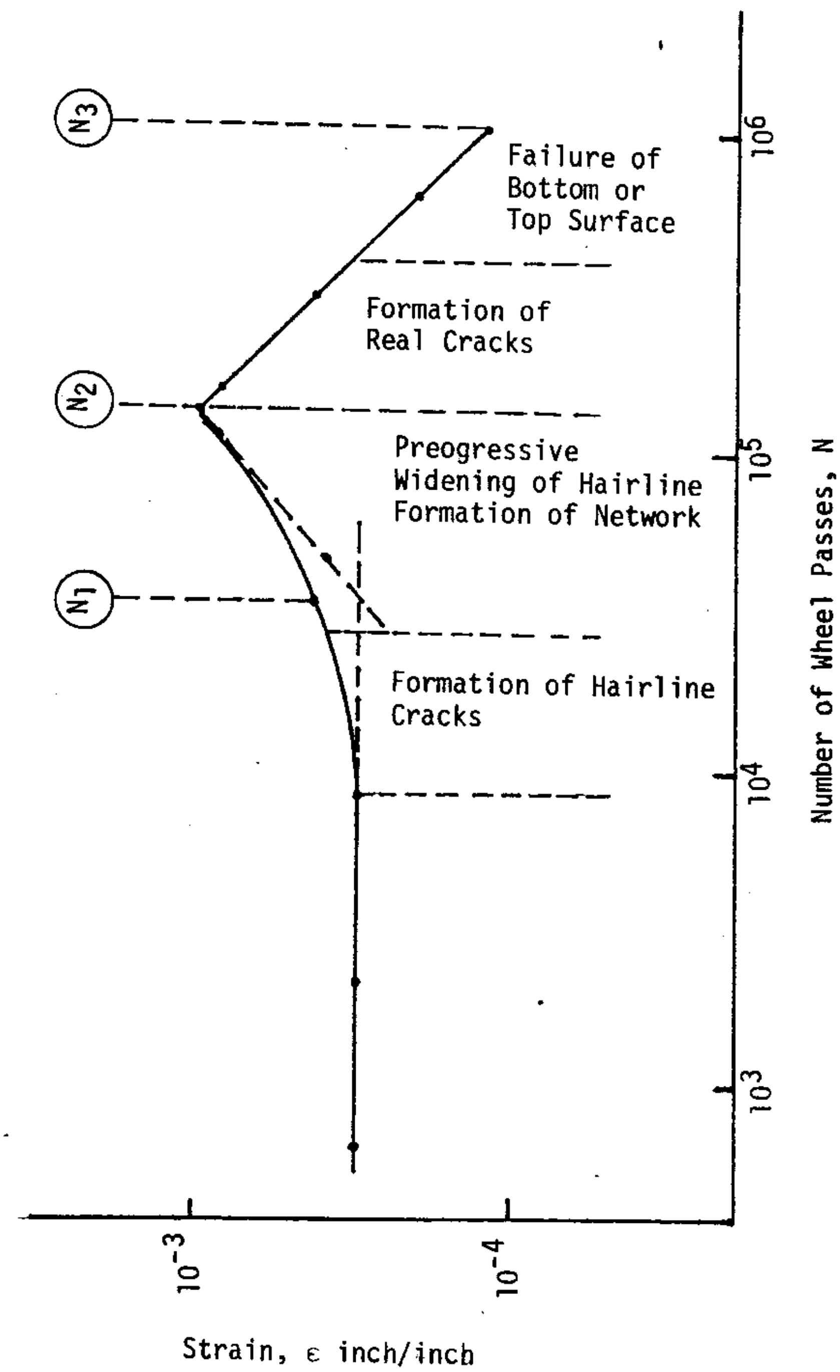

Figure V-5. Development of Cracking as a Function of Strain at Equivalent Number of Wheel Passes [after Ref. V-21].

Table V-3. Summary of Fatigue Shift Factors.

Ref No.	Researcher	Relationship
V-18	Brown and Pell	$N_{field} = 20\ N_{lab}$
V-21	Van Dijk	$N_{field} = 3\ N_{lab}$
V-22	Pickett, et al.	$N_{field} = K\ N_{lab}$, where $K = 0.516 \times 10^{0.0147T}$, T = Temperature, °F
V-23	Finn, et al.	$N_{field} = 13.03\ N_{lab}$
V-24	Santucci	$N_{field} = N_{lab} \times 10^{m}$, where $m = 4.84 \frac{V_b}{V_v + V_b} - 0.69$ V_b = asphalt volume, V_v = air voids volume

As stated earlier, the relationship between fatigue life and initial strain can be expressed as: $N_f = K_1 (\frac{1}{\varepsilon})^{K_2}$ where K_1 and K_2 are fatigue parameters. The fatigue parameters K_1 and K_2 have been reported for a variety of design mix conditions [V-22, 25]. The values of K_1 and K_2 depend on many factors such as: type of aggregate, asphalt-content, temperature, and type of testing (i.e. flexure, rotating flexure, or diametral). The lowest value of K_2 reported in the literature is 1.83 ($K_1 = 4.99 \times 10^{-4}$) for diameteral controlled stress fatigue testing for crushed gravel mix with asphalt content of 6.0 percent at 70 F [V-26]. And the maximum value reported is $K_2 = 6.43$ ($K_1 = 1.87 \times 10\text{-}18$) for flexure controlled stress fatigue testing for crushed granite with limestone filler [V-27].

Several of the fatigue models reported in the literature are shown in Figures V-6 through V-8. Figure V-6 provides a representative sample of laboratory data based on the controlled stress testing. Data in Figure V-7 are based on the controlled strain testing. It is obvious that these models are more conservative than those in Figure V-6.

Models in Figure V-8 are based on field observations of pavement performance. The discussion herein will be limited to the field models in Figure V-8. For other models consult References V-22 and 25. Figure V-8 shows the fatigue life prediction curves based on a stiffness of 7.0×10^5 psi. The model reported by Mahoney and Terrel [V-38] involved the determination of initial bending strains by measuring in situ strains with strain coils installed in the pavement of the Washington State University (WSU) test track and by using the

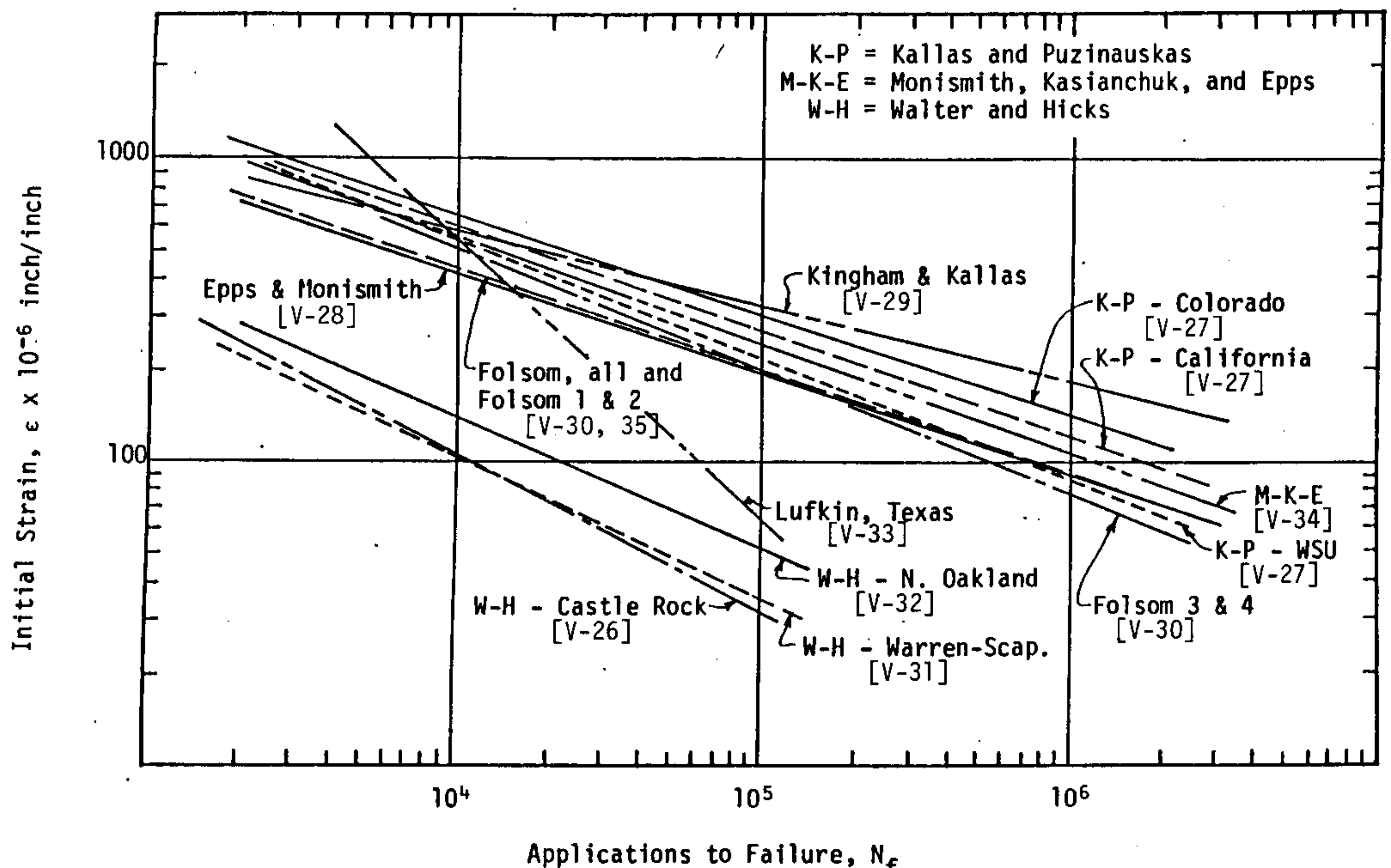

Figure V-6. Fatigue Models for Controlled Stress Testing [after Ref. 25].

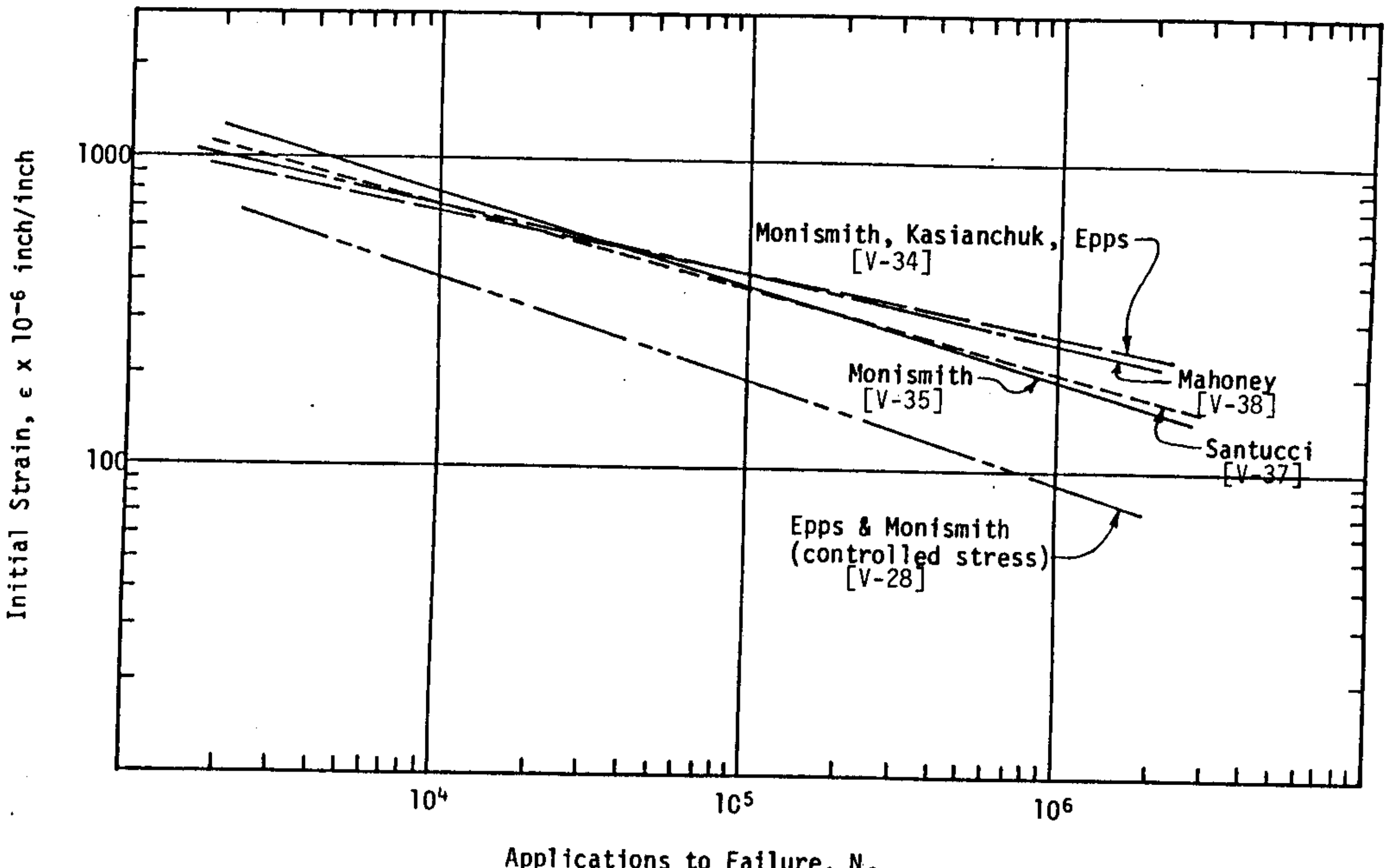

Figure V-7. Fatigue Models for Controlled Strain Testing [after Ref. 25].

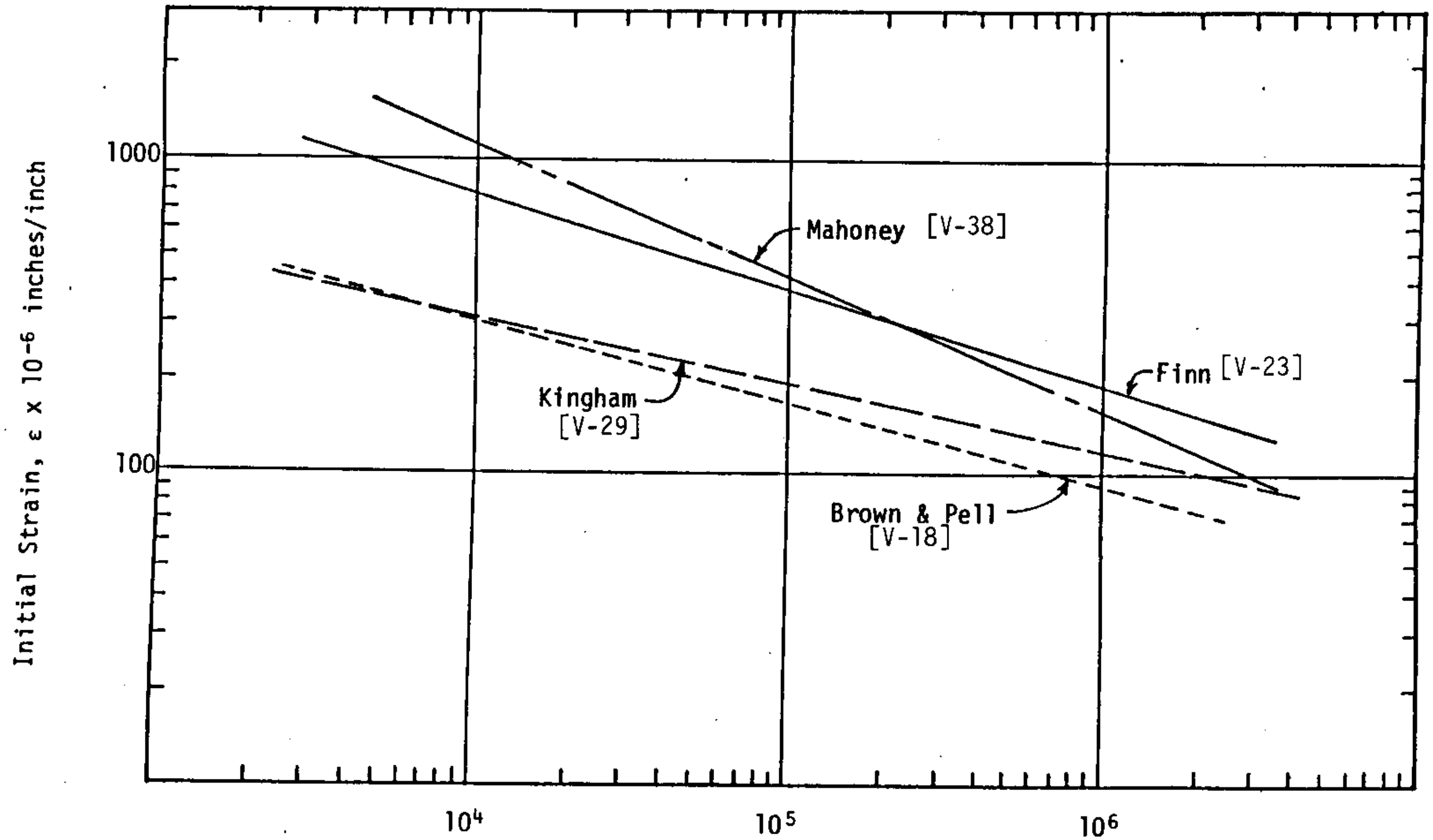

Figure V-8. Fatigue Prediction Models (E - 7.0 x 10^5 psi) [after Ref. 25].

BISAR layered elastic computer program. The fatigue curve was developed as the best fit of the combined data points of these two approaches.

The model developed by Finn, et al. [V-23] was based on AASHO Road Test data. The Kingham fatigue model [V-29] was also based on evaluation of AASHO Road Test data. The curve reported by Brown and Pell [V-18] is also shown in Figure V-8 even though the stiffness is unknown. This model is based on a laboratory determined curve displaced to give actual lives 20 times those obtained on test specimens. The other fatigue model based on a stiffness of 4.6×10^5 psi (not shown in Figure V-8) was developed by Austin Research Engineers (ARE) [V-39]. This model is based on data from 27 AASHO Road test sections which included traffic repetitions and predicted tensile strains.

In Figure V-8 the relationships by Kingham, and Brown and Pell appear to be conservative at this level of stiffness, while the relationships by Mahoney and Terrel, and Finn, et al. provide similar estimates of pavement life. The slightly steeper slope of the Mahoney and Terrel model may have been influenced by the reported heavy rains which occurred at the test track shortly after initial cracks had formed. Moisture may have entered the pavement subsurfaces weakening the pavement system [V-38].

CURRENT PRACTICE USING PAVEMENT RATING

The most important need for condition surveys is to establish trends of pavement condition with time for use in pavement rehabilitation decisions. Reasons for making condition surveys are either to determine rideability or to determine structural adequacy.

There are three important aspects of the pavement condition rating:

1. Evaluating pavement condition on a systematic basis.
2. Defining unacceptable pavement conditions in terms of parameters used to evaluate the pavements.
3. Using the conditon rating criterion in the pavement design procedures.

In the following sections a discussion of different methods used to perform pavement condition is presented.

Present Serviceability Index.

The most widely used criterion is Pavement Serviceability Index (PSI). The criterion was developed during the AASHO Road Test. The PSI is based upon the concept of correlating user opinions with measurements of road roughness, cracking, patching, and rutting (this condition was discussed in Chapter IV).

Pavement Condition Data

The broad definition of "pavement condition data" includes the following [V-40]:

1. Roughness (ride),
2. Surface distress,
3. Structural evaluation (surface deflection), and
4. Skid resistance.

Each of the above types of condition data (individually or in combination) have been used as "failure criteria". However, the roughness data type was discussed in Chapter IV and skid resistance is not directly considered in structural overlay design (and hence will not be discussed in this report).

Pavement surface distress and deflections can be used as failure criteria in overlay design and therefore merit discussion.

Surface Distress

Distress surveys are commonly grouped into three categories:

1. Fracture,
2. Distortion, and
3. Disintegration.

For each category of distress data, it is necessary to identify individual distress types, corresponding amount and severity, and locations. Detailed definitions for individual distress types have been provided by Smith et al. [V-41].

The types of distress data commonly collected by various agencies are shown in Tables V-4 and 5 for a group of nine agencies surveyed within the last three years (including WSDOT) [V-43]. As shown in Table V-4 (flexible pavements), essentially all of the surveyed agencies use some measure of cracking. Specifically, transverse longitudinal, and alligator cracking are commonly used. Most agencies measure rutting, raveling, patching, and flushing.

There appears to be less uniformity among the agencies regarding rigid pavement distress (refer to Table V-5). General measures of cracking, spalling, faulting, settlement, pumping, joint separation, raveling, popouts, scaling, and patching are the most common types of data collected.

A wide range of methods are used to condense distress data into useful information. However, a common procedure is to associate deduct (penalty) points with specific distress type,

Table V-4. Types of Distress Data Collected for Flexible Pavements [Ref. V-43]

Distress Mode	Distress Type	Agency								
		Ariz.	Calif.	Fla.	N.Y.	Ont.	Pa.	USAF	Utah	Wash.
Fracture	Cracking									
	Generalized	●		●						
	Transverse		●			●	●	●	●	●
	Longitudinal		●			●	●	●	●	●
	Alligator		●			●	●	●	●	●
	Block		●				●	●		
	Other					●	●	●		
Distortion	Rutting	●	●	●		●	○	●	●	●
	Corrugations					●	●	●		●
Disintregation	Raveling		●	◐		●	○	●	●	●
	Stripping						○	●		
	Polishing							●	●	
Other	Patching		●	●		●	○	●	●	●
	Potholes			◐			●			
	Flushing			◐		●	○	●	●	●

[a] ● Required; ○ optional; ◐ data collected on specific projects only.

Table V-5. Types of Distress Data Collected for Rigid Pavements [Ref. V-43]

Distress Mode	Distress Type	Agency								
		Ariz.	Calif.	Fla.	N.Y.	Ont.	Pa.	USAF	Utah	Wash.
Fracture	Cracking									
	General	●	●[a]					●		●
	Transverse			●		●	●	●		
	Longitudinal			●		●	●	●		
	Diagonal			●		●				
	D					●		●		
	Corner			●		●		●		
	Other					●				
	Spalling		●	●		●	●	●		●
	Shattered Slab			●			●			
Distortion	Rutting									●
	Settlement					●	●	●		●
	Faulting		●	●		●	●	●		●
	Pumping			●				●		●
	Joint Separation		●	●		●				
	Blow Up					●	●	●		●
	Warping					●				●
Disintegration	Raveling			●[b]		●				●
	Popouts			●[b]		●		●		●
	Scaling			●[b]		●		●		●
	Polishing					●				
Other	Patching		●	●		●	●	●		●
	Potholes					●	●			

[a] Composed of 1st, 2nd, and/or 3rd stage cracking.
[b] Evaluated by use of one distress type termed "surface deterioration."

severity, and extent combinations. These points can then be summed and subtracted from some upper limit or maximum value (usually 100). A generalized relationship for this concept was described by Shahin and Darter [V-42]:

$$\text{Rating (distress) Score} = C - \left[\sum_{i=1}^{n} \sum_{j=1}^{m} a(T_i, S_j, E_j) \right]$$

where:

C = initial rating (distress) number, and

$a(\ )$ = weighting factor or deduct points, which is a function of distress type T_i, severity of distress S_j, and extent of distress E_{ij}.

The use of such information by a number of surveyed agencies include the following [V-43]:

1. Establish priorities (priority programming),
2. Determine maintenance or rehabilititation strategies, and
3. Predict performance.

Initially, similar data developed and collected by WSDOT was used principally for priority programming. However, with the amount of data now being acquired, performance history and future performance projections are being developed. These projections are being used to prepare recommendations for scheduling highway maintenance and rehabilitation.

Due to the amount and accessibility of the WSDOT surface distress data, it is planned to use this information in developing "failure criteria" such as fatigue relationships from state maintained pavements.

The principle of the WSDOT system is to assign a weighting value for the severity and extent of several types of distress [V-44]. Assuming a value of 100 (no distress), the sum of the weighted value is subtracted from 100 to establish the overall distress rating. The general formula for calculating the Combined Pavement Rating (CPR) is:

$$CPR = (100 - \Sigma D)(1.0 - 0.3(\frac{CPM}{5000})^2)$$

where:

CPR = combined pavement rating,

D = weighted distress value, and

CPM = Counts per mile acquired with a Cox Ride Meter.

It is obvious that the CPR formula emphasizes the distress rating more than the ride rating. In Washington State, it has been found that there is little correlation between the progression of distress and the deterioration of ride. Even though extensive longitudinal cracking or alligator cracking exists, the pavement may ride well. Only when the pavement begins to break up does it demonstrate poor rideability.

A performance curve can be established for each road section by plotting the CPR with time or traffic. The general form of the performance equation adopted by Washington State Department of Transportation is:

$$R = C - mA^B$$

where: R = Combined pavement rating,

A = Age of the pavement,

C = Model constant for maximum rating (100),

m = Score coefficent, and

B = Constant that controls the degree of curvature.

Two considerations are emphasized in identifying the significant categories of distress:

1. The consistency of rating with time, and
2. The relationship of each to rehabilitation criteria.

Types of distress which have been found to agree with these two considerations are: transverse cracking, longitudinal cracking, alligator cracking and patching. All other categories of distress are presently unweighted and employed as supplemental information only. Figure V-9 represents the distress weightings presently being used in WSDOT.

WSDOT uses two performance levels. The "should" level and the "must" level. The "should" level is at a score of 60 where a pavement shows some type of distress. The "must" level is set at a score of 40, where the pavement distress is apparent and some type of rehabilitation must be scheduled.

TYPES OF DATA COLLECTED BY WSDOT

Pavement data in the Washington State Department of Transportation are stored in several files. These files are:

1. Road life history file,
2. Roadway inventory file,
3. Annual traffic file,
4. Surface friction file, and
5. Pavement condition rating files.

The following is a brief description of these files which are related to overlay design. The reader is advised to consult Reference V-44 for more information.

PAVEMENT CONDITION RATING
BITUMINOUS PAVEMENTS
DEFECT DEDUCTIONS

PMS

Negative Values are Assigned to the Failures by Degree

RUTTING PAVEMENT WEAR	Average Depth in Inches		Throughout Rated Section					
			None	1/4-1/2	1/2-3/4	3/4+		
		(1) 1/4-1/2"		0				
		(2) 1/2-3/4"			0			
		(3) Over 3/4"				0		Negative Values
CORRUGATIONS WAVES SAGS HUMPS	Percent of Roadway		Change Per 10 Feet in Inches					
			None	1/4-2	2-4	4+		
		(1) 1-25		0	0	0		
		(2) 26-75		0	0	0		
		(3) 76+		0	0	0		Negative Values
ALLIGATOR CRACKING			Percent of Wheel Track Per Station					
			None	1-24	25-49	50-74	75+	
		(1) Hairline		20	25	30	35	
		(2) Spalling		35	40	45	50	
		(3) Spalling & Pumping		50	55	60	65	Negative Values
RAVELING OR FLUSHING				Local-ized	Wheel Paths	Entire Lane		
		(1) Slight		0	0	0		
		(2) Moderate		0	0	0		
		(3) Severe		0	0	0		Negative Values
LONGITUDINAL CRACKING	Lineal Feet Per Station		Average Width in Inches					
			None	1/8-1/4	1/4+	Spalled		
		(1) 1-99		5	15	30		
		(2) 100-199		15	30	45		
		(3) 200+		30	45	60		Negative Values
TRANSVERSE CRACKING	Number Per Station		Average Width in Inches					
			None	1/8-1/4	1/4+	Spalled		
		(1) 1-4		5	10	15		
		(2) 5-9		10	15	20		
		(3) 10+		15	20	25		Negative Values
PATCHING	Percent Area Per Station		Average Depth in Inches					
			None	0-1/2	1/2-1	1+		
		(1) 1-5		10	15	20		
		(2) 6-25		15	20	25		
		(3) 26+		20	25	30		Negative Values

Figure V-9. Weighting Values for Bituminous Pavements [Washington State Department of Transportation]

Roadlife History

The Roadlife History is basically a milepost by milepost breakdown of the entire construction history of every mile of roadway on the state system. Record layout is such that each record represents a homogeneous roadway section. Every surfacing action from the date of original construction to the most recent rehabilitation is noted by type, depth, and date. Also recorded are functional classification, type of highway configuration, base material types and depths, and provisions for locating added lanes and old P.C.C. pavements.

Annual Traffic File

The Annual Traffic File provides traffic volumes at all locations throughout the network. Items contained in this file include: average annual daily traffic, growth rate, single unit truck percentages, combination truck percentages, K factor for reducing AADT to a design hour volume, D factor for splitting the DHV into a directional volume, and three previous year's AADT.

Pavement Condition Rating Files

These files (one for each pavement condition survey) are a collection of the raw coded ratings for each of several surface distress categories, together with roughness data. Items coded in this file include ratings for flexible and rigid pavements. Pavement rutting and wear, alligator cracking, ravelling or flushing, longitudinal cracking, transverse cracking, and patching are recorded for bituminous pavements. Cracking, ravelling-disintegration, popouts-scaling, joint spalling, pumping and blowing, blowups, faulting-curling warping-settlement, patching, and pavement rutting and wear are recorded for portland cement concrete pavements. This file has been the basis for establishing pavement rehabilitation priorities.

CURRENT PRACTICES USING SURFACE DEFLECTION MEASUREMENTS

Deformation

Deformation can be classified as permanent and transient. Further, permanent deformation can be divided into consolidation and plastic deformation. Also, the transient deformation can be divided into viscoelastic deformation and elastic deformation.

Transient deformation is one which disappears when the load producing it is removed. In viscoelastic deformation, a certain time lapse exists between the removal of the load and the complete recovery of the deformation. In elastic deformation, the recovery occurs immediately after removal of the load.

The discussion here will be limited to the transient deformation (deflection) since this is the type of deformation is used to design and evaluate the pavement.

Two definitions need to be stated in this regard: critical deflection and deflection attenuation. The critical deflections represent the maximum pavement deflection allowable before failure occurs in form of fatigue cracks. Deflection attenuation is the ability of a given thickness of asphalt concrete overlay to reduce measured deflections to a tolerable level. This can be determined either by (1) determining the ratio of the strength of asphaltic concrete to the strength of gravel and then developing the deflection attenuation relationship based on the reduction capabilities of gravel, or (2) by measuring pavement deflection before and after overlay and observing the actual percent reduction. The tolerable deflections represent the end of the design life of a pavement which has been subjected to an indicated number of applications of 18 kip equivalent load. A comparison of the tolerable deflection with the existing pavement deflection defines the required deflection reduction necessary to prevent fatigue cracking during a selected design period.

Critical Deflection Values.

Pavement strength can be defined as the ability of a pavement structure to support an imposed load without permanent deformation. Therefore, critical deflection values must be established at a level less than that at which permanent deformation or rupture will occur.

Since the strength of a flexible pavement section is a function of temperature and moisture content, the critical deflection should be defined as a time dependent variable to account for the seasonal variation which the pavement will experience during its life. A summary of the deflection critical values found in the literature along with their associated agencies were listed in Table V-6.

Structural failure is often related to functional failure. An indication of structural failure is increasing surface deflection under a given load. However, since maximum deflection under the load does not indicate in which layer the failure is occurring, some current practices allow for use of deflection basins. Some of these methods may be found in Chapter III, Table III-2.

Table V-6. Criteria Deflection Values.

Reference	Deflection Criteria	Remarks
Huculak [V-45]	Spring Δ_{max} = 35 mils Summer Δ_{max} = 50 mils	Conventional flexible pavement. Deflection measured under 18 kip axle.
Ruiz [V-45]	Δ_{all} = 20 to 28 mils	Conventional flexible pavement. BB. Deflection under 18 kip axle.
Aratangy [V-46]	Δ = 20 mils 20 mils $<\Delta_{all} <$ 39 mils $\Delta_{all} >$ 39 mils	Conventional flexible pavement. Pavement in good condition. Pavement in fair condition. Pavement in poor condition for an axle load of 12,000 lb.
Lassal and Langumier [V-46]	31 mils $\leq\Delta_{all}\leq$ 20 mils	Conventional flexible pavement. Deflection measured under 13 kip axle.
Hveem [V-47]	Δ_{all} = 17 mils Δ_{all} = 20 mils Δ_{all} = 25 mils Δ_{all} = 12 mils Δ_{all} = 50 mils	Conventional flexible pavement. 4 in. of AC on granular base. 3 in. of AC on granular base. 2 in. of AC on granular base. Cement treated based with Surface treatment deflection measured under 15 kip axle load.
Road Research Lab. Britain [V-46]	20 mils $\leq\Delta_{all}$ <80 mils 5 mils $\leq\Delta_{all}\leq$12 mils	Granular-base pavement with bituminous surfacing. Cement-treated base pavement with bituminous surfacing.
Carneria [V-46]	Δ_{all} = 20 mils Δ_{all} = 28 mils	For heavy traffic and medium to heavy traffic highway.
California [V-48]	Δ_{all} = 17 mils Δ_{all} = 20 mils Δ_{all} = 25 mils Δ_{all} = 36 mils Δ_{all} = 50 mils	4 in. AC. 3 in. plant mix on gravel base. 2 in. plant mix on gravel base. 1 in. road mix on gravel base. 1/2 in. surface treatment.

Table V-6. Criteria Deflection Values (continued)

Reference	Deflection Criteria	Remarks
Asphalt Institute [V-49]	$\Delta_{all} = f(T_i, N)$	DTN = Design traffic = average daily 18 kip axle loads. Δ_{all} = Allowable maximum deflection (plus two standard deviations) Examples: Δ_{all} = 70 mils for DTN = 8 Δ_{all} = 30 mils for DTN = 230 Benkelman Beam Deflections
Nagumo [V-50]	$\text{Log } N = 0.179\,\Delta^2 - 1.117\,\Delta + 6.772$	N = Accumulated traffic volume of large commerical vehicles over 18 kips Δ = Benkelman Beam Deflections
Joseph and Hall [V-51]	$\Delta = 1.1315/N\ 0.233$	Δ = Initial deflection (mils) under a given load. N = Repetition to failure of that load.
Norman, et al. [V-52]	$\Delta = f(\text{pavement type})$	Graphical relations between N and Δ_{all} Example: Pavement with unbound base $N = 10^6$, Δ_{all} = 35 mils Pavement with cemented base $N = 10^6$, Δ_{all} = 25 mils Pavement with bituminous base $N = 10^6$, Δ_{all} = 35 mils
Lister [V-53]	$N = f(\Delta\ \text{in, pavement type})$	N = Cumulative number of 18 kip axle repetitions Δ in = Initial Benkelman beam deflection (14 kip axle) Graphical relation between N and Δ in for different pavement types. For AC pavement with granular base layer: Δ in = 20 mils, $N = 4.5 \times 10^6$ Δ in = 40 mils, $N = 0.5 \times 10^6$
Whiffin, et al. [V-54]	20 mils $\leq \Delta_{max} \leq$ 30 mils 5 mils $\leq \Delta_{max} \leq$ 15 mils	AC over granular base AC over cement treated base. Benkelman beam deflections under 14 kips.
State of Louisana [V-55]	$\Delta_{all} = f(N)$	Graphical relations between N = number of load repetitions and Δ_{all} = Dynaflect deflection Example: $N = 10^6$, Δ_{all} = 0.5 mils $N = 10^5$, Δ_{all} = 1.3 mils
Kansas [V-56]	$DMD = f(ADL_{18})$	DMD = Dynaflect maximum deflection. ADL_{18} = Average daily load, EQ.18K Example: ADL_{18} = 20, DMD = 1.7 mils ADL_{18} = 100, DMD = 0.8 mils

RELATIONSHIPS BETWEEN PERFORMANCE AND NUMBER OF EQUIVALENT LOAD REPETITIONS

Serviceability can be defined as the ability of a pavement to serve the traffic for which it was designed. Pavement performance is the ability of a pavement to adequately serve traffic over a time period. There are many variables affecting pavement performance. Some of these are: present serviceability, thicknesses of the pavement layers, stiffness of the pavement layers and subgrade, traffic loads carried by the pavement over time, climatic or environmental effects, construction effects, and type and degree of maintenance.

The most widely used method of relating performance to number of equivalent load repetitions is the AASHO method. The basic performance equation for flexible pavement at the AASHO Road Test is:

$$G_t = \beta(\log W_t - \log \rho)$$

where:

G_t = a function of the ratio of loss in serviceability at time t to the potential loss taken to a point where PSI = 1.5,

β = a function of design and load variables that influence the shape of the performance curve, equal to $0.40 + 1094/(SN + 1)^{5.19}$, for the AASHO Road Test conditions, and for an 18,000 lb. single axle load,

SN = structural number,

W_t = axle load application to time t, and

ρ = a function of design and load variables denoting the expected number of axle load applications to a serviceability index of 1.5.

$\log \rho = 9.36 \log (\overline{SN} + 1) - 0.20$ for the AASHO Road Test condition, and for 18,000-lb single axle load.

Combining the above equations, gives the number of axle loads, W_{t18}:

$$\log W_{t18} = 9.36 \log (\overline{SN} + 1) - 0.20 + \frac{G_t}{0.40 + 1094/(\overline{SN} + 1)^{5.19}}$$

where:

$G_t = \log (4.2 - P_t)/(4.2 - 1.5)$

P_t = serviceability index at time t

This equation is directly applicable only to the AASHO Road Test conditions in this form. The concepts of soil support and the regional factor were developed to make the equation more applicable for different soil types and climatic conditions. Using this relation in different climatic and soil areas may not give satisfactory performance predictions. Other models relating the performance to number of equivalent load repetitions can be found in Reference V-57.

One of the most acceptable and widely used distress prediction model is the one developed by Finn, et al. [V-23]. The model was developed for fatigue cracking at the AASHO Road Test:

$\log N_f = 15.947 - 3.291 \log (\varepsilon/10^{-6}) - 0.854 \log (E/10^3)$ for fatigue cracking up to 10 percent of the wheel path area, and

$\log N_f = 16.086 - 3.291 \log (\varepsilon/10^{-6}) - 0.854 \log (E/10^3)$ for fatigue cracking more than 45 percent of the wheel path area.

where:

N_f = number of load application to failure,

ε = tensile strain at the bottom of asphalt layer, and

E = complex modulus.

Finn, et al. [V-23] indicated that in theory, these relationships could apply to areas other than the AASHO Road Test, with or without spring thaw, providing the fatigue properties of the asphalt concrete do not vary significantly from those of AASHO Road Test.

CHAPTER SUMMARY AND FUTURE DEVELOPMENTS

This chapter has been used to present an overview of the development to date of pavement failure criteria based on laboratory and field conditions. Of particular interest to WSDOT is a field fatigue criterion. In a pavement section, pavement failure may be defined in terms of:

1. pavement condition ratings,
2. deflection measurements, or
3. a combination of pavement condition ratings and deflection measurements.

The last option should be studied carefully in the establishment of WSDOT failure criteria. This is the most desirable method since deflection measurements will not necessarily be indicative of serviceability and pavement condition ratings provide no measurement of structural adequacy. By reviewing pavement condition rating files and deflection histories (where available), relationships between serviceability and structural deterioration with time may be developed for different types of highway construction. These can be used in pavement maintenance forecasting as well as in the design of pavement overlays.

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CHAPTER VI
SEASONAL VARIATIONS

INTRODUCTION

The response of a pavement system to a loading condition will vary with changes in moisture and temperature. Moisture will affect subgrade and base course behavior in a pavement. Generally speaking, as the moisture contents of a base and subgrade increase, these layers become weaker. The degree of weaking will depend upon the drainage characteristics of the pavement as well as the type of soil.

Moisture may also influence the behavior of an asphalt concrete surface course if the drainage of the pavement is such that the surface remains in contact with water for an extended period of time. The behavior of asphalt concrete in the presence of water depends mainly upon the hydrophyllic or hydrophobic nature of the aggregate and the permeability of the mixture.

The effects of temperature upon subgrade and base responses are due to mainly to the freezing and thawing of water present in these layers. The presence of ice will stiffen the pavement system. The system will weaken considerably upon thawing of the ice. Evaporation of moisture will be controlled by the ambient temperature and relative humidity. Therefore, more moisture may accumulate in the subgrade and base in the winter than in the summer.

Asphalt is a temperature dependent material. The stiffness of asphalt concrete will decrease with increasing temperature. Thus, the amount of vertical or horizontal strain that an asphalt concrete can withstand without fracture will be greater in the summer than in the winter. This also means that there is a greater potential for rutting in the summer.

An acceptable pavement evaluation and design system must account for seasonal variations in temperature and moisture. A variety of studies have been conducted in various parts of the country to investigate the effects of seasonal changes. Some of these may be found in References VI-1 through VI-12. Likewise, a number of methods for dealing with seasonal changes in pavement evaluation and design have been developed for specific environmental conditions (VI-5, VI-6, VI-7, VI-9, VI-10, VI-11, VI-12, VI-13 and VI-14). This chapter will examine these studies and procedures.

ESTABLISHMENT OF CRITICAL PERIODS

A critical period for a pavement may be thought of as the time or times of a year when the system is weakest and, consequently, most susceptible to damage from traffic. In many cases this period may occur in the spring when snow and ice melt and the water infiltrates into underlying layers of soil. In agricultural areas, irrigation water may run off into roadside ditches and saturate subgrade and soil base course material in the summer. This would have to be considered in addition to a greater potential for deformation in the asphalt concrete during the summer. Some areas which are not subject to freezing receive more precipitation in the winter than other times of the year. This moisture, coupled with less evaporation, may make the critical period sometime in the winter. Thus, for a particular pavement, the critical period is dependent upon temperature changes, moisture amounts, moisture sources and drainage.

The Asphalt Institute [VI-13] recommends two methods for determining the critical period of a pavement. The preferred method is to continuously monitor pavement deflections on similar pavements in similar environments with similar subgrades. The less desireable option is to use engineering judgment to estimate the critical period.

The former method was used by Scrivner, et. al., in a study of pavements in Minnesota and Illinois [VI-1]. The purpose of this research was to investigate the use of Dynaflect measurements to detect seasonal variations in flexible pavement performance. The results were then used to develop load limit criteria to protect asphalt concrete pavements from overloading during the critical period. Figure VI-1 shows the typical yearly deflection history for a flexible pavement in the Minnesota-Illinois study. Also shown are the depth of frost and axle load limits for various times of the year. It should be noted that the critical period occurred in the spring. The surface curvature index (SCI) and maximum Dynaflect deflection (DMD) both increased during this period. It was decided to use the SCI as the parameter for establishing load limits. This will be discussed later in this chapter.

Several years later, Metwali investigated the relationships between deflection measurements made in spring and other times of the year using data he had collected as well as data presented in Reference VI-1 [VI-7]. These relationships are shown in Figure VI-2. Here it can be seen that for Illinois, deflections are

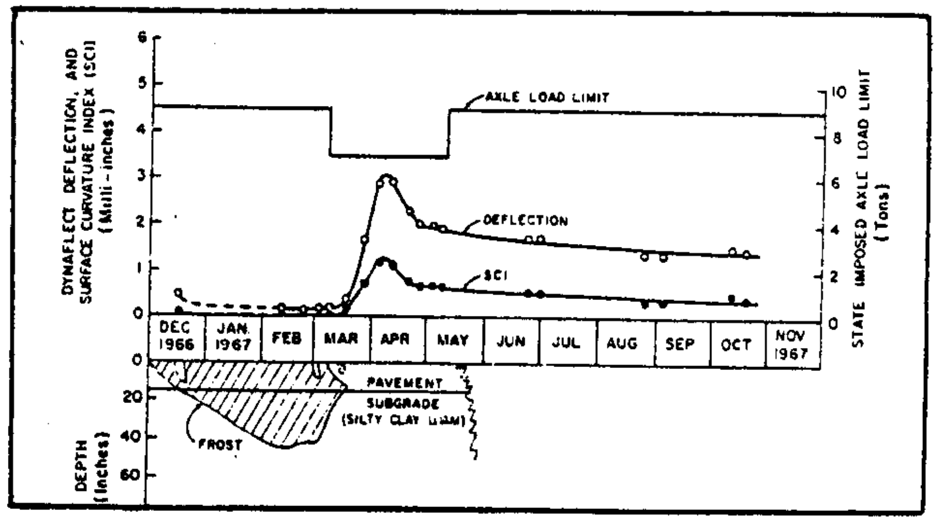

Figure VI-1. Yearly Deflection History for a Typical Flexible Pavement in Minnesota and Illnois [after Ref. VI-1]

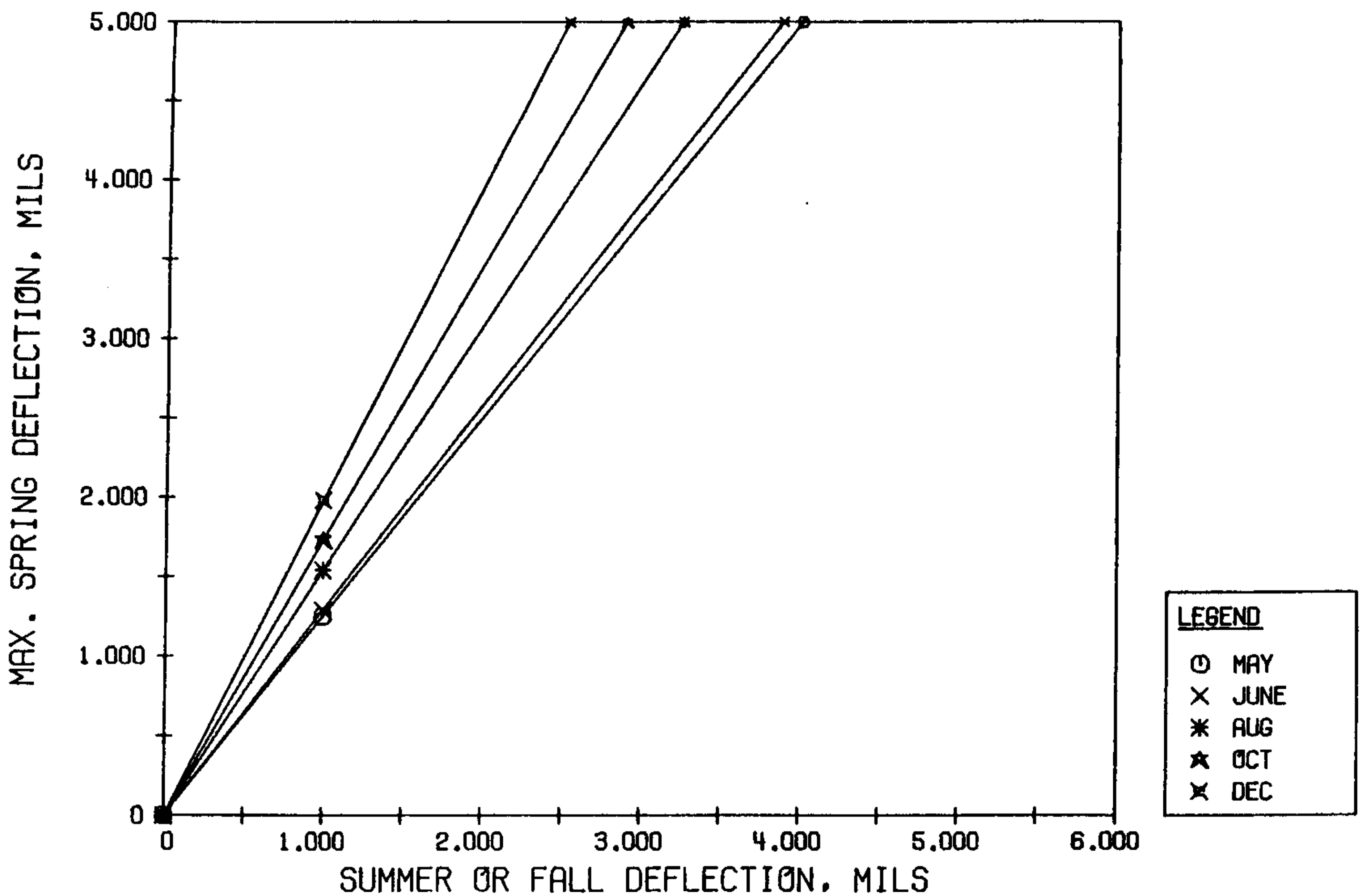

Figure VI-2. Relationship Between Summer and Fall Maximum Deflections and Maximum Spring Deflection for Asphalt Pavements for Illionis [after Ref. VI-7] higher in the spring than in other times of the year.

A similar study using Dynaflect deflections was conducted in Texas [VI-2]. The higher deflections were found to occur during the spring and summer. A sine curve model was developed to describe yearly deflection histories. This model was:

$$W_1(t) = A_1 + A_2 \sin \left[\frac{2\pi(t-A_3)}{365}\right]$$

where: $W_1(t)$ = expected deflection, mils, at time t,

t = time of year, days ($0 \leq t \leq 365$),

A_1 = estimated mean annual deflection,

A_2 = amplitude of sine curve (difference between maximum or minimum and mean,

A_3 = time of year, days, during period of increasing deflection, when $W_1(t) = A_1$

One of the interesting points of the Texas study was that deflection changes within a particular length of road will vary more due to random changes in the pavement and subgrade than for seasonal variations.

Stubstad and Connor [IV-6] used FWD data and the FROST computer program to determine critical periods for Alaskan highways. The FROST program uses the FWD load and seven deflection basin values to compute 1) the estimated depth of thaw, 2) the corrected center deflection for a 9,000 lb half-axle load with no frozen materials and adjusted to a surface temperature of $70^{\circ}F$ and 3) the approximate vertical strain in a granular base under a 9,000 lb load. It was found that although there was a large difference between center deflections measured on partially frozen and unfrozen materials, the vertical strain

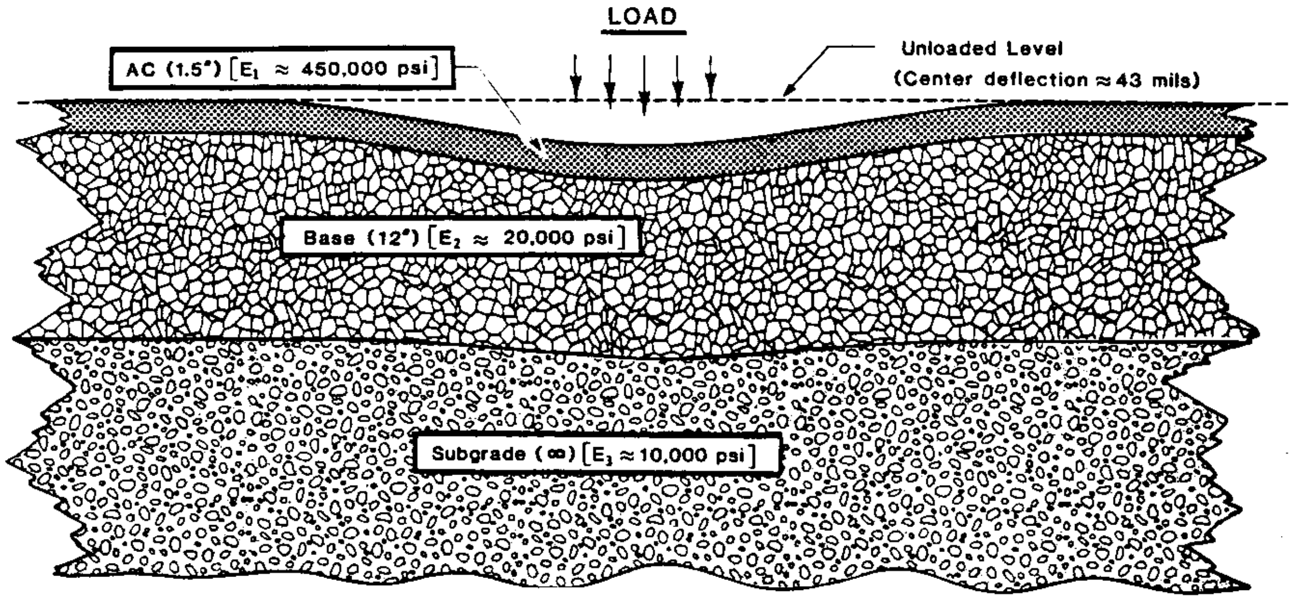

Figure VI-3. Hypothetical Case for an Unfrozen Asphalt Pavement Under a 9,000 lb. Load [after Ref. IV-6]

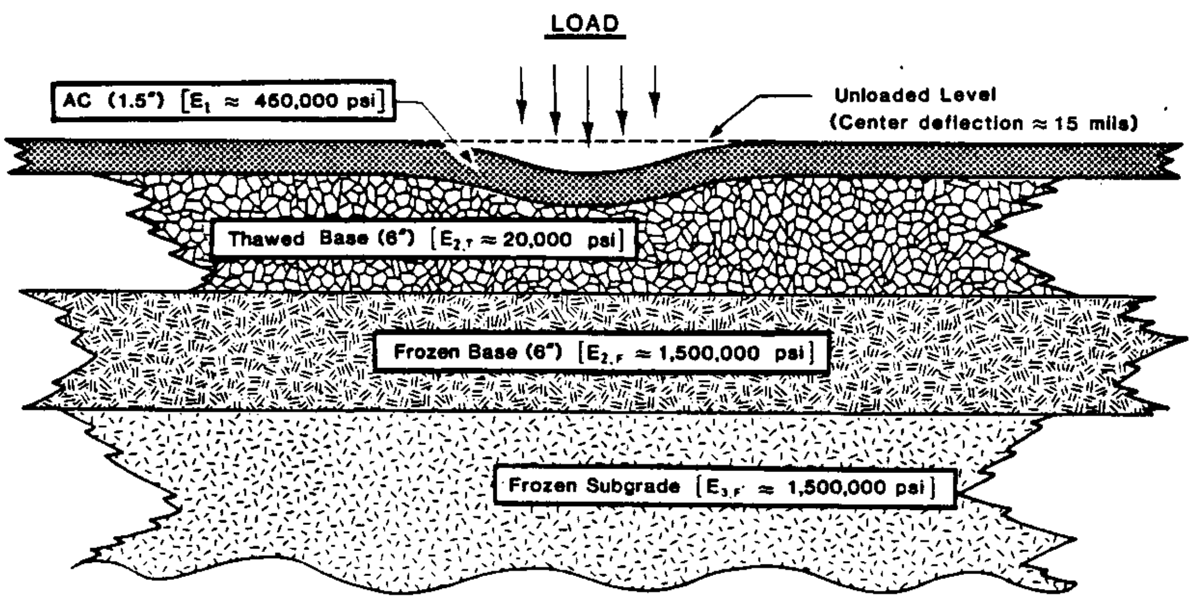

Figure VI-4. Hypothetical Case for a Partially Frozen Asphalt Pavement Under a 9,000 lb Load [after Ref. VI-6] at the top of the granular base and horizontal strain at the bottom of the surface course were approximately equal for the two conditions. The frozen and unfrozen cases are shown in Figure IV-3 and IV-4. The FROST program was able to determine the depth of the thaw to within one foot at shallow thaw depths and to within two feet at greater depths. The conclusion from this study was that the vertical strain at the top of a granular base course was a good indication of structural integrity.

In the development of a pavement overlay design procedure for the state of Kansas, a very thorough study of seasonal variations was conducted for different regions of the state [VI-12]. The state was divided according to average annual rainfall, subgrade soil type, and average temperature. The flexible pavement test sites within each region were chosen according to pavement thickness, traffic, pavement condition and subgrade modification. Pavement temperatures were adjusted to a baseline of 80°F according to the Asphalt Institute's method [VI-13]. The multiplier used was referred to as the temperature adjustment factor (TAF). Seasonal adjustment factors (SAF) were developed from continuous monitoring of the pavement sites in each region. These were developed for the Dynaflect parameters of maximum deflection, surface curverature index, and base curvature index. The representative deflection equations were of the following form:

$$x = (\bar{x} + z\sigma_x) \times TAF \times SAF$$

where: x = representative deflection parameter,

$\bar{x}$ = mean of deflection parameter,

z = distance from mean to a selected significance level on a normal distribution curve and

σ_x = standard deviation of deflection parameter.

It was found that the fifth sensor deflection and the spreadability factor required no seasonal adjustments.

A study of seasonal effects was conducted for Region 6 of the U.S. Forest Service on four Forest Service roads in Oregon and Washington [VI-3 and VI-4]. Extensive laboratory and field data on local weather conditions and material properties were gathered during the course of this research. It was found that an excellent correlation exists between soil suction and subgrade moisture content. Excellent predictions of base and subgrade moduli were obtained from the parameters of bulk or deviator stress, moisture content and dry density. This is interesting since it means that a series of relatively simple tests could be used to estimate modulus values. A dual paramtric approach [VI-11] using the Dynaflect parameters of spreadability and maximum deflection was found to reasonably predict subgrade modulus.

SEASONAL CONSIDERATIONS IN PAVEMENT DESIGN

There are two general methods of accommodating seasonal variations in pavement design. The first method is to design the pavement such that traffic induced deflections will not exceed a selected value in the critical period. The second method is to establish a load limit for the critical period of the year. Both of these approaches have merits which would make them attractive in certain instances. The first method may require more initial construction cost than the second. However, placing a load limit on a pavement may cause an economic hardship for road users.

A study conducted in Idaho attempted to compare load limit alternatives based upon predicted asphalt fatigue behavior [VI-5]. It was stated that the magnitude of critical tensile strain caused by loads during various seasonal conditions could be calculated using layered elastic theory with seasonal values of resilient modulus and Poisson's ratio. Load limit alternatives were compared using Miner's law:

$$\frac{D_A}{D_B} = \frac{\left(\sum\limits_i \sum\limits_j \frac{n_{ij}}{N_{ij}}\right)_A}{\left(\sum\limits_i \sum\limits_j \frac{n_{ij}}{N_{ij}}\right)_B}$$

where: D_A = damage factor for load limit A,

D_B = damage factor for load limit B,

n = number of accumulated repetitions,

N = number of repetitions to failure,

i = number of repetitions of a particular load, and

j = various seasonal physical conditions

This relationship was used to establish axle load limits during the critical period that produced the same rate of fatigue consumption as normal load limits during other times of the year.

In the Minnesota-Illinois study, Scrivner, et al. established a load limit using the maximum value of the surface curvature index, SCI. The relationship was:

$$L_s = \frac{k}{\max SCI}$$

where: L_S = safe axle load

k = proportionality constant

The proportionality constant was found to be approximately 6.3. Figure VI-1 shows how the safe axle load changed for the critical period.

Kinchen and Temple [VI-9] developed an overlay design procedure for Louisiana which accounted for seasonal variations in temperature and subgrade moisture. A percent reduction in deflection was calculated for a given overlay thickness based upon a combination of temperature and subgrade moisture effects. This method was proposed in order to allow the pavement designer to select an appropriate thickness for the design life of the overlay.

Thompson and Hoffman [VI-14] proposed the use of weighted mean monthly air temperature to adjust the temperature of the asphalt concrete to a standard spring temperature. This was done in order to compare asphalt modulus values taken at different times of the year. They also developed climatic adjustment factors for a variety of subgrade soil types. These adjustment factors are presented in Table VI-1 along with an explanation of how to use them in adjusting subgrade moduli.

A method for establishing load limits on Alaskan highways with FWD data is presented in Reference VI-6. In this procedure, the maximum measured deflection for a particular time of year is compared to an acceptable deflection level for a particular class of road. If the measured deflection exceeds the acceptable

Table VI-1. Climatic Adjustment Factors for Various Subgrade Conditions [after Ref. VI-14]

USDA Soil Types	USDA Internal Drainage Class			
	Well Drained or Better		Other USDA Drainage Classes	
	Freeze-Thaw	No-Freeze-Thaw	Freeze-Thaw	No-Freeze-Thaw
silt, silt loam, loam, sandy loam	.70	.85	.50	.60
silty clay loam, clay loam, sandy clay loam, sandy clay, silty clay, clay	.65	.85	.50	.75

Notes:

1. E_{ri} determined for the SUMMER/FALL period is assigned a factor of 1.0 (no adjustment required)
2. To predict SPRING E_{ri} from SUMMER/FALL data, multiply by the appropriate adjustment factor from Table 1.
3. To predict SUMMER/FALL E_{ri} from SPRING data, divide the Spring E_{ri} by the appropriate adjustment factor from Table 1.

deflection, the load limit is reduced accordingly. This is illustrated in Figure VI-5.

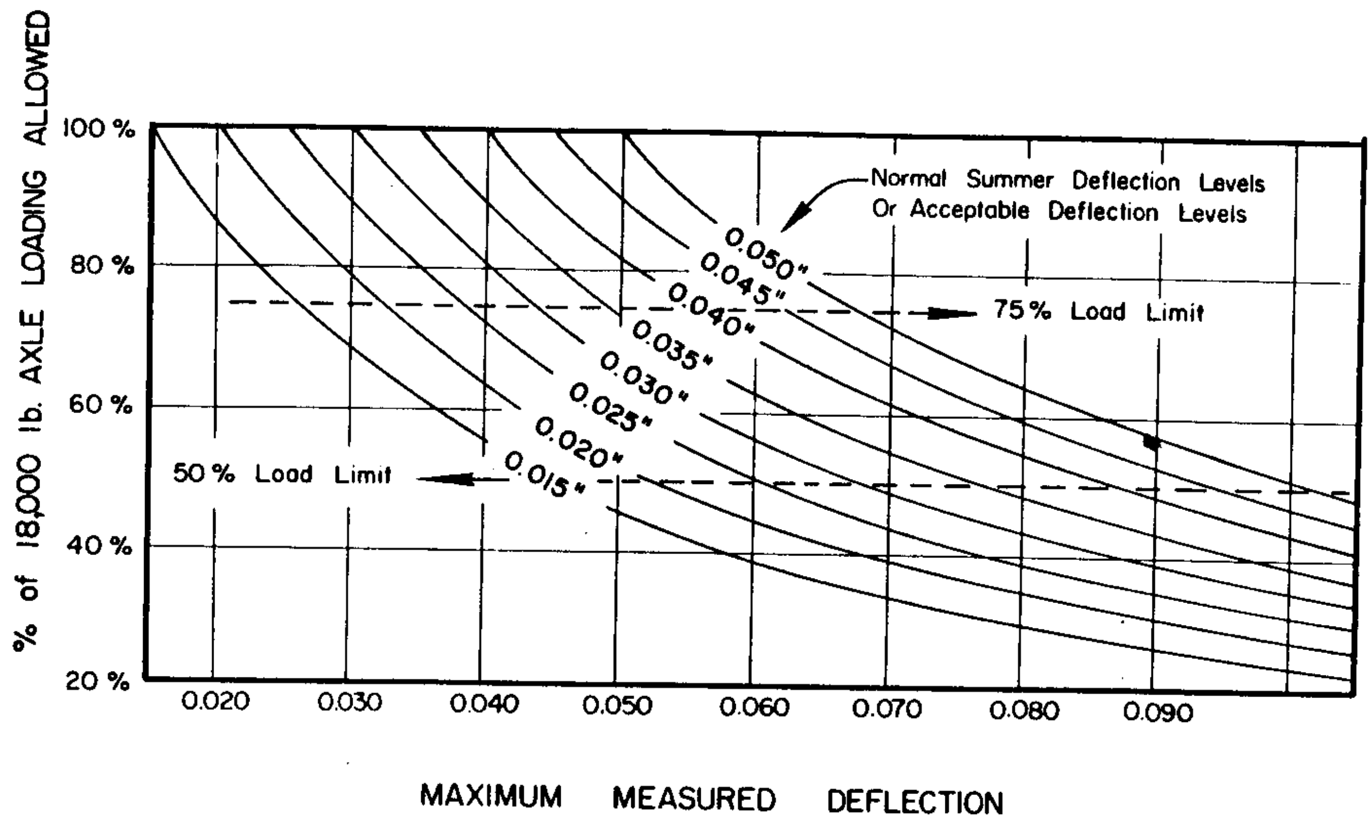

Figure VI-5. Load Limit Criteria Based Upon FWD Deflections [after Ref. VI-6]

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CHAPTER VII
RESEARCH PLAN

INTRODUCTION

The fundamental goal of the research will be the development of a mechanistic based overlay design procedure for pavement rehabilitation which can employ deflection data gathered with the Falling Weight Deflectometer. An important result of this study will be an improved understanding of environmental effects upon pavement structures. In order to accomplish this, a number of objectives must be met during the course of the project. The major components of the study will be the development and implementation of the overlay design procedure. Each of these will consist of a logical sequence of tasks.

It should be noted that the work will be accomplished in cooperation with the WSDOT. The interaction between WSDOT and the University of Washington will be valuable since it will allow the researchers to more efficiently integrate the overlay design procedure into the existing pavement rehabilititation system.

STUDY OUTLINE

The following projects, phases, tasks and subtasks have been identified for incorporation into the study:

PROJECT 1 - Develop Overlay Design Procedure

- PHASE 1.1 - Assemble overlay design components
  - TASK 1.1A - Evaluate existing computer programs
    - SUBTASK 1.1A1 - Layered elastic
    - 1.1A2 - Finite element

TASK 1.1B - Develop analysis procedure compatable with the FWD

TASK 1.1C - Identify pavement monitoring sites

TASK 1.1D - Establish pavement failure criteria

SUBTASK 1.1D1 - Identify seasonal changes

1.1D2 - Establish load equivalencies

1.1D3 - Verify construction history

1.1D4 - Incorporate pavement ratings

TASK 1.1E - Mateials testing

SUBTASK 1.1E1 - In situ (FWD) - WSDOT

1.1E2 - Laboratory - WSDOT

1.1E3 - Establish NDT - Material Property Relationships

TASK 1.1F - Evaluate analysis procedure on existing pavement sections

PHASE 1.2 - Integrate overlay design components

TASK 1.2A - Identify pavement overlay design concept

TASK 1.2B - Combine performance criteria and analysis procedure

TASK 1.2C - Conduct preliminary evaluation of overlay design procedure

TASK 1.2D - Prepare interim report to document progress to date

PROJECT 2 - Implement Overlay Design Procedure

PHASE 2.1 - Implement overlay design procedures on WSDOT projects

TASK 2.1A - Develop guidelines for implementing Overlay Design Procedure

2.1B - Develop operational capability within WSDOT

2.1C - Initiate use of Overlay Design Procedure

PHASE 2.2 - Monitor selected projects

TASK 2.2A - Develop criteria for selection of monitoring sites and samples

2.2B - Monitor selected sites and perform analysis

2.2C - Prepare final Overlay Design Procedure document

Phase 1.1 shows considerably more detail than the rest of the study. This is because the later parts of the study are dependent upon the findings of the first phase.

Task Description - Project 1 - Phase 1.1

Task 1.1A. Evaluate existing computer programs. Four programs are currently under consideration for use in the overlay design procedure. Two of these,BISDEF and BISAR, are based upon layered elastic theory. The other programs, ILLI-PAVE and ILLI-SLAB, are finite element programs. The identification of these four programs will not necessarily eliminate the consideration of other available programs. This task will be used to develop the best approach or combination of approaches for the overlay design procedure.

Task 1.1B. Develop analysis procedure compatible with FWD. Computer programs identified in Task 1.1A will be used in conjunction with other analysis techniques to develop an algorithm for use with FWD deflection basins. This algorithm will be used to back-calculate elastic moduli as well as other material properties for any pavement structural section.

Task 1.1C. Identify pavement monitoring sites. In order to test the algorithm from Task 1.1B and successfully accomplish Task 1.1D, it will be necessary to carefully choose pavement sections in various stages of service condition. The selection of these sections will depend upon the consideration of subgrade type, construction type, traffic and environmental conditions. It may be necessary to use statistical procedures to establish a partial factorial experiment in order to accommodate these variables.

Task 1.1D. Establish pavement failure criteria. There are four elements within this task. These are 1) identification of seasonal changes, 2) establishment of load equivalencies, 3) verification of construction history and 4) incorporation of pavement rating scores.

Seasonal changes in pavement response will be measured on roads of differing construction, subgrade and environmental condition. As was stated in the previous task, these roads will be identified at the outset. Various environmental conditions will be chosen according to available climatological data on temperature and precipitation. Other sources of subgrade moisture will also be considered in choosing sites. This part of the task will result in an understanding of how in situ material properties change with time.

It may be necessary to verify traffic counts on the monitoring sites. This would include weight and axle count data.

These data would then be used to compute the number of repetitions for a given level of pavement rating. This relationship would have to be established for a number of pavement construction types. Thus it will be necessary to verify the construction histories of the sites by sampling cross-section material.

Task 1.1E. Materials testing. Both in situ and laboratory materials characterization will be conducted by the WSDOT Materials Laboratory. The in situ properties will be derived from an analysis of FWD deflection data. These values will be compared to those measured in a laboratory using a triaxial repeated loading device. With this correlation, it should be possible to predict material properties directly from FWD data. The emphasis in this task is to establish standard test procedures for the determination of modulus values for all types of pavement materials. This will complement the work to be accomplished in Task 1.1B and be necessary for the successful completion of Task 1.1F.

Task 1.1F. Evaluate analysis procedure on existing pavement sections. FWD deflection basins will be used with the analysis procedure (computer program and other techniques) to evaluate the precision and accuracy of the analysis system. The FWD deflections will be obtained on the sites selected in 1.1C. These may include the reconstructed WSU test track and all ten of the Long Term Monitoring (LTM) sites. This will serve to minimize the amount of material sampling as well as laboratory and field testing.

Task Decription - Project 1 - Phase 1.2

Task 1.2A. Identify pavement overlay design concept. This will require the logical stepwise construction of the pavement overlay design procedure. This model will become the premise for using the data obtained in the first phase. In other words, this becomes the "first cut" in the development of the procedure. This procedure will then be tested in Task 1.2C.

Task 1.2B. Combine performance criteria and analysis procedure. This task will combine the analysis procedure, performance (failure) criteria and other required overlay design inputs into an overall, operational system. Presumably this will be in the form of a computer program (but does not exclude development of a hand-analysis method).

Task 1.2C. Preliminary evaluation of overlay design procedure. A maximum of 10 projects will be used to perform a preliminary evaluation of the overlay design system. These projects will have FWD deflection basin data avialable for use with the overlay design procedure. Further, these estimates will be compared to conventional, existing overlay design methods. If adjustments are required in the new overlay design procedure, they will be made during this task. These 10 projects should be ones previously selected for rehabilitation. Thus, at a minimum, FWD deflection basins will be obtained prior to rehabilitation at a spacing of not less than 100 ft nor more than 500 ft. Further, borings will be made to determine layer thicknesses and sample the principal structural materials (at a minimum the asphalt or portland cement concrete materials). Follow-up FWD deflection basins will be obtained approximately one and five months after rehabilitation.

Task 1.2D. Prepare interim report to document Project 1. All findings of Project 1 are to be presented in this document. This will include a detailed discussion of the pavement analysis procedure and pavement failure criteria chosen for incorporation to the WSDOT overlay design procedure. Pavement monitoring sites will be identified and the results of NDT-material property relationships will be discussed. The overlay design concept and procedure will be presented in its preliminary form.

Task Description - Project 2 - Phase 2.1

Task 2.1A. Develop guidelines for implementing design procedure. All required steps will be defined that are necessary to fully implement and field validate the new overlay design procedure. It is anticipated that the process will require adding specific steps (or subtasks) in the tasks which will follow in Project 2.

Task 2.1B. Develop operational capability within WSDOT Materials Laboratory. The overlay analysis package including the computer program(s) will be fully transferred from the University of Washington to the WSDOT Materials Laboratory and its associated computer system. Final debugging of the computer program(s) will be accomplished in this task.

Task 2.1C. Initial use of design method. At the commencement of this task, the WSDOT Materials Laboratory will start to use the new overlay design procedure on a regular basis.

Task Description - Project 2 - Phase 2.2

Task 2.2A. Develop criteria for selection of monitoring sites and sample. To field validate the overlay design procedure, detailed and long-term monitoring of several projects is required. These sites should represent a range of traffic, environmental and structural conditions for the State of Washington; thus, sample size will be carefully considered. Following this process, the actual sites will be selected, and data types and collection schedules will be prepared. It is anticipated that 25 to 50 sites will required.

Task 2.2B. Monitor selected sites and perform analyses. The selected sites will be monitored by use of data such as crack surveys, FWD deflection basins, truck counts and weights, etc. This information will be used as feedback into the overlay design procedure to validate, update and improve the overlay analysis.

Task 2.2C. Prepare final report for overlay design procedure. This task will culminate in a final report which will provide the following:

1. Provide an overview of all activities associated with the development and implementation of the overlay design procedure.
2. Provide an overview of the performance of the overlay design procedure.
3. Include final judgment concerning the future use of and modification of the overlay design procedure.

RESEARCH ADMINISTRATION

Appendix A contains all data pertinent to the administration the proposed study. Scheduling of the projects and phases are shown in Figure A1, Appendix A. The first project is scheduled to be completed after 36 months. Project 2, Implement Overlay Design Procedure, is scheduled to be completed after 11 years. Figures A-2 and A-3, Appendix A, show the details of Projects 1 and 2, respectively. The percentages of the projects to be completed by WSDOT and UW are shown in Table A-1. Table A-2 lists the effort estimated for the principals involved in the research.

CHAPTER VIII
CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

The following conclusions are warranted:

1. Available computer programs based on layered elastic theory or finite element computation methods can be used in the proposed WSDOT overlay design procedure.
2. Of the nondestructive pavement testing equipment reviewed, the falling weight deflectometer appears to best simulate traffic loads for determining in situ pavement material properties.
3. The traffic input necessary for the proposed overlay design procedure can be the currently used Traffic Index (based on the equivalent number of 5,000 lb. wheel loads) or 18,000 lb. equivalent single axle loads.
4. A number of failure criteria have been developed in previous research studies (field and laboratory). The most common and useful failure criteria is fatigue. Thus, fatigue (as a failure criterion) will receive the greatest emphasis in the proposed overlay design procedure.
5. The seasonal variation of pavement strength is an important parameter for design. Additional field data are required to better define the impact on overlay design in the State of Washington.
6. A research plan has been presented for the development of an overlay design procedure for use in Washington State. This plan encompasses a number of investigations

which will be necessary for the successful completion of the activity.

RECOMMENDATIONS

The following recommendations are warranted:

1. Additional test sites on the WSDOT maintained highway network are necessary for obtaining falling weight deflectometer deflection basins (and associated analysis). Such data will improve the design of pavement rehabilitation projects.
2. Full development of the proposed WSDOT pavement overlay design system should proceed. This is in part based on the fact that no known major obstacles exist which would preclude the system's successful development and implementation.
3. Further investigation is needed to properly ascertain the effects of seasonal changes upon pavement response. This is incorporated in the proposed work.
4. Specific failure criteria for Washington pavements need to be developed in order to optimize pavement maintenance management procedures.
5. A judicious review of pavement overlay design computer programs should be conducted to select the most appropriate programs for use by WSDOT.

APPENDIX A

SCHEDULE,

TASK RESPONSIBILITIES,

PERSONNEL COMMITMENTS (PROJECT 1) AND

PROPOSED BUDGET (PROJECT 1)

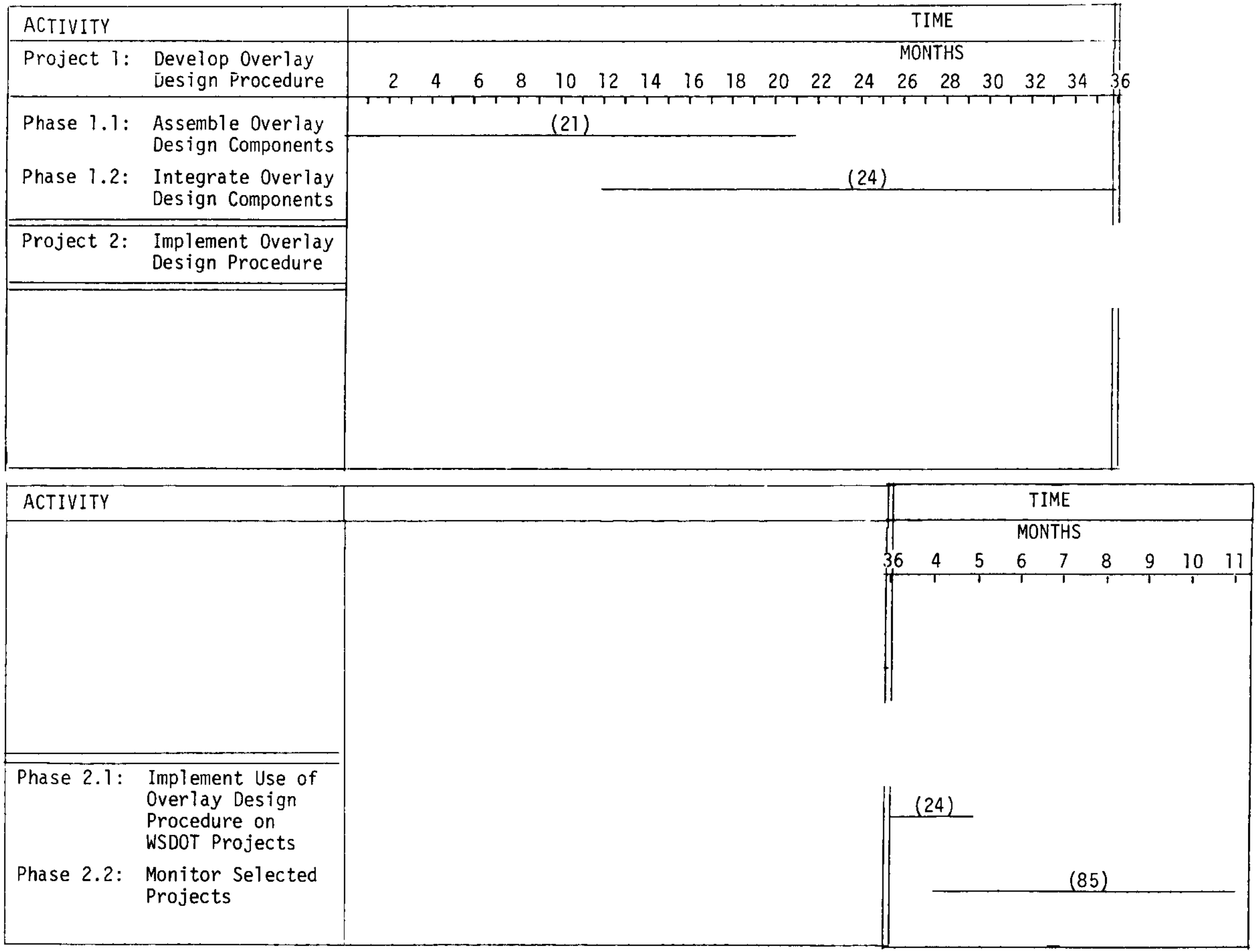

Figure A-1. Pavement Design and Performance Program Time Schedule

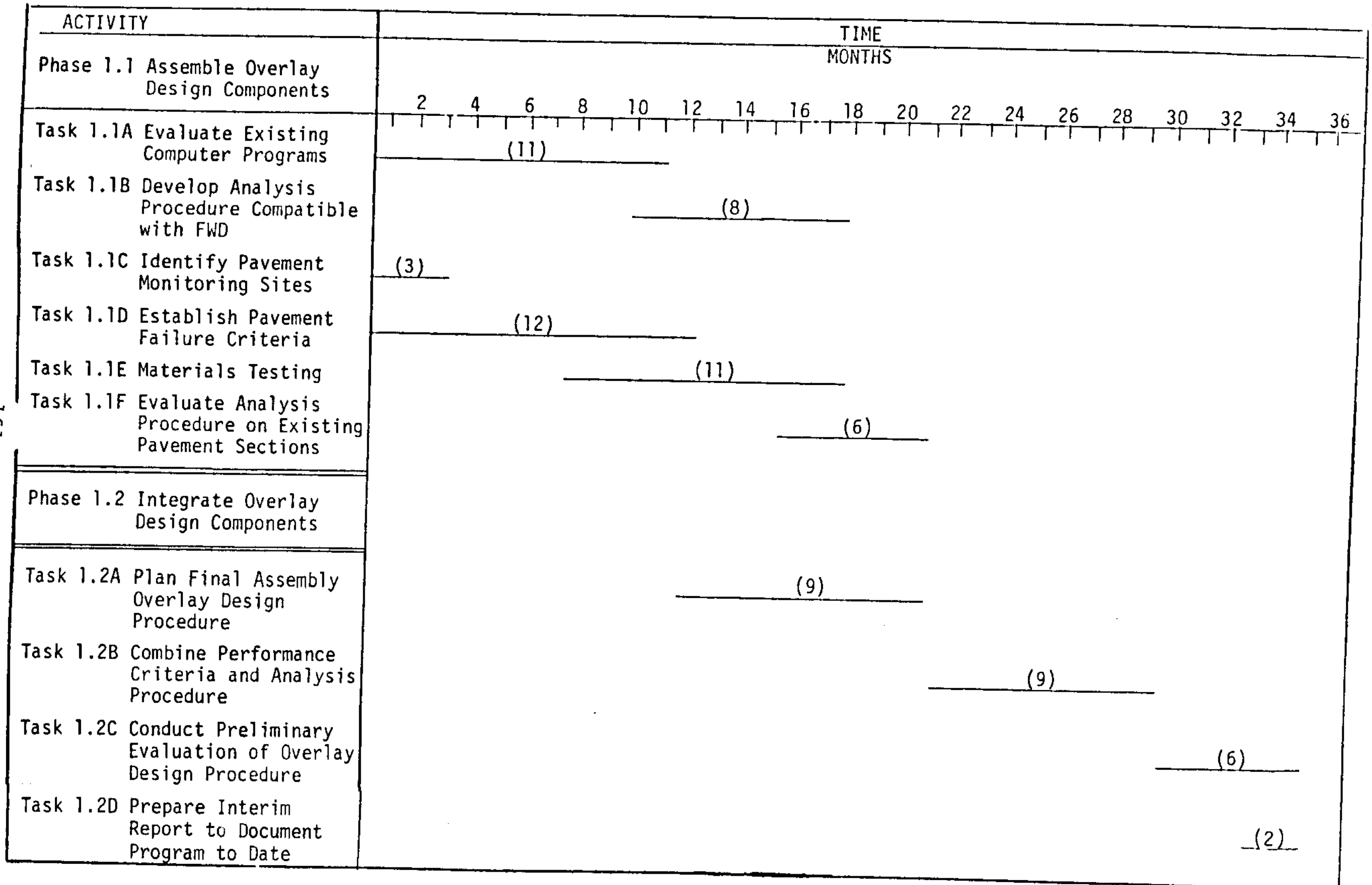

Figure A-2. Pavement Design and Performance Program Time Schedule - Project 1 (Develop Overlay Design Procedure).

TIME SCHEDULE - PROJECT 2 (IMPLEMENT OVERLAY DESIGN PROCEDURE)

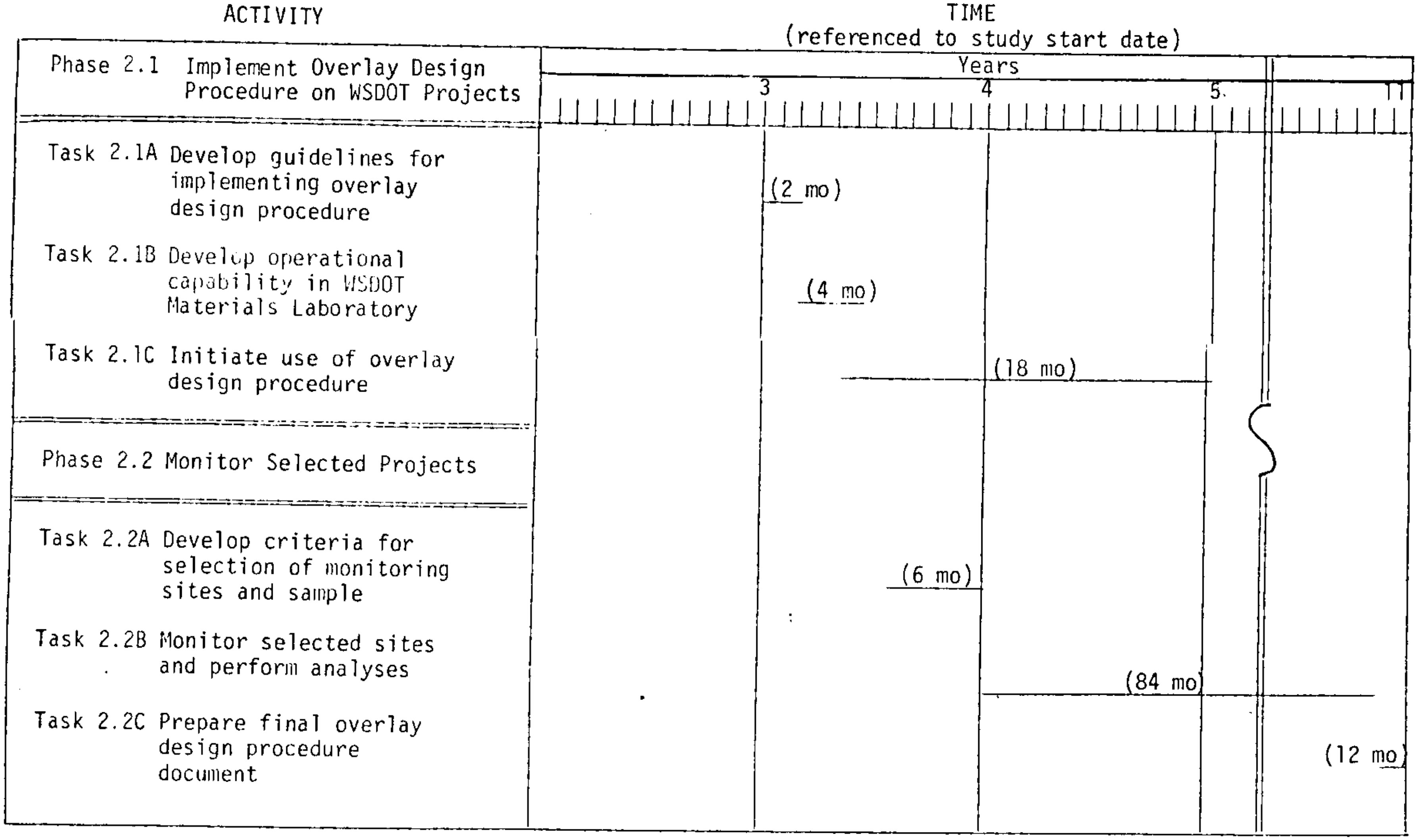

Figure A-3. Pavement Design and Performance Program

Table A-1. Task Responsibilities

Activity	Approximate Percent of Task Responsibility	
	WSDOT	UW
1. Project 1	30%	70%
(a) Phase 1.1	20%	80%
(i) Task 1.1A (ii) Task 1.1B (iii) Task 1.1C (iv) Task 1.1D (v) Task 1.1E (vi) Task 1.1F		
(b) Phase 1.2	40%	60%
(i) Task 1.2A (ii) Task 1.2B (iii) Task 1.2C (iv) Task 1.2D		
2. Project 2	85%	15%
(a) Phase 2.1	80%	20%
(i) Task 2.1A (ii) Task 2.1B (iii) Task 2.1C		
(b) Phase 2.2	95%	5%
(i) Task 2.2A (ii) Task 2.2B (iii) Task 2.2C		

Table A-2. Estimated Level of Effort and Task Breakdown for Primary Program Personnel.

Project Personnel	Role in Study	Tasks (Man-Months)										
		1.1A	1.1B	1.1C	1.1D	1.1E	1.1F	1.2A	1.2B	1.2C	1.2D	Total
1. Washington State DOT												
(a) A.J. Peters	Program Manager	0.0	0.1	0.1	0.0	0.1	0.2	0.2	0.1	0.1	0.1	1.0
(b) N.C. Jackson	PI - WSDOT	0.1	0.4	0.3	0.2	0.2	0.5	0.7	0.3	0.6	0.1	3.4
(c) D.M. Crimmins	Project Engineer	0.0	0.0	0.0	0.0	0.5	2.0	1.5	1.0	0.4	0.1	5.5
2. University of Washington												
(a) J.P. Mahoney	PI - UW	0.2	1.0	0.3	0.5	0.5	1.0	1.0	0.7	0.5	0.3	6.0
(b) D.E. Newcomb	Res Engr/Res Asst	2.8	3.0	1.5	2.0	0.0	2.0	3.3	3.5	3.0	0.5	21.6
(c) J. Sharma	Project Engineer	1.0	1.0	0.0	0.5	0.0	1.5	0.7	1.4	1.0	0.1	7.2