DEVELOPMENT OF AN IMPROVED OVERLAY DESIGN PROCEDURE FOR THE STATE OF ALASKA
Volume II: Final Report

by

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Oregon State University

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FOR THE STATE OF ALASKA

VOL. II - FINAL REPORT

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16. Abstract

The state of Alaska is currently developing a pavement management system for their road system. An important input to the system is the determination of flexible overlays based on the structural adequacy of the existing pavement. The current overlay procedure (the Asphalt Institute method) does not always show a need for overlays, despite the poor surface condition of the pavement and high traffic volumes. Therefore, an improved procedure that would consider not only traffic but also surface distress, the structural properties of the pavement, and, most importantly, the effects of a freezing and thawing of the base, subbase, and subgrade layers are needed. This report recommends two such methods for further consideration: a simplified mechanistic method using equations developed in Pennsylvania, and a mechanistic procedure employing a backcalculation computer program known as ELSDEF. The results of these analyses are compared with those of the Asphalt Institute procedure. It was determined that the Asphalt Institute procedure was inappropriate and underdesigned the overlay for the road sections. The Pennsylvania equations tended to be slightly more conservative than the mechanistic method using ELSDEF except for cases where the pavement fatigue life had been utilized completely by past traffic loads. It is recommended that Alaska utilize the mechanistic approach to design overlays and ELSDEF as the backcalculation procedure. The Pennsylvania equations can be used in those cases not requiring considerable accuracy.

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A NOTE FROM THE PROJECT MANAGER

During the past few years the Alaska Department of Transportation and Public Facilities' (DOT&PF) need for improved overlay design procedures has grown tremendously due to increasing traffic and reduction of available funds to maintain the highway network. Previous methods have had only limited success in adequately analyzing the alternatives available to the designer. This project does not and cannot stand alone. It is the combination of millions of dollars of research performed throughout the world. This report represents the state of the art in pavement design. Gary Hicks and Margot Yapp have done an excellent job of gleaning from the literature information which can be applied in Alaska.

The design approach recommended here allows the designer considerable flexibility in overlay design. The design approach may not be as rigid as some would prefer. It is hoped that by applying engineering principles, expertise, and judgment, more cost effective pavement overlay projects can be produced. As designers gain experience in the approach recommended here, they will find that they are no longer confined to the limits of past experience. This will allow more innovation in pavement design through the ability to analyze the performance of new materials and material response through the seasons. However, experience will continue to guide the designer in determining which design alternative is most cost effective for any project.

The principles presented in this report will provide new dimensions to the ability of the pavement designer's ability to predict pavement performance thereby allowing the selection of the best alternative within political and budget constraints. Although the ideal would be an overlay design without such constraints, we feel the resulting design process will offer significant benefits for Alaska DOT&PF.

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DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented. The contents do not necessarily reflect the official views of policies of the Alaska Department of Transportation and Public Facilities or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation. Also, the use of manufacturer's names does not constitute an endorsement of their products.

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1.0 INTRODUCTION

1.1 Problem Definition

The State of Alaska is currently developing a Pavement Management System to assist highway managers in optimizing decisions regarding what, when and where rehabilitation actions are needed for the state's road network (Woodward-Clyde, 1986). An important input to this system is the determination of the thickness of a flexible overlay based on the structural adequacy of the existing pavement. At present, Alaska uses the Falling Weight Deflectometer to measure pavement deflections and then determines overlay thickness by following the Asphalt Institute procedure (Asphalt Institute, 1983). This official procedure does not consider the remaining life of the pavement, nor the effects of frozen base and subgrade materials. Therefore, the resulting design thicknesses have turned out to be inadequate and on occasion, the procedure has shown no need for an overlay. An improved procedure using the current state-of-the-art would remedy this situation and may prove beneficial in terms of cost savings. It would also ensure good performance in future construction and rehabilitation activities and provide a procedure that is applicable to all regions in Alaska.

1.2 Objectives

In the development of an improved procedure for overlay thickness design, the following objectives are undertaken:

1. Evaluate existing overlay thickness design methods, both empirical and mechanistic, including that used by the state of Alaska,
2. Develop an improved framework for overlay design,

3. Collect data on selected projects including resilient elastic modulus and surface deflections,
4. Test the new overlay thickness design procedure and compare results with current overlay procedures, and
5. Develop a flowchart for implementation of the improved procedure.

1.3 Study Approach

The study approach consisted of two phases as shown in Figure 1.1. Phase I evaluated existing methods of overlay thickness design and developed a framework for a new procedure to be used in Alaska. In Phase II, testing was performed to modify the design procedure as needed. Once tested and evaluated, the overlay design procedures will be documented in a field manual.

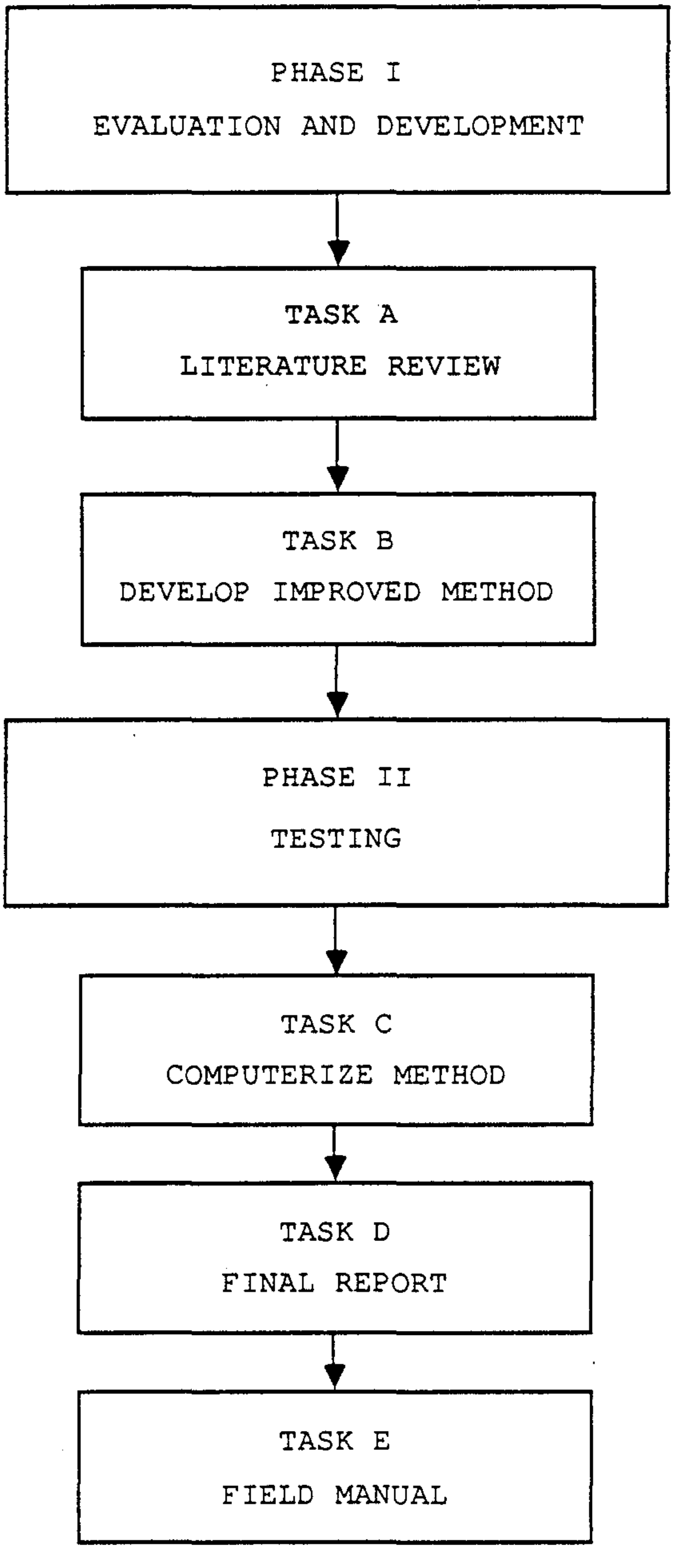

Figure 1.1. Flowchart of Study Approach.

2.0 BACKGROUND

This chapter describes Alaska's new Pavement Management System as well as their overlay design procedures. It describes the current overlay design procedures and discusses problems associated with this method. In addition, other overlay design procedures, deflection-based, simplified mechanistic and mechanistic, are reviewed and summarized. These procedures are used by other state and research agencies and give a perspective of the current state-of-the-art with regard to overlay design.

2.1 Pavement Management System For Alaska

As part of an effort to improve the cost-effectiveness of management policies applied to state-wide transportation facilities, the Alaska Department of Transportation and Public Facilities (ADOT&PF) has developed a Pavement Management System (PMS) (Woodward-Clyde, 1986). The PMS is a tool to assist highway managers in selecting the optimum rehabilitation or maintenance actions and scheduling where and when these actions should be applied. The objectives of the PMS include:

1. Quantifying current roadway conditions.
2. Determining optimal maintenance policies.
3. Providing correlations between pavement performance and construction parameters.
4. Assessing short- and long-term funding needs.
5. Providing a consistent, objective basis for funding requests and determining the consequences of alternative budget levels.
6. Providing decision-makers with the information needed to make informed decisions on road rehabilitation.

7. Improving agency credibility by showing that the consequences of choices have been objectively analyzed.

The PMS only manages in-service paved highways in Alaska. Unsurfaced roads and airport pavements have not been included in the system at the present time. Additionally, the focus of the PMS is at the network level, and not the project level.

The Markov decision process, a special class of the dynamic decision model, was selected as the basis for developing the PMS for Alaska. This model recognizes that the future pavement condition is not known with certainty, but the probabilities of future condition "x" occurring may be estimated. The model is illustrated in as a decision tree in Figure 2.1. A decision tree has two nodes, a decision node and a chance node, with several alternatives shown as branches at each node. At a decision node, the branches represent feasible alternative actions, while the branches at a chance node represent the possible outcomes of the action taken at the previous decision node. To analyze a decision tree, the probabilities and costs of different outcomes at each chance node must be estimated.

The main components of a Markov decision process as applied to a PMS are:

1. Condition states.
2. Alternative pavement preservation actions.
3. Cost of alternatives.
4. Performance of alternatives.

A condition state is defined as a combination of specific levels of the variables that affect the pavement performance. For example, if the relevant variables include pavement roughness and cracking, then one condition state

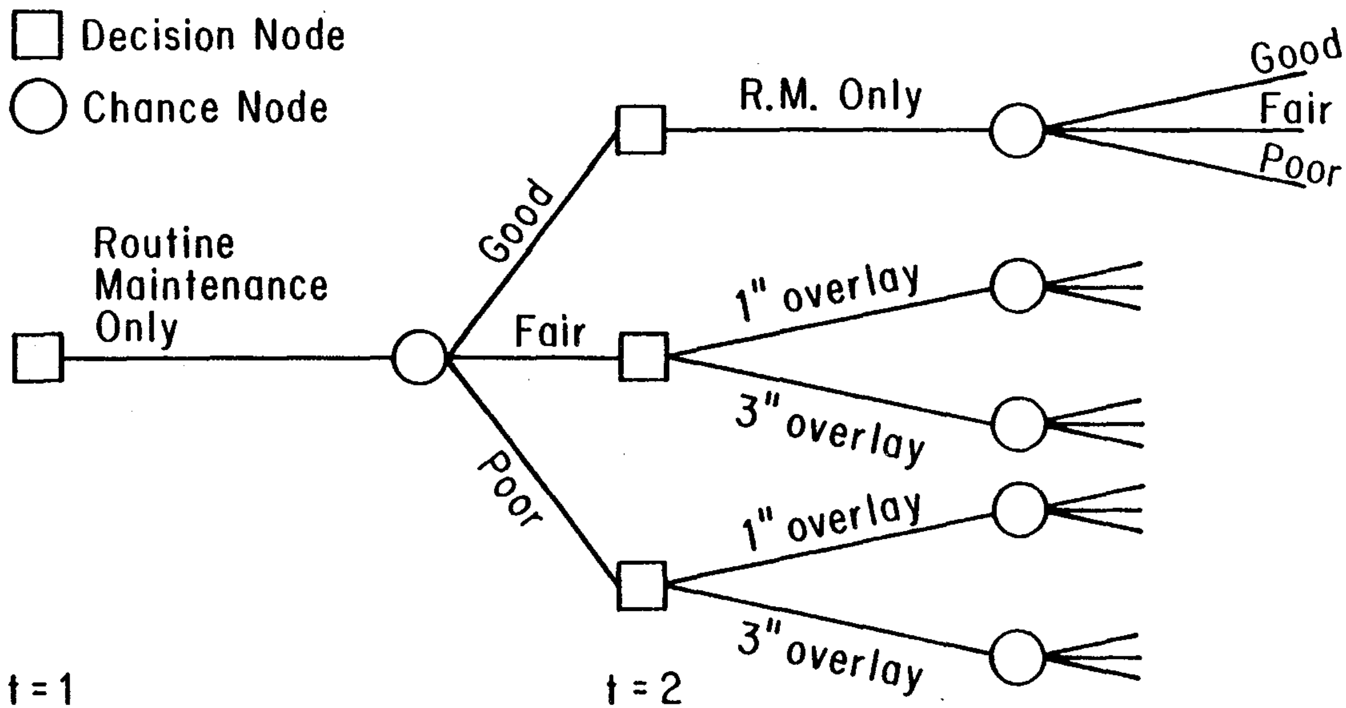

Figure 2.1 An Example of a Decision Tree (Woodward-Clyde, 1986).

might be the combination of roughness = 50 in./mile (79 cm/km) and cracking = 5%.

To implement the PMS, the following steps are required:

1. Divide network into uniform road segments: A segment length of one mile (1.6 km) is selected, which is consistent with present pavement condition survey procedures. Road segments also terminate at maintenance district boundaries.

2. Define road categories: All road segments in the network are grouped into mutually exclusive road categories. Road categories provide the means to separate road segments into groups that perform differently, or have different costs, or are of different relative importance. Four factors are used to define road categories:

   a. Six performance classes, ranging from an urban interstate to rural minor collectors.

   b. Foundation condition beneath the pavements; whether it is stable or unstable. Geologic information and soils data should be used unless not available, in which case the unstable condition may be defined as roughness greater than 250 in./mile (3.9 mm/m).

   c. Volume of traffic, measured in 18-kip (80kN) EALs.

   d. Maintenance districts - ADOT&PF is organized into five districts: Southeast, Interior, Western, Central and South-central.

3. Define distress and condition states for each road category: A distress state is a combination of specific levels of relevant distress types for pavements in a given road category. A condition

state expands the definition of a distress state by adding the variables that influence the rate of change of each distress type. The distress states are defined by discrete levels of roughness, fatigue cracking plus patching, and rut depth. Table 2.1 illustrates the 24 distress states available. To define condition states, two influence variables are added to the distress states, and they are the:

a. Index to first crack of the last major action, and

b. Change in fatigue cracking plus patching during the last time period.

Table 2.2 illustrates the criteria used in the above-mentioned variables. Table 2.3 gives a few examples of condition states for the Alaska PMS.

4. Select appropriate maintenance alternatives: Alternative maintenance actions are selected ranging from "do nothing" to "substantial corrective measures". Table 2.4 illustrates the list of alternative maintenance actions developed for the Alaska PMS. The basis for selecting the rehabilitation actions are not clear at this point. This report is only concerned with the overlay alternatives which form part of the available selections in the PMS.

5. Develop performance prediction models: Pavement performance prediction models were developed to estimate the probabilities of pavement deterioration. For the Markov decision process, the transition probability, p_{ij} (a_k) is used. This is the probability that a road segment will move from condition state i to j in unit

Table 2.1. Distress States for the Alaska PMS (Woodward-Clyde, 1986).

Distress States No.	Roughness Level	Roughness Range of Values (in./mile)	Fatigue Cracking & Patching Level	Fatigue Cracking & Patching Range of Values %	Rut Depth Level	Rut Depth Range of Values (in.)
1	1	0-55	1	0-5	1	0-0.5
2	1	0-55	1	0-5	2	> 0.5
3	1	0-55	2	6-15	1	0-0.5
4	1	0-55	2	6-15	2	> 0.5
5	1	0-55	3	16-40	1	0-0.5
6	1	0-55	3	16-40	2	> 0.5
7	1	0-55	4	> 40	1	> 0.5
8	1	0-55	4	> 40	2	> 0.5
9	2	56-95	1	0-5	1	0-0.5
10	2	56-95	1	0-5	2	> 0.5
11	2	56-95	2	6-15	1	0-0.5
12	2	56-95	2	6-15	2	> 0.5
13	2	56-95	3	16-40	1	0-0.5
14	2	56-95	3	16-40	2	> 0.5
15	2	56-95	4	> 40	1	0-0.5
16	2	56-95	4	> 40	2	> 0.5
17	3	> 95	1	0-5	1	0-0.5
18	3	> 95	1	0-5	2	> 0.5
19	3	> 95	2	6-15	1	0-0.5
20	3	> 95	2	6-15	2	> 0.5
21	3	> 95	3	16-40	1	0-0.5
22	3	> 95	3	16-40	2	> 0.5
23	3	> 95	4	> 40	1	0-0.5
24	3	> 95	4	> 40	2	> 0.5

1 inch = 2.54 cm
1 mile = 1.6 km

Table 2.2. Influence Variables Used to Define Condition States.

Level	Index to First Crack (Avg. No. of Years Before Occurrence of First Crack)
1	0 - 4
2	4.1 - 8
3	8.1 - 12
4	12.1 - 16

Current Fatigue Cracking Plus Patching (% area)	Rate of Change in Fatigue Cracking and Patching	
	Level	Range of Values (% area)
0 - 5	1	0
	2	1 - 3
	3	4 - 5
6 - 15	1	0 - 5
	2	6 - 10
	3	11 - 15
16 - 40	1	0 - 5
	2	6 - 10
	3	> 10
> 40	1	any change

Table 2.3. Condition States for the Alaska PMS (Woodward-Clyde, 1986).

Condition State No.	Index to First Crack	Roughness	Fatigue Cracking & Patching	Change in Fatigue Cracking & Patching	Rut Depth
1	1	1	1	1	1
2	1	1	1	1	2
3	1	1	1	2	1
4	1	1	1	2	2
5	1	1	1	3	1
6	1	1	1	3	2
7	1	1	2	1	1
8	1	1	2	1	2
9	1	1	2	2	1
10	1	1	2	2	2
11	1	1	2	3	1
12	1	1	2	3	2
13	1	1	3	1	1
14	1	1	3	1	2
15	1	1	3	2	1
16	1	1	3	2	2
17	1	1	3	3	1
18	1	1	3	3	2
19	1	1	4	1	1
20	1	1	4	1	2
21	1	2	1	1	1
:					
:					
40	1	2	4	1	2
41	1	3	1	1	1
:					
:					
60	1	3	4	1	2
:					
:					
240	4	3	4	1	2

Table 2.4. Alternative Maintenance Actions for the Alaska PMS.

Action Index	Action Description
1	Do nothing
2	Routine maintenance - crack sealing, patching
3	Seals - chip or slurry
4	Bituminous surface treatment (BST)
5	1 in. AC overlay
6	2 in. AC overlay
7	3 in. AC overlay
8	4 in. AC overlay
9	1 in. recycling AC surface
10	2 in. recycling AC surface
11	3 in. recycling AC surface
12	4 in. recycling AC surface
13	Structural strengthening
14	Cold recycling

1 inch = 2.54 cm

time if action a_k is taken at the present time. There are three component probabilities:

a. R_i, R_j - roughness levels for condition states i and j.

b. u_i, u_j - combination levels of fatigue cracking and patching for condition states i and j.

c. t_i, t_j - rut depth levels for condition states i and j.

However, because the highway system is relatively new and not many roads have required rehabilitation actions, available data are not adequate to estimate transition probabilities directly for most of the rehabilitation actions. Therefore, the experience of the department's engineers was used in addition to whatever data are available from past studies to estimate the probabilities.

6. <u>Develop cost estimation studies</u>: At present, only maintenance and construction costs are included in the PMS with the possibility of adding user costs in the future if needed. The model allows both costs to vary as a function of road category and prior pavement condition. From past construction records, costs were derived for different rehabilitation actions and for different regions. Maintenance cost factors were also derived for different road classes.

7. <u>Develop optimization model</u>: The optimization model uses the cost-minimization approach because it permits dividing the state-wide network into road categories and solving the optimization problem for each road category separately. There are two inter-related models incorporated in the PMS; long-term and short-term models. The long-term model is first solved to determine the

stationary policy that maintains the road network in a steady state. It also provides the target performance goal for the road network. There are two methods of specifying long-term performance standards:

a. By specifying minimum equivalent proportion segments to be maintained at x condition, or,

b. By identifying desirable and undesirable condition states, and then defining the minimum proportion of segments to be maintained at the desirable condition and the maximum proportion of segments to be in the undesirable condition state.

The short-term model standards may be different in order to permit a transition from present to the desired network condition. The model finds maintenance policies that would minimize total expected cost during the transition period subject to meeting short-term performance standards as well as long-term target performance goals.

This report interfaces with the PMS in step 4, the selection of appropriate maintenance alternatives. Specifically, this report only concerns itself with the overlay rehabilitation alternatives in Table 2.4. It should be emphasized that the PMS acts at a network level, with the entire road network in mind, whereas this report is concerned with roads at the project level. Also, the PMS is used to make budgetary decisions which this report is not equipped to do. Nonetheless, one of the aims of this report is to ensure that the overlay design procedure is consistent with the goals of the PMS, which is to provide an overlay that is efficient, cost effective and that meets performance standards.

2.2 Current Overlay Design Method In Alaska

The official flexible overlay design method (McHattie, 1985) currently used by ADOT&PF is essentially that contained in the Asphalt Institute's MS-17 (Asphalt Institute, 1983). Using this procedure, the Representative Rebound Deflection (RRD) at the center-of-load is measured with a Falling Weight Deflectometer (FWD). The average of the deflection values plus twice the standard deviation is used to enter Figure 2.2. The design traffic, in terms of equivalent 18-kip axle loads (EALs) is estimated; then the chart in Figure 2.2 is used to obtain the overlay thickness required.

The overlay design curves in Figure 2.2 were determined by solving Kirk's (1964) two-layer elastic theory equation for tolerable pavement deflection for various thicknesses (Kingham & Jester, 1983). The existing pavement and subgrade were represented by an effective modulus which is computed using the RRD with a one-layer Boussinesq equation. The thickness of the overlay required to reduce the RRD to the tolerable deflection was calculated using two-layer elastic theory. It was assumed that the modulus of the asphalt concrete overlay was 500 ksi (3.4 GPa). The charts have also corrected the deflections to 70°F (21°C). In addition, the frequency of loading was assumed to be 1 Hz which is equivalent to 3 to 5 mph (4.8 to 8 km/hr).

Using a more elaborate procedure, FWD deflection basin shapes may be fed into back calculation programs such as MODCOMP2, ISSEM4, or ELMOD, which determines the elastic resilient moduli for each of the pavement structure layers. At the present time, ADOT&PF uses MODCOMP2 although it may not necessarily be the best program available. It contains problem which, however, may not be overcome by other program (McHattie, 1985). Appendix A discusses several of these backcalculation programs in detail.

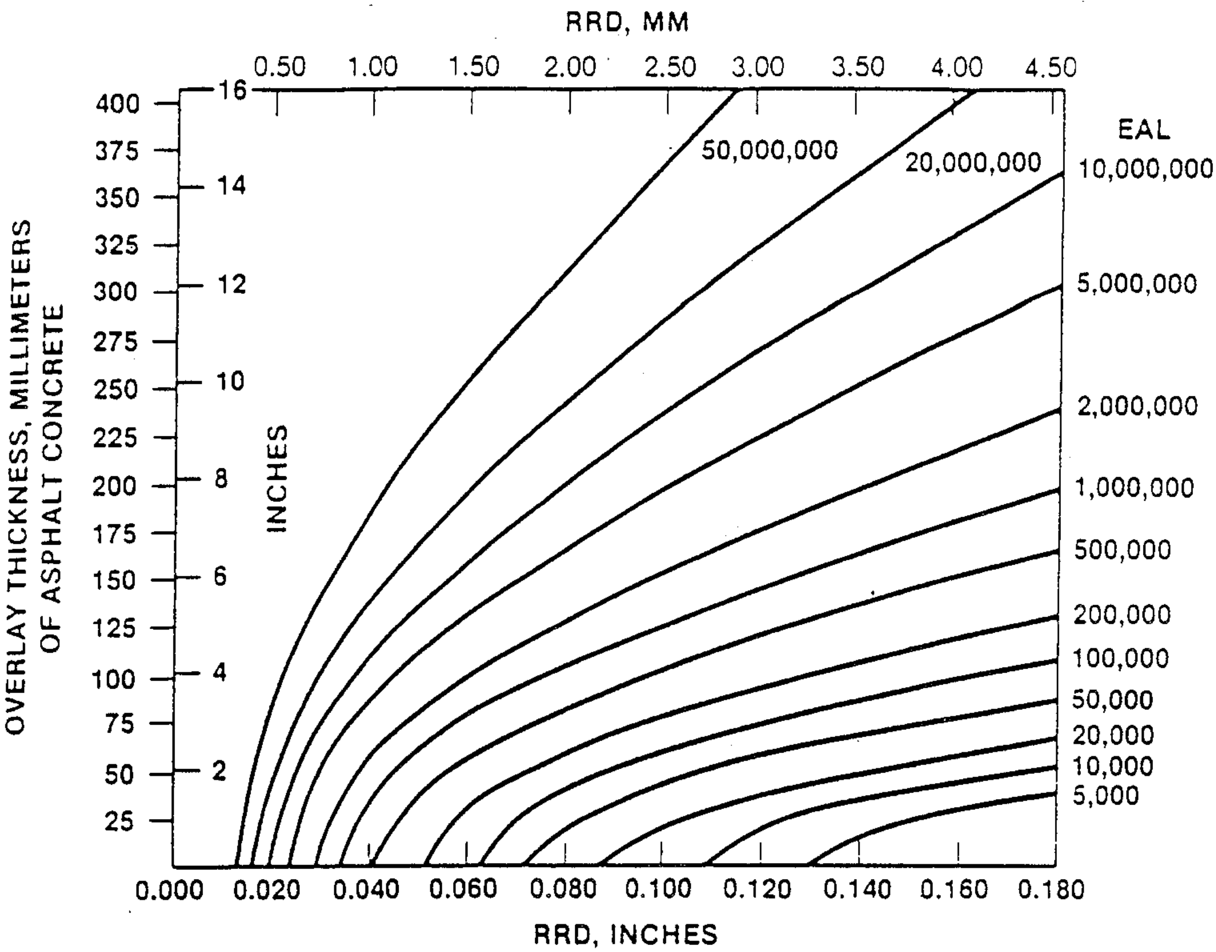

Figure 2.2 Asphalt Concrete Overlay Thickness Required to Reduce Pavement Deflections from a Measured to a Design Deflection Value (Rebound Test).(Asphalt Institute,1983).

The backcalculation procedures recalculate deflection basin shapes using progressively refined assumptions of elastic moduli values for each structural layer until the calculated deflection basin shape fits that measured in the field. When this occurs, the layer moduli values are assumed to have been correctly estimated. It is then possible to use forward calculating programs such as ELSYM5 or PSAD2A to estimate pavement response stresses, strains and deformations for any given loads. Higher moduli values may also be used to represent the substitution of higher quality materials for certain loadings, and then a determination of the load-center deflections is possible. Then entering Figure 2.2 with these calculated deflections allows the pavement designer to see how variations in the thickness or elastic properties of granular layers will affect the overlay thickness needed. In a sense, this is a form of sensitivity analysis.

Although the official approach is the Asphalt Institute (Asphalt Institute, 1983) procedure in theory, other approaches have been used by ADOT&PF. For example, in the Central Region (Anchorage), the official procedure indicates that no overlay is required despite the fact that roadways have high traffic volumes. This is true for the Southeastern Region (Juneau) as well. However, in the Interior Region (Fairbanks), the official procedure is used and interestingly enough, Fairbanks obtains higher deflections than Anchorage despite having a colder climate and lower traffic volumes. One possible explanation could be that Fairbanks might have weaker pavements.

Fairbanks is the only region that currently employs the Asphalt Institute procedure. Typical overlays are 1.5 to 2 in. (3.8 to 5.1 cm) with an expected life of 7 to 10 years. Currently, FWD deflection measurements are taken at a frequency of 5 per mile, but this is strictly for purposes of creating a data base only, and not for pavement design use.

Anchorage employs a mechanistic approach with the help of two computer programs, ELSYM5 and PSAD2A. The FWD obtains deflections during the most critical period of the year (typically Spring), and cores are taken to obtain layer moduli and thicknesses. Anchorage also notes that the design approach used is often determined by the funding source. State projects are heavily influenced by politics, while federal aid projects require the use of an established procedure. Typical overlays are 1.5 to 3 in. (3.8 to 7.6 cm) thick, but the mechanistic approach has designed overlays up to 7 in. (17.8 cm) thick. The design life is expected to be 10 years, although thermal cracks are expected within one to two years.

Juneau, on the other hand, employs neither the Asphalt Institute nor a mechanistic procedure. Instead, historical deflection data, from the Road Rater or Benkelman Beam, are used. However, Juneau is currently reconstructing roads more often then overlaying them. Typical overlays are 1.5 to 2 in. (3.8 to 5.1 cm), with an expected life of 10 years.

2.3 Problems With Current Overlay Design Methods

The problems associated with the current ADOT&PF overlay design procedure are summarized in Table 2.5. The following paragraphs discuss the problems in greater detail.

The existing official procedure does not always show a need for overlays, particularly on roads with a higher traffic volume as noted in the preceding section. Further, surface distress, neither its type, extent nor severity, is considered in the design process. Specifically, although it is known that

Table 2.5. Problems with the Current Overlay Design Method.

1. Current procedures do not always show a need for overlays despite high traffic volumes.
2. Pavement surface condition is not considered, especially cracking.
3. Effects of frozen subgrade and base not considered.
4. Remaining life.
5. Pavement layer dimensions not used.
6. Use of new additives in asphalt concrete.
7. Peak center deflection not a good indicator of distress.

cracking reduces the remaining life of the pavement, most existing procedures do not address this issue very well (Coetzee et al., 1986).

The data collected from current procedures are insufficient to provide accurate indications of the contributions of each structural layer, particularly if the pavement structure is partially frozen (Stubstad & Connor, 1982). Figure 2.3a shows an unfrozen pavement section. Under load, the pavement deflects according to the elastic layer theory, with the magnitude of each deflection value along the deflection basin being a function of the elastic properties of the materials in the pavement section. The deflection furthest from the load approximately reflects the subgrade condition, since the compression of the pavement layer above the subgrade is negligible compared to the vertical movement of the subgrade itself (Ullidtz, 1977). The center-of-load deflection, however, may be thought of as the sum of the vertical strains in each layer and is therefore affected by all layers.

The same pavement section, but in a partially frozen state, is shown in Figure 2.3b. Here, the depth of thaw is 6 in. (15 cm) below the asphalt concrete layer and the center-of-load deflection is seen to be approximately one-third that of the unfrozen section. However, the results of a comparative analysis of these two pavement sections using CHEVRON N-LAYER reveal that the horizontal tensile strain at the bottom of the asphalt concrete layer is approximately equal to the vertical strain at the top of the base layer, despite the difference in deflections (Stubstad & Connor, 1982). That is to say, despite the smaller deflection values for the partially frozen pavement, the stresses and strains that the pavement endures is much greater. Therefore, it may be concluded that the center-of-load deflection alone would be a poor indicator of the potential for pavement distress.

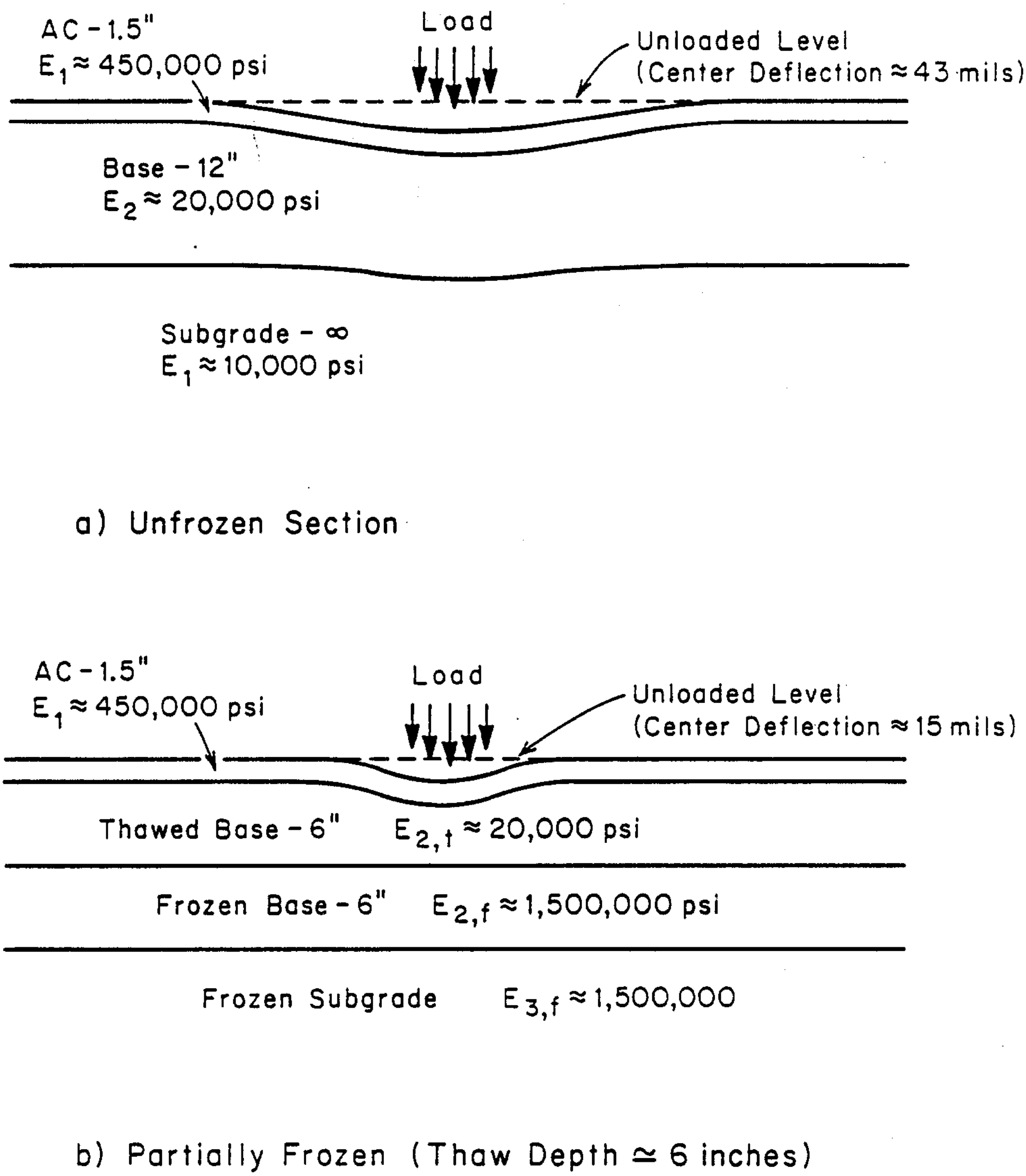

Figure 2.3. Typical Pavement Cross Sections (Stubstad & Connor, 1982).

It should also be noted that the maximum damage potential may occur long before peak deflection occurs. This may be remedied if the depth of thaw is known, in which case an adjustment to the deflection value is made. However, the thaw depth generally varies greatly from point to point, depending on factors such as exposure to sunlight, types of materials present, and water content. Frost tube measurements used for approximate thaw depths may be in error by several feet (Stubstad & Connor, 1982) and the adjusted deflection values may err by a factor of two or more as well.

Presently, the FROST program can be used to adjust the measured FWD deflections so that the adjusted deflections are available to design overlays (Stubstad & Connor, 1982). The program scales the FWD-measured deflection basin to the 9,000 lb (4080 kg) design load since the measured FWD load varies from 8,500 to 9,500 lbs (3860 to 4310 kg). Then, it compares the measured basin with the theoretical ones in a solution table and selects the three "best fit" theoretical basins. The solution table was generated by executing approximately 350 runs of the CHEVRON N-LAYER program. A likely range of conditions present in Alaska pavements were used and the results then stored in the solution table used by FROST. In addition, FROST also adjusts the deflection to a standard 70°F (21°C) using the following equation (Asphalt Institute, 1983):

$$d_{70} = d_{adj}(0.64 + 25.2/t) \tag{2.1}$$

where:

d_{70} = deflection adjusted to 70°F (21°C),

d_{adj} = adjusted field deflection (for a thawed section),

t = pavement temperature, °F.

It should be noted that this equation is found in Figure 2.4 as the 25 in. (63.5 cm) aggregate base curve.

However, the FROST program does not cover all conceivable solutions in its solutions table. Therefore, only an approximation of the results can be expected. The range of solution table parameters used in the FROST program was intended only for Alaskan roads. Other cold-climate regions require additional ranges of parameters.

Further, the Asphalt Institute method as given in MS-17 (Asphalt Institute, 1982) does not take into consideration the remaining life of the pavement. Work completed by Kennedy & Lister (1978) indicates that in many pavements, deflections remain relatively constant for most of the serviceable life of the pavement and only increase near the end of the pavement life. Therefore, a fatigue relationship that utilizes both past and future traffic should be used in the analysis. Also, the tolerable deflection as used may not be adequate to quantify the desired deflection criteria properly (Smith et al., 1986). This tolerable deflection is a function of the materials in the pavement structure, subgrade support and the layer thicknesses; however, the deflection method is used without reference to the layer thickness.

In addition, the use of new additives in asphalt concrete such as rubber and other polymers have added new dimensions to the overlay problem, which are is not addressed by the current method. Recall that the design charts were derived with an asphalt concrete layer of assumed modulus 500 ksi (3.4 GPa).

2.4 Other Overlay Design Methods

There are two major approaches to overlay design methods. One is based on surface deflection measurements and the other based on mechanistic principle. The deflection methods are based on the philosophy of providing an

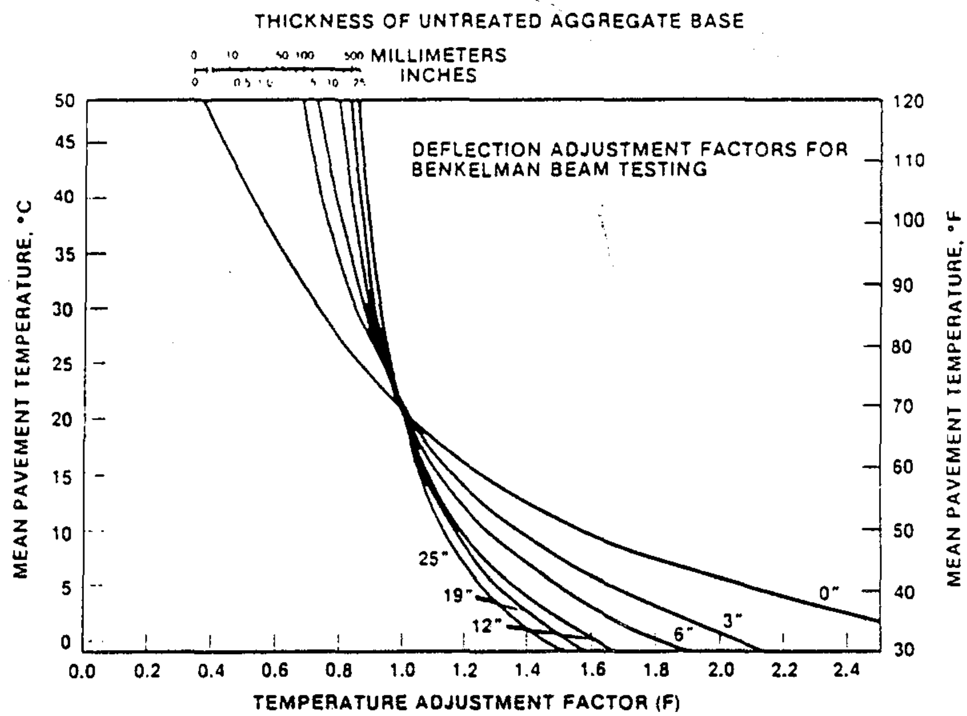

Figure 2.4. Temperature Adjustment Factors for Benkelman Beam Deflections for Various Thicknesses of Dense-Graded Aggregate Bases (Asphalt Institute, 1983).

overlay thickness sufficient to reduce the deflections to an acceptable level. The deflection criterion is normally determined empirically by the agency based on research in its own locale. These results are usually unique to the agency's own climate, pavement and soil materials. As a result, such procedures are generally only applicable to pavements maintained by other agencies if the design assumptions and inputs are similar to those used in the original research. Additionally, it is difficult to upgrade such procedures when changes in technology occur.

The mechanistic method uses the deflection data with theoretical analytical procedures to determine the characteristics of pavement layers and material properties. These pavement properties are then used together with traffic and environment data to determine the effects of various overlay materials on stresses and strains in the resurfaced pavement. These procedures are more widely applicable because of the theoretical basis which underlies their development. This is the major advantage of this method - the overlay requirements can be determined for any pavement for which the stresses and strains can be calculated. Therefore, the analysis is not limited to only pavements for which there is extensive experience; instead, the expected performance of new designs and the influence of new materials can be analyzed. This requires that the user be able to model the new designs and characterize the new materials with the mechanistic models to calculate stress or strain. Should the critical stress or strain occur in new material, a new relationship between the stress or strain and failure would also be required.

The major shortcoming of the mechanistic procedure is that in order to characterize properly the pavement structure and material properties, it requires either:

1. Extensive materials testing and evaluation, or
2. Backcalculation of material properties from deflection measurements to determine all the required inputs.

However, the long-term effect of environment on material properties and cracks often make it difficult to characterize accurately in-place pavements (Smith et al., 1986). Further, the backcalculated moduli values are not always unique (Uddin, 1984). Errors may also be present due to the possible variation in pavement layer thicknesses and the non-linear behavior of the granular layers and subgrade. Finally, if the input values used are out of the range for which the model was calibrated, the results will not be accurate. Table 2.6 summarizes the advantages and disadvantages of both these approaches.

The simplified mechanistic approach, a subset of the mechanistic procedure is also described in this section. This approach employs both mechanistic and empirical principles. Elastic layered theory together with regression techniques are employed to derive relationships between strain and surface deflections or more directly, between layer moduli and surface deflections. Therefore, instead of undergoing a time-consuming, iterative deflection basin fitting procedure, these regression equations are used to compute layer properties directly such as modulus and strain as a function of deflection and layer thickness.

2.4.1 Deflection Based Procedures

Deflection based overlay design procedures use the basic concept that a pavement with higher deflections will fail more quickly than a similar pavement with lower deflections under the same loading. This is combined with the concept that increased asphalt thicknesses decrease deflections to a tolerable level (Smith et al., 1986). These concepts can then be combined to

Table 2.6. Advantages and Disadvantages of Overlay Design Procedures.

Procedure	Advantages	Disadvantages
Deflection-Based	Areal coverage. Measurements representative of in situ conditions. Relatively inexpensive. Relatively fast. Relative high degree of reliability possible.	Does not measure material properties. Limited to materials and construction for which correlations are established. Related to one mode of distress e.g. fatigue cracking.
Mechanistic	Appropriate distress modes can be considered individually e.g. fatigue, rutting, low-temperature cracking. Capable of considering: - changed loading & tire pressure effects - new materials - environmental influences - aging effects - influence of changed subsurface drainage conditions	Unfamiliar to most current designers. Requires new and different equipment. Limited experience to date. Considerable computer time required. Non-unique moduli values. Variations in layer thickness affect moduli values. Nonlinear behavior of pavement layers. Input ranges may not be within model's calibrated values.

give a chart similar to Figure 2.2, which gives the overlay thickness needed for an existing deflection for different traffic levels.

There are three basic elements included in deflection-based overlay procedures. They are:

1. Surface pavement deflection measurements,
2. Pavement condition, and
3. Traffic.

The objective of the deflection testing is to measure the structural properties of the pavement. A known load is applied to the pavement and the deflection response is measured. The response is a function of the thickness of the pavement layers, subgrade strength, environmental conditions, and the loading conditions such as contact pressure, total load and time of loading. The overlay thickness is normally calculated from the maximum deflection under the load, although the use of the deflection basin is gradually increasing. Several types of equipment are used to measure deflection, such as the Benkelman Beam, Dynaflect, the Road Rater and the Falling Weight Deflectometer (FWD). It is important that the method of measurement be compatible with the overlay design procedure.

A pavement condition survey is an important part of an efficient overlay design procedure. It establishes the need for maintenance and rehabilitation, identifies homogeneous sections of roadway and points out special considerations such as drainage. A homogeneous section refers to a segment of pavement that has nearly the same traffic, age, structural capacity, and performance. Since there are different scales used for classifying pavement damage, the intervals are a matter of judgment and experience.

Most deflection-based procedures rely on determining two deflections; the design deflection and the tolerable deflection. The design deflection (d) is a function of the mean (x) and the standard deviation (s) of the measured deflection values as follows:

$$d = x + zs \tag{2.2}$$

The Asphalt Institute (1983) uses a z value of 2.0 while CalTrans (Monismith & Finn, 1984) uses a z value of 0.84. The Asphalt Institute also makes adjustments for measurements taken other than at the critical time of the year. The temperature at which the deflection values are measured are often normalized to a temperature of 70°F (21°C). CalTrans performs temperature adjustments only when the temperature falls below 50°F (10°C) (Lytton & Smith, 1985).

The tolerable deflection is a function of traffic level and was established based on data from several agencies, including data from the AASHO and WASHO road tests, Benkelman's results and data from other countries (Kingham & Jester, 1983). This is a conservative relationship in that the probability of an unsatisfactory design is very low.

Traffic is an important consideration in both overlay design procedures, and is expressed in terms of 18-kip equivalent single axle loads (EALs). Mixed traffic may be converted into a single design factor by summing all the load combinations. The error in estimating overlay thickness or the remaining life may be significant depending upon the reliability of the historical information and future traffic estimates. The tolerable deflection is based on the design EALs. The basic philosophy for deflection-based overlay design is to reduce the measured deflection to a tolerable level. This is accomplished by one of the following three methods. Table 2.7 summarizes the characteristics of deflection based overlay design procedures.

2.4.1.1 CalTrans Procedure. The CalTrans procedure (Monismith & Finn, 1984; Finn & Monismith, 1984; Lytton & Smith, 1985) employs the Benkelman Beam, Dynaflect, Road Rater or California (Traveling) Deflectometers to measure pavement deflections. Correlations between nondestructive testing (NDT) equipment have been developed by many agencies, and Table 2.8 illustrates a few of these relationships. Homogeneous sections are chosen based on the length of the project. If the project is less than a mile (1.6 km) in length, then the entire project is treated as one section. If the project length is greater than one mile, then 1000 ft (305 m) sections are selected to represent each mile (1.6 km).

General conditions from a visual survey are documented. Patching, rutting, raveling, and cracking are all noted as to the type and severity. The design deflection for each test section is computed from the following equation:

$$D_{80} = x + 0.84s \tag{2.3}$$

where:

D_{80} = design deflection value (80th percentile deflection),

x = mean deflection, and

s = standard deviation.

This procedure does not determine the remaining life of the pavement, and there is no temperature correction factor for temperatures less than 50°F (10°C) (Lytton & Smith, 1985). The representative deflection for a particular project length is compared with a tolerable deflection obtained from

Table 2.7. Characteristics of Deflection-Based Overlay Design Procedures (Finn & Monismith, 1984).

Method	Deflection Measurement	Condition Survey	Establishment of Analysis Sections	Design Deflection	Provision for Remaining Life Estimate	Overlay Thickness Determination
California Department of Transp.	Dynaflect; Traveling Deflectometer	Yes	Yes	$\delta + 0.84s$	No	Based on relation between permissible deflection as a function of asphalt layer thickness and repetitions of 18-kip EAL and reduction in deflection achieved by different thicknesses of overlay materials.
Transport and Road Research Laboratory	La Croix Deflectograph	Yes	Yes	85^{th} percentile	Yes*	Observed damping effect on deflections under 18-kip EAL for various overlay thicknesses used to develop design charts as a function of repetitions of 18-kip EAL.
Arizona Department of Transp.	Dynaflect	Yes	No	Deflection basin used.	Yes	Prediction model derived from regression equations. Spreadability Index represents load carrying capacity. Temperature corrections not used. FWD will eventually be integrated.

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

Table 2.8. Conversions to Benkelman Beam Deflections.

Organization	Source	Relationship
Utah	Peterson & Shepherd, 1986	$BB = 22.5\ D$
USFS Region 6	Whipple, 1986	$BB = 72.627675(D^{1.114702})$ (multiple geophones) $BB = 0.001024 + 17.153689\ R - 469.9866\ R^2$
AASHTO T-256	AASHTO, 1986	$BB = 23.52\ R + 0.0071$ $BB = 28.48\ D - 0.0029$
Asphalt Institute	Asphalt Institute, 1983	$BB = 22.30\ D - 2.73$
Louisiana	Lytton & Smith, 1985	$BB = 20.63\ D$
Virginia	Lytton & Smith, 1985	$BB = 30.5\ D - 12.3$
Arizona	Lytton & Smith, 1985	$BB = 22.5\ D$

All deflections are in mils except for USFS equations, which are in inches.
BB = Benkelman Beam deflections
D = Dynaflect deflections
R = Road Rater deflections
(1 mil = 0.0254 mm)

Figure 2.5. If the tolerable deflection is greater than the representative deflection, then an overlay is not needed. If the tolerable deflection is less than the representative deflection then the percent reduction in deflection is calculated as follows:

$$\% \text{ reduction} = 100*(D_{80} - D_t)/D_{80} \quad (2.4)$$

where:

D_t = tolerable deflection, inches.

Figure 2.6 is then used to determine the gravel equivalency value. The gravel equivalency factor is converted to an equivalent thickness of asphalt concrete by division with a factor of 1.9. It should be at least half the thickness of the existing asphalt concrete for an untreated base.

2.4.1.2 Transport and Road Research Laboratory. In the TRRL procedure (Monismith & Finn, 1984), deflections are measured with either the Benkelman Beam or a modified version of the La Croix deflectograph. An axle load of 14,000 lbs (63 kN) is used for both devices. Deflection measurements using the Benkelman Beam are taken every 40 to 80 ft (12 to 24 m) depending on the condition of the pavement. Measured deflections are adjusted for temperature effects (Figure 2.7). If deflections are obtained from the deflectograph, they are correlated to Benkelman Beam measurements using Figure 2.8.

An assessment of remaining life requires the use of the representative design deflection, an estimate of traffic and a selected probability of attaining the design life (Figure 2.9). Given the current deflection and traffic estimate, a point (e.g. Point A) is located in Figure 2.9. The remaining life (in standard axles) is then the dashed line extending from A to the probability lines. For this example, the probability selected is 0.50.

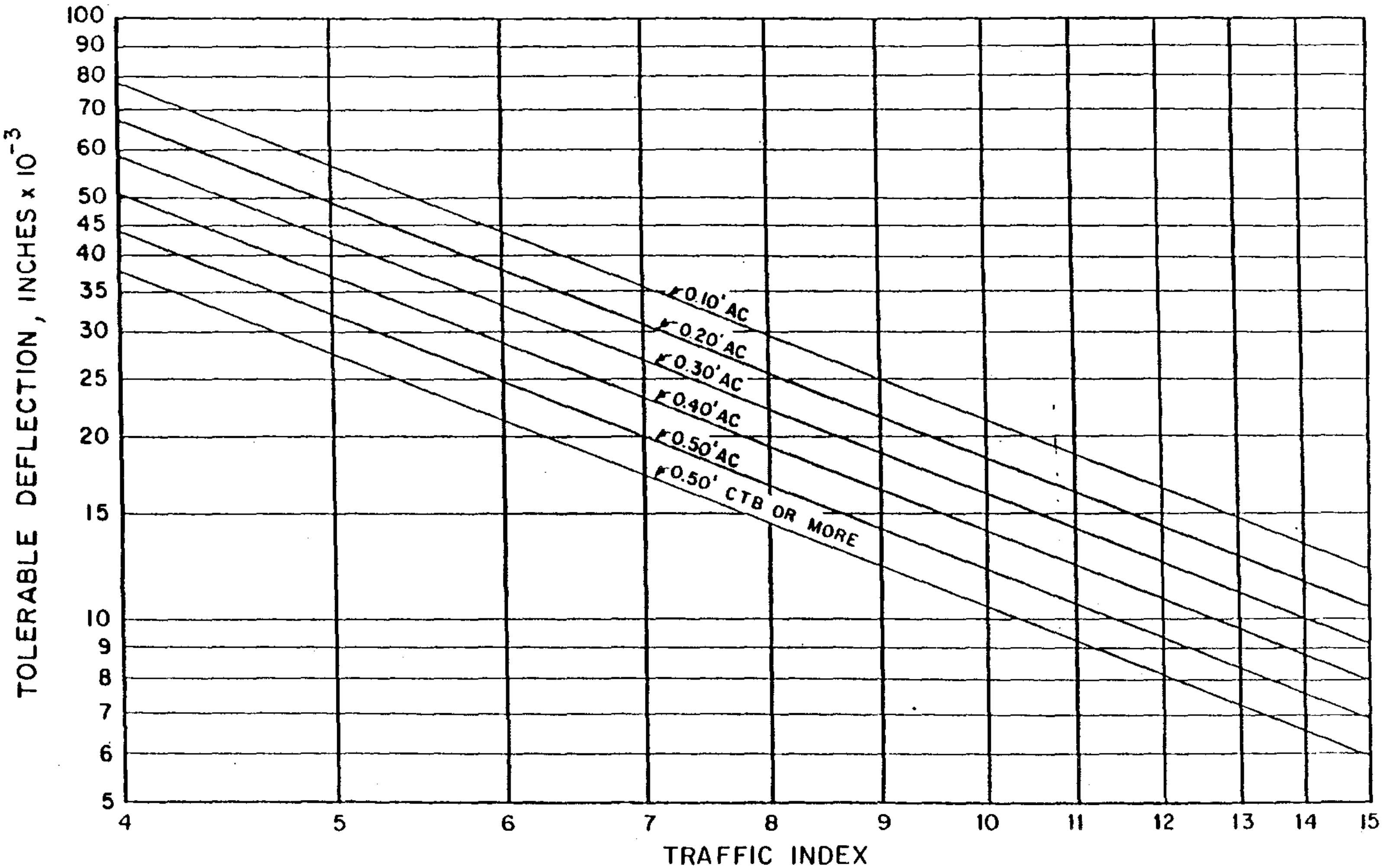

Figure 2.5 Tolerable Deflection Chart (CalTrans, 1979).
(1 in. = 2.54 cm)

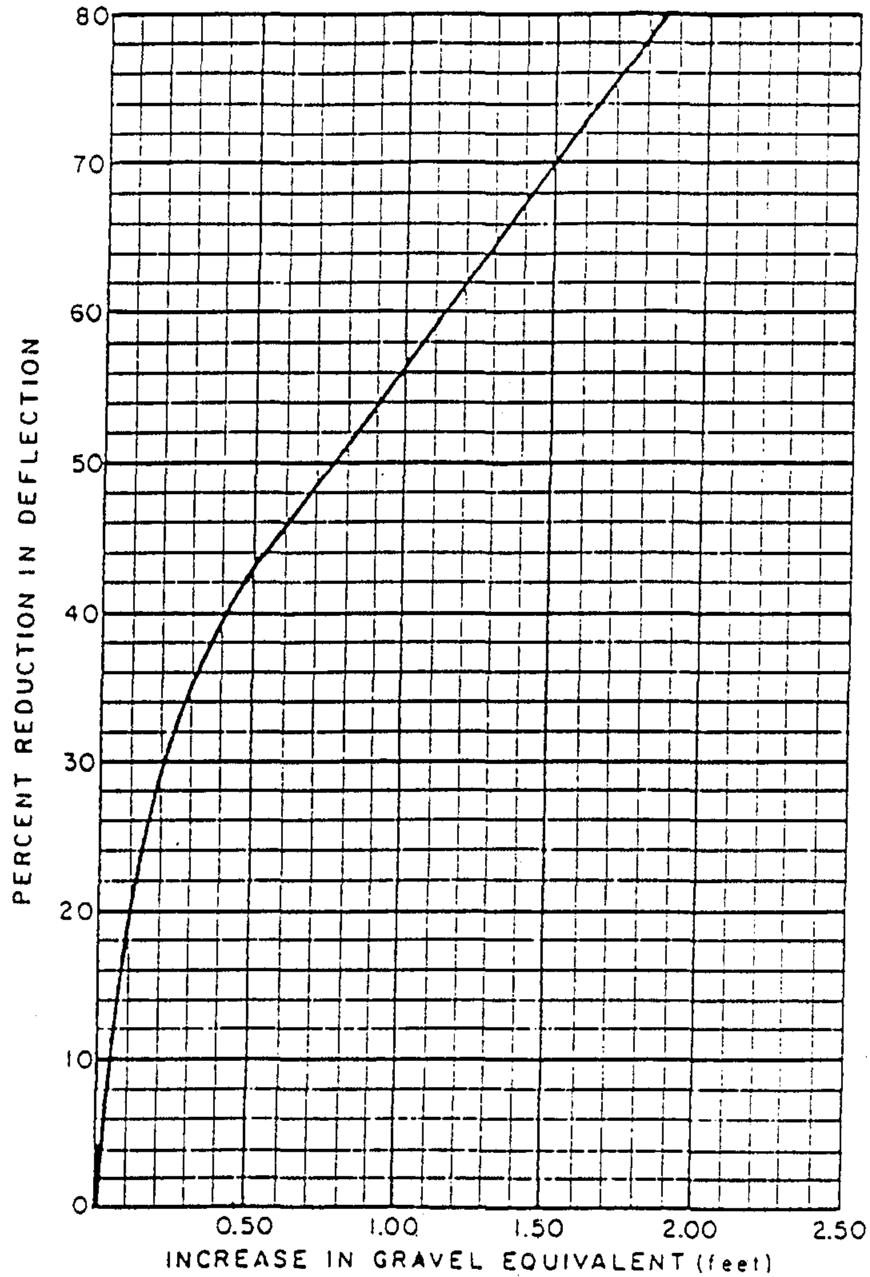

Figure 2.6 Reduction in Deflection Resulting from Pavement Overlays (CalTrans, 1979). (1 ft = 30.5 cm)

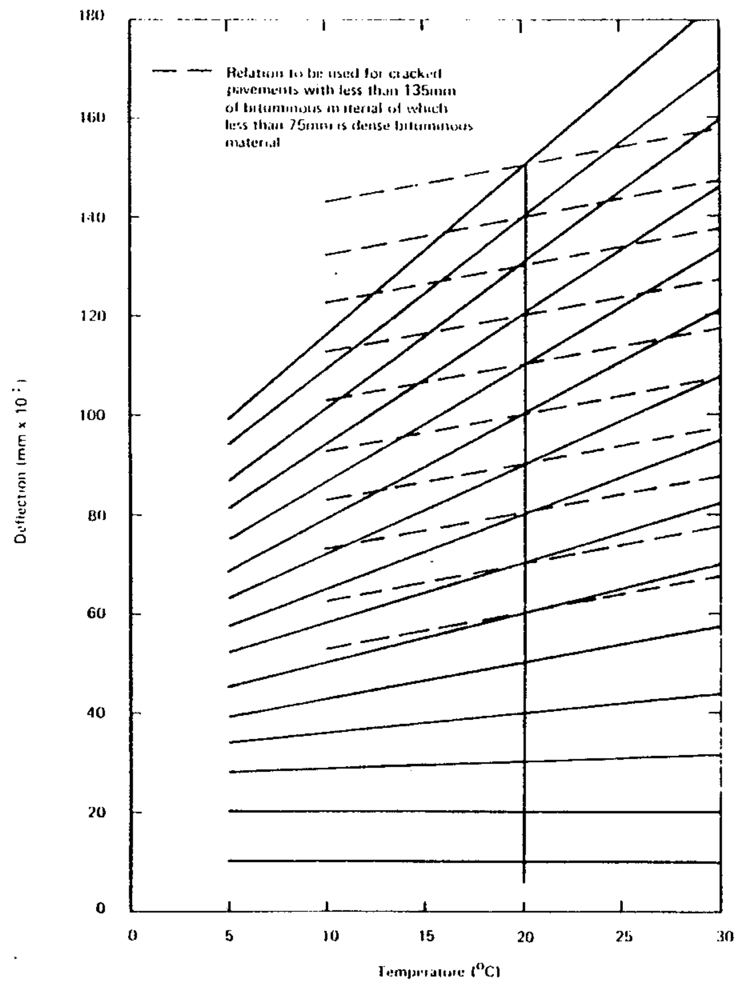

Figure 2.7 Relation Between Deflection and Temperature with less than 135 mm of Bituminous Material, of which less than 75 mm is Dense Bituminous Material. (Kennedy & Lister, 1978).

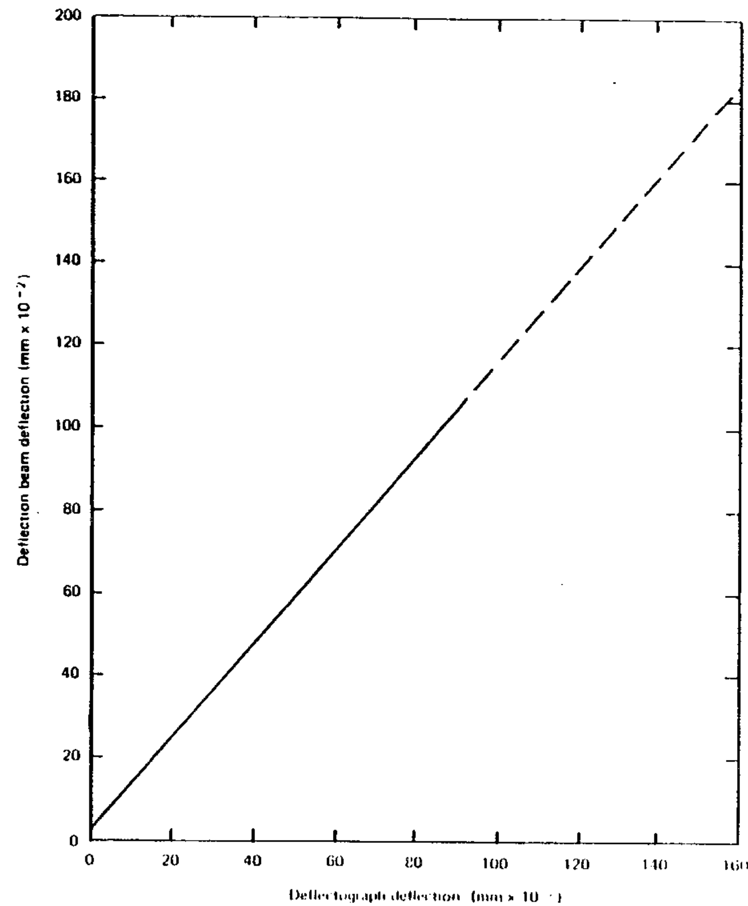

Figure 2.8 Correlation Between Deflection Beam and Deflectograph. (Kennedy & Lister, 1978).

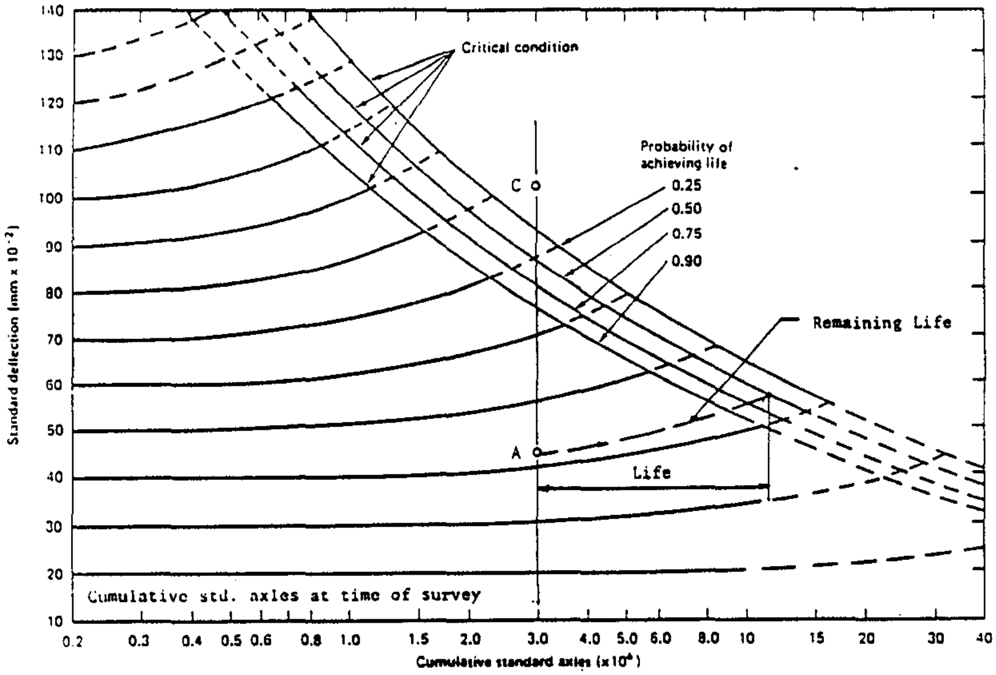

Figure 2.9. Relation Between Standard Deflection and Life for Pavements with Non-Cementing Granular Road Bases (Kennedy & Lister, 1978).

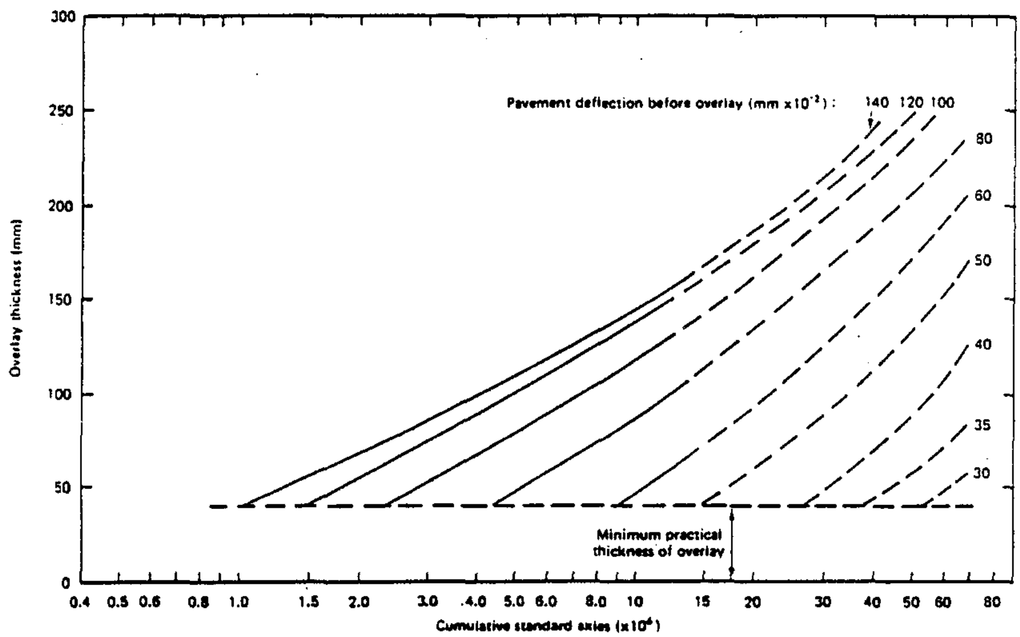

Figure 2.10. Overlay Design Chart for Pavements with Non-Cementing Granular Road Bases (.50 probability) (Kennedy & Lister, 1978).

If the life expectancy is in the critical condition (as Point C is), strengthening is required immediately. Overlay thicknesses are selected from Figure 2.10. The expected traffic on the horizontal axis is projected to the pavement deflection, and the overlay thickness is read off the vertical axis. A more detailed overlay design may also be performed to identify critical areas of the road that may require thicker overlays.

2.4.1.3 Arizona. The Arizona State Department (Way et al., 1984) developed a new overlay design method that involves the use of the Dynaflect based on empirical and theoretical concepts. Thirty-one variables were considered for their various effects upon the thickness of the overlay for 170 locations. The intent of this analysis was to determine which values were significant in the overlay design. The multiple regression analysis produced the following relationship:

$$T = \frac{\log L + 0.104R + 0.00578P_o - 0.0653(SI)}{0.0587\ (2.6 + 32D_5)^{0.333}} \tag{2.5}$$

where:

- T = the asphalt concrete overlay thickness, (in.),
- L = expected traffic loading for the overlay during time t (18^k EALs),
- t = time until Mays Ridemeter reaches a value of 260 in. (PSR = 2.5),
- R = regional factor from AASHTO Road Test,
- P_o = Mays Ridemeter Roughness of existing pavement, (in.),
- D_5 = fifth Dynaflect sensor deflection (mils), and

SI = spreadability index of existing pavement before overlay as determined from the following equation:

$$SI = (100/5D_1*(D_1+D_2+D_3+D_4+D_5)) \quad (2.6)$$

where:

$D_{1,2,3,4,5}$ = deflection measurements for 5 Dynaflect locations (mils).

The spreadability index is a function of the moduli values for each layer. The equation also assumes the modulus of the asphalt concrete to be 200 ksi (1380 MPa). Arizona does not apply temperature correction factors to the deflections since their results showed that corrections were unreliable. Equation (2.5) was developed only through extensive research, and it must be realized that it is only applicable to Arizona's special conditions, climatic and otherwise.

2.4.2 Mechanistic Methods

A mechanistic design procedure characterizes the response of the pavement to a load in terms of strains and/or stresses in various pavement layers. A fatigue relationship between that response and number of load repetitions to a designated failure criterion is used to determine pavement life. Most mechanistic procedures use deflections as the response (Smith et al., 1986). The difference between such a procedure and the deflection-based approach is that the deflection used to develop the performance relationship is based on a mechanistic model rather than empirical data.

As in deflection-based procedures, nondestructive pavement evaluation condition surveys and traffic are required inputs for mechanistic methods. However, pavement distress and stiffness (modulus) properties of the various pavement layers are needed as well. Distress is defined as permanent deformation or fatigue cracking of the pavement. Stiffness may be determined through testing of representative samples or estimated from nondestructive measurements. Reliance should not be placed solely on nondestructive testing, particularly for major projects. Table 2.9 gives suggested guidelines for field testing frequency. When the sections have been selected, the design deflection is established. This value is usually set in the 80^{th} to 90^{th} percentile range. At present, ADOT&PF uses the 95^{th} percentile for their design deflection. Representative material characteristics are determined from pavement cores, layer samples, thicknesses, and undisturbed subgrade samples.

Multilayer elastic analysis is used to estimate deflections under known loadings for a given laboratory-determined stiffness. These deflections are compared to the actual measured deflections; adjustments are then made to the stiffness values until the predicted and measured deflections are in reasonable agreement. Stiffness characteristics can also be estimated from surface deflection measurements. The shape of the deflected surface at various radii from the applied load is used as input for a computer program to determine the modulus values that will give the best fit to the data. The influence of the various layers in the pavement structure may be determined using the following approach given a pavement section as represented in Figure 2.11 and characterized by layer thickness (h), Young's moduli (E) and Poisson's ratio (υ). When a load of known intensity is applied over a known area, deflections are created at some distance from the center of the loaded area. The load is assumed to be distributed through the pavement system by a truncated zone (dashed line). The deflection d_4 at a distance r_4 from the center of load can only be due to the "elastic" compression of layer 4 because layers 1, 2, and 3

Table 2.9. Suggested Spacings for Nondestructive Measurements (Finn & Monismith, 1984).

Organization	Highways	Airfields
Asphalt Institute	20/mile (minimum) 10/analysis section (minimum)	N/A
Transport and Road Research Laboratory	50 to 80 ft (Benkelman Beam) 12 ft (La Croix Deflectograph)	N/A
U.S. Army Corps of Engineers (WES)	N/A	250 ft - runways & primary taxiways 250 to 500 ft grid patterns - aprons
Shell Research	25 to 50 m	25 to 100 m
FHWA-ARE	100 to 250 ft depending on terrain and material uniformity.	N/A

1 ft = 30.5 cm

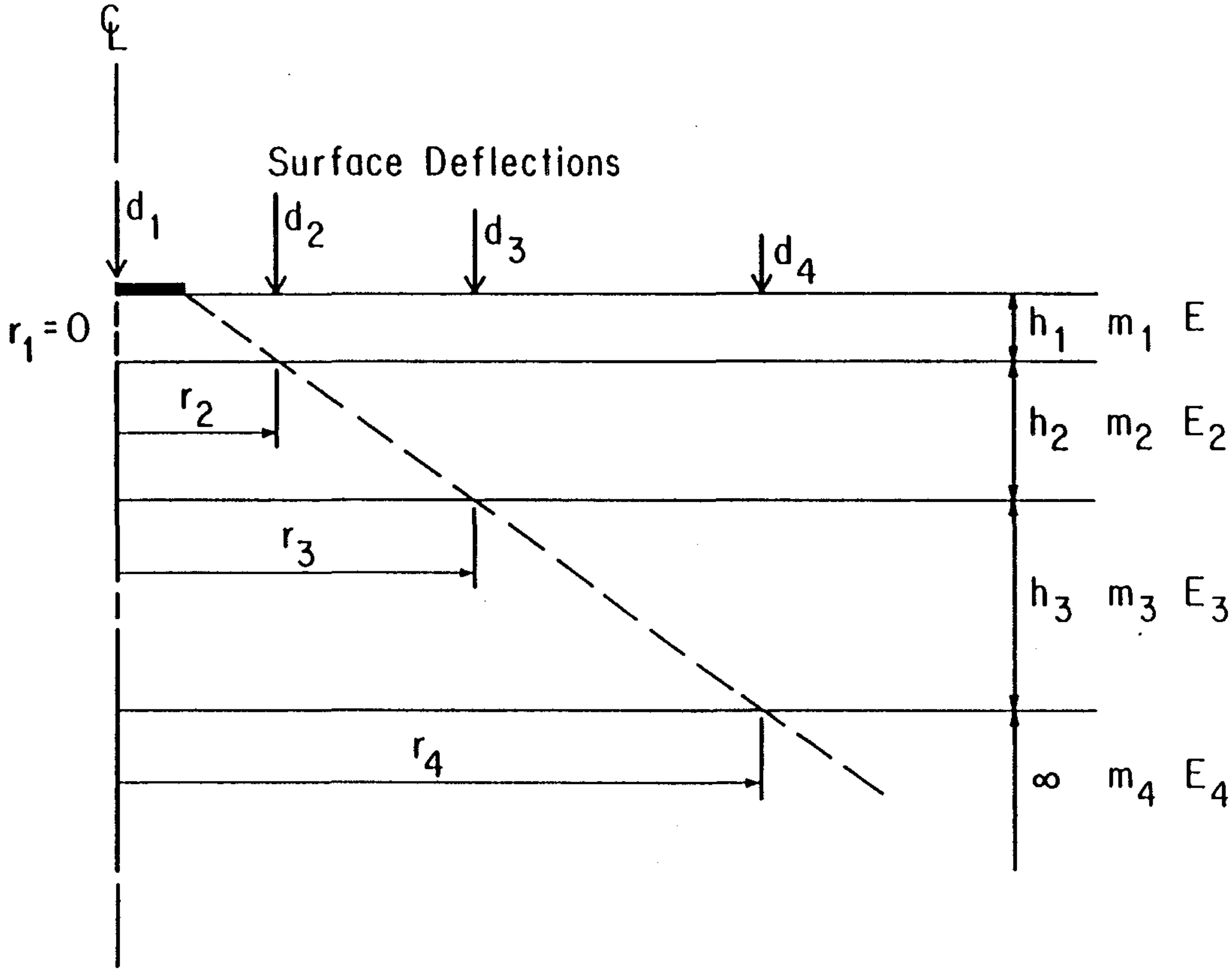

Figure 2.11 Four Layer Representation of a Pavement System (Lytton & Smith, 1985).

are outside the influence cone created by the load (Lytton & Smith, 1985). Similarly, deflection d_3 is due to the compression of layers 3 and 4. This general approach is used to backcalculate the properties of pavement layers.

Traffic considerations include not only the equivalency concept previously discussed but also a distribution of traffic across the lanes and the concentrations of truck traffic in the outer lane. Remaining life is computed from Miner's Hypothesis, a cumulative damage theory using a fatigue relationship based on strain:

$$N_f = A(1/\epsilon_t)^b (1/S_{mix})^c \tag{2.7}$$

where:

N_f	=	number of applications to failure,
ϵ_t	=	tensile strain in asphalt concrete (in./in.),
S_{mix}	=	stiffness modulus of concrete (psi), and
A,b,c	=	constants for specific asphalt mix.

A simple form of the cumulative damage theory to determine remaining life is:

$$N_r/N_{D1} = 1 - N_{A1}/N_{D1} \tag{2.8}$$

where:

N_r/N_{D1}	=	remaining life,
N_{A1}	=	number of EALs to date,
N_{D1}	=	allowable number of EALs according to fatigue, and
N_r	=	additional 18^k EALs that can be applied to existing pavement.

Given the tensile strain value and the allowable number of repetitions, it is possible to define a relationship between overlay thickness and additional load applications from a fatigue expression. By applying additional thicknesses of overlay to the surface, the strains or stresses in the existing pavement is decreased. The following procedures demonstrate the amount of input and the calculations necessary for determining overlay thicknesses using a mechanistic approach. In the following sections, four procedures are briefly described. Table 2.10 summarizes the characteristics of analytically based overlay design procedures.

2.4.2.1 Shell Oil. The Shell Oil procedure (Monismith & Finn, 1984; Shell, 1978) uses the Falling Weight Deflectometer (FWD) to measure the structural response of the pavement. The deflections define the response of the pavement to the applied load. The maximum deflection and the shape of the deflection basin are used to determine the stiffness characteristics of each layer, defined by the resilient modulus (E) and Poisson's ratio (υ). For the typical three-layer pavement, the modulus of the asphalt bound layer is estimated from Shell nomographs, and the thickness of the granular layer is estimated from construction reports or determined from coring. The subgrade modulus and the effective thickness of the asphalt concrete layer are then determined using the BISAR program, an iterative process in which the values of the parameters are adjusted until measured and computed values are in reasonable agreement.

Overlay thicknesses are selected to mitigate excessive permanent deformation and fatigue cracking. Distress in the form of excessive permanent deformation is limited by controlling the vertical compressive strain at the subgrade surface:

Table 2.10. Characteristics of Analytically Based Overlay Design Procedures (Finn & Monismith, 1984).

		Stiffness Determinations			Distress Mechanisms			
Procedure	Nondestructive Pavement Evaluation	In Situ Measurement	Lab Testing	Analysis Procedure	Fatigue	Rutting	Provision for Existing Pavement	Overlay Thickness Determination
Shell Oil	Falling Weight Deflectometer	Yes	No	BISAR Computer Program	Yes	Yes	Yes	Overlay thickness selected to a) limit fatigue and b) limit rutting for anticipated traffic; thickness also selected assuming existing pavement is cracked.
FHWA-ARE	Dynaflect; Benkelman Beam	No	Yes	ELSYM Computer Program	Yes	Yes	Yes	Overlay thickness selected to a) limit fatigue and b) limit rutting for anticipated traffic; asphalt concrete assigned different stiffness values depending on conditions.
New Mexico Department of Transp.	Road Rater	Yes	Yes	Modified BISAR	Yes	No	Yes	Overlay thickness selected to limit a) fatigue and b) rutting for anticipated traffic.

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

$$\epsilon_v = (2.8 \times 10^{-2})\ N^{-0.25} \tag{2.9}$$

where:

ϵ_v = vertical compressive subgrade strain, and

N = number of load applications.

Fatigue cracking is controlled by limiting the tensile strain:

$$N = A(1/\epsilon_t)^a(1/E_1)^b \tag{2.10}$$

where:

ϵ_t = tensile strain,

E_1 = modulus of the asphalt bound layer, and

A,a,b = constants for the asphalt mix.

The remaining life is estimated from a knowledge of the layer thicknesses, design values, and traffic volumes. If the design life of the existing section has been exceeded so that the section can not accommodate the anticipated traffic, an overlay thickness must be ascertained. This is accomplished by considering three separate thickness determinations:

1. Thickness to satisfy the subgrade strain criterion,
2. Thickness to satisfy fatigue strain in the asphalt layer, and
3. Thickness assuming the existing pavement has deteriorated to the extent that it acts as an unbound granular layer.

The subgrade modulus together with the effective asphalt concrete thickness is used to enter Figure 2.12 to obtain the original design life based on subgrade strain. This value is compared with the actual number of axle loads that the pavement has seen to determine the residual life. The additional number of axle loads expected is re-entered into Figure 2.12 to

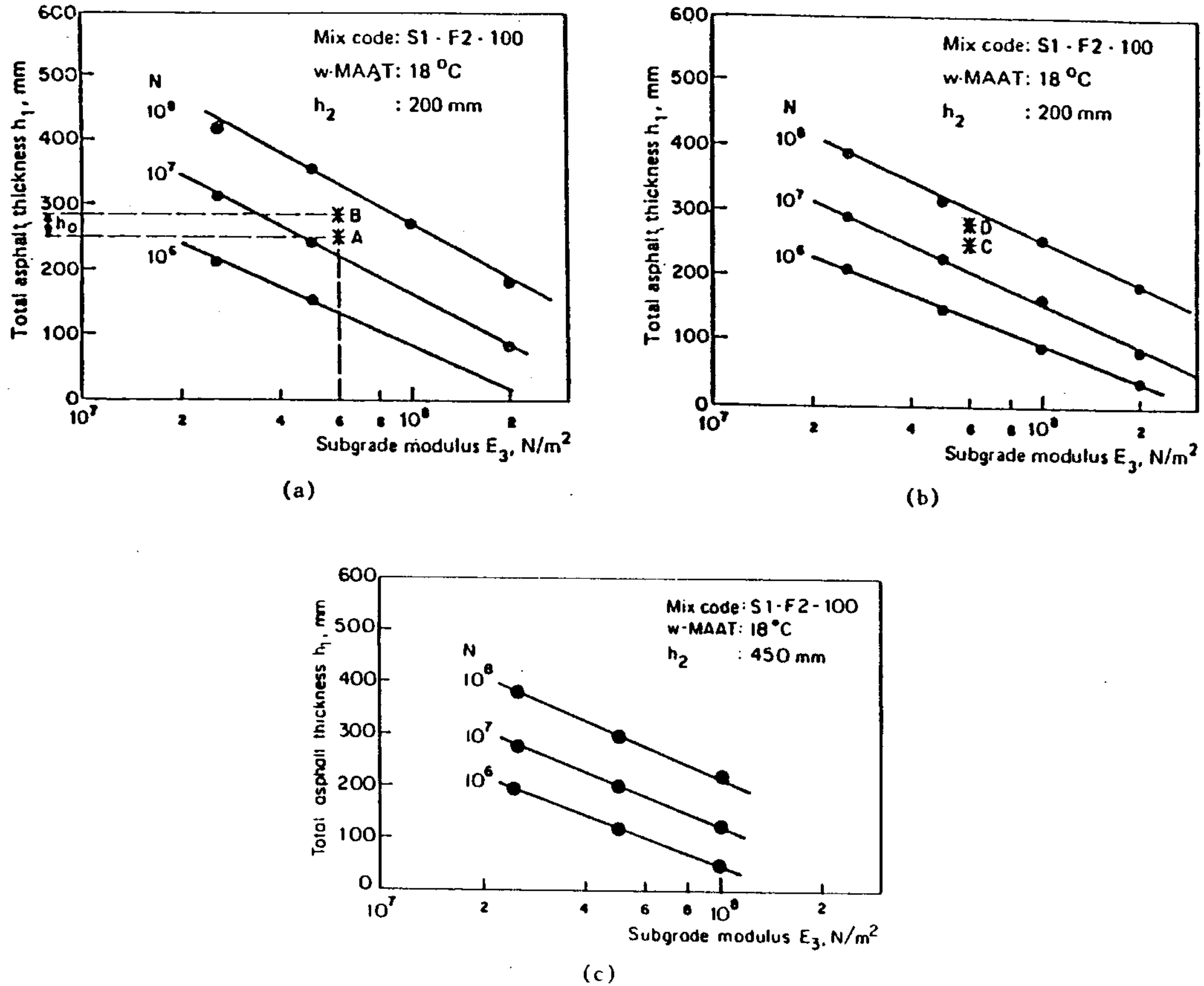

Figure 2.12 Interpolated Design Charts: (a) Based on Subgrade Strain Criterion, (b) Based on Asphalt Strain Criterion, and (c) Based on the Assumtion that the Existing Asphalt Concrete Behaves as Granular Base (Shell, 1978).

ascertain the additional amount of asphalt needed to satisfy the subgrade criteria.

Using much the same procedure for condition 2 (fatigue strain) and employing the use of the equation below to find the design number, an alternate thickness can be found which is compared to the previous one to determine which condition should govern.

$$N_{D2} = N_{D1} * N_{A2} / (N_{D1} - N_{A1}) \tag{2.11}$$

where:

N_{D2} = design number,

N_{A2} = number of additional standard axles anticipated,

N_{D1} = original design life based on asphalt criteria, and

N_{A1} = actual number of repetitions at the present.

2.4.2.2 Federal Highway Administration - ARE. This method was developed for the Federal Highway Administration by Austin Research Engineers, Inc. (Finn & Monismith, 1984). In this procedure, any type of deflection equipment may be used for obtaining deflections. The frequency of deflections depend on the terrain, and guidelines for determining the frequency of NDT measurements are given below:

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

Condition surveys are performed at the same time the deflection measurements are made. Deflection profiles are generated for the entire length of the project to assist in establishing the analysis sections. The stiffness modulus and Poisson's ratio are determined for each layer. Subgrade specimen modulus can be determined from:

$$M_R = A\sigma_d^{\,b} \tag{2.12}$$

where:

M_R - resilient modulus, psi,

σ_d - applied deviator stress, psi, and

A,b - laboratory determined coefficients.

The asphalt concrete is assigned a modulus of 70,000 psi (310 kN) when cracked or defined over a range of temperatures corresponding to a loading time which simulates traffic. Fatigue is defined by the magnitude of the tensile strain:

$$N = 9.73 \times 10^{-15}\ (1/\epsilon_t)^{5.16} \tag{2.13}$$

where:

N - allowable number of 18^k EALs, and

ϵ_t - horizontal tensile strain on the underside of the asphalt bound layer.

Rutting is controlled by the following variables:

1. $\epsilon_{z1}, \epsilon_{z4}$ - vertical strain at bottom of layers 1 and 4,
2. $\sigma_{z1}, \sigma_{z2}, \sigma_{z3}$ - vertical stress at bottom of layers 1, 2, and 3,
3. $\epsilon_{z5}, \sigma_{z5}$ - vertical strain and stress at top of layer 5,
4. σ_{x2} - horizontal stress parallel to load axle at bottom of layer 2, and

5. Number of days per year when average daily temperature is greater than 64°F (18°C), five-year average.

The remaining life is determined for an uncracked pavement to estimate the amount of traffic (18^k EALs) applied to date, and the tensile strain(s) on the underside of the asphalt bound layer. The ELSYM computer program computes the strains, and together with the modulus values and Poisson's ratios, the total number of load repetitions, N, can be estimated from Equation 2.13. The remaining life is then calculated using the linear enumeration of cycle ratios cumulative damage hypothesis, i.e.,

$$N_r = N_1(1 - N_D/N_1) \qquad (2.14)$$

where:

N_r = number of additional 18^k EAL for computed strain level,

N_D = design traffic volume, and

N_1 = total number of load repetitions.

The total traffic is modified by the regional factor according to:

$$N_D = RF*(18^k \text{ EAL}) \qquad (2.15)$$

where:

RF = regional factor.

If n_r is less than the expected traffic then an overlay is needed. The relationship between fatigue and rutting versus equivalent load applications is shown on Figure 2.13. The value of N for each criterion is used to determine the overlay thickness depending upon the amount of cracking. Two computer programs DEFANL and OVANL are employed for this procedure.

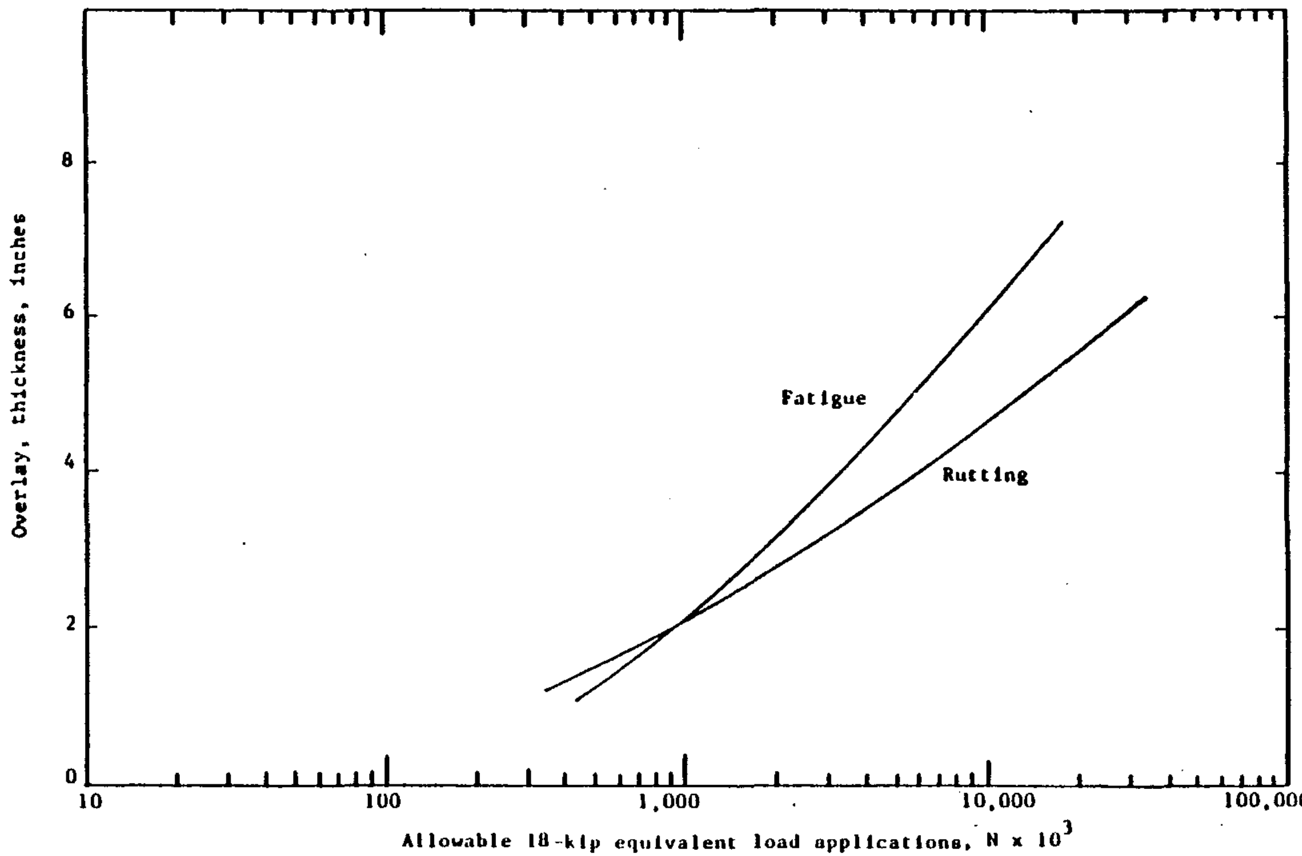

Figure 2.13 Sample Overlay Thickness Design Curves (Finn & Monismith, 1984).

2.4.2.3 <u>New Mexico</u>. This New Mexico method (Tenison, 1984) uses AASHTO principles and is simplified with the use of several computer programs. Deflection measurements are obtained from a Road Rater device for each mile (1.6 km) of the project length. A temperature correction factor is applied to the deflection prior to the calculation of the layer moduli, and these factors are either one of the following:

$$F_{70} = 29.56T_{AC}^{-0.7935} \quad (E_{subgrade} \geq 3500 \text{ psi } (24 \text{ MPa})) \tag{2.16}$$

$$F_{70} = 194.15T_{AC}^{-1.231} \quad (E_{subgrade} < 3500 \text{ psi } (24 \text{ MPa})) \tag{2.17}$$

where:

F_{70} = correction factor multiplied to measured deflections for normalization.

The average asphalt temperature may be calculated from Equation 2.18. These equations are based on regression analysis and work done by Southgate (1964) of the Kentucky Department of Highways.

$$T_{AC} = (0.7077T - 58.9)D_{AC}^{-0.138} + 42.8 \tag{2.18}$$

where:

T_{AC} = average asphalt temperature (°F),

T = measured surface temperature plus the average air temperature (°F) for the preceding 5 days, and

D_{AC} = depth of the existing asphalt layer (in.).

These standardized deflection values together with existing layer thicknesses are then fed into a deflection analysis program which is based on

BISAR and backcalculates layer moduli using multilayer elastic theory. The overall layer moduli for the one-mile section is determined as shown below:

$$LE_i = E_i - 0.7(SD) \tag{2.19}$$

where:

LE_i = elastic modulus for layer i (psi),

E_i = mean elastic modulus of layer i (psi), and

SD = standard deviation for layer i.

This procedure is repeated for each mile of project length and the results summarized. From this summary, the design engineer determines whether some sections should be designed separately. Then the resilient modulus of the asphalt concrete for the desired overlay is determined by one of the following methods:

1. Coring a completed construction project that uses the same aggregate and asphalt source,
2. Testing of laboratory samples composed from the same aggregate and asphalt source, or
3. Using an empirical equation that is a function of the fines content, percent air voids, asphalt content, temperature, viscosity, and penetration of the asphalt cement.

The traffic is all converted to 18^k EALs. Estimates of prior traffic, future traffic, and allowable traffic are employed to determine the remaining life of the existing pavement. It is desirable to have the existing pavement structure and the asphalt overlay fail at the same time. These remaining life factors, as well as the modulus values for the in situ pavement, and the proposed overlay are all input into the modified DAMA program, a pavement analysis program that operates on the fatigue and compressive strain criteria for failure of the pavement structure. DAMA was developed at the University of Maryland and is an elastic-layered pavement analysis program used to ascertain the repetitions to failure in the deformation and fatigue cracking distress modes for a given pavement structure in a given environment subjected to a given design load configuration. Since DAMA analyzes a given pavement cross section structure, it is not a design program per se (Asphalt Institute, 1982). An initial thickness for the overlay is assumed. Iteration of this value takes place until the desired design life criteria is met. That specific assumed thickness then becomes the overlay design thickness.

2.4.3 Simplified Mechanistic Methods

The simplified mechanistic methods employ mechanistic principles to arrive at empirical relationships. Linear elastic theory together with regression analysis techniques develop equations that describe the relationships between surface deflections and pavement layer properties.

2.4.3.1 Pennsylvania. This strain-deflection approach is a simplified mechanistic procedure that was developed by Fernando et al. (1986) at Pennsylvania State University. Essentially, the linear elastic-layered theory was used to develop strain vs. deflection relationships for the direct calculation of pavement strains from measured FWD deflections rather than using a deflection basin fitting procedure to backcalculate moduli values. The multilayer linear elastic program, BISAR, was used in a large factorial study to develop these relationships. Three pavement layers were assumed, and a wide range of moduli and thicknesses of each layer were used. Strain versus deflection relationships were then developed for the tensile strain (ϵ_t) at the bottom of of the asphalt layer and the compressive strain (ϵ_c) at the top of the subgrade. They are:

$$\log [\epsilon_t/\log (H_1+1)] = - 2.261 - 0.944 \log (\delta_1-\delta_2) + 1.947 \log [(\delta_1-\delta_3)/\delta_2] + 0.175 (\delta_1*H_2) + 0.926 \log (\delta_1*\delta_2) \quad (2.20)$$

$$\log \epsilon_c = - 0.054 + 1.941 \log (\delta_1-\delta_2) - 2.004 \log [(\delta_1-\delta_3)/\delta_2] - 1.465 \log (H_1+H_2) - 0.136 (H_2)^{0.5} + 0.725 \log (\delta_1*H_2) + 0.285 (\delta_1*H_1)^{0.5} - 0.910 \log (\delta_1*\delta_2) \quad (2.21)$$

ϵ_t = tensile strain at bottom of the existing AC layer,

ϵ_c = compressive strain at top of subgrade,

δ_i = deflection at i^{th} sensor of the FWD (in.),

H_1 = thickness of existing AC layer (in.), and

H_2 = thickness of existing subbase (in.).

Using the calculated strain values, performance prediction estimates are possible. The tensile strains were used with the Austin Research Engineers fatigue equation (ARE, 1975), and the subgrade strains were used with the performance model developed by Luhr et al. (1983) to obtain the following equations. However, it was found that the performance predictions from the deflection basin fitting (i.e. mechanistic, backcalculation) procedures match more closely the predictions generated from the theoretical strains than from this simplified strain-deflection procedure. The two performance equations are defined as follows:

$$W_{18} = 9.73 \times 10^{-15} (\epsilon_t)^{-5.16} \tag{2.22}$$

$$\log N_x = 2.15122 - 597.662\epsilon_x + \log [(PSI_i - TSI)/(4.2 - 1.5)]^{0.5} \tag{2.23}$$

where:

W_{18} = weighted 18-kip applications prior to Class 2 cracking,

ϵ_t = tensile strain at bottom of the AC layer,

N_x = allowable applications of axle load x,

ϵ_x = subgrade compressive strain due to axle load x,

PSI_i = initial PSI of pavement and,

TSI = terminal serviceability index.

Once the strains are known, it is possible to input various thicknesses of the overlay and then to recompute the strains. The required overlay thickness would be the one that reduces the strains to a specified tolerable level. The relationships developed for estimating the pavement's tensile and compressive strains are given by the following:

$$\log (\epsilon_t)_{ov} = - 0.689 + 0.793 \log \epsilon_t - 0.041 (H_{ov} + H_1)^{0.5} - 0.057H_{ov} \tag{2.24}$$

$$\log (\epsilon_c)_{ov} = - 0.359 + 0.870 \log \epsilon_c - 0.051H_{ov} - 0.109 [(H_{ov}\ H_1)/H_1]^{0.5} \tag{2.25}$$

where:

$(\epsilon_t)_{ov}$ = tensile strain at bottom of original AC layer after overlay,

$(\epsilon_c)_{ov}$ = compressive strain of subgrade after overlay,

ϵ_t = tensile strain of AC layer before overlay,

ϵ_c = compressive strain before overlay,

H_{ov} = overlay thickness (in.), and

H_1 = thickness of original AC layer, in.

2.4.3.2 Washington. The Washington State DOT procedure was developed by Newcomb (1986) at the University of Washington. It provides a means of estimating material properties in a two- or three-layer flexible pavement structure from FWD deflection data for use in a mechanistically based pavement design procedure.

Layered elastic theory was applied to two- and three-layered pavements with a variety of pavement conditions. FWD deflection data was used to determine the material moduli. A step-wise regression analysis was performed on the results of this layered elastic analysis. The regression equations thus derived may then be used to determine layer moduli directly without the need of using backcalculation programs, which are iterative in nature (and therefore time-consuming) and which may result in non-unique solutions.

ELSYM5 was chosen to characterize the pavement response of two- and three-layer pavements. The variables in the ELSYM5 runs were FWD load, layer thicknesses and moduli. Table 2.11 summarizes the input variables. Several restrictions were defined in the course of developing the regression equations, and they include:

1. The modulus of an underlying layer is less than or equal to the layer immediately above it. This is made on the assumption

Table 2.11. Variables Used in ELSYM5 for Three-Layer Case (Newcomb, 1986).

Load, P lb	Surface Thick. h_{AC}, in.	Base Thick. h_B, in.	AC Mod. E_{AC}, psi	Base Mod. E_B, psi	Subgrade Mod. E_S, psi
5,000	2	4	2,000,000	100,000	50,000
10,000	6	10	500,000	50,000	30,000
15,000	12	18	100,000	30,000	10,000
				10,000	5,000
					2,500

1 psi = 6.89 kPa
1 in. = 2.56 cm

that FWD testing does not occur during periods of spring thaw or when the base and subbase may be frozen.

2. The base course modulus is not greater than six times the subgrade modulus. Claessen et al. (1976) have conducted studies which suggest that the unbound base course to subgrade modular ratio ranges from 2 to 4.

The parameters investigated in the regression analysis included deflection basin readings with up to 6 geophones, and the area under the deflection curves. From the regression analysis, the following equations for a three-layer pavement were obtained:

$$E_s = -530 + 0.00877*(P/D_3) \tag{2.26}$$

$$E_s = -111 + 0.00577*(P/D_4) \tag{2.27}$$

$$E_s = -346 + 0.00676*[2P/(D_3+D_4)] \tag{2.28}$$

where:

E_s = subgrade modulus, psi,

P = applied FWD load, lbs, and

D_i = deflection at i^{th} feet from the center of load, mils.

The R^2 for each of these equations was 0.99. Equation 2.28 uses the average value of D_3 and D_4, and while the equation has no theoretical advantage over the other two, it may have the practical advantage of being robust estimators.

Once the subgrade modulus has been determined, the asphalt concrete moduli can be found:

$$\log E_{AC} = -4.13464 + 0.25726*(5.9/h_{AC}) + 0.92874*(5.9/h_B)^{0.5} - 0.69727*(h_{AC}/h_B)^{0.5} - 0.96687*\log E_s + 1.88298*\log (PA_1/D_o^2) \quad (2.29)$$

where:

E_{AC} = modulus of AC layer, psi,

h_{AC} = thickness of AC layer, in.,

h_B = thickness of base layer, in.,

E_s = subgrade modulus, psi,

P = applied FWD load, lbs,

D_o = deflection under center of applied load, mils, and

A_1 = area under deflection basin out to 3 ft. This may be calculated from Equation (2.30) below:

$$A_1 = 4D_o + 6D_{0.67} + 8D_1 + 12D_2 + 6D_3 \quad (2.30)$$

where:

D_i = deflection at i^{th} ft from the applied load, mils.

Similarly, the base modulus may also be found from:

$$\log E_B = 0.50634 + 0.03474*(5.9/h_{AC}) + 0.12541*(5.9/h_B)^{0.5} - 0.09416*(h_{AC}/h_B)^{0.5} + 0.51386*\log E_s + 0.25424*\log (PA_1/D_o^2) \quad (2.31)$$

However, there are limitations to these equations. As with all other regression relationships, the equations are:

1. Valid only for the parameter ranges for which they were developed.
2. Only two- and three-layered pavements may be analyzed with these equations.
3. The FWD configuration must have the 5 or 6 geophone arrangement as described.

The assumptions inherent in elastic layered theory also apply, such as a semi-infinite subgrade, homogeneous and isotropic soils and layers are infinite horizontally. The moduli of the pavement layers follow the relationship: $E_1>E_2>E_3$, and Poisson's ratio for the AC is 0.35, and 0.45 for unbound materials.

Further, as a result of the sensitivity analysis performed, it was found that the subgrade modulus can best be determined when the overlying layers are thin and have a low stiffness. In using Equation 2.29 to estimate E_{AC}, results are best when the surface thickness exceeds 2 inches, $E_{AC} > 10^5$ psi and the subgrade is at its weakest. The base modulus equation (Eqn. 2.31) works best when E_{AC} and E_s are low and E_B is in the mid-range.

2.5 Summary

This chapter has provided a background to the need for an improved overlay design procedure as part of a new Pavement Management System. The problems with the current overlay design procedures are discussed and that clearly indicates the need for an improved procedure. Then, the current state-of-the-art is described, and this encompasses not only the deflection-based and mechanistic procedures, but also simplified mechanistic methods based on regression analysis.

3.0 DEVELOPMENT OF IMPROVED METHOD

This chapter discusses the developmental process used to obtain a new and improved overlay design method. The basic ADOT&PF requirements are also discussed. A two-tiered approach is described in the following sections.

3.1 Preliminary Planning

This new overlay design procedure is to interface with Alaska's new Pavement Management System at the project level. Design will be mostly performed by project managers in each region.

3.1.1 Design Approach

In July 1986, a planning meeting with ADOT&PF was held in Alaska to discuss the project. The advantages and disadvantages of the current overlay procedures were debated, and the outlines of the improved procedure were drawn. Project sites and data needs were also identified at that time. Chapter 2 discusses in detail the problems associated with the current methods.

A two-tiered concept was also determined to be most desirable at that point. The first tier would be a simpler and easier approach than the second. Originally, it was decided that the current deflection-based procedure (Asphalt Institute, 1983) would form the first tier. This procedure could possibly include a modified temperature correction chart to take into consideration Alaska's temperature extremes. This method, using the Asphalt Institute's current temperature correction chart, is available in FORTRAN (Kingham & Jester, 1983) for use on an IBM microcomputer. The program listing and documentation may be found in Appendix B. However, at a later meeting in February of 1987, it was felt that there would be no benefits to retaining the

Asphalt Institute procedure due to the problems discussed in Chapter 2. Also, a move away from the deflection-based procedure towards a mechanistic approach was viewed as desirable, and the fact that ADOT&PF had already begun developing a data base of FWD deflections would ease the initial problems of implementing the new design procedure.

Therefore, it is proposed that the first tier be a simplified-mechanistic approach developed by Fernando et al. (1986) at Pennsylvania State University. This is a simpler and more straightforward approach than backcalculation. However, the range of layer properties used in the study by Fernando et al. were noted as probably not completely appropriate for Alaska's conditions. Therefore, it may prove necessary to develop strain versus deflection relationships specifically for Alaskan pavements. Similarly, performance equations can and should be developed. Chapter 4 evaluates the appropriateness of this procedure for Alaskan conditions.

The second tier is the full mechanistic approach where layer moduli are backcalculated from surface deflections. Several backcalculation programs are discussed in detail in Appendix A. Such backcalculation procedures would require an iterative basin-fitting procedure, which implies the considerable use of computer time. On the other hand, a knowledge of in situ moduli is sometimes needed, in which case this procedure is justified.

3.1.2 Other Items Considered

Other items considered during this stage of the planning included the following:

1. Layer Properties. Accurate resilient moduli values are needed to determine the structural capacity of a pavement. To reduce the number of unknowns in the backcalculation procedure, cores

should be used to determine the surface layer properties. Presently, a minimum of 5 cores per mile (1.6 km) are taken in the Anchorage region. Backcalculation programs can then be used with deflection readings to obtain the resilient moduli values of the other layers. A recent study by Rwebangira et al. (1987) has also indicated that the backcalculated moduli from three such computer programs are sensitive to variations in layer thicknesses. Therefore, care should be taken to determine thicknesses - if possible, cores are to be taken, otherwise construction records may also prove helpful. In particular, the thickness of the asphalt concrete layer plays an influential role in the determination of the modulus.

2. Construction Practices. The standard roadway construction practices of Alaska were also taken into consideration, such as the use of insulation. If an insulation layer exists in cross sections, then this would affect the thermal regime of the pavement and thus the resilient moduli and surface deflection values. In the mechanistic approach, the appropriate range in modulus for the pavement layers can be input to account for the difference in the thermal regime.

3. Thaw Depths. The location of the thaw depth directly influences resilient moduli values and deflection readings as discussed in Section 2.3. However, because methods of determining the thaw depth are inexact and have a high percentage of error (Stubstad & Connor, 1982), present ADOT&PF policy is to take weekly deflection readings during the critical time of the year

(usually Spring) until a peak deflection is obtained. Then the 95^{th} percentile is used for design together with the moduli values for this period. The modified Berggren equation may not be be used to calculate the thaw depth as it was developed to calculate thaw for complete seasons, not partial seasons.

4. Seasonal and Geographical Effects. Deflection measurements vary depending on the time of the year (summer vs winter) and on the geographical location (e.g. winter in Fairbanks vs winter in Juneau). Therefore, to develop a consistent data base of deflection readings, some guidelines need to be set. Currently, all readings are obtained between March and July and then normalized to 70°F (21°C) using the Asphalt Institute relationship (Asphalt Institute, 1983). The seasonal correction data available at present are unreliable. Therefore, it was concluded that seasonal correction factors should only be used as a last resort. However, because it may not always be possible for all five maintenance districts to conduct their testing at the same time every year (e.g. because of budgetary or equipment limitations), some correction factors may still be required. These correction factors would, in effect, be temperature correction factors tailored specifically for each district in Alaska. It should also be noted that the term "critical" indicates the time of the year when the pavement is structurally weakest, and the pavement strains, not surface deflections, would be indicative of this condition.

5. <u>Traffic Volume</u>. Pavements fail due to a combination of many factors, of which the loads carried by the pavement are the most important. As a wheel load passes over a pavement, elastic and plastic deformations occur. Elastic deformations may lead to fatigue and plastic deformations to excessive rutting. The higher the volume of traffic, the greater the cumulative damage that is done. Typically, the loads due to passenger cars are insignificant, with truck loads being the most significant. In designing an overlay, a knowledge of the remaining life is desirable, and remaining life is measured in 18-kip equivalent axle loads (EALs) applications. It is important to recognize that the error in estimating remaining life can be rather large depending on the reliability of historical traffic information and future traffic estimates (Finn & Monismith, 1984).

Therefore, it cannot be emphasized strongly enough that an accurate and reliable traffic count is essential for a good overlay design. In addition, future traffic volumes and growth rates also need to be estimated or predicted, and a reliable data base of past traffic volumes facilitates this procedure. Also, to predict the life cycle of the overlaid pavement, traffic volumes are needed. Finally, the effect of traffic distribution across lanes needs to be examined. On four or more lane highways, the outermost lane typically carries the majority of the truck loads and therefore may require rehabilitation before the inner lanes.

6. Remaining Life. Pavements deteriorate, i.e. have a reduced life, after exposure to traffic for extended periods of time. In effect, a pavement "uses" a part of its total life as a result of load repetitions imposed by traffic. By the time distress conditions appear, a certain amount of the useful life of the pavement has been used and must be accounted for in the design process. At the same time the remaining life of the existing pavement can be utilized in designing the pavement for future conditions.

   Remaining life in the existing pavement can be estimated using some form of a cumulative damage hypothesis. In the case of asphalt pavements, fatigue in the asphalt-bound layer is usually the most important distress mode (Finn & Monismith, 1984), and the linear summation of cycle ratios cumulative damage hypothesis permits an estimate of the fatigue damage to be made. This is also known as Miner's Hypothesis. The Asphalt Institute's deflection-based method does provide for an estimate of remaining life using effective thickness procedures. However, ADOT&PF does not use this in their current design procedure.

7. Temperature. The surface deflection of a pavement varies depending on the temperature. Generally, the lower the temperature, the lower the deflection measurement because the pavement is more rigid. Therefore, deflections taken at different temperatures need to be normalized to some standard temperature, usually 70°F (21°C). Currently, ADOT&PF uses the

Asphalt Institute's temperature correction chart (Figure 2.4). However, these charts were not developed with Alaska's extreme temperatures in mind, and new correction factors are probably justified. New temperature correction factors that were developed from Fairbanks data are discussed in Appendix D. However, there were many assumptions made on temperature effects and pavement structures, and these factors should be developed in greater detail before being applied as a standard.

8. Deflection Basin versus Maximum Deflection. In the mechanistic procedure, the deflection basin is a better indicator of pavement distress (Stubstad & Connor, 1982; Kulkarni et al., 1986) than maximum deflection. However, the maximum deflection is still currently used as the criterion for deciding the location of the critical analysis section. In the Asphalt Institute procedure, the basin is also not used. Instead, the representative rebound deflection (mean + 2 standard deviations) is used for design. The use of the maximum deflection is obviously simpler than the deflection basin and requires less time and effort. But, as has been discussed, this is simply not an accurate indicator of pavement distress. The strains in the pavement would provide the best indicator. This report attempts to include this in the design procedure.

9. Other Overlay Procedures. Many other state and federal agencies as well as private institutes have developed their own overlay design procedures. In an effort to prevent duplication

of their efforts and mistakes, other procedures should be reviewed and lessons learned from their mistakes. Section 2.4 summarizes and discusses some of these procedures.

10. Life of Overlay. Obviously, the desired design life of the overlay affects the overlay design. The longer the design life, the greater the number of loads that are applied over the pavement and, therefore, the thicker the overlay.

11. Analysis Section. There does not appear to be any clear criteria used in the selection of an analysis section for design. Currently, the center-of-load deflections (after being adjusted for temperature and load) are summarized for every mile (1.6 km). There are five readings per mile and these are averaged and the mean plus two standard deviations obtained. The maximum value is then theoretically used for design, despite the fact that peak center-of-load deflections are not always a good indicator of pavement performance, as discussed in Chapter 2. However, This value is not always used in actual design. It appears that some "anomalies" are thrown out by designers, and the mean recalculated. This can prove misleading, particularly if what is deemed "excessively high" deflections are thrown out. The mean deflection is then reduced, and the overlay designed with this value, although the "high" deflection could have been representative of a section over 1000 ft (320 m) long. If this is the case, the overlay is under-designed, and may fail earlier than expected.

Ideally, the strains should be calculated for each deflection basin since strain is a much more accurate indicator of distress and would form a better criterion for selecting an analysis section. However, given the literally thousands of deflection basins available, this would be too time-consuming and expensive. Therefore, some form of screening procedure is needed to select a few deflection basins with ease and alacrity. This, however, is the crux of the problem. There does not appear to be any easy and reliable method of screening the deflection basins to obtain the critical deflection basins.

In discussions with various professionals, three factors were mentioned again and again. They are:

a. Highest and lowest subgrade modulus,

b. Highest and lowest center-of-load deflections, and

c. An area factor of the deflection basin. One such factor is that developed by Hoffman and Thompson (1981) shown below:

$$A_f = 6(1 + 2S3/S1 + 2S5/S1 + S6/S1) \tag{3.1}$$

where:

A_f = Area factor, mils, and

S_i = deflection reading at i^{th} sensor, mils.

If the area factor has a maximum value of 36, then the pavement is perfectly flexible. Typically, the factors range from 7 to 15 for the projects analyzed.

Appendix C details the recommended procedure for selection of the critical section. Briefly, FWD deflections for the same location over a period of time, preferably March to July, are needed. This is if the overlay is to be designed for the weakest and therefore most critical pavement condition. The maximum strain may be calculated, and this indicates the critical time of the year. Once this has been determined, only deflections taken at this time are considered for analysis. The designer then selects the sections with the lowest and highest center deflections and area factors as the design sections.

12. Condition Surveys. Condition surveys include not only the type of distress that has occurred but also its severity, its extent and the location (Haas & Hudson, 1982). Reflection cracking, in particular, is an important pavement distress consideration. Cracking represents materials failure in the form of a fracture. Although a crack when it first occurs may not immediately affect the serviceability of a pavement, it may lead to rapid losses in serviceability with small subsequent increments of time and/or load repetitions. Currently, there are no documented mechanistic procedures that can be used routinely to select overlay designs to minimize reflection cracking (Finn & Monismith, 1984). However, McCullough and Seeds (1982) have presented an analytical procedure that analyzes the reflection cracking problem. Essentially, the procedure defines shear and tensile strain in the overlay resulting from differential

vertical movements at the crack due to traffic loads and due to horizontal movements at the crack because of a drop in temperature, respectively. Finite-element methods are also possible in this analysis, and some investigators such as Coetzee et al. (1986) have made some studies in this area. However, such approaches are not yet available on a routine basis at this time because of the need for computers with relatively large capacities (Finn & Monismith, 1984).

13. Reliability. During the course of discussions with ADOT&PF, the subject of reliability was addressed. The agency felt that they might not necessarily have the resources to design low-volume roads, particularly those in the rural areas, to standards that an arterial or high volume highway would be designed. Therefore, some measure or indicator of reliability would be needed.

Reliability, as defined by the proposed AASHTO design guide (AASHTO, 1985), is "the probability that a pavement section ... will perform satisfactorily over the traffic and environmental conditions for the design period." A reliability factor, F_R, is used to incorporate this concept into the design process based on a shift in the design traffic. The predicted traffic, in EALs, for the design period (typically 20 years), is increased by multiplying a selected reliability factor, F_R, that is greater than 1. Appendix E describes this procedure in detail and contains the necessary tables.

14. Project Sites. Finally, six project sites were initially selected to evaluate the improved overlay procedures. The sites were to cover a range of geographical, seasonal, structural and functional characteristics. Input data and output requirements were also listed. This is further discussed in Chapter 4.

3.2 Basic ADOT&PF Requirements

This section describes the basic requirements that ADOT&PF required for the improved overlay design procedure. There are essentially three subsections discussed here: the expectations, inputs, and outputs of the procedure.

3.2.1 Expectations

From meetings and other communications with ADOT&PF, the following expectations were discussed and an attempt was made to accommodate as many of these factors as possible in developing the framework. Table 3.1 summarizes the expectations of the improved method. This discussion is essentially limited to the mechanistic approaches since the deflection-based method is not treated in this report.

Simplicity was to be a prime requirement of the new procedures, particularly the complex mechanistic procedure. The new procedures should preferably be adaptable for use in a microcomputer rather than the more awkward and cumbersome mainframe computer. To facilitate the use of these computer programs, clear instructions should be available regarding the inputs. As was discussed earlier, a two-tiered approach was proposed.

The new Pavement Management System in Alaska has proposed that condition surveys be used to evaluate the current condition of pavements and are to be

Table 3.1. Expectations of Improved Method.

Item		Description
Simplicity	1.	Ease of use.
	2.	Adaptable for microcomputers.
	3.	Two tiers - simple and complex.
Effects of Surface Condition on Rehab. Strategy	1.	Effect on Rehab. Strategy (e.g. overlay, recycling, removal).
	2.	Type and extent of distress.
	3.	Relations to remaining life.
Deflections & Coring	1.	Use FWD to obtain deflections.
	2.	Maximum deflection value as well as deflection basin used.
	3.	Base and surface thickness from cores and/or plans.
	4.	Surface modulus from cores.
Determination of Layer Contribution	1.	Use of backcalculation procedures.
	2.	Contributions of individual structural layers.
Benefits	1.	Savings over existing methods.
	2.	Change in level of effort required.
	3.	Foundation for design has theoretical basis.

used as a basis for selection of the appropriate rehabilitation strategy (see Section 2.1). Also, because surface condition affects the remaining life of the pavement the new procedures should take surface condition into account. Consequently, surface condition surveys are necessary to determine the type, extent and severity of distress, especially reflection cracking. However, as discussed in the previous section, there are currently no simple nor easy methods of incorporating reflection cracking considerations without being bogged down into a detailed, long and drawn-out process using finite element methods. Hence, a choice has to be made between simplicity and a more accurate (but more complicated and expensive) overlay design procedure. For this report, simplicity became the more important factor.

The FWD has replaced the Benkelman Beam and other NDT equipment and will be the only NDT device used in obtaining surface deflection measurements. The maximum deflection value as well as the deflection basin is used both in the backcalculation analysis and in the simplified procedure. To reduce the number of unknowns in the pavement layer system and to serve as a check on the backcalculation procedures, cores are to be taken to obtain the surface layer modulus and layer thickness. If cores are not available, construction plans should be employed to determine the layer thicknesses as accurately as possible.

In the backcalculation procedure, the effect of layer contribution is included in the analysis, so the location of the sensors are important. Specifically, the sensors should be placed far enough away from the load to obtain the sole contribution of the subgrade to the pavement strains and to acquire a full description of the deflection basin. The furthermost sensor at present is located at 47.2 in. (1200 mm) from the load.

Finally, the benefits expected from these improved procedures would include not only cost savings in terms of more efficient roads, but also a reduction in the level of effort required for design and maintenance. In addition, with the use of the mechanistic procedure, the design has a strong theoretical foundation.

3.2.2 Inputs

Table 3.2 summarizes the inputs needed for these procedures. They include the deflection measurements from the FWD, both maximum deflection and the deflection basin. Measurements should be taken during the critical time of the year, which would be during the period of spring thaw. At that time, the pavements are weakest, and overlays are to be designed for the most conservative condition. If measurements are not possible during that time of the year, then seasonal correlations are required. These would have to be developed. Also, pavement surface temperatures should be taken at the same time so that deflections may be normalized to one standard temperature, usually 70°F (21°C). In addition, geographical correction factors may be needed.

It cannot be emphasized strongly enough that accurate counts of the anticipated traffic volumes and loads are necessary for an accurate overlay design. Traffic growth rates are also needed, and again, there is a necessity for accuracy. At present, there is a weigh-in-motion (WIM) project at Golden River. Portable automatic vehicle classification (AVC) units are taken out and left on site for a week at a time to obtain traffic counts. However, it is likely that such short periods of time would yield an unrepresentative value of the annual traffic volumes. It is suggested that some units be left

Table 3.2. Inputs to Overlay Design Procedure.

Item	Description
Deflection Measurements	1. Taken by FWD. 2. Deflection basin using standardized sensor positions. 3. Maximum deflection. 4. Surface temperature. 5. Seasonal and geographical correction factors.
Traffic Volumes	1. Historical traffic data to date. 2. Anticipated traffic loads and volumes. 3. Distribution of loads. 4. Growth rates.
Layer Properties and/or Contribution	1. Modulus of AC from cores. 2. Thickness of layers from cores or plans. 3. Poisson's Ratio is assumed.
Surface Condition (For future procedures)	1. Extent and intensity of cracking. 2. Also rutting, roughness, and patching.
Current Construction & Rehab. Practices	1. Type of pavement structure. 2. Feasible rehabilitation strategies.
Other Items	1. Design schedule - how to collect data within appropriate time frame. 2. Financial/political factors - affects rehabilitation strategy.

on site for at least two weeks at different times of the year to obtain a more representative profile of the traffic volumes and distribution.

Layer properties also play a major role in the validity of the final overlay design. Cores should be used to ascertain the surface layer's modulus through laboratory testing and should be taken. Currently, Anchorage obtains 5 cores per mile (3 per km). Layer thickness may be determined through cores or construction plans. An assumption of Poisson's ratio are consistent with previous research; typically, 0.35 is used for the asphalt concrete and base layers and 0.45 for the subgrade.

Condition surveys are needed to obtain the necessary data on the type, extent and severity of distress of the pavement. This, in turn, is to be used as a basis for selecting the appropriate rehabilitation method as discussed in the documentation for the PMS (Woodward-Clyde, 1986). Furthermore, as techniques become more sophisticated in the future, reflection cracking may be incorporated into the estimate of remaining life and a database of historical condition survey information will be needed.

Finally, other factors include a knowledge of the current pavement structures constructed, feasible rehabilitation strategies, and design schedule. Financial and political factors may also play an important role in the design process as well but they are too complex to be predictable and are not included in this report.

3.2.3 Outputs

Table 3.3 summarizes the outputs of these improved procedures. As expected, a fairly accurate description of the pavement properties will be obtained, including moduli, layer thicknesses and stresses and strains. The remaining life of the pavement would be (ideally) accurately predicted, and

Table 3.3. Outputs of Overlay Design Procedure.

Item	Description
Layer Properties	1. Reslilient modulus. 2. Layer thickness. 3. Stresses and strains.
Remaining Life	1. Accurate prediction. 2. Correction due to surface condition.
Overlay Thickness	1. Thickness required.

the thickness of the overlay needed to bring the pavement back up to structural standards.

3.3 Developed Framework

Figure 3.1 illustrates, in a simplified manner, the developed framework. The following sections describe in greater detail both of the proposed approaches.

3.3.1 Simplified Mechanistic Procedure

This simplified mechanistic procedure was developed by Fernando et al. (1986) at Pennsylvania State University. The relationships and equations discussed in Section 2.4.3.1 were developed for Pennsylvanian conditions, and so these equations may not be valid for Alaska. Some experimentation on the validity of the procedure should be performed. However, it should be possible to derive similar relationships utilizing Alaskan conditions if needed. To do this, a program such as ELSYM5 or BISAR may be used to analyse a wide variety of pavement conditions and to calculate deflections. A comparison with actual field conditions and backcalculated moduli should also be performed as a check. The simplified mechanistic procedure is, after all, a much easier approach than the backcalculation procedure, and the results of such relationships appear to be closely related to that of backcalculation procedures. Figure 3.2 shows the flowchart of the simplified-mechanistic procedure.

Once the critical analysis section has been determined for design (following the recommended procedure described previously), design of the overlay can begin. The past and future traffic over the life of the overlay together with the AC modulus is needed to compute the tolerable strains using appropriate fatigue and permanent deformation criteria. From the analysis

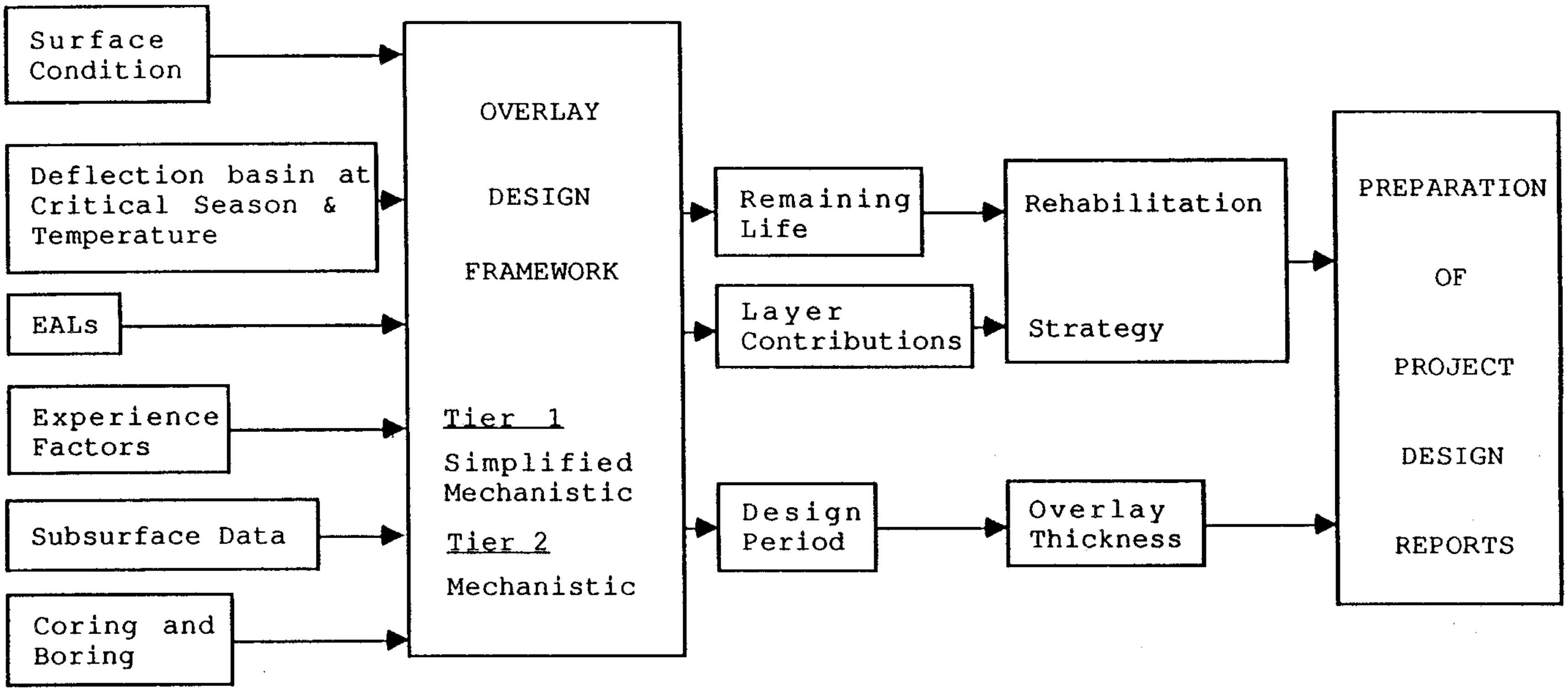

Figure 3.1. Simplified Framework for the Improved Overlay Design Method.

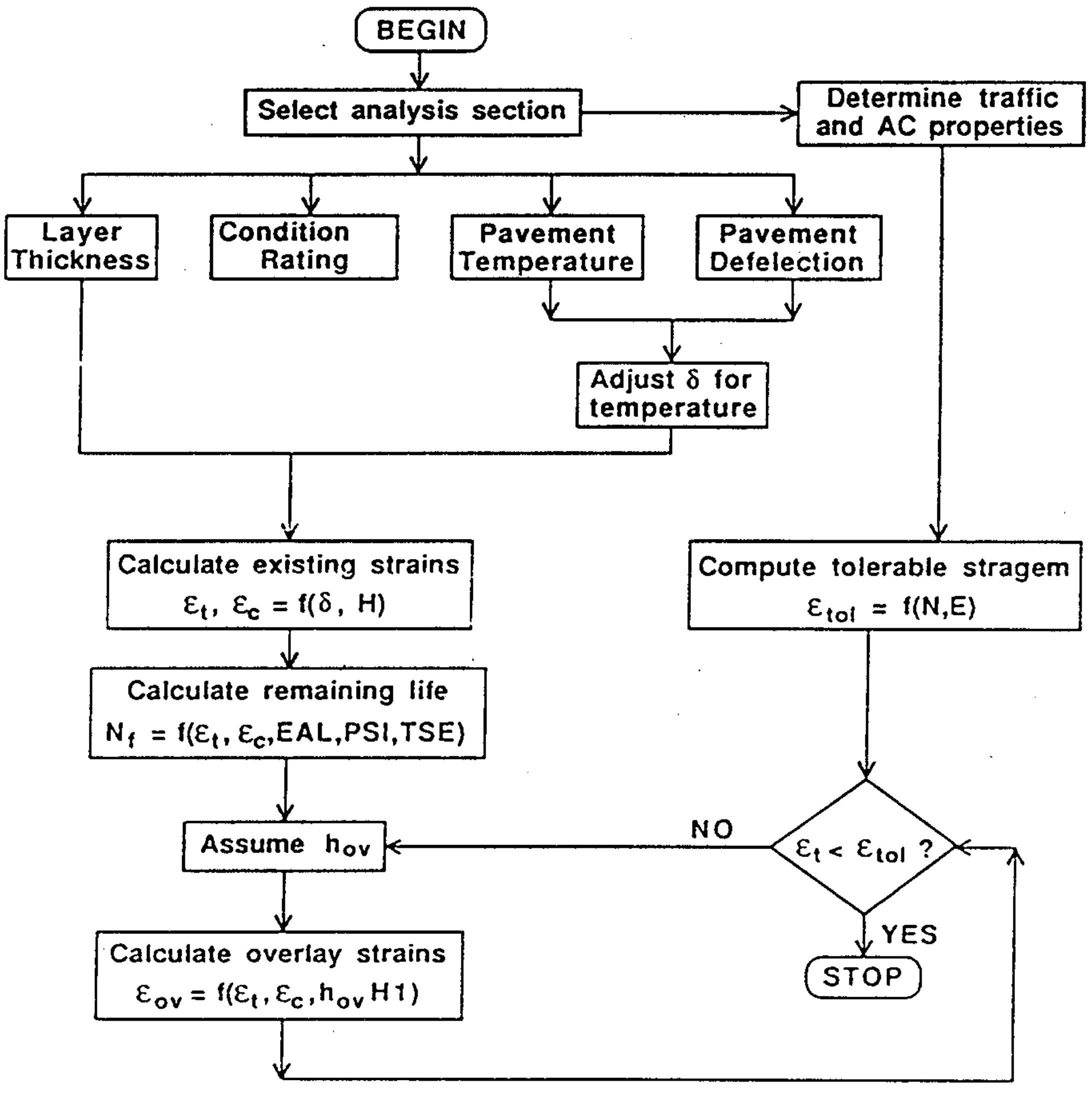

Figure 3.2. Flowchart for the Simplified Mechanistic Procedure.

section, layer properties, condition ratings, pavement temperatures and FWD deflections must be obtained. Once this has been completed, the strains and remaining life of the existing pavement can be computed. An overlay thickness is then assumed, and the strains recalculated. If the recalculated strains are less than or equal to the tolerable strains, then the selected overlay thickness is sufficient to meet the needs of the design criteria.

The inputs needed to utilize Fernando's relationships and to perform the above steps are summarized in Table 3.4. They are discussed in greater detail in the following paragraphs.

1. Layer properties. They include the thicknesses of the existing AC and base layers. These may be obtained from sample cores or from design and construction records.
2. Deflections. Deflections are obtained from the FWD. The equations developed were for load sensors that are located 1 ft (30.5 cm) apart. Only the first three deflection readings (0, 1 and 2 ft) were found to be statistically significant.
3. Design Criteria. Fernando et al. use the performance equations developed by Austin Research Engineers, Inc. (1975) (Equation 2.22) for tensile strains and that developed by Luhr et al. (1983) (Equation 2.23) for subgrade strains.
4. Pavement condition. The initial pavement serviceability index (PSI) and the terminal serviceability index (TSI) are required if the performance equation developed by Luhr et al. (1983) is to be used.

Table 3.4. Inputs to the Simplified Mechanistic Procedure.

Symbol	Description
1. <u>Design Criteria</u>	
ϵ_{tol}	Tolerable strains from design criteria.
2. <u>Pavement Properties</u>	
H_1	Thickness of AC layer, in.
H_2	Thickness of base and subbase, in.
3. <u>Pavement Condition Rating</u>	
PSI	Pavement serviceability index of existing pavement.
TSI	Terminal serviceability index of overlay.
4. <u>Deflections</u>	
δ_1	FWD deflections at center-of-load.
δ_2	FWD deflections 1 ft from load.
δ_3	FWD deflections 2 ft from load.
T	Pavement temperature.

1 in. = 2.54 cm
1 ft = 30.5 cm
1 psi = 6.89 kPa

3.3.2 Mechanistic Procedure

A fully mechanistic design procedure characterizes the response of the pavement to a load in terms of strains and/or stresses in various pavement layers. A fatigue relationship between that response and number of load repetitions to a designated failure criteria is used to determine pavement life. Mechanistic and deflection procedures are not mutually exclusive. Most procedures use stress or strain level based on deflection testing as the pavement response that is related to performance. The difference between such a system and a "deflection" approach is that the deflection used to develop the performance relationship is based on a mechanistic model rather than an empirical one.

Figure 3.3 illustrates the flowchart of the complex mechanistic procedure. As in the deflection-based procedures, nondestructive pavement evaluation, condition surveys and traffic are required as inputs. In addition, some knowledge of the stiffness properties and distress characteristics (such as fatigue cracking and plastic deformation) of the various materials comprising the pavement structure are needed. Stiffness characteristics of the various pavement components can either be defined by tests on undisturbed or representative specimens of the pavement components, or inferred from NDT measurements.

The FWD used to measure the structural response provides a measure of the surface deflection under an impulse load. Measurements should be obtained at reasonable intervals throughout the project. The condition of the pavement must be carefully ascertained. This is used to determine the analysis sections and to help establish performance criteria for related distress.

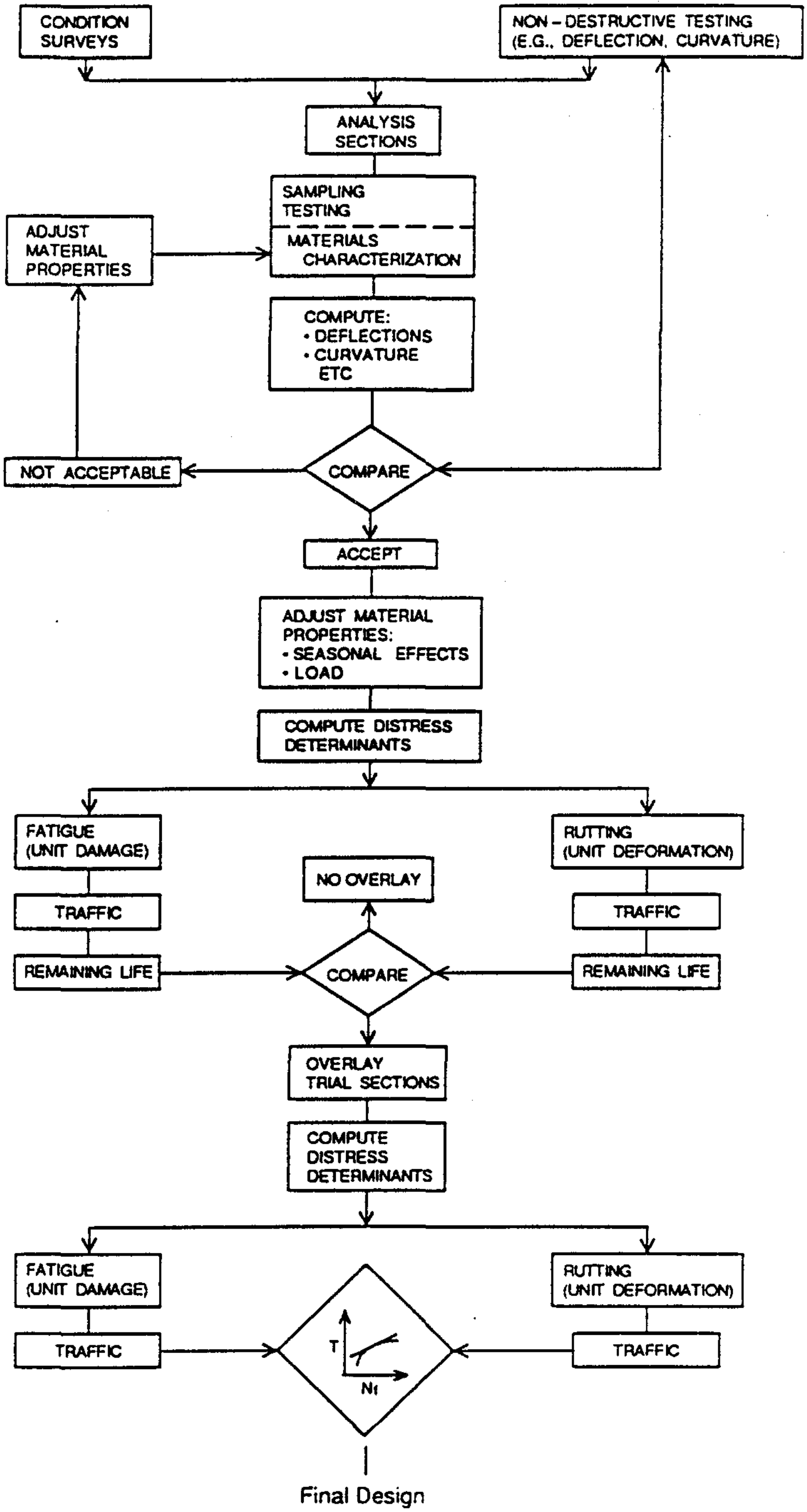

Figure 3.3. Flowchart for the Mechanistic Method (Finn & Monismith, 1984).

Once the analysis sections have been established, it is then necessary to establish a representative or "design" deflection for that section. It is recommended that this value be set somewhere in the 80 to 90 percentile range, i.e. 80 to 90 percent of the deflections in the section will be equal to or less than the values chosen (Finn & Monismith, 1984).

The stiffness characteristics of the various layers can be estimated from surface deflection measurements. The shape of the deflection basin is defined by deflections measured directly under a load and at a number of radii. By use of a computer program for selection of stresses and deformations in a multi-layered elastic system, a set of modulus values is determined that provides the best fit between the measured and computed deflection basins of the pavement surface. Normally, the procedure involves assuming a set of modulus values and then iterating with the computer until the measured and computed deflections are in "reasonable" agreement. Various programs are available to perform the backcalculation analysis, as described in Appendix A. It is recommended, however, that some laboratory testing be performed to verify the results (Finn & Monismith, 1984).

Traffic volumes using the facility should be known. The distribution of traffic across lanes and the concentration of truck traffic in the outer lane should be recognized. With the traffic information and stiffness properties, critical performance parameters can be determined using layered elastic analysis. The parameters can be related to "acceptable" and "not acceptable" performances observed in the condition survey as well as to laboratory defined distress criteria.

Since Alaska has not developed their own design criteria at present, those developed by the Asphalt Institute (1982) may be used. These were selected because of their widespread use. For control of fatigue in the asphalt layer, Equation 3.2 (English units) is used:

$$N = 18.4*C*0.004325*\epsilon_t^{-3.291}*E^{-0.854} \tag{3.2}$$

where:

N = number of 18-kip (80 kN) equivalent single axle loads,

ϵ_t = horizontal tensile strain on underside of existing AC layer,

E = modulus of AC layer, psi, and

C = a function of voids and volumes of asphalt in the mix design, and can be calculated:

$$C = 10^M \tag{3.3}$$

and $$M = 4.84*[V_b/(V_v+V_b) - 0.69] \tag{3.4}$$

where:

V_b = volume of asphalt, %, and

V_v = volume of air voids, %.

Similarly, the vertical compression strain criterion is used to control permanent deformation:

$$N = 1.365 \times 10^{-9}*\epsilon_c^{-4.477} \tag{3.5}$$

where:

ϵ_c = vertical compression strain at the subgrade surface.

If the future anticipated traffic for the life of the overlay were known, it is possible to rearrange Equations 3.2 and 3.5 to obtain the tolerable strains.

The remaining life of the pavement can be determined using Miner's Hypothesis. A simple form of the expression is:

$$N_r/N_{D1} = 1 - N_{A1}/N_{D1} \qquad (3.6)$$

where:

N_r/N_{D1}	=	remaining life,
N_{A1}	=	number of applications of EALs to date,
N_{D1}	=	allowable number of applications of EALs according to fatigue relationships, and
N_r	=	additional number of applications of EALs that can be applied to the existing pavement.

If an overlay is needed it must then be designed to resist fatigue and rutting (if these are the distress criteria). For a specific thickness of overlay to minimize fatigue, the tensile strain is determined on the underside of the existing layer. The allowable number of applications may be estimated from some form of a fatigue relationship and modified by the remaining life ratio. It is possible to define the relationship between overlay thickness and additional load applications. At the present time, the Asphalt Institute criteria are recommended.

Permanent deformation is of concern only at the surface of the overlay. It can be assumed that the overlay will fill any existing ruts. As before, a relationship between overlay thickness and load applications can be determined.

If other distress modes are considered, similar relationships between thickness and load applications can be developed. The design overlay thickness is the maximum values required to satisfy the various conditions (Finn & Monismith, 1984).

3.4 Summary

This chapter has focused on developing a framework for the improved overlay design procedure. During the preliminary planning, there were discussions with ADOT&PF on what was expected from this project and what was required.

In the final outcome, there is not just one procedure but two procedures proposed. To recapitulate, the first is a simplified mechanistic approach, employing both empirical and mechanistic principles. This is the method developed by Fernando et al.(1986) at Pennsylvania. An empirical strain versus deflection relationship is obtained from a factorial study and overlay equations are developed. The second is the fully mechanistic procedure which uses backcalculation programs to determine pavement layer properties that will result in the same deflection basin as the measured deflection basin. Based on the distress criteria selected, an overlay thickness that meets them is obtained.

4.0 EVALUATION OF DEVELOPED METHODOLOGIES

This chapter describes the project sites selected as experimental sites for the new overlay design methodologies. The project data are then evaluated using the developed methodologies from Chapter 3.0. Furthermore, the results of the new procedures are compared with the current official ADOT&PF procedure and the Washington procedure, and the findings are discussed.

4.1 Projects Sites Evaluated

Originally, six project sites were planned; two each from the following three regions: the Central Region (Anchorage), the Interior (Fairbanks) and the South Eastern Region (Juneau). The two sites in each region were to reflect the diversity in climate and temperature as well as reflect a diversity in the roadway's functional and structural class. The selected projects were to come from both urban and rural areas, representing different pavement strengths and traffic loadings.

However, only three sites were evaluated, two from Anchorage and one from Fairbanks. The following sections describe each project site in detail. Figure 4.1 shows the location of the three project sites. Table 4.1 summarizes the parameters of all three projects.

4.1.1 Sterling Highway, Anchorage

The Sterling Highway project is 54 miles (87 km) long and is located southwest of the city of Anchorage. It extends from Clam Gulch (MP 117) to Homer High School (MP 171). This is a rural highway that hugs the coastline of Cook Inlet. Figure 4.2 shows the location of this project. Based on FWD deflection readings, pavement condition surveys, materials testing, pavement cores and traffic volumes (measured in EALs), the project was divided into

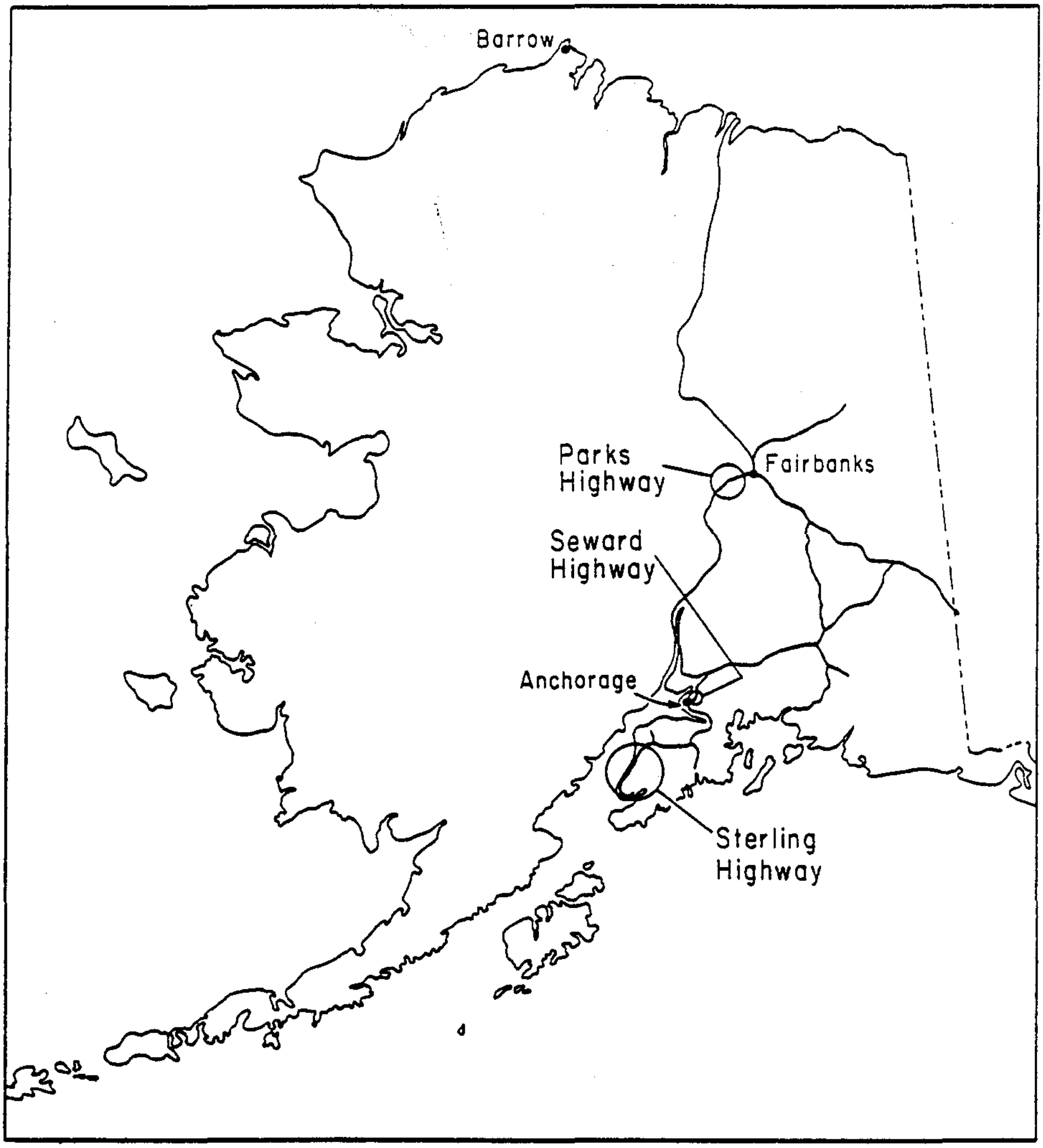

Figure 4.1 Location of Project Sites in Alaska.

Table 4.1. Summary of Parameters for Project Sites.

Project Location	Pavement Structure[1]	Traffic To Date	Traffic 20 yr. EAL	Pavement Condition
Sterling Highway				
MP 117-130	AC - 1.5 in. Base - 4.0 Subbase - 6.0 Borrow - 24.0	130,000[2]	1,800,000	Good
MP 130-157	Same as above	130,000[2]	1,800,000	Extensive alligator cracking and rutting
MP 157-162	AC - 1.5 Base - 6.0 Borrow - 0 to 36.0	130,000[2]	1,800,000	No fatigue cracking
MP 162-166	Same as above	unknown	2,770,000	5-26% cracking
MP 166-171	Same as above	unknown	7,980,000	100% cracked
Seward Highway				
36th to 4th Ave.	AC - 2 to 5.25 Base - 6.0 Subbase - 18.0	4,400,000	5,083,000 (10 years)	Extensive cracking and rutting
Parks Highway				
North Section	AC - 2.0 Base - 4.0 to 6.0 Subbase - 6.0 to 12.0 Borrow - 24.0 to 36.0	78,723[2]	390,521	Severe rutting and alligator cracking
South Section	Same as above	76,069[2]	345,526	Severe rutting and alligator cracking

[1]For Parks Highway, these are assumed dimensions.
[2]Traffic data are assumed.
1 in. = 2.54 cm

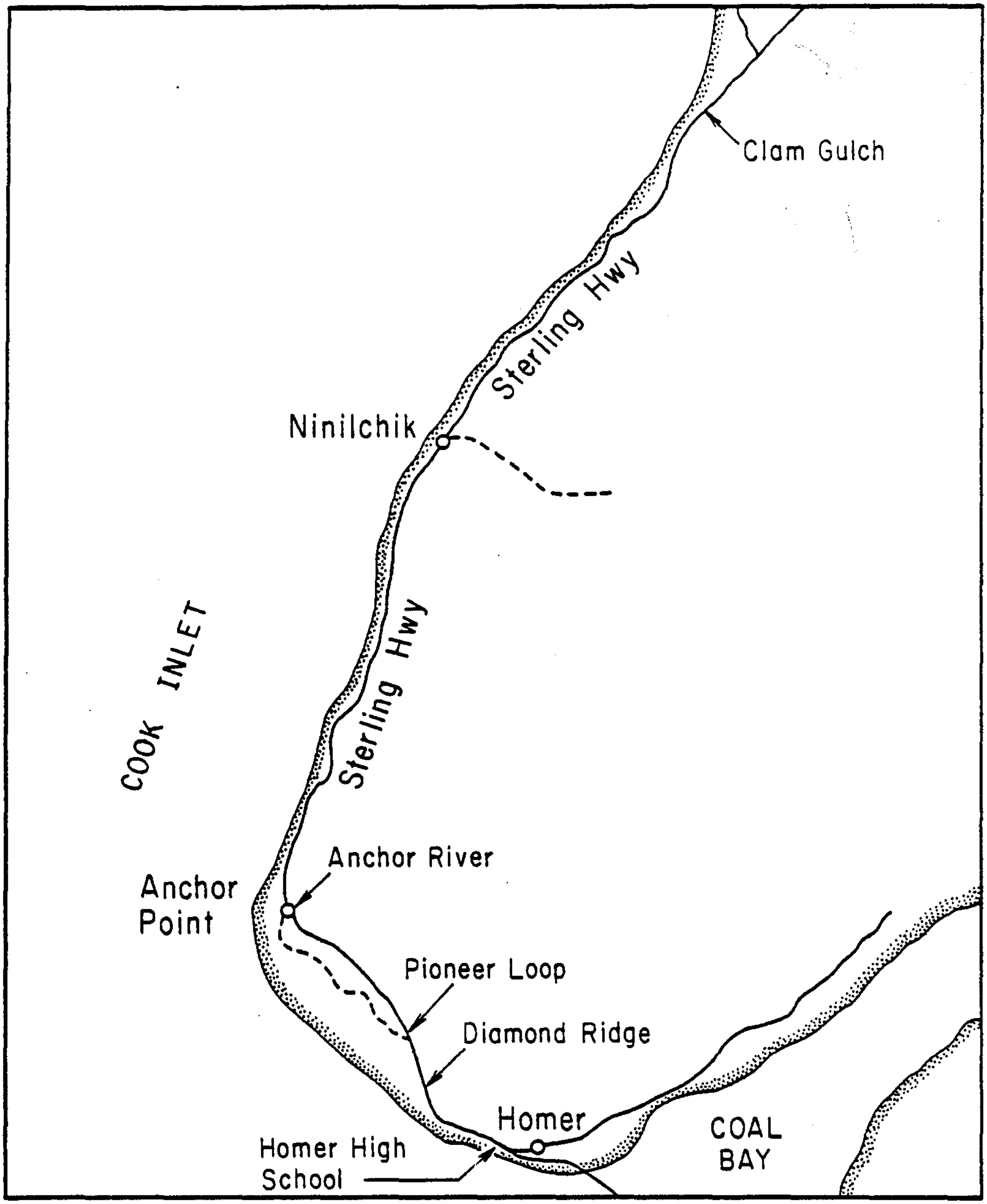

Figure 4.2 Sterling Highway Project Site.

five sections by the regional pavement engineer. The overlays required for each section were designed separately. Appendix C gives further information on the pavement parameters and details the calculations performed to obtain the overlays. The traffic volumes were obtained from a portable WIM site in December 1986. Because up to 30% of the trucks at that time exceeded the legal weight limit by substantial amounts, the EALs were almost doubled. Since the traffic data collection occurred over a short period of time (a few days), it is also possible that a seasonal variation in the traffic accounted for the high volumes. Therefore, another traffic count has been planned for the summer of 1987, and for a longer period of time. Construction on this project is not expected to be undertaken for another three to four years. The thaw depth estimated from the modified Berggren equation was 30 in. (76 cm). However, due to a lack of accurate data, the field estimate of 15 in. (38) was used for analysis.

The first section extends from Clam Gulch (MP 117) to 2 miles (3.2 km) north of Ninilchik (MP 130). The pavement is in good condition and no alligator or fatigue cracking was found when the survey was conducted in 1986. The existing pavement structure consists of a 1.5 in. (3.8 cm) asphalt concrete layer, a 4 in. (10 cm) base course, a 6 in. (15 cm) subbase and 24 in. (61 cm) of select borrow material. The design traffic volume was projected to be 1,800,000 EALs for a 20 year design life. The remaining life of the pavement is estimated to be four years.

The second section continues from MP 130 to Anchor River (MP 157). Although this section has an identical pavement structure and the same traffic volumes as the preceding section, it exhibits extensive alligator cracking and severe rutting. This is possibly due to the percent of fines in this section exceeding the specified maximum value of 6% (see Appendix C).

The third section continues on to a mile north of Pioneer Loop (MP 162). There is no fatigue cracking. The pavement structure consists of a 1.5 in. (3.8 cm) AC layer, a 6 in. (15 cm) base, and from 9 - 3 ft (0 - 91 cm) of select material. Again, traffic volumes are the same as the preceding two sections.

The fourth section extends from Pioneer Loop to Diamond Ridge Road (MP 166), and 5 to 26 percent of the pavement is cracked. It is expected that there will be 100% cracking by the time this pavement is rehabilitated. The pavement structure is similar to that in the third section. However, because of the higher traffic levels (2,770,000 in 20 years), the road is in worse condition.

Finally, the last section extends from Diamond Ridge Road to Homer High School (MP 171) and is 100% cracked. The pavement structure is similar to that of the third section but because of much higher traffic volumes, this section exhibits a greater percentage of surface distress. The 20 year EAL is 7,980,000.

4.1.2 Seward Highway, Anchorage

The Seward Highway project is located in downtown Anchorage, between 36th and 4th Avenues (see Figure 4.3). From 36th to 22nd Avenues, this is a 6-lane facility, separated by a median. Beyond 22nd Avenue, the highway splits into two 3-lane one-way streets, Gambell and Ingra. Figures 4.4 and 4.5 indicate the pavement distress that is readily obvious on this project.

Pavement cores were taken at approximately every 1000 ft (305 m). From the cores, it was determined that the asphalt layer thicknesses ranged from 2

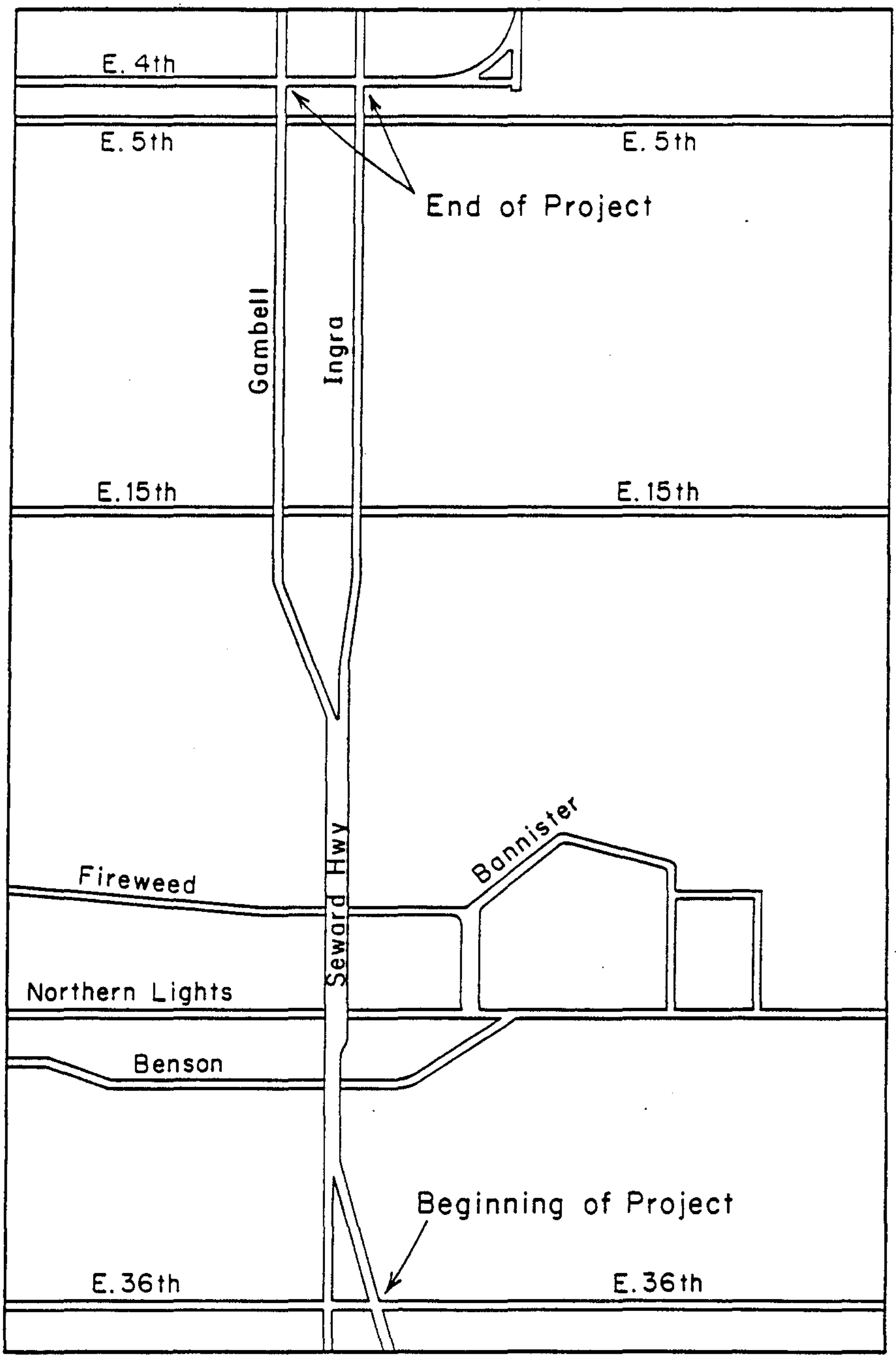

Figure 4.3 Seward Highway Project Site, Anchorage.

Figure 4.4 Seward Highway along Gambell.

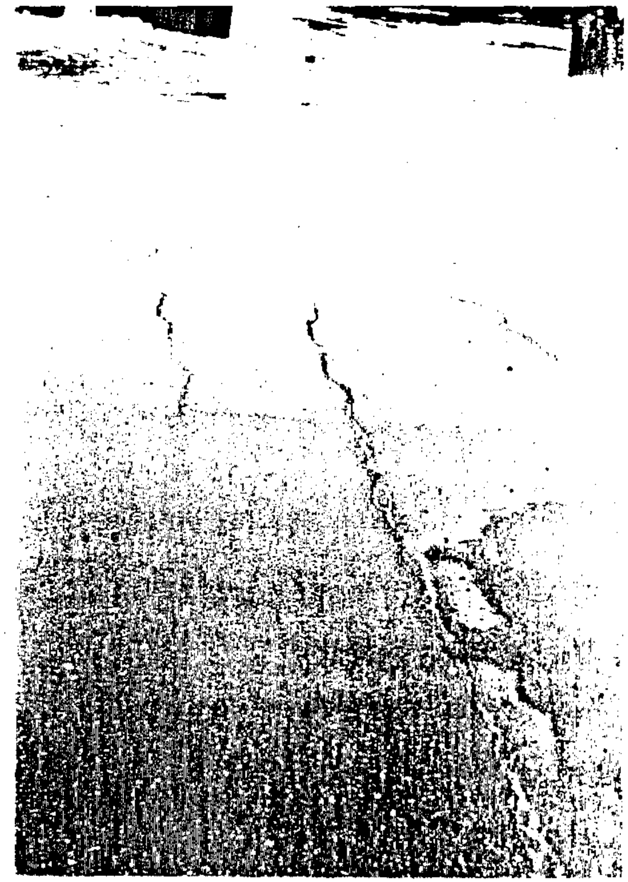

Figure 4.5 Pavement Surface Distress, Seward Highway.

to 5.25 in. (5 - 13 cm). The remainder of the pavement structure had typical dimensions of 6 in. (15 cm) base and an 18 in. (46 cm) subbase (38 cm) below the surface during the critical period. From the modified Berggren equation, the thaw depth was approximately 31 in. (79 cm). However, the assumptions made in the analysis could have been inaccurate due to a lack of sufficient climatoligical and pavement data. For purposes of analysis, the thaw depth was assumed to be 15 in. (38 cm) as ADOT&PF field estimates was in this region.

The section between 36th and Benson was selected for analysis. This pavement was constructed in 1969. It is conservatively estimated that 4.4 million EALs have traveled over it since then. The predicted 10 year EAL is expected to reach 5,083,000. This project is expected to be overlaid in the summer of 1987.

4.1.3 Parks Highway, Fairbanks

Parks Highway is a major arterial and serves as the main highway route between Anchorage and Fairbanks in the interior of Alaska. As such, it carries substantial traffic volumes. Much of the oilfield and pipeline traffic related to the North Slope activities since 1972 has used this highway. The highway was constructed from 1968 to 1974. The high usage, especially the heavy loads, plus the advanced age of the pavement has caused severe rutting and alligator cracking in several areas. Figures 4.6 and 4.7 illustrate the pavement surface condition.

The project is approximately 35.6 miles (57 km) long and is split into two sections. The north section begins 10 miles (16 km) west of Fairbanks near Ester and extends 24.4 miles (39 km) to the southwest. The south section begins 83 miles (134 km) further at Dragonfly Creek and ends 11.2 miles (18

Figure 4.6 Permanent Deformation, Parks Highway.

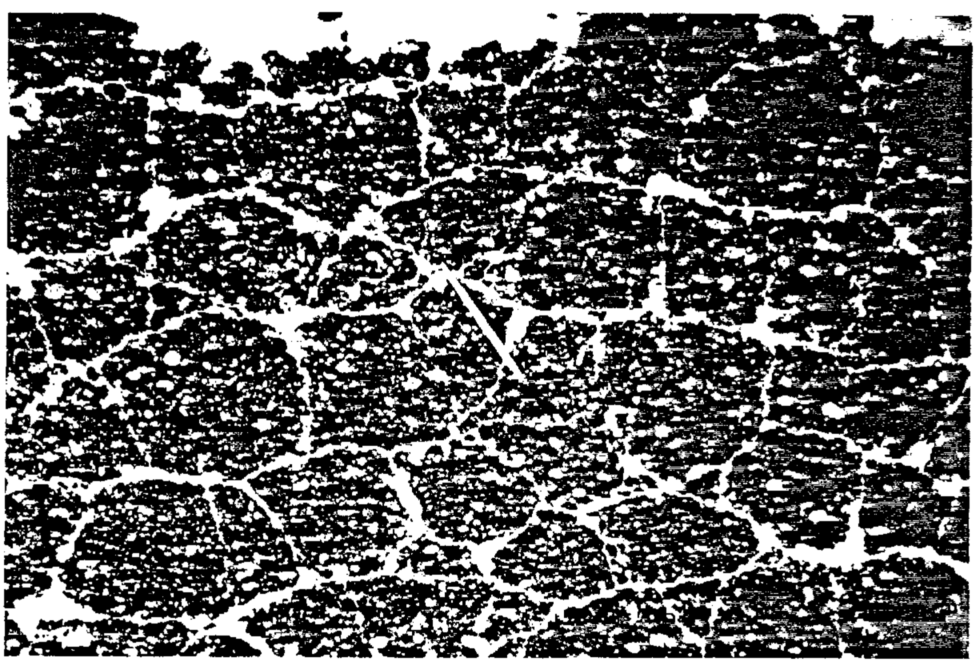

Figure 4.7 Alligator Cracking, Parks Highway.

km) later at McKinley Village. Figure 4.8 shows the location of the two sections. For design, the predicted 20 year EALs are 390,521 and 345,526 for the north and south sections, respectively. The design deflections used for this project ranged from 66 to 92 mils (1680 - 2340 μm). The typical pavement structure consists of a 2 in. (5 cm) asphalt concrete layer, a 4 to 6 in. (10 to 15 cm) base, a 6 to 12 in. (15 to 30 cm) subbase and 24 to 36 in. (61 to 91 cm) of select material. At the critical season, the thaw depth is approximately one to two feet (30 to 61 cm) below the surface. The modified Berggren equation estimates the thaw depth to be approximately 20 in. (51 cm). For this analysis, 15 in. (38 cm) was used.

4.2 Results of Analysis

Essentially, four overlay design procedures are evaluated in this section. All four methods have been previously discussed in Chapters 2 and 3. The first was the Asphalt Institute MS-17 (1983) procedure, followed by the regression equations developed by Newcomb (1986) in the state of Washington. Following these are the strain versus deflection relationships developed by Fernando et al. (1986) at Pennsylvania and lastly, two backcalculation programs, ELSDEF and BISDEF, were used to determine the layer moduli of the pavements.

The results of the analyses are summarized in Tables 4.2 to 4.8. Also, Appendix C presents in detail the parameters and inputs used for all procedures, and discusses the assumptions made. The intermediate steps of the analyses are shown in Tables C.1 to C.16, and worked examples are illustrated. This section presents only the end results of the analyses.

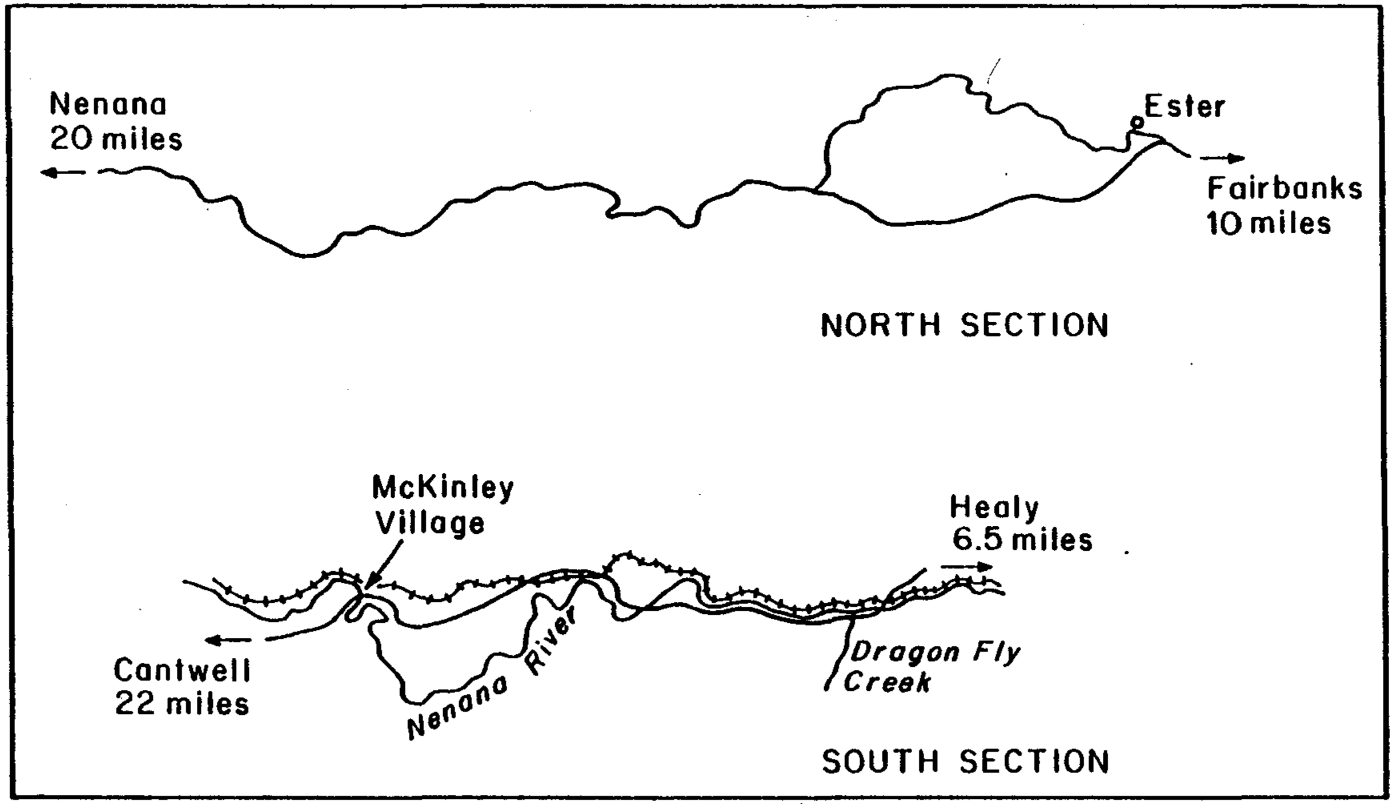

Figure 4.8 Parks Highway Project Site, Fairbanks.

4.2.1 Sterling Highway

Table 4.2 compares the layer moduli that were calculated from three methods, the computer programs BISDEF and ELSDEF and also that from Newcomb's regression equations. It is readily obvious that the BISDEF results show values that were input as the minimum and maximum ranges, particularly for E_1 where the maximum modulus was 2000 ksi (13,780 MPa) and the minimum was 80 ksi (551 MPa). Further, the author encountered various problems with this program, including an error message that indicated that the computations had been suspended due to a lack of Gaussian polynomials. However, despite this message, the final moduli values were still output. This occurred in more than half of the deflection basins that were analyzed. In a conversation with Don Alexander of the Waterways Experiment Station, it was indicated that the shape of the deflection basin and the presence of frozen layers could have been contributing factors. Of the remaining basins that did not have this error message, the 10% difference in calculated and assumed moduli were usually not met. Therefore, the results from this program may not be reliable. Additional information is needed before further analysis can be performed. In any case, further analysis was not carried out after this point.

The layer moduli backcalculated from ELSDEF appeared more reasonable. Although up to 9 iterations were needed to achieve a 10% tolerance, the majority of the pavement sections reached tolerance within 3 iterations. This translates to an average of 20 minutes of computer time per section on an IBM-compatible XT, with an 8087 mathematics coprocessor chip. In over half of the pavements, the subgrade shows substantial stiffness ranging from 150 to 650 ksi (1030 to 4480 MPa), indicating the presence of a frozen to partially

Table 4.2. Comparison of Layer Moduli[1] for Sterling Highway.

Location[2] MP	BISDEF[3]			ELSDEF			NEWCOMB[4]		
	E1	E2	E3	E1	E2	E3	E1	E2	E3
117.5	1688	93	125	399	71	120	289	136	148
118.0	1719	36	103	919	33	76	112	119	146
118.5	657	51	93	304	37	91	97	115	144
119.0	2000	34	346	971	41	165	-	-	-[5]
119.5	398	42	189	274	37	161	98	115	143
120.0	2000	13	81	107	31	60	31	240	567
120.5	80	27	1000	332	27	372	42	123	188
121.0	80	24	916	969	21	285	37	158	286
121.5	80	91	311	297	47	647	75	131	186
122.0	80	29	898	956	19	229	62	94	116
122.5	188	92	1000	379	150	100	28	232	550
123.0	1398	16	76	1184	15	41	53	92	116
123.5	317	29	705	356	31	428	73	130	183
124.0	80	18	945	1000	20	294	38	162	296
124.5	80	37	1000	363	30	415	31	152	281
125.0	80	14	656	899	29	374	87	118	152
125.5	80	29	768	324	25	347	58	105	138
126.0	2000	49	102	866	60	56	131	187	285
127.0	2000	12	1000	1263	17	41	55	219	584
127.5	80	25	1000	460	28	390	55	134	205
128.0	2000	32	293	1029	30	165	48	164	290
128.5	2000	120	209	331	67	335	165	149	191
129.0	107	24	186	948	20	192	50	128	194
129.5	80	65	1000	190	150	1000	23	153	305

[1]All moduli values are in ksi (1 ksi = 6.89 MPa).
[2]One section (MP 126.5 was not included because the moduli values were not found within 9 iterations.
[3]BISDEF values die not converge due to difficulties within the program. The results included here are not reliable, and are only for general comparison purposes. Most of the moduli are at the extremes of the ranges specified.
[4]Newcomb's regression equations specifically do not consider the effects of a frozen base and subbase, whereas these sections are partially frozen.
[5]Moduli values were not calculated due to a zero deflection reading.

frozen layer. Over half the sections have an asphalt concrete layer moduli in the 400 to 1200 ksi (2750 to 8270 MPa) range, which would be in accordance with pavements that were tested in March when temperatures were 37°F (2.8°C) (See Table C.3). For the remaining sections, temperatures averaged 60°F (15.6°C), and the moduli are correspondingly lower, in the 200 to 350 ksi (1380 to 2400 MPa) range. The trend is the lower the pavement temperature, the higher the stiffnesses. The base moduli tend to be on the low side, averaging 40 ksi (275 MPa). Recent research (Johnson & Hicks, 1987) on aggregate bases for this region has indicated that a base modulus of 100 ksi (690 MPa) would be closer to the mark.

The layer moduli obtained from Newcomb's equations are substantially different from that from ELSDEF. As Table 4.2 shows, E_1 values tend to be lower, in the 30 to 100 ksi (210 to 690 MPa) range. In contrast, the base moduli are much higher, almost all of which are in the 100 ksi (690 MPa) region. However, the subgrade moduli (E_3) values are similar for each method. Since the subgrade modulus is inversely proportional to the value of the outermost load sensor, very small deflections would give high modulus values, which is a reasonable assumption. However, if the readings are zero, as can occur when the pavements are frozen, the modulus would then be infinite. The other two moduli (E_1 and E_2) are additionally dependent on the ratio of the layer thicknesses and the subgrade modulus. More importantly, the "area" of the deflection basin (actually a measure of the basin shape), and the deflection at the center of the load play a larger role in determining these values. It should be noted that Newcomb's equations were not designed for pavements with frozen layers. This is explicitly stated in his dissertation. Also, the models were built with the assumption that $E_1 \geq E_2 \geq E_3$, and this is not true for the sections under review here, particularly when the frozen subgrade is typically of a magnitude up to 10 times greater in value. These two points probably explain much, if not all, of the differences between these moduli and those calculated from ELSDEF.

Fernando's equations and the Asphalt Institute procedure do not give layer moduli values and are therefore not included in Table 4.2.

Table 4.3 compares the existing pavement tensile strains calculated from 3 procedures, from Newcomb, Fernando and the mechanistic (ELSDEF) method. These tensile strains are measured at the base of the asphalt concrete layer, and the program ELSYM5 was used for the computations. Note that the strains from Newcomb's equations are much smaller than the other two methods, and this is expected since the base moduli values were so much greater than that found with the mechanistic method. The strains for the other two appear to compare well with each other for the majority of the sections. However, there are three exceptions, for MP 121.5, 122.5, and 129.5. This could be due to the fact that the deflection basins for these locations have readings of zero or near zero for intermediate sensors while the outermost sensors have higher readings. Figure 4.9 illustrates such a deflection basin; however, other locations also show similar shapes, and yet do not exhibit such a great difference in the strains.

The tensile strains lead to a determination of the overlay thicknesses as shown in Table 4.4. Here, the overlays for all 4 procedures are listed. As can be seen, Newcomb's equations leads one to believe that no overlays are required, given the very small strains that were obtained. The Asphalt Institute procedure recommends the next smallest overlay of 2 in. (5 cm) for this project. The overlays from Fernando's equations range anywhere from 2

Table 4.3. Comparison of Tensile Strains[1] for Sterling Highway.

Location MP	Tensile Strain (x 10^{-6}) Newcomb	Fernando	Mechanistic[3]
117.5	29.5	150	106
118.0	6.0	249	265
118.5	3.0	281	231
119.0	-[2]	268	220
119.5	3.1	280	231
120.0	9.3	236	270
120.5	14.5	407	344
121.0	13.2	340	371
121.5	4.8	436	179
122.0	6.0	423	394
122.5	9.5	365	38.8
123.0	10.3	462	434
123.5	5.4	272	294
124.0	13.2	328	385
124.5	14.5	431	308
125.0	0.6	320	295
125.5	8.8	384	365
126.0	0.9	147	148
127.0	5.3	392	395
127.5	11.0	352	327
128.0	10.8	288	281
128.5	9.7	230	114
129.0	12.0	397	308
129.5	17.1	886	246

NB: The minimum overlay thickness is 1 inch, and values are rounded up to the nearest 0.5 inch. (1 inch = 2.54 cm)

[1]Tensile strain is measured at the bottom of the asphalt layer.

[2]There is no value because Newcomb's equations cannot deal with zero deflections.

[3]The mechanistic method uses the results of the ELSDEF and ELSYM5 programs.

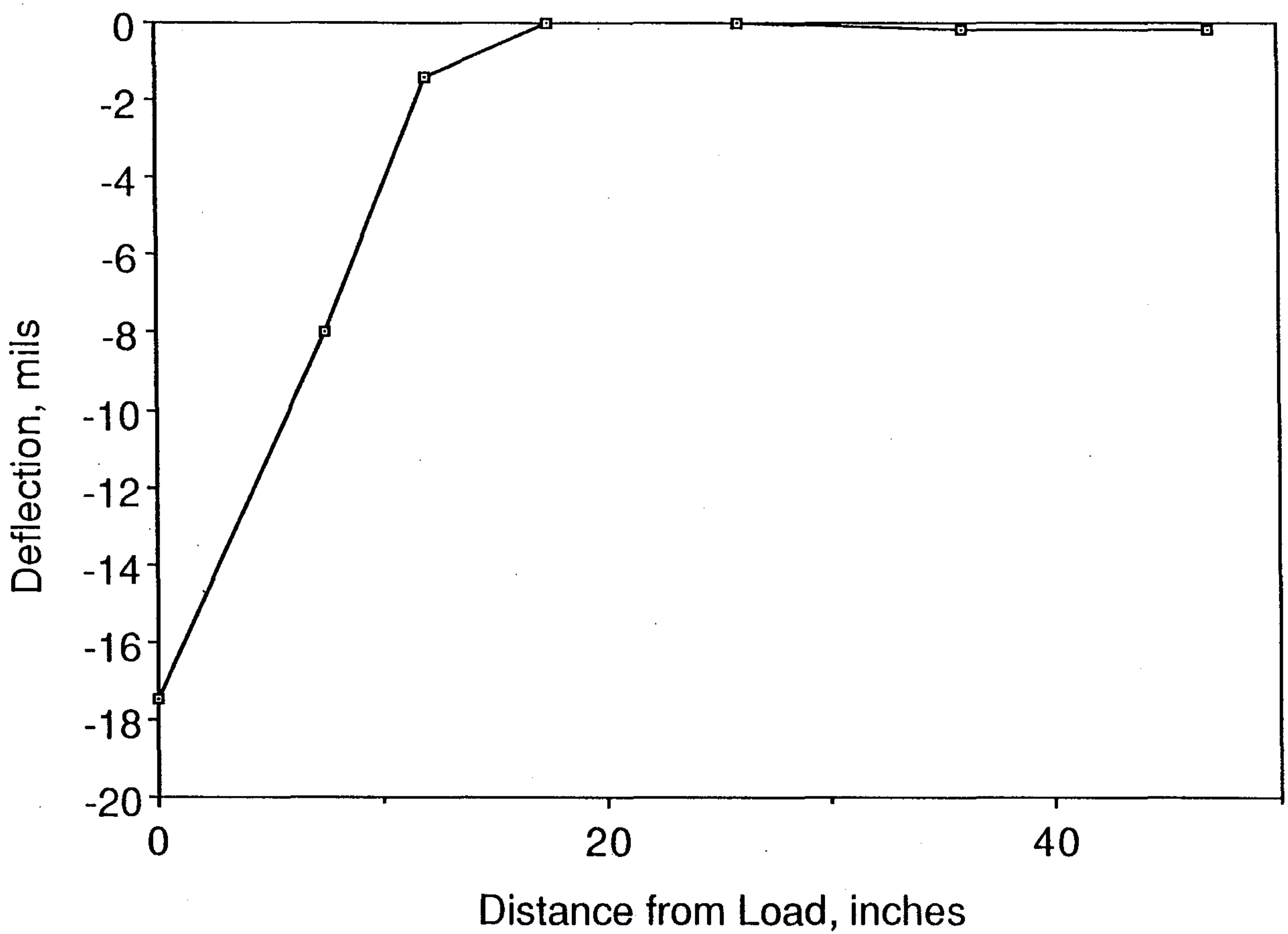

Figure 4.9 Deflection Basin for MP 129.5, Sterling Highway.

Table 4.4. Comparison of Overlay Thicknesses for Sterling Highway.

Location[1] MP	TAI[2]	Overlay Thicknesses (in.) Newcomb[5]	Fernando	Mechanistic[3] 50%[4]	90%
117.5	The maximum representative rebound deflection for this section is 40 mils.	0	2.5	0.0	0.0
118.0		0	5.0	4.0	6.5
118.5		0	5.5	5.0	8.0
119.0		-	5.5	-	-
119.5	The 20-year EAL is 1,800,000.	0	5.5	5.5	8.0
120.0		0	5.0	4.5	6.5
120.5	The overlay thickness required is 2.0 inches.	0	8.0	6.5	>8.0
121.0		0	7.0	-	-[6]
121.5		0	8.0	4.0	7.0
122.0		0	8.0	-	-
122.5		0	7.0	1.0	1.0
123.0		0	>8.0	-	-
123.5		0	5.5	5.5	8.0
124.0		0	6.5	-	-
124.5		0	8.0	5.5	8.0
125.0		0	6.5	4.5	7.0
125.5		0	7.5	6.5	>8.0
126.0		0	2.5	2.5	5.0
127.0		0	7.5	-	-
127.5		0	7.0	5.5	8.0
128.0		0	5.5	4.5	6.5
128.5		0	5.0	2.0	5.5
129.0		0	7.5	-	-
129.5		0	>8.0	0.0	0.0

NB: The minimum overlay thickness is 1 inch, and values are rounded up to the nearest 0.5 inch. (1 inch = 2.54 cm)

[1]MP 126.5 is not included because the moduli did not reach tolerance after 9 iterations.

[2]This is the Asphalt Institute Procedure.

[3]The mechanistic method uses the results of the ELSDEF and ELSYM5 programs.

[4]These are the 50% and 90% reliability levels as defined by AASHTO.

[5]This is for both 50% and 90% reliability levels.

[6]These sections had a negative remaining life so overlay thicknesses could not be computed.

in. (5 cm) to more than 8 in. (20 cm) and has an average of 6 in. (15 cm). When comparing this with the overlays from the mechanistic method, there is clearly a trend. The overlays from Fernando's equations are generally at least as thick as or thicker than those designed by the mechanistic method. Figure 4.10 illustrates this trend in a bar chart.

Note also that the overlays for the 90% reliability level (see Appendix E for more on reliability) are consistently higher than those for the 50% reliability level. For a 90% reliability level, a reliability design factor, F_R, was found to be 4.25. On average, the overlays tend to be a little over 2 in. (5 cm) thicker than for the 50% level.

4.2.2 Seward Highway

Table 4.5 compares the layer moduli obtained from three methods. Again, the results from BISDEF appear unreliable due to operational errors in the program. Note that the values for locations TH3 and TH34 did not achieve the specified tolerance even after 9 iterations with ELSDEF. Further iterations were not performed because the author felt that they would probably not have helped. Also, time constraints played a role in the decision as it takes substantially more computer time for more than three iterations.

The subgrade moduli from ELSDEF are extremely low when compared with that for Sterling Highway. From Table C.8 in Appendix C, it can be seen that the deflections are substantially higher and the pavement temperatures higher (53°F). This would seem to indicate that the sections are unfrozen. This could be true since this portion of Seward Highway lies close to downtown Anchorage and sees much higher traffic volumes. The frequent loadings could have increased the air temperature over the pavement traffic and increased the likelihood that the pavement is unfrozen. Except for TH1, E_1 values are in

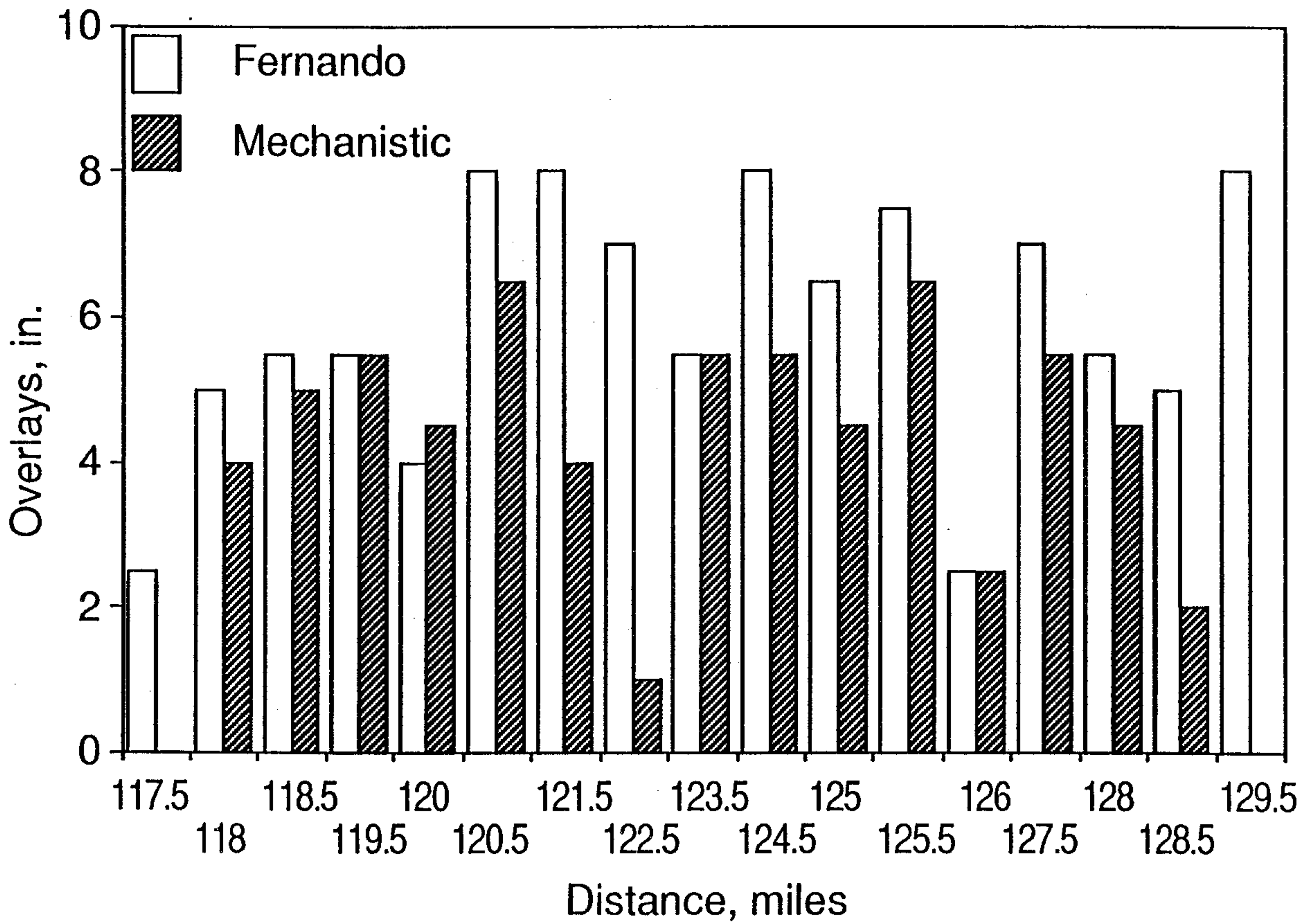

Figure 4.10. Comparison of Overlays Calculated from Fernando's Equations and the Mechanistic Method for Sterling Highway.

Table 4.5. Comparison of Layer Moduli[1] for Seward Highway.

Location[2] MP	BISDEF[3]			ELSDEF			NEWCOMB[4]		
	E1	E2	E3	E1	E2	E3	E1	E2	E3
TH 1	2000	46	29	1038	52	24	247	88	78
TH 2	2000	52	16	634	62	9	307	76	54
TH 3	2000	16	47	1892	21	12	198	79	74
TH 4	1488	37	14	438	39	10	252	58	40
TH 45	570	26	12	485	21	11	131	46	34
TH 34	2000	32	14	1269	40	6	359	64	44
TH 35	1522	42	24	620	44	15	169	75	65

[1]All moduli values are in ksi (1 ksi = 6.89 MPa).
[2]TH 3 and TH 34 are included even though the moduli values were not found within 9 iterations.
[3]BISDEF values did not converge due to difficulties within the program. The results included here are not reliable, and are only for general comparison purposes. The values shown also tend to be in the extreme ranges.

the 400 to 600 ksi (2760 to 4130 MPa) range. The base moduli are a little higher than that for Sterling Highway and seem to be more realistic.

The layer moduli from Newcomb's equations show much higher subgrade values, although still not as high as those for Sterling Highway. This reflects the larger deflection readings. This time, the base moduli agree fairly well with the ones from ELSDEF, although the E_1 values tend to be smaller.

Table 4.6 summarizes the tensile strains and overlay thicknesses. The tensile strains appear to be similar for all three procedures although there is a tendency for those calculated by the mechanistic method to be greater. With regard to overlay thicknesses, the Asphalt Institute recommends overlays of 2 and 3 in. (5 and 7.6 cm) for the 50% and 90% reliability levels, respectively. Fernando's equations recommend higher overlays, from 2 to 6 in. (5 to 15 cm).

The mechanistic method indicates a different outcome. Because of the high traffic this highway has carried (conservatively estimated at 4.4 million EALs) to date, the fatigue life remaining to the pavement is negative (see Appendix C for detailed calculations). This implies that the traffic has far exceeded the fatigue life of the pavement, and regardless of the thickness of the overlay applied, the fatigue life of the pavement cannot be restored. Unlike permanent deformation, an overlay will not solve the fatigue problem. Therefore, with the mechanistic method, a complete reconstruction of this section is recommended based on these results. This is true regardless of the reliability level. The results from Newcomb's equations, however, are not as conservative. They reflect the extremes in the range of overlay thicknesses and while it indicates that an overlay of 6 to 8 in. (15 to 20 cm) or more is

Table 4.6. Comparison of Tensile Strain and Overlays for Seward Highway.

	Tensile Strain[1] (x 10^{-6})				Overlay Thicknesses (in.)			
							Newcomb	
Location	Fernando	Newcomb	Mechanistic[2]	TAI[3]	Fernando	Mechanistic[4]	50%[5]	90%
TH 1	130	116	196	RRD = 26 Overlay is 2 in. (50%) & 3 in. (90%)	3.5	Reconstruction is recommended for this section.	0	8
TH 2	111	127	172		2.0		6	>8
TH 3	158	175	-		4.0		Reconstruction	
TH 4	172	190	278		5.0		Reconstruction	
TH 45	250	298	354		6.0		Reconstruction	
TH 34	133	177	-		4.5		Reconstruction	
TH 35	159	145	241		4.0		6	>8

NB: The minimum overlay thickness is 1 inch, and values are rounded up to the nearest 0.5 inch. (1 inch = 2.54 cm)

[1]Tensile strain is measured at the bottom of the asphalt layer.

[2]The mechanistic method uses the results of the ELSDEF and ELSYM5 programs. The moduli values of TH 3 and TH 34 did not reach tolerance after 9 iterations and so strains were not calculated.

[3]This is the Asphalt Institute Procedure.

[4]This is for both the 50% and 90% reliability levels.

[5]These are the reliability levels as defined by AASHTO. Overlays could not be found for some sections because of the negative remaining life. (Appendix C).

needed for three sections, it also indicates that none is needed for TH1. Because four sections also had a negative remaining life, no overlays could be calculated. Instead, reconstruction is required. In general, it could be said that Newcomb's results tend to agree with the mechanistic method, in that either very thick overlays or reconstruction are needed.

4.2.3 Parks Highway

Table 4.7 compares the layer moduli, and again, the results from BISDEF do not appear reliable and were not analyzed any further. ELSDEF shows base moduli in the range of 20 to 75 ksi (140 to 510 MPa) which is reasonable for an unfrozen base. The subgrade moduli, on the other hand, have a much greater range, indicating both frozen and unfrozen sections. It is fairly obvious which sections are frozen. The E_1 values also tend to be in the 200 to 500 ksi (1380 to 3440 MPa) range which also appears reasonable.

Newcomb's equations, however, illustrate dramatic differences. The asphalt concrete and base moduli are very low, and in some cases, unreasonably so. For example, CDS 206.2 has a surface layer modulus of only 5,000 psi (34 kPa). The base moduli compare reasonably well with the mechanistic method, but the subgrade moduli are substantially higher. Overall, there is a tendency for the moduli to increase with depth. This could be due to the fact that the outermost sensors have deflections on the order of 0.2 mils (5 μm) while the center deflections are almost 16 to 25 mils (400 to 635 μm). Figure 4.11 illustrates sample deflection basins. Therefore, this deflection basin seems to indicate that there is a very stiff subgrade while the top layer is weak. In any case, the computed moduli presents a problem. Due to the weak asphalt concrete moduli, it turns out that an increase in thickness of this

Table 4.7. Comparison of Layer Moduli[1] for Parks Highway.

Location[2] CDS	BISDEF[3] E1	BISDEF[3] E2	BISDEF[3] E3	ELSDEF E1	ELSDEF E2	ELSDEF E3	NEWCOMB E1	NEWCOMB E2	NEWCOMB E3
293	513	42	101	587	22	223	35	23	240
293.2	198	57	99	520	31	116	62	19	160
293.4	992	23	40	1245	16	36	33	16	138
293.5	1812	17	39	1451	17	24	32	17	160
293.6	1431	29	33	645	20	31	61	14	97
304.2	2000	96	59	653	67	54	211	23	164
304.4	307	117	274	521	74	267	63	40	509
304.6	80	128	92	451	75	1000	120	19	147
304.8	100	80	389	460	43	603	28	25	301
206	2000	18	1000	422	24	17	18	22	240
206.2	286	25	1000	383	24	45	5	37	684
206.4	1035	24	171	381	26	32	15	21	225
206.6	1194	21	552	352	23	31	14	21	233
206.8	554	25	89	243	20	203	23	21	225
198	1368	31	185	243	22	660	7	40	730
198.2	2000	66	1000	254	44	401	13	42	695
198.4	2000	30	158	351	44	18	62	22	183
198.6	972	36	44	286	34	29	75	14	88
198.8	340	33	50	239	22	51	31	16	138

[1]All moduli values are in ksi (1 ksi = 6.89 MPa)
[2]Three sections (CDS 293.8, 304, and 304.5) are not included because the moduli values were not found within 9 iterations.
[3]BISDEF values did not converge due to difficulties within the program. The results included here are not reliable, and are only for general comparison purposes.

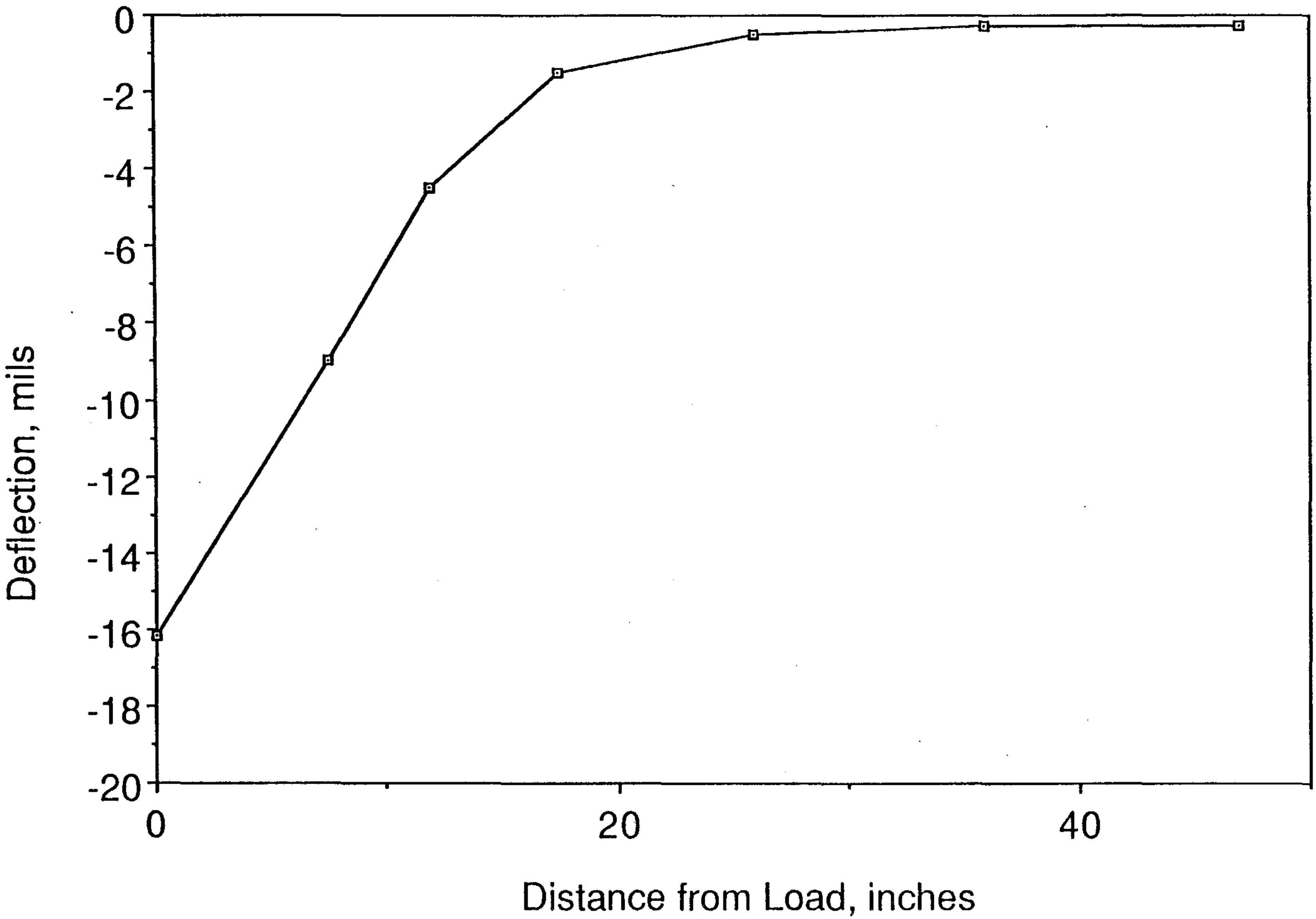

Figure 4.11 Deflection Basin for CDS 293, Parks Highway.

layer actually INCREASES the tensile strain instead of decreasing it. Therefore, the overlay thicknesses could not be found.

From the Asphalt Institute procedure (Table 4.8), overlays of 1 and 2.5 in. (2.5 to 6.3 cm) are needed for the north section, and none for the south section. However, both Fernando's and the mechanistic method result in higher overlays, by as much as 6 in. (15 cm). Again, the general trend is that Fernando produces equal or greater overlays than the mechanistic method except for a few exceptions. Figure 4.12 compares the overlays produced from the two procedures. The higher reliability level is reflected by a thicker overlay of approximately 2 in. (5 cm).

4.3 Discussion of Results

From the preceding sections, there do not seem to be any clear trends or conclusions that can be drawn. Overall, it is clear that the use of the program BISDEF was not very successful. The author speculates that this may be due to the presence of frozen layers, the presence of zero deflections or a combination of both. It would seem unlikely that the copies of the program the author used were all faulty or damaged, as a total of 6 copies were tried with the same results. Further, an updated version of the program was requested from the Waterways Experiment Station as recently as April 1987, and the same results occurred.

On the subject of backcalculation programs, ELSDEF appears to give more reasonable values. Still, there are some anomalies. In the case of Seward Highway, more sections should probably have been analyzed to obtain a better sample of the population. Base moduli seem to be on the low side, but the program seems to handle the presence of a frozen layer well.

Table 4.8. Comparison of Tensile Strain and Overlays for Parks Highway.

Location CDS	Tensile Strain[1] (x 10^{-6}) Fernando	Newcomb	Mechanistic[2]	Overlay Thicknesses (in.) TAI[3]	Fernando	Mechanistic 50%[4]	Mechanistic 90%
293	330	371	419	RRD = 52	4.5	4.5	6.5
293.2	303	547	327	Overlay is 1.0	4.0	2.0	4.5
293.4	375	583	364		5.0	8.0	8.5
293.5	372	536	332		5.0	3.5	6.0
293.6	363	774	4221		5.0	6.0	8.5
304.2	137	481	160	RRD = 78	1.0	1.0	2.0
304.4	223	217	150	Overlay is 2.5	2.5	1.0	1.0
304.6	512	587	150		7.0	1.0	1.0
304.8	565	308	255		7.5	2.0	4.0
206	484	205	416	RRD = 26	6.5	4.0	6.5
206.2	508	10.4	430	Overlay is 0.	6.5	4.5	6.5
206.4	511	191	405		6.5	3.5	6.5
206.6	538	180	440		7.0	4.5	7.0
206.8	386	362	502		5.0	7.0	>8.0
198	440	10.4	502	RRD = 29	6.0	6.0	8.5
198.2	290	36.7	246	Overlay is 0.	3.5	1.0	4.5
198.4	277	413	239		3.5	1.0	3.5
198.6	336	756	472		4.5	4.0	6.5
198.8	470	572	481		6.5	5.0	8.0

NB: The minimum overlay thickness is 1 inch, and values are rounded up to the nearest 0.5 inch. (1 inch = 2.54 cm)

Note also that overlays for Newcomb's method were not included because they were either zero or could not be calculated due to inaccurate layer moduli.

[1]Tensile strain is measured at the bottom of the asphalt layer.

[2]The mechanistic method uses the results of the ELSDEF and ELSYM5 programs.

[3]This is the Asphalt Institute Procedure.

[4]These are the 50% and 90% reliability levels as defined by AASHTO.

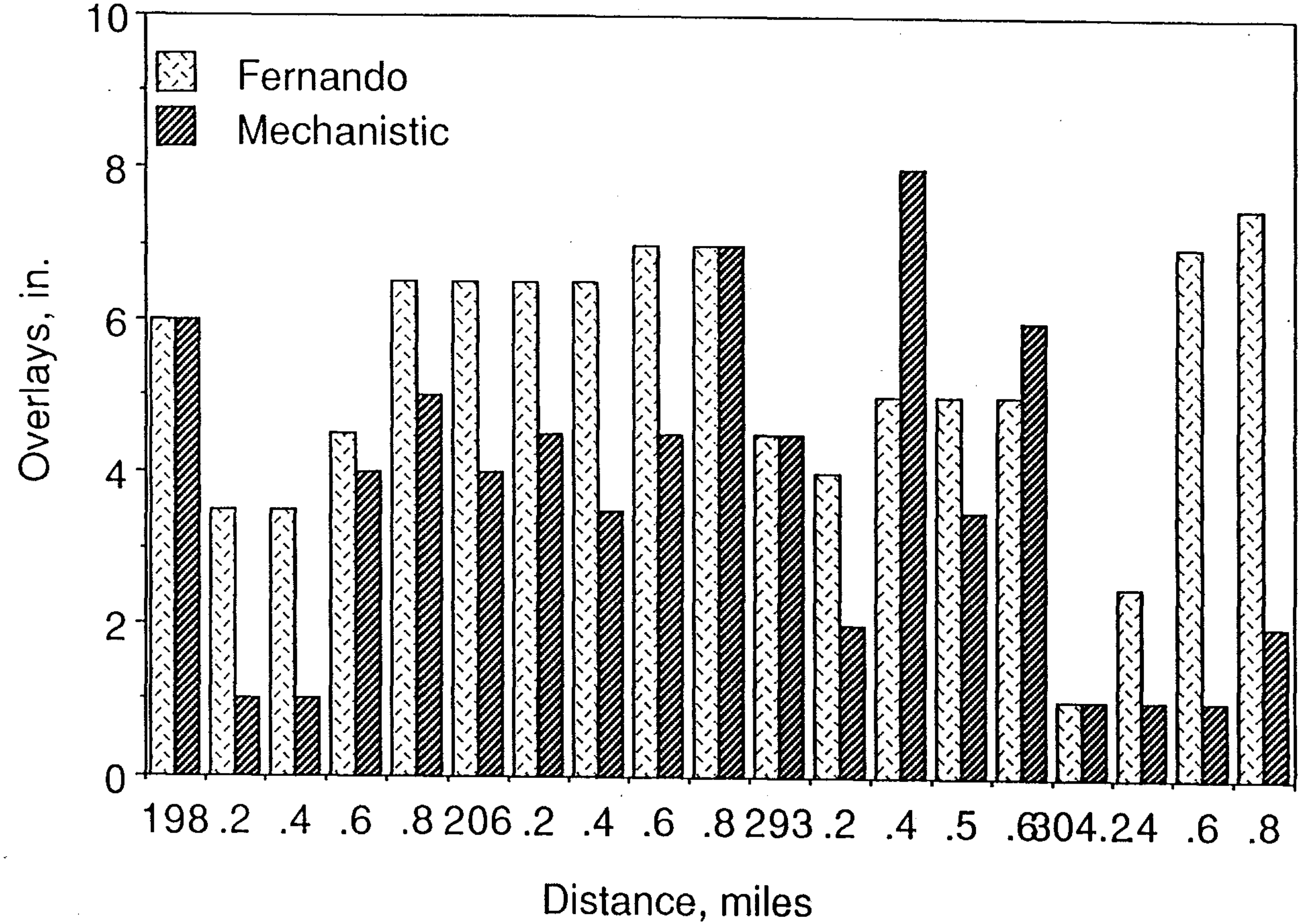

Figure 4.12 Comparison of Overlays Calculated from Fernando's Equations and the Mechanistic Method for Parks Highway.

Newcomb's equations are not recommended at all for Alaskan conditions. It should be noted they were specifically developed for non-frozen pavements and probably should not have been included in this report. For Sterling Highway, Newcomb's equations indicated no overlays were needed which was completely opposite to the results of the other procedures. For Seward Highway, either 8 in. (21 cm) of overlay or reconstruction was called for, which agrees reasonably well with the mechanistic method. However, for Parks Highway, it recommended the asphalt concrete layer be removed to strengthen the pavement. This is an unreasonable solution but which makes sense given the layer moduli it calculated. From the three projects, a zig-zag of results are obtained, and there is no consistency. Also, the presence of a frozen layer and a very thick base and subbase result in shallower deflection basins, and the outermost sensors should probably be extended further than 48 in. (122 cm). If equations were developed for deflections beyond this point, the layer moduli may be more reasonable.

Fernando's method suffers when the pavement has used up its fatigue life. Without being able to consider this in the equations, it determines that overlays are sufficient for Seward Highway although the pavement fatigue life is used up. For the other projects, it generally appears that this procedure produces overlays at least equal to or greater than those determined by the mechanistic method in the majority of cases. The presence of the frozen layers generally does not appear to distort the results from the regression equations. Frozen sites were probably considered in the original factorial study in Pennsylvania. However, the conservative tone of the overlay designs should prove sufficient for Alaska to develop their own equations.

In the case of the reliability levels, the 90% level increases the traffic by a factor of 4.25 for flexible pavements, and this is reflected in the design by a consistent increase of approximately 2 in. (5 cm) for the overlay.

The results of the Asphalt Institute procedure confirms that this procedure is inadequate and should not be used in Alaska. The lack of consideration for remaining life or the frozen layers are obvious in the results. The overlay thicknesses obtained are either significantly lower than the other procedures or indicate no need for an overlay.

The fatigue relationships used here are those from the Asphalt Institute, and the tensile strain at the base of the asphalt concrete layer was used. Other criteria and relationships are available. The use of the compressive subgrade strain is one. There are various performance relationships that have been developed by agencies and individuals that are also available. As with all models, they may not be readily transferable outside the conditions for which they were developed. The ideal situation would be for Alaska to develop their own.

Before concluding, the author would also like to comment on the present procedure of using center deflections to determine the critical section for analysis. Figures 4.13 and 4.14 graph both the overlay thicknesses (for the 50% reliability level) and center deflections at the same location for Parks Highway and Sterling Highway, respectively. They show little, if any, coherent relationship between the center deflection (corrected to 70^G#^GF and normalized to a 9000 lb load) and the overlay thickness (from the mechanistic method). On the other hand, Figure 4.15 indicates a clearer relationship between the overlays and the tensile strains. This again emphasizes the point

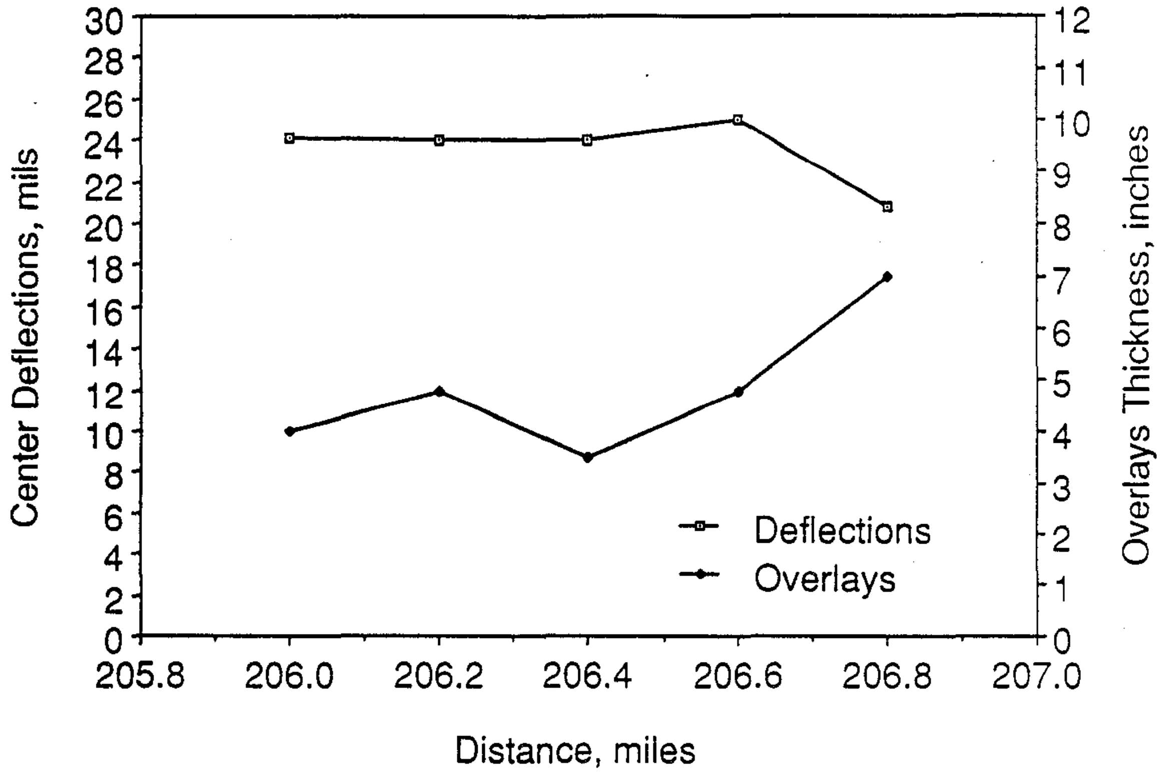

Figure 4.13 Comparison Between Overlay Thickness and Center Deflections, Parks Highway

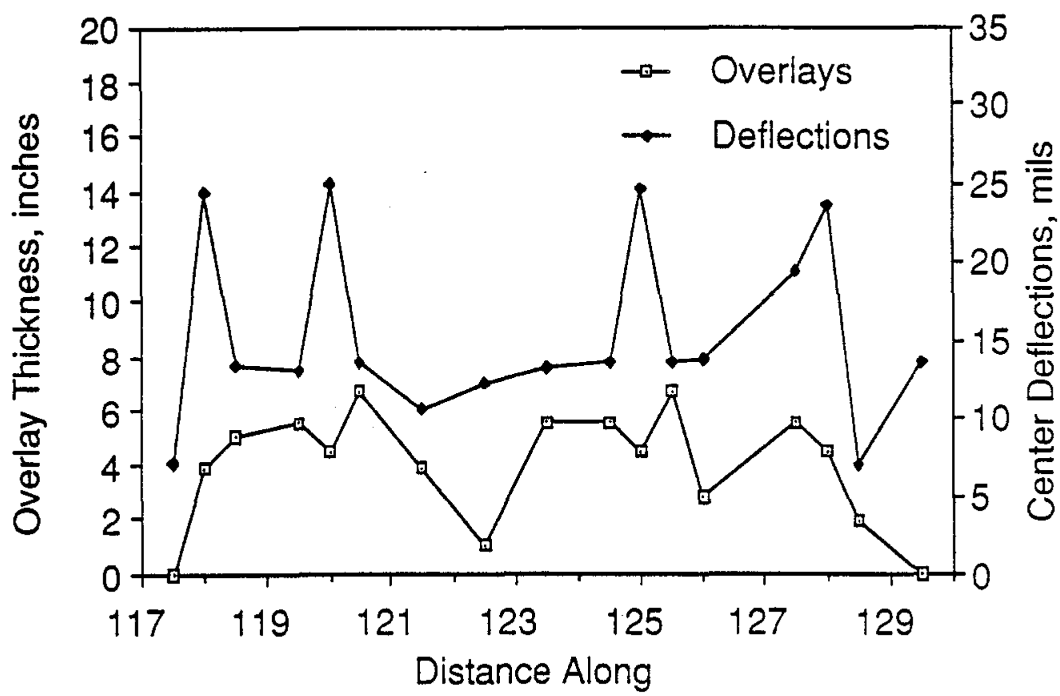

Figure 4.14 Comparison Between Overlay Thickness and Center Deflection, Sterling Highway.

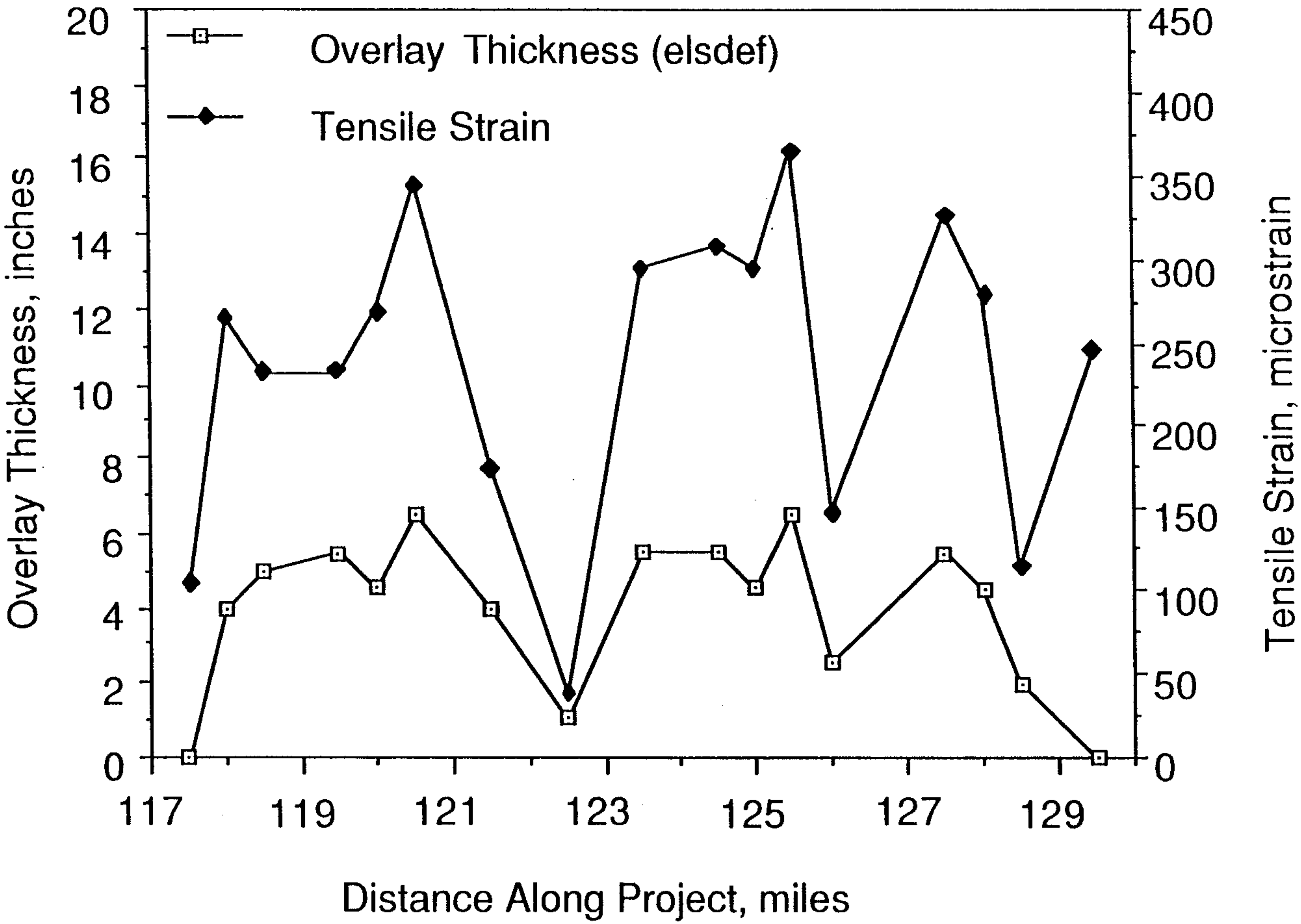

Figure 4.15 Relationship Between Overlay Thickness and Tensile Strain for Sterling Highway

that center deflections should not be used as a criterion for selection of an analysis section.

As a final note, the lack of accurate data on the historical traffic EALs and the pavement layer thicknesses proved to be a major obstacle in the analyses. Many assumptions were made which may or may not be correct in the calculations, and the results presented herein reflect the accuracy of those assumptions. It cannot be emphasized strongly enough that an accurate knowledge of the traffic and layer thicknesses are very important. The thickness of the asphalt concrete layer, in particular, affects markedly the backcalculated results (Rwebangira et al., 1987), and an inaccurate traffic counts serve to disguise the actual remaining life of the pavement.

4.4 Summary

This chapter has presented the results of the analyses performed on three project sites, two in Anchorage and one in Fairbanks. Four overlay design methods were applied, the Asphalt Institute, Fernando et al. (1986), Newcomb (1986) and a mechanistic procedure using ELSDEF and BISDEF. The details of the calculations are documented in Appendix C. This chapter only summarizes the highlights of the analyses.

From the discussions above, it can be said that three procedures, the Asphalt Institute, Newcomb's equations and the use of BISDEF would lead the engineer to errors in the design of the overlays. The layer moduli calculated from BISDEF do not appear reliable, and the other two procedures would under-design the pavements significantly. However, ELSDEF appears to yield reasonable results, and Fernando's procedure tends, if anything, to be on the conservative side.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Based on the findings of this study, the following conclusions appear warranted:

1. The present overlay design procedure used by the state of Alaska does not acknowledge the special problems that the climate presents in that region. It also does not utilize the concept of remaining life and tends to underdesign the pavement by specifying a thinner overlay than needed, if at all.
2. Four methods of overlay design were analyzed in this report. They included the Asphalt Institute procedure; Fernando et al.'s strain-deflection relationships developed in Pennsylvania; a mechanistic method employing two backcalculation programs, BISDEF and ELSDEF; and one set of regression equations developed in Washington State.
3. The tensile strain at the base of the asphalt concrete layer was the criterion used in determining the remaining life as fatigue appeared to be the predominant failure mode. The fatigue equation developed by the Asphalt Institute was also used. Other criteria such as the compressive subgrade strain and corresponding equations are available. However, the pavement structure designed in Alaska are usually so thick due to frost considerations that compressive subgrade strain is rarely a problem.

4. There were insufficient traffic and pavement structural data for the projects. This led to assumptions used in the analysis that may be inaccurate and as a result, the overlay thicknesses could be misleading. However, the data are representative of what is available for most design situations. Often, more accurate data simply are not available.
5. The BISDEF program did not always close because of apparent operational errors in the software; therefore, it was determined that the backcalculated moduli were not always reliable.
6. The ELSDEF program appeared to give the most reasonable results when compared with prior laboratory test results. However, additional work is needed to verify these backcalculated values for the specific projects evaluated.
7. The Asphalt Institute procedure appears inappropriate for use in Alaska in that overlays designed are very thin. This method indicates that overlays are not required for several sections despite contrary results from the other methods.
8. Fernando et al.'s equations appear to perform reasonably well compared with the mechanistic method; if anything, they are more conservative. However, it appears that when a pavement is badly fatigued, the procedure gives misleading results. On the other hand, there probably should be more sections analyzed where the remaining life is close to zero before a definitive conclusion can be reached.

9. The basis for selection of the critical section should not be the center deflection alone because the overlay thicknesses obtained are not related to the magnitude of these deflections. Instead, tensile strain is a better indicator, and the procedure discussed in this report is recommended.
10. For the projects evaluated, an increase in the AASHTO reliability levels from 50% to 90% generally increases the overlay thickness by approximately 2 in. (5 cm).

5.2 Recommendations for Implementation

From the conclusions discussed above, the following items are recommended for implementation:

1. The Asphalt Institute procedure should not be used for overlay design.
2. The mechanistic procedure using ELSDEF should be considered as a replacement for the Asphalt Institute procedure in the design of flexible overlays. For thin asphalt layers (< 2 inches), it is recommended that asphalt cores and aggregate samples be obtained and tested.
3. The tensile strain criteria and the Asphalt Institute fatigue relationships appear adequate at present. However, further analysis is needed before it can be decided if they are completely appropriate.
4. For a simpler procedure, Fernando's equations may be used. However, the results would be more conservative than that from ELSDEF.

5. More accurate traffic (EALs) data are needed, particularly historical data. Efforts should also be concentrated on collecting traffic data for future use and making growth projections. It is recommended that WIM and AVC units be left in the field for several weeks at different times of the year to obtain a better representation of the traffic profile.
6. Similarly, accurate layer thicknesses are needed, as this strongly influences the backcalculated moduli. If construction records have not been kept, then cores should be taken, approximately at a frequency of 5 per mile.
7. The 50% reliability level is recommended for design on roads with low volumes. For roads with significant traffic volumes or those which may expect a substantial increase in traffic, a 90% level may be more appropriate.

5.3 Recommendations for Research

The following items are recommended for further research:

1. The use of BISDEF as a backcalculation program should be further studied. The results from this report do not appear promising. Other programs such as MODCOMP2 should also be analyzed.
2. To better evaluate Fernando's procedure, more sections that are badly fatigued should be analyzed.
3. ADOT&PF should develop their own strain versus deflection relationships eventually rather than using Fernando's equations or a more cost-efficient overlay design method. The discussion for this procedure is in the text of this report.

4. Alaska should evaluate the suitability of the performance criteria developed by the Asphalt Institute that was used in this analysis. If not completely appropriate, they should consider developing their own performance criteria and equations that may be more appropriate for their conditions.

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APPENDIX A

LITERATURE REVIEW OF SELECTED BACKCALCULATION METHODS

LITERATURE REVIEW OF SELECTED BACKCALCULATION METHODS

1.0 INTRODUCTION

1.1 Problem Statement

The major problem facing highway engineers today is not the design and construction of new pavements, but the evaluation, maintenance and upgrading of existing pavement systems. This evaluation is necessary to meet today's demands for higher magnitudes of traffic loading and intensity. Therefore, there exists a need for a reliable, quick and nondestructive tool that permits the evaluation of pavements to obtain accurate information about existing structural conditions. The magnitude of stresses and strains induced in a pavement system are universally recognized as indicators of pavement performance. The most common method of determining these quantities is by laboratory characterization of materials sampled from the pavement section. This approach, however, suffers from many serious deficiencies. It is destructive to the pavement section, slow and expensive. Since sampling and testing for each site requires considerable effort, it is probable that only a few sites will be used to characterize several miles of roadway.

In lieu of stress and strain measurements in the field and laboratory, measurement of pavement surface deflection and curvature have been universally recognized as indicators of pavement performance. Numerous field and laboratory investigations have been performed to develop relationships between pavement performance and deflection (Hveem, 1955; Lister & Kennedy, 1977; Majidzadeh et al, 1976; Bergan & Monismith, 1972). In contrast to laboratory and field evaluation of pavement layer materials, pavement deflection testing is nondestructive and can be very rapid. Using deflection testing a through evaluation of the response of a given pavement can be obtained from closely spaced sites. Statistical techniques can be used to develop representative deflection values characterizing structural condition.

1.2 Purpose

The purpose of this appendix is to investigate and document the various methods that have been used in the nondestructive evaluation of pavements. It includes methods that have been developed to utilize NDT deflection data, especially those used to backcalculate layer moduli as well as those used in overlay design. Factors that affect deflection measurements are also included. Further, it looks at the correlations that have been established between the various NDT devices.

2.0 FACTORS AFFECTING DEFLECTION MEASUREMENTS

Several factors affect the deflection of a pavement system. These include season, temperature and equipment type. The effects of each of these factors are discussed in the following sections.

2.1 Season

The effect of season on deflection is mainly related to the changes in the moisture content of the pavement material. This change is noted most in the subgrade soil or granular base where an increase in moisture content results in the reduction of the modulus value. The amount of moisture or water content in a highly plastic soil can have a large effect on the deflection measured by impact type devices as compared to static load devices. This is dependent on the pore pressure that are created in the clay soils during impact. As the pore pressure accumulates,the effective stresses caused by the applied load decrease and the strength of the soil decreases. Even in unsaturated soils the increase in moisture content has been shown to result in a decrease in the resilient moduli. This reduction in moduli results in an increase in the amount of surface deflection.

2.2 Temperature

Most areas of the country experience significant changes of temperature throughout the year. A temperature increase tends to soften the asphalt concrete while a temperature decrease tends to stiffen asphalt concrete. This in turn affects deflection measured by NDT devices. Low temperatures that result in the freezing of the base and subgrade reduces the deflection value of the pavement. Most agencies have developed procedures for the correction of deflection measurements taken at any temperature to an equivalent maximum deflection at a standard temperature. Alaska uses the Asphalt Institute temperature correction charts at present. However, temperature correction factors should be developed for Alaska to consider the effects of frozen granular layers in the pavement structure. Appendix D discusses in greater detail the development of these correction factors.

2.3 Equipment Type

The magnitude of deflections measured on a pavement surface is influenced by the type of NDT device used. This results from various factors related to equipment type that affect the modulus of layer materials and include the following:

1. Stress-sensitivity of each material within the pavement structure and subgrade,
2. Load duration,
3. Contact pressure and area, and
4. Number of loads.

The stiffness of most fine-grained soils and granular base materials are functions of the confining pressure and vertical stresses imposed upon them. For soils that are highly stress sensitive, the stiffness can vary significantly under different loads. As a result, the change in deflection magnitude and basin may not be directly proportional to change in load.

The effect of loading time or load duration depends on the creep and viscoelastic characteristics of each material within the pavement structure. For purely elastic materials, load duration would not be a dependent variable. However, most pavement materials are viscoelastic and do creep under loads. Typically larger deflections occur in pavements built on clay soils that are normally consolidated with larger loading times. However, for many types of clay soils, the responses are essentially elastic under the short term dynamic loads that simulate wheel load duration and low deviator stresses typical of subgrade soils. For cases where the subgrade soils are predominantly granular, the loading duration is not an important factor, because these types of soils are less viscoelastic.

The contact pressure, and number of loads applied by an NDT device also affects the magnitude of stresses in the soil depending on its stress sensitivity as explained above.

3.0 DETERMINATION OF MATERIAL PROPERTIES FROM NDT DATA

Nondestructive test results can be used directly with a minimum of analysis, in designing overlay thickness, or they can be used to "backcalculate" material properties using mechanical analyses. Backcalculation is, to an extent, an inverted design process. If the cross section and properties of the paving materials and support system are known, it is possible to compute the pavement response (stresses, strains, and displacements) for a given loading condition. In the evaluation process the response of the pavement is observed and the material properties are backcalculated.

Among the different load responses, only surface deflections are easily measurable. Deflection is a basic response of the whole system to the applied load. It is frequently used as an indicator of the load carrying capacity of the pavement. Also, surface deflection measurements are rapid, relatively cheap, and nondestructive.

There are a number of different analysis methods that can be used to determine the moduli of pavement layers using the measurements made with an NDT device. They fall broadly into three categories namely; 1) equivalent thickness methods, 2) layered elastic methods, and c) finite element methods. Most of the procedures currently in use fall in one of the above categories and will be discussed in some detail below. Table A.1 shows some of the methods that can be used to automatically determine the modulus from NDT data.

3.1 Equivalent Thickness Methods

This group of methods is based on Odemark's assumption (1949) which converts a multi-layered pavement into an equivalent pavement having only a

Table A.1. Summary of Self-Iterative Procedures for Evaluation of Pavement Moduli from Deflection Basins.

Procedure Title	Source	Pavement Model[1]	Layered Theory Program for Analysis	NDT Device	Input[2]	Output
*	Anani & Wang 1979	4-layers, flexible	BISAR	RR400	W_i i=1 to 4	E_1 to E_4
ISSEM4	Sharma & Stubstad 1980	4-layers, flexible	ELSYM5	FWD	W_i i=variable	E_1 to E_4 for 4-layer input
ELMOD	Dynatest	4-layers, flexible	ELSYM5	FWD	W_i i=variable	E_1 to E_4 for 4-layer input & overlay thickness
CHEVDEF[3]	Bush-WES 1980	4-layers, not to exceed no. of deflections, flexible	CHEVRON	RR2008	W_i i=1 to 4 max (i=1+n)	E_j j=1 to n
ELSDEF	Lytton, Roberts & Stoeffels, 1985	4-layers, flexible	ELSYM5	Dynaflect FWD	W_i i=variable	E_j j=1 to n
OAF	Majidzadeh & Ilves, 1981	3 or 4-layers, flexible	ELSYM5	Dynaflect RR, FWD	W_i i=variable	E_j j=1 to 3 or overlay thickness
MODCOMP2	Irwin, 1983	8-layers, flexible	CHEVRON	Dynaflect FWD	W_i i=variable	E_j j=1 to n
MODULUS	Lytton, Roberts & Stoeffels, 1985	3-layers, flexible	BISAR, ELSYM5 or CRANLAY	Dynaflect FWD	W^i i=1 to 3	E_j j=1 to 3
SEARCH	Lytton, Roberts & Stoeffels, 1985	3-layers, flexible	-	Dynaflect FWD	W_i i=1 to 4	E_j j=1 to n
*	Tenison-NMSHD 1983	3-layers, flexible	CHEVRON's n-layer	RR2000	W_i i=1 to 4	E_j j=1 to 3
RPEDDI	Uddin et al. 1985	3 or 4-layers, rigid	ELSYM5	Dynaflect FWD	W_i i=1 to 5 or i=1 to 7	E_j j=1 to 3 or 4
FPEDDI	Uddin et al. 1985	3 or 4-layers, flexible	ELSYM5	Dynaflect FWD	W_i i=1 to 5 or i=1 to 7	E_j j=1 to 3 or (remaining life)

[1]Semi-infinite subgrade assumed in input.
[2]Thickness, Poisson's ratio, initial seed modulus of each layer (except the thickness of bottom layer) are required input. Allowable ranges of moduli are also required.
[3]Can be easily modified to handle other NDT devices.

single layer. Odemark's formula for converting a multilayer pavement into a single layer is:

$$h_e = c*h_i*(E_1/E_o) \tag{A.1}$$

where:

h_e = the equivalent thickness, in.,

h_i = the actual thickness of the i^{th} layer, in.,

E_i = the elastic modulus of the i^{th} layer, psi,

E_o = the datum modulus i.e. the modulus of the single material into which the multilayered pavement is converted, psi, and

c = a constant which Odemark found to be approximately 0.8 to 0.9.

The new single layered pavement is assumed to have the same vertical deflections at equivalent depths as the original pavement. The major advantage of this procedure is in the speed of computation that can be achieved.

Ullidtz (1978) developed a method by which to calculate the elastic parameters of the pavement layers. He employed the Odemark (1949) assumption to transform multiple layered pavements into equivalent pavements of a single layer. Then he used the Boussinesq equations to solve for surface deflections. The measured deflection basin can be matched if the proper elastic parameters are assumed. The validity of the Ullidtz model has been verified through full scale experiments and actual traffic loading.

Another method (SEARCH) which utilizes Odemark's assumption was developed by Lytton and Michalak (1979) and is built upon two previous approaches to multilayered pavement analysis:

1. multilayered pavement on a rigid base developed by two Russian authors, Vlasov and Leont'ev (1966), and

2. a more general version of Odemark's assumption.

By using the form of equations suggested in these two sources and finding the constants by fitting empirical field deflection data, equations were developed which have been shown to fit the surface deflection of the pavement with a mean-square error that is very small compared to other conventional methods.

The advantage of the equivalent thickness methods is that they are empirical relations that can be used in making rapid trial and error calculations of layer moduli. If the initial assumptions are reasonably close to the actual values, the resulting moduli will be satisfactory.

3.2 Elastic Layer Methods

In the elastic layer approach, the pavement is usually represented by elastic layers (as shown in Figure A.1) of known thicknesses (except for the lowest layer which is assumed to have infinite depth). The layer materials are characterized by Young's Moduli (E) and Poisson's ratio (υ). When a load of known intensity is applied over a known area, deflections are created at some distance from the center of the loaded area. It is normally assumed that the load is distributed through the pavement system by a truncated zone represented by the dashed line in Figure A.1.

Based on this concept, the deflection d_4 at a distance r_4 from the center of the load can only be due to the elastic compression of layer 4 since layers 1, 2 and 3 are outside the influence zone created by the load as shown in Figure A.1. Likewise, the deflection, d_3, at distance r_3 is due to compression of layers 3 and 4; the deflection at distance r_2 is due to compression in layers 2, 3 and 4 and the deflection, d_1 is due to the compression of all layers. This general approach is used to backcalculate properties of pavement

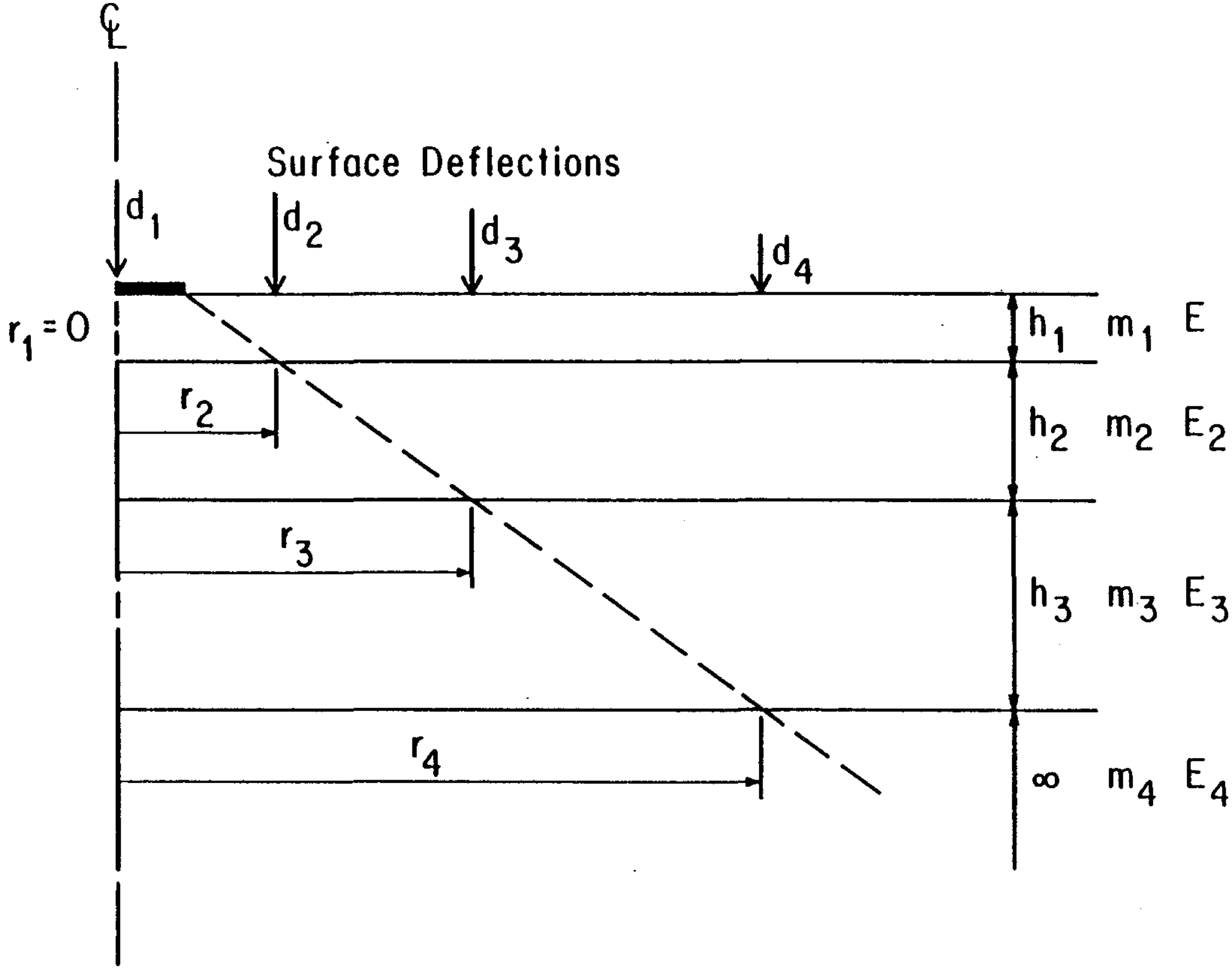

Figure A.1 Four Layer Representation of a Pavement System (Lytton & Smith, 1985).

layers. Examples of commonly used equivalent layer and elastic layer programs are described in the following sections.

3.2.1 CHEVDEF/BISDEF

This computer program was developed by the U.S. Army Corps of Engineers, Waterways Experiment Station (Bush, 1980). It uses a deflection basin from nondestructive testing (NDT) results to predict the elastic moduli of up to four pavement layers. This is accomplished by matching the calculated deflection basin to the measured deflection basin.

The basic assumption of the method is that dynamic deflections correspond to those from the layered elastic theory. This method uses the Chevron (Michelow, 1963) layered elastic program to compute the deflections, stresses and strains of the structure under investigation. The procedure was verified using only one device, the Model 2008 Road Rater. To test the applicability of the deflection basin to the layered elastic analysis, analysis was carried out on test sections using both the BISAR (SHELL, 1972) and CHEVIT (Chevron program with iteration) programs. It was found out that there was good agreement between computed and measured deflections when a rigid layer 20 ft. (6.1 m) from the surface was assumed. The effect of the static load applied to the pavement as a preload with the Model 2008 Road Rater was investigated. It was found that the effect of the static preload for computer modeling of the Road Rater results was practically negligible for most comparisons.

To determine layer moduli, the basic inputs for analysis include the elastic layer pavement characteristics as well as deflection basin values. The inputs for each layer are:

1. Poisson's ratio,
2. Thickness of each layer,

3. Range of allowable modulus, and

4. Initial estimate of modulus.

For the deflection basin the required input includes:

1. Deflection at a number of sensor locations (ND), and

2. Maximum acceptable error in deflections

The modulus of any surface layer may be assigned or computed. If assigned, the value will be based on the type of material or properties of the material at the time of testing. The number of layers with unknown modulus values cannot exceed the number of measured deflections. Best results are obtained when not more than three layers are allowed to vary. As mentioned earlier, a rigid layer is placed 20 ft (6.1 m) from the pavement surface.

The program, by an iterative process, provides the best fit between measured deflection and computed deflection basins. This is done by determining the set of E's that will minimize the error sum, between the computed deflection and measured deflections. A flowchart of the program is given in Figure A.2. The basic steps in the analysis are discussed below.

1. A set of modulus values (E_i) is assumed and the deflection (δ) is computed corresponding to the measured deflection (RRD_j).

2. Each unknown modulus is varied and a new set of deflections is computed for each variation.

3. Using the two computed deflections and the two values of each E, a relationship is determined for each deflection as a function of slope and intercept of the log Modulus vs Deflection curve. Figure A.3 is an illustration for one deflection and one layer. An equation is developed that define the slope

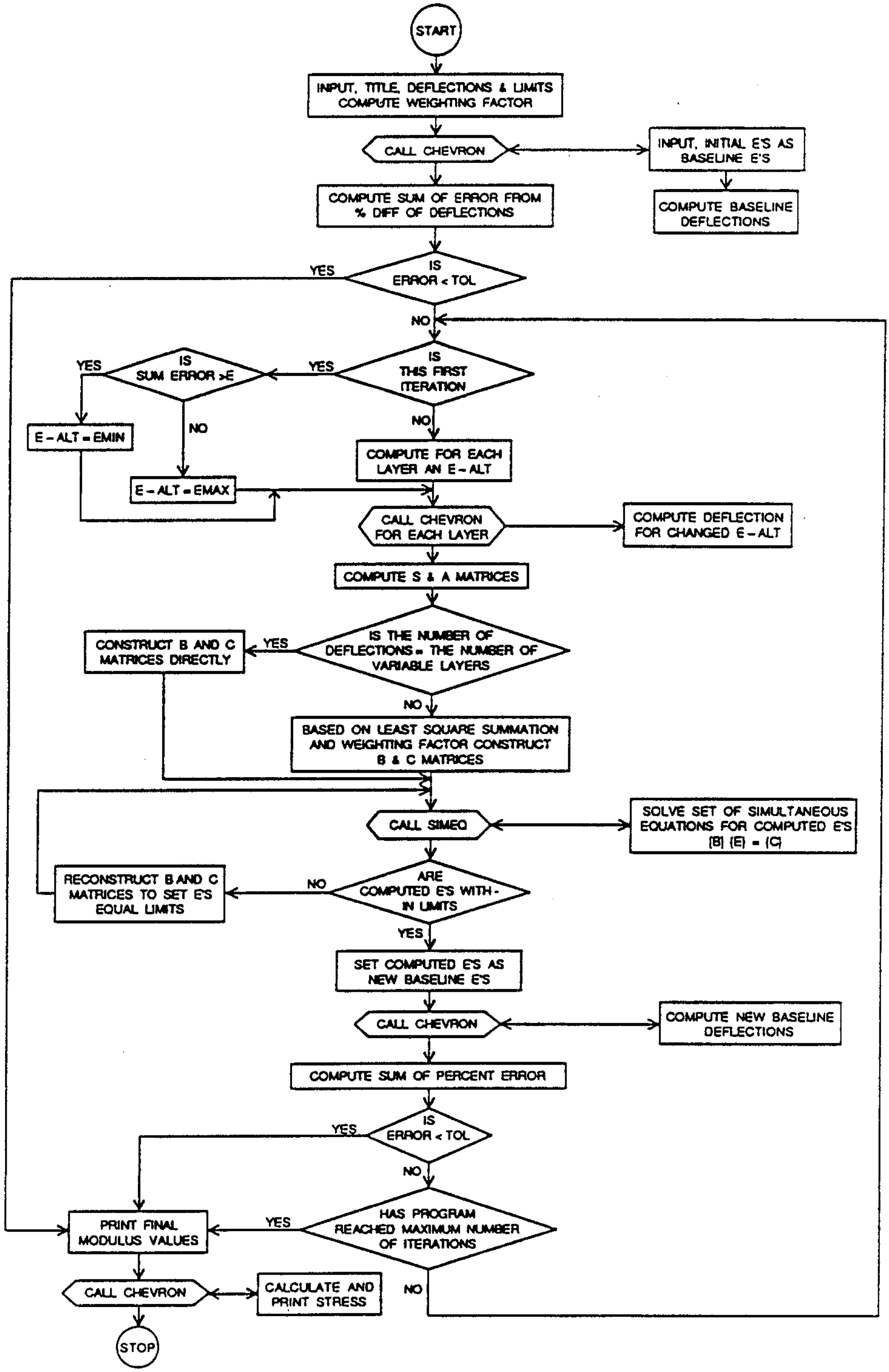

Figure A.2. CHEVDEF/BISDEF Program Flowchart (Bush, 1980).

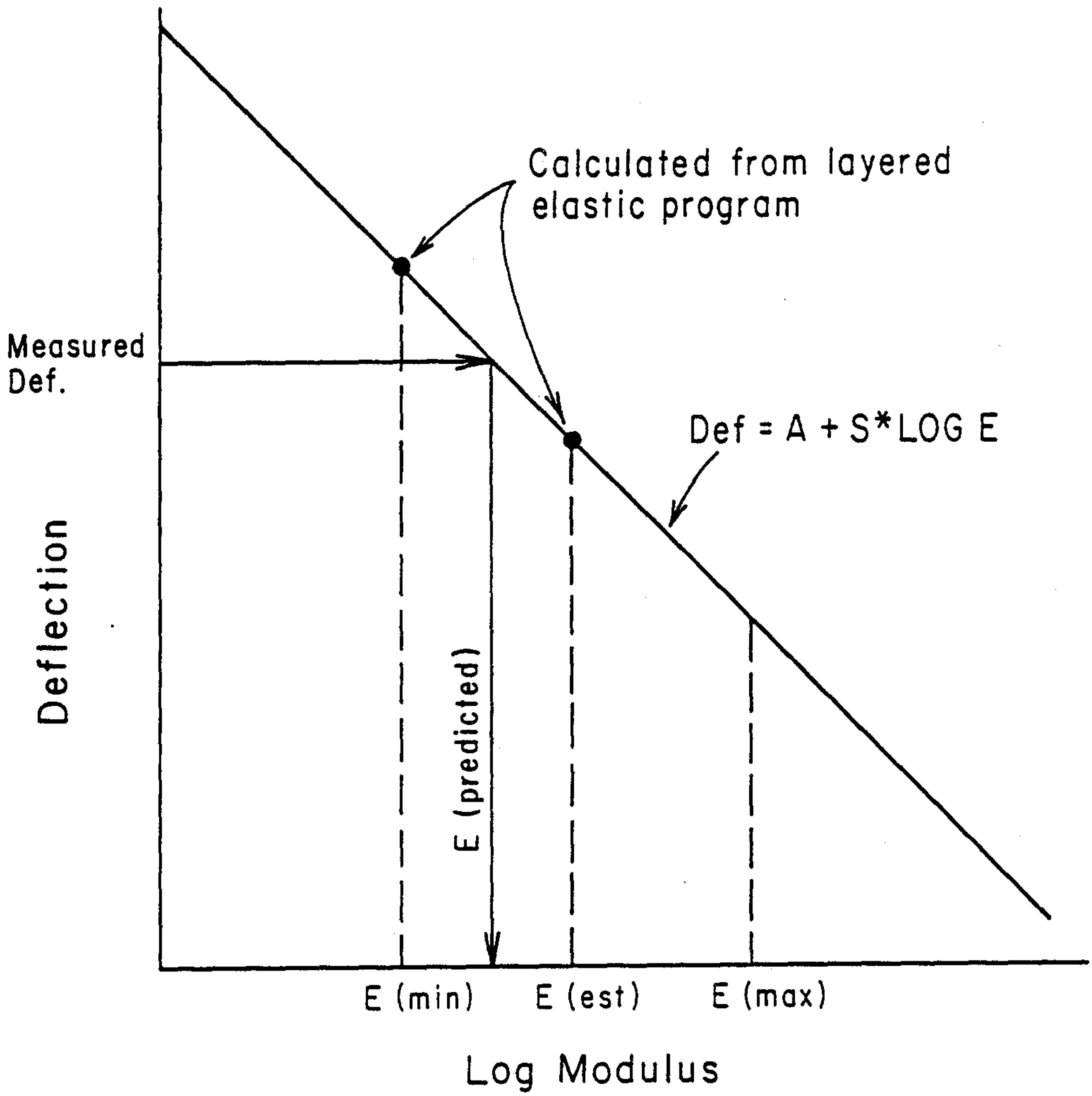

Figure A.3 Simplified Description of the Deflection Matching Procedure in BISDEF/CHEVDEF (Bush, 1980).

and intercept for each deflection and each variable layer as follows:

$$\Delta_j = A_{ji} + S_{ji} (\log E_i) \tag{A.2}$$

where:

Δ_j = surface deflection,

E_i = modulus of layer i,

A = intercept,

S = slope,

j = 1 to number of deflections, and

i = 1 to number of variable layers.

4. For multiple deflections and layers, the solution is obtained by developing a set of equations similar to the above:

$$\Delta_j = \Delta_j + \Sigma S_{ji} (\log E'_i - \log E_i^o) \tag{A.3}$$

where:

Δ_j = computed deflection at first assumed value of E_i, and

NL = number of variable layers.

5. Next the error between the calculated and measured value is determined:

$$RRD_j - \Delta_j = RRD_j - [\Delta_j + \Sigma S_{ji}(\log E'_i - \log E_i^o)] \tag{A.4}$$

where:

RRD_j = measured deflection value.

6. Rearrangement of the above expression produces an equation of the form:

$$[B][E] = [D] \tag{A.5}$$

where:

D = the constant term, and

B = a function of S_{ij}.

7. Solution of the above equations for minimum error cases yields the values of E's.

Errors are minimized by weighing deflections so that the smaller deflections away from the applied load contribute equally to those near the load. Normally three iterations within the program produce a set of modulus values that yield a deflection basin that is within an average of three percent difference of the measured deflections. This accuracy appears to be well within the accuracy of most NDT deflection measuring sensors.

The limitations of this approach are mostly related to the use of the elastic layer theory. First, the elastic layer theory assumes a uniform pressure applied to the surface of the pavement. With the model 2008 Road Rater, the load is applied through a rigid circular plate with the center deflection measured on top of that plate. Therefore, a difference does exist in the measured center deflection and the deflection computed from layer elastic procedures at the center of the load area. Use of the linear elastic layer theory also limits the approach in that it cannot characterize the nonlinear behavior of granular and subgrade materials. The final limitation of this procedure and all deflection curve fitting procedures is that the modulus derived is not unique. It is generally sensitive to the initial assumed seed moduli, especially if these values are drastically different from actual moduli. For gravel roads the program has difficulty matching the computed to measured deflections even after more than five iterations.

3.2.2 ELSDEF

ELSDEF (Lytton et al., 1986) was modified from the program BISDEF, which was developed by the U.S. Army Corps of Engineers at the Waterways Experiment Station. The modification was performed by Brent Rauhut Engineers and instead of using the BISAR subroutine in BISDEF, ELSYM5 was substituted. The Elastic Layered System computer program (ELSYM5) was written at the University of California at Berkeley. It determines the various component stresses, strains and displacements along with principal values in a three-dimensional ideal elastic-layered system (Hicks, 1982). The layered system can be loaded with one or more identical uniform circular loads normal to the surface of the system. ELSDEF, therefore incorporates several improvements over BISDEF, especially its greater efficiency in the use of computer time.

ELSDEF has been compiled with the Microsoft Fortran Compiler to run on IBM-compatible microcomputers. Two versions are available, the standard version and an 8087 math coprocessor chip version.

3.2.3 MODCOMP2

The MODCOMP2 (Irwin, 1983) program was developed at Cornell University. The purpose of the program is to derive the moduli of elasticity for pavement layers from surface deflection data. The program specifications are:

1. The program can deal with up to eight layers in the pavement system, including the bottom layer which is assumed to be infinitely deep.

2. The combination of the layers may be linearly elastic or nonlinearly stress-strain dependent. For the nonlinear case the program presumes an exponential constitutive relationship of the form:

$$E = k1*S^{k2} \tag{A.6}$$

where:

E = modulus of elasticity,

S = stress-strain parameter,

K1 = a coefficient, and

K2 = an exponent.

3. The program is capable of accepting data from several typical nondestructive testing devices such as the Falling Weight Deflectometer, the Road Rater, and the Dynaflect.
4. It can take up to eight surface deflections for each load level, measured at various radial distances from the center of the load.
5. Although the program can take up to 8 layers, good results are obtained for pavement systems having four unknown, linearly elastic layers.
6. Given three or more different load levels the program is capable of deriving the K1 and K2 parameters when they are unknown.
7. The program is capable of accepting up to six load levels.
8. To determine the moduli of deep layers, surface deflections must be measured at relatively large radial distances from the

load. Generally the program will be able to determine the moduli for layers which lie at a depth that is no more than two thirds of the distance from the load to the outermost measured deflection.

9. The computed results of the program are sensitive to variations in the layer thickness. The layer thicknesses should be determined to a degree of precision of five percent or better.

MODCOMP2 utilizes the Chevron elastic layer computer program for determining the stresses, strains and deformations in the pavement system. Since there is no closed-form solution for determining layer moduli from surface deflection data, an iterative approach is used in the computations. The procedure is as follows:

1. Input a set of "seed" moduli from which surface deflections are computed using the Chevron program.
2. The computed deflections are compared to the measured deflections and the seed moduli adjusted as a function of the magnitude of the difference in deflections.
3. The modulus for the layer is interpolated to obtain one which agrees with the measured deflection (Figure A.4).
4. This process is repeated for each layer until the agreement between the calculated and measured deflection is within the specified tolerance or until the specified number of iterations has been exhausted.

Where unknown nonlinear models are to be determined, the program evaluates a modulus for the layer for each of several load levels. The moduli and associated stresses in the layer are then passed to a subroutine which

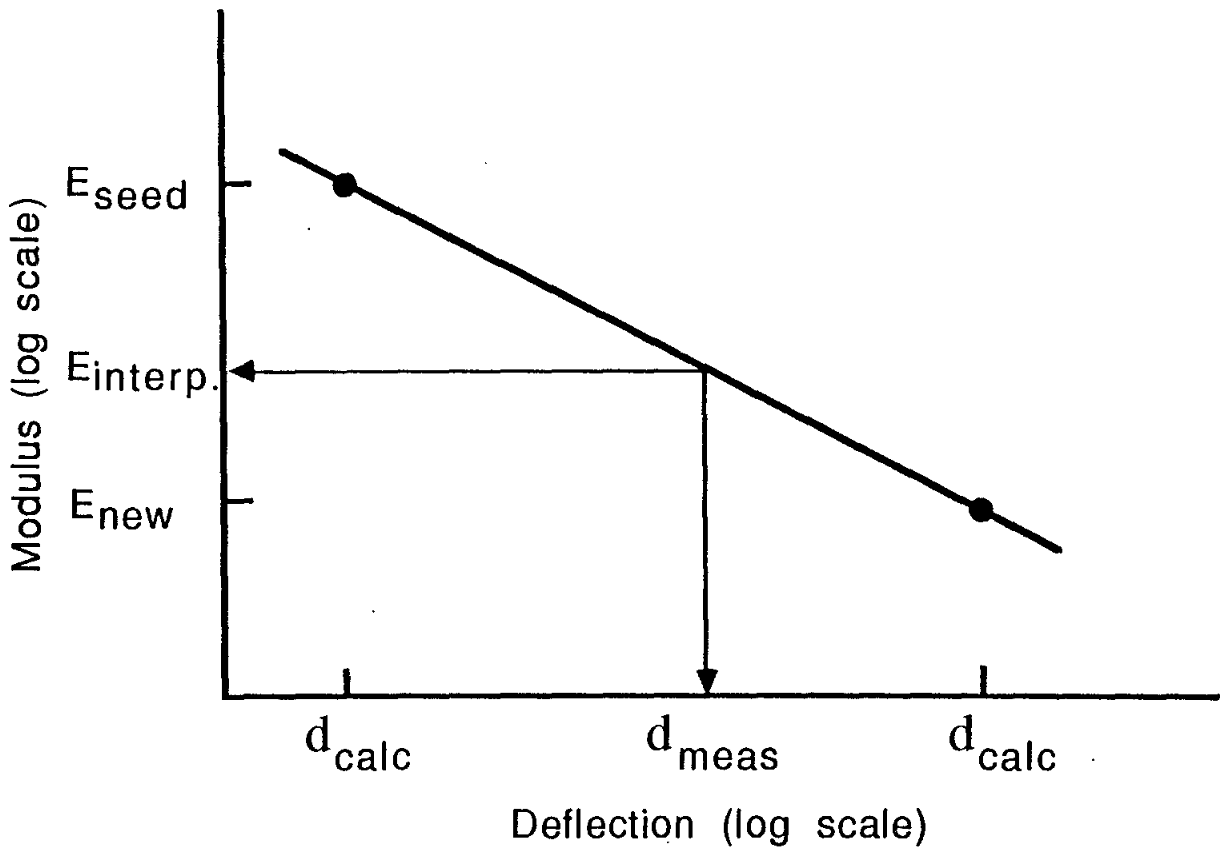

Figure A.4. Interpolation of Modulus Using Calculated and Measured Deflections in MODCOMP2 Program (Irwin, 1983).

performs a regression analysis to determine the K1 and K2 parameters. A hypothesis test is performed to assure that the nonlinear model is significant. If the model is not found to be significant the layer is treated as being linearly elastic for the rest of the iteration. If the model is significant it is used for the remainder of the calculations in the iteration. One of four nonlinear model types can be specified.

Figure A.5 shows the depth beneath which 95% of the surface deflection occurs. The actual shape and position of this line is a function of the moduli and thicknesses of the pavement layers. Most of the registered surface deflection is attributable to compression that occurs in the layers that are below this line. While the actual location of the line is unknown for a particular program, in MODCOMP2 its position is approximated by a 34-degree line. Deflections are assigned to given layers from the set of input data using this line. The deflection that falls closest to the intersection between the upper layer interface and 34-degree line will generally be used.

Sensitivity analyses with the MODCOMP2 program have found that an extremely small tolerance must be specified in order to get accurate results. In general a deflection tolerance on the order of 0.5 percent is required. This is recommended to avoid compounding measurement uncertainties with calculation uncertainties.

The number of iterations required to converge to a solution varies depending on the number of variable layers and whether a linear or nonlinear solution is required. Simple problems with all linear layers and two or three unknowns usually require five or so iterations. More complex problems with four unknowns, perhaps with one or two of them nonlinear, may require ten to twenty iterations.

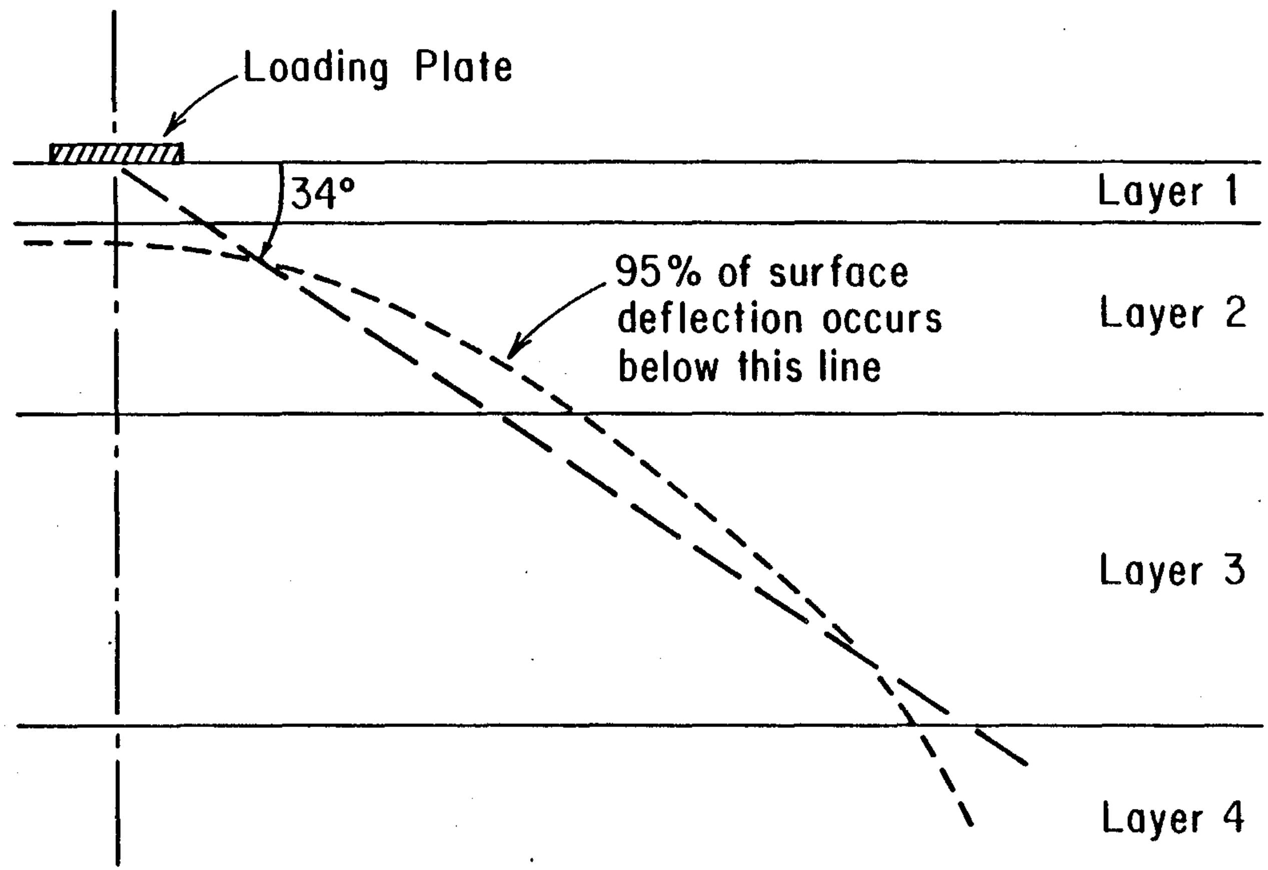

Figure A.5. Pavement Model Showing Line of 95% Deflection (Irwin, 1983).

3.2.4 SEARCH

SEARCH was developed at the Texas Transportation Institute by Lytton and Michalak (1979). This program uses a pattern-search technique to fit deflection basins with elliptic integral function-shaped curves. These curves are solutions to the differential equations used in elastic layered theory. Lytton and Michalak (1979) discuss in detail the theoretical development of the deflection equation used in SEARCH. The deflection equation is based on work that was done by two Russians, Vlasov and Leont'ev (1966), who were interested in the behavior of an elastic layer resting on a rigid incompressible layer. In addition, a generalized form of Odemark's assumption is used. This assumption transforms the thickness of all layers to an equivalent thickness of a material having a single modulus.

The nonlinearity of the response of pavement materials to a load is accounted for by letting the coefficients of vertical displacement distribution with depth and radius depend upon the geometry of the pavement. These coefficients were determined by nonlinear regression analysis upon displacements that were measured at the Texas Transportation Institute's Pavement Test Facility.

The program searches for the elastic moduli that fits the measured deflection basin to the calculated deflection basin with the least average error. The outputs of the program include the calculated moduli, computed and measured deflections, force applied and the squared error of the fitted basin.

3.2.5 MODULUS

MODULUS is an interpolation program that was written by Jacob Uzan (Lytton, Roberts & Stoeffels, 1985). It is based on data calculated using an elastic layered program such as BISAR, ELSYM5 and CRANLAY. However, numerous elastic layered problems must be run for the specific layer thicknesses and loading radii for the pavement sections in question. Therefore, MODULUS is recommended for use when a large number of pavements with similar cross-sections are to be run or when an appropriate data base is already available. MODULUS is written in FORTRAN and compiled by the Microsoft FORTRAN compiler for use on an IBM microcomputer. Two versions are available, one utilizing the presence of an 8087 math coprocessor chip and the other without.

A maximum of three pavement layers (e.g. asphalt concrete layer, base and subgrade) and four sensor locations.

3.2.6 ISSEM4

ISSEM4 is an acronym for In Situ Stress-dependent Elastic Moduli, 4 layers maximum and was developed for use on data generated by the Dynatest FWD by Sharma & Stubstad (1979, 1980). The original concepts used in the program were first published by Ullidtz (1977).

The ISSEM4 program uses a quasi-finite element approach to backcalculate, through a process of iteration, resilient modulus values for layered, nonlinear elastic system from the surface deflections generated by a FWD (Dynatest, 1986). The nonlinear relationships were developed for those "finite cylinders" within the conically-shaped volume of influence of the applied load. This is an iterative process, with a set of seed moduli values used to initiate each run.

From the deflection basin, a deflection reading which reflects the contribution of the subgrade alone is picked. A minimum of seven deflection readings is advised to obtain a full deflection basin profile. Using the ELSYM5 subprogram, the subgrade modulus value is then obtained when the calculated deflection basin fits the measured deflection basin. This process is then repeated with another deflection reading that is yet further away from the load, and another subgrade modulus value obtained. The two moduli are then combined to obtain a composite modulus which can be related to the major principal stress level at or near the surface of the subgrade. Similarly, two E-values can be found for layer i and so on. The finite cylinder modulus relationship is of the general form:

$$E_i = K1_i[SIG(1)_i]^{K2_i} \tag{A.7}$$

where:

E_i = modulus of the i^{th} layer,

$SIG(1)_i$ = principal stress at or near surface of layer i, and

$K1_i, K2_i$ = constants for layer i.

The underlying layer moduli represented by the above equation are appropriately adjusted to reflect their actual moduli at the deflection position being processed. Finally, the above E relationships for each layer below the surface layer are used to calculate the corresponding centerline E-values, and the E-value under the load for the surface layer (E_1) is derived.

The above process describes the first iteration to arrive at a set of modulus vs stress levels relationships for layers 2 to the subgrade, and a set of centerline modulus values for all layers. Next, the ISSEM4 program uses the matrix of E-values obtained from the first iteration loop to reinitiate the next iteration. The relationships and modulus values derived from the second iteration are then compared with those from the first iteration and if the percentage tolerance is less than the user-specified amount, a satisfactory solution has been obtained for the given deflection basin and structural cross section. If not, a new iteration loop is initiated until the percentage tolerance is met. The better the initial seeded modulus values, the quicker the convergence to a unique solution.

All values of stress used in the above equation are calculated based on the linear elastic theory. However, the nonlinearity of a material will not significantly affect the major principal stress magnitudes in a layered, nonlinear elastic system, although the strains may be affected markedly (Dynatest, 1986).

As with all backcalculation programs, ISSEM4 is not perfect. In particular, there are a few danger points to look out for:

1. If the AC layer is less than 3 in. (75 mm), the modulus value for that layer may be quite unreliable.
2. The thickness of layer 2 should be greater than layer 1, or the results may likewise be unreliable.
3. Each layer in the pavement should have a decreasing modulus from the top on downwards, unless E_1 is fixed, in which case E_1 may be less than E_2. The results obtained otherwise may be correct, or it may not.
4. If a four-layer system is to be analyzed, the results for layer 3 may be inaccurate unless it is constrained. ISSEM4 functions most reliably in two- or three-layered systems.

5. A unique solution may not always be possible, due to the fact that the models used in the layered-elastic programs are merely an approximation of actual pavement layers conditions.

3.2.7 ELMOD

ELMOD, or Evaluation of Layer Moduli and Overlay Design (Dynatest, undated), is also a proprietary program of Dynatest Consulting, Inc. The method of equivalent thicknesses is used together with Boussinesq's equations (Ullidtz & Stubstad, 1986) to calculate the layer moduli of a pavement structure using load deflection data generated by a FWD. Once the deflection basin has been input, the ELMOD program automatically calculates the modulus for each layer and will also carry out an overlay design for given loading and climatic conditions.

Two empirical relationships are used in ELMOD, one for predicting cracking of bound layers and one for predicting permanent deformations, and they are of the exponential form:

$$N = KS^a \qquad (A.8)$$

where:

N = the number of loads to cause a certain deterioration at a stress or strain level,

S = stress or strain level at the critically loaded position in the layer, and

K,a = user-controlled input parameters.

Seasonal variation of the critical stresses and strains are also considered. As many as 12 "seasons" may be specified in the program, and the moduli of all layers (including the subgrade) may be varied with season. The damage caused in each season is calculated and summed using Miner's Hypothesis.

If the residual life of a pavement is insufficient, the program will determine the needed overlay thickness of a given material to satisfy the empirical equation above as specified for each layer in the structure. In addition, the program uses the following model to predict the future functional condition of the pavement (Ullidtz & Stubstad, 1986):

$$N = K \times S^a \times E^b \times (P_I - P_T)^c \tag{A.9}$$

where:

- N = the number of load repetitions to cause the performance measure to change from:
- P_I = the initial level to
- P_T = the terminal level, and
- S = critical stress or strain,
- E = the modulus of the material, and
- K,a,b,c = constants.

If the pavement layers are sufficiently nonlinear, the ISSEM4 and MODCOMP2 programs are available. For bedrock or frozen layers close to the surface, the ELMOD program also contains a subprogram called ELROC which calculates the (equivalent) depth to any hard layer, along with the requisite E-values of the materials above this layer.

In summary, it may be said that ELMOD could be useful for the maintenance and rehabilitation of a road network because of its simplicity. For more complex structures, particularly where the nonlinear elastic properties of granular materials are important, Dynatest recommends that other programs be used.

3.2.8 FPEDDI

FPEDDI (Uddin et al., 1985, Uddin, 1984) is a flexible pavement structural evaluation system using dynamic deflections. It evaluates NDT data to determine in situ pavement moduli and applies relevant corrections for the temperature dependency of the asphalt concrete layer and the nonlinear strain-dependent behavior of granular layers and subgrades. An option for remaining life is also provided. The system utilizes the ELSYM5 computer program for calculation of theoretical response of a pavement structure. FPEDDI is designed to handle a three or four layer flexible pavement. Currently the program is capable of analyzing 50 deflection basins in one run.

Input Variables: The input data include the number of sensors, peak force and radius of loading, measured deflection basin, number of layers, test temperature and design temperature for the asphalt concrete layer, and radial distance of geophones from the loading plate. A summarized list of input variables is presented below:

1. Number of total deflection basins for analyses.
2. Test site and date.
3. Station (test location) and name of NDT device.
4. Switch for NDT device, number of deflection sensors, peak force, peak stress of NDT device, and radius of loading.
5. Options for:
    a) summary output of basin fitting subroutine,
    b) remaining life analysis,

c) default procedure for creating a rigid layer at a finite depth of subgrade,

d) type of base material,

e) average unit weight of subgrade soil,

f) surface condition of pavement, and

h) deleting the equivalent linear and remaining life analyses.

6. Measured deflections in mils.

7. Number of layers including subgrade layer, pavement test temperature (°F), and design temperature (°F).

8. Information about each layer, starting from the top layer. Layer number, thickness, Poisson's ratio, initial seed modulus (generally, zero should be entered), maximum allowable modulus, and minimum permissible value of modulus.

9. Maximum allowable number of iterations and five types of tolerances for use in the self-iterative basin fitting procedure.

10. Indicator for user specified design load configuration, design load per tire, tire pressure, and past traffic in cumulative 18-kip ESAL.

<u>Determination of In Situ Moduli</u>: A simplified flow chart of FPEDDI is presented in Figure A.6. The detailed discussion related to different analytical models used in FPEDDI are found in Uddin, 1984. Only the principal analysis models and methodology are briefly described here.

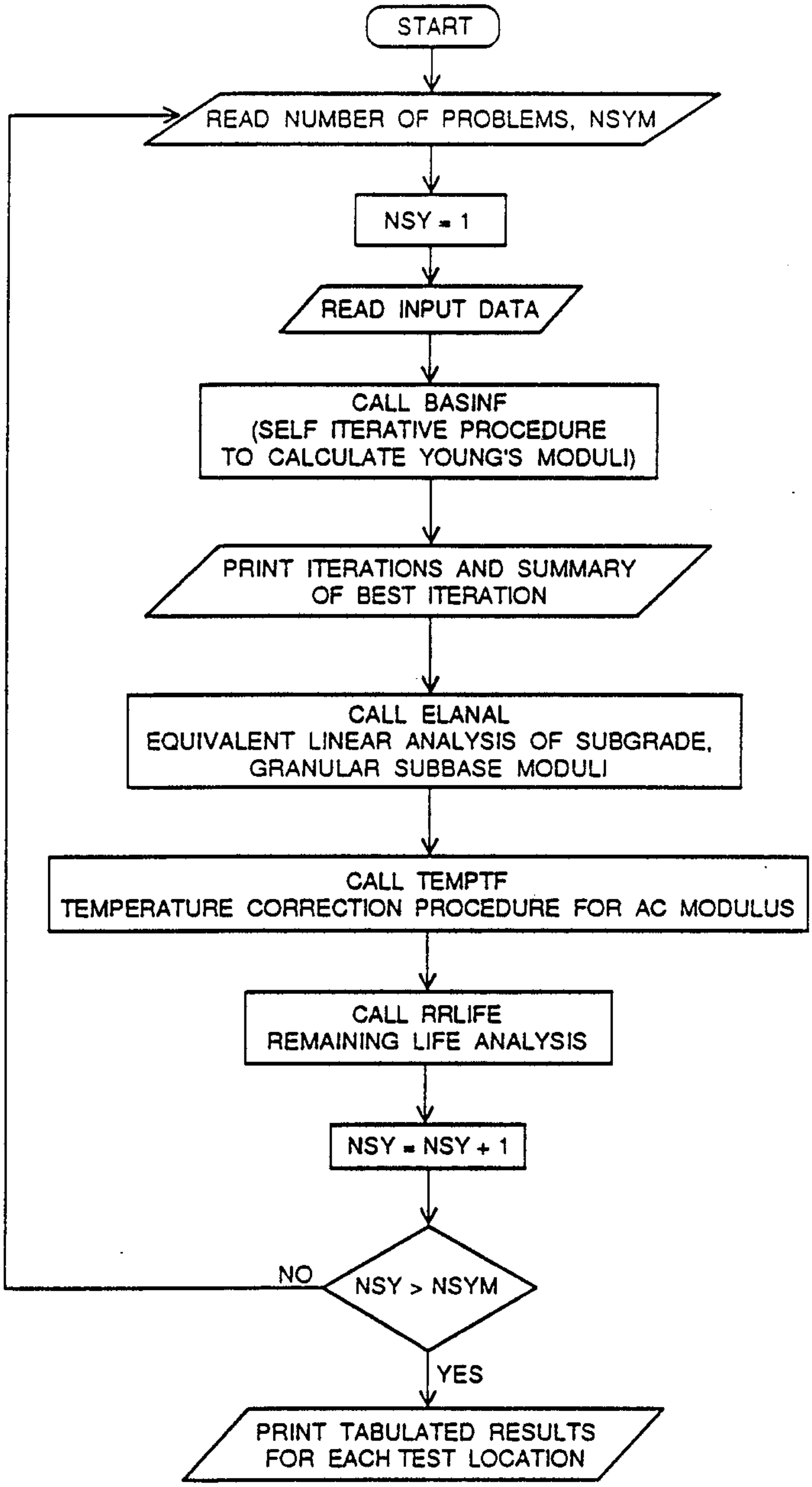

Figure A.6. Simplified Flowchart of FPEDDI (Uddin, 1984).

The following set of assumptions are made in order to validate the application of layered elastic theory for use in determining in situ moduli. These are listed below:

1. The existing pavement is considered to be a layered elastic system. Therefore, the principle of superposition is valid for calculating response due to more than one load.
2. The peak to peak dynamic force of the Dynaflect is modeled as two pseudo-static loads of 500 lb (2200 N) each uniformly distributed on circular areas (3 in.^{2} or 19 cm^2 each). The peak dynamic force of the FWD is assumed to equal the static load uniformly distributed on a circular area representing the FWD loading plate.
3. Thickness of each layer is assumed to be known.
4. Subgrade is characterized by assigning an average value to its modulus of elasticity.

The methodology of determining the in situ moduli relies on generating theoretical deflection basins with ELSYM5 and changing the initial values of assumed moduli through a procedure of successive corrections until a best fit of the measured basin is obtained. A conceptual treatment of the procedure of successive corrections is presented in the following paragraph.

To start with, deflections are calculated from the initial input or default values of moduli. In the first cycle, the number of iterations is equal to the number of layers in the pavement. In each cycle, the first iteration is made to correct the subgrade modulus. ELSYM5 is then called to calculate theoretical deflections. Corrections are then applied to the modulus of the next upper layer and ELSYM5 is again called to calculate theoretical deflections. This procedure of successive corrections is continued until the moduli of all layers have been checked for corrections. Then another cycle of iterations begins anew from the subgrade layer. The generalized form of the relationship used in the procedure of successive corrections is given as:

$$ENEW_i = E_i \, (1.0 - CORR_i \times ERRP_k \times 0.5) \tag{A-10}$$

where:

$ENEW_i$	=	corrected value of Young's modulus of i^{th} layer,
E_i	=	value of Young's modulus of i^{th} layer in the previous iteration.
$CORR_i$	=	correction factor for the i^{th} layer, and
$ERRP_k$	=	discrepancy in measured deflection and predicted deflection as percent error.

The discrepancy in measured and theoretical deflections at the furthest sensor can be used to correct the subgrade modulus. The moduli of intermediate layers are related to discrepancies in the deflection of one or more of the intermediate sensors. Finally, the surface layer modulus can be corrected using the discrepancy at the first sensor and Equation A-10.

Only half of the discrepancy is removed in each iteration. A set of three factors is used in the self-iterative procedure; one is for the subgrade modulus, the second is for the intermediate layers, and the third is associated with the surface layer. Iterations are stopped when one of the following criteria is reached: (1) the maximum absolute discrepancy among calculated and measured deflection is equal to or less than the permissible tolerance, (2) any further correction in the modulus value causes the discrepancies in calculated and measured deflections to increase, (3) the allowable number of iterations is at the maximum specified.

Uniqueness of the Backcalculated Moduli: A severe limitation in any deflection basin fitting method is the non-uniqueness of in situ moduli. As mentioned earlier, the subgrade modulus can be uniquely related to the furthest sensor. However, for a three- or four-layered pavement more than one combination of moduli can predict theoretical deflection basins which will match the measured deflection basin with reasonable closure tolerance. Additionally, a basin matching procedure is generally sensitive to initially assumed seed moduli, especially if these values are drastically different from actual moduli.

Using FPEDDI, a unique set of in situ moduli can be obtained by activating the default procedure for seed moduli. This is done by entering zero values in the input for seed moduli. Predictive equations have been developed for the Dynaflect and the FWD. The procedure adopted for this purpose was to generate numerous theoretical deflection basins for combinations of pavement based on fractional factorial design. Data generated in this way were later used to develop nonlinear predictive equations for seed modulus, E for each layer, with R^2 values ranging from 0.70 to 0.99. In the functional form of these equations, an estimated seed modulus is a nonlinear function of measured deflections, radial distances of geophones and thickness of pavement layers. The provision for default seed moduli eliminates guesswork in selecting initial moduli and ensures a unique result which is not user-dependent.

Nonlinear properties of granular materials and subgrade can be determined based on the concept of Equivalent Linear Analysis. This involves using the relationship between normalized shear modulus with shearing strain. This analysis is necessary for all Young's moduli calculated for nonlinear granular material and subgrade from loads which are less than the design load. An equivalent linear analysis subroutine (ELANAL) has been developed for this task. Another subroutine TEMPFT is used to apply temperature correction to the backcalculated surfacing moduli.

3.3 Finite Element Methods

Linear elastic layer assumptions do not consider the stress dependent nature of the modulus of most pavement materials. It has been shown that the modulus of granular materials is a function of the bulk stress and also that the subgrade material modulus is a function of the deviator stress. The obvious advantage of using a finite element program is that nonlinear stress-strain properties of each of the layers may be used and these properties can be changed with stress levels varying from one element to the next. However, the computing time required to reach an iterative solution using a finite element program is greater than for the linear elastic layer programs.

There are no known automated methods which use a finite element program to calculate layer moduli to match a measured deflection basin. Instead, the approach that is commonly followed is to select a typical pavement type and NDT loading device and make a series of computer runs to determine the surface deflections of that type of pavement as the layer thicknesses and material properties of the layer materials change. An experimental design is used to set the high, low and medium levels of the pavement properties that vary. The surface deflections are then related to thickness and material properties by linear regression analysis. A widely known method utilizing this approach is the set of equations developed by Hoffman and Thompson (1982) which will be discussed in detail below. A somewhat similar approach was adopted by

Marchiona et al. (1985) in developing regression equations and incorporating them into computer programs for use in matching deflection basins.

3.3.1 ILLI-CALC

ILLI-CALC (Hoffman & Thompson, 1981, 1982) is a method developed at the University of Illinois and is used to evaluate nonlinear resilient moduli based on the interpretation of the measured surface deflection basin. The method is not a true backcalculation procedure in the sense of the methods mentioned earlier. Instead it utilizes regression equations and nomographs developed from selected pavement types and materials. The regression equations and nomographs are based on the results of the stress-dependent finite element model ILLI-PAVE. Solutions are possible for conventional flexible pavements composed of an asphalt concrete layer with a typical crushed stone base layer and a fine grained subgrade soil.

The method is based on a deflection basin measured with either the Road Rater or the Falling weight Deflectometer. The Road Rater deflection values are converted to FWD values using the correlations developed during the Illinois study (Hoffman & Thompson, 1981). The deflection basin is characterized as follows:

1. D0 - The maximum deflection at the center of the applied load.
2. D1,D2, D3 - Deflections at 1, 2 and 3 ft. (0.3, 0.6 & 0.9 m) from the center of the load plate.
3. The deflection basin "area" is defined as follows:

$$\text{Area (in}^2) = 6*(1+2D1/D0 + 2D2/D0 + D3/D0)$$

4. The deflection basin shape factors, F1 and F2 are defined as: F1 = (D0-D2)D1 and F2 = (D1-D3)/D2.

In the evaluation procedure, Road Rater center deflections (D0) at 8 kips (35.6 kN) and 15 Hz are converted to equivalent FWD deflections by using the given correlations between the two devices.

The greatest advantage of this procedure is its ability to characterize the nonlinear stress-strain relationships exhibited by most paving materials. The ILLI-PAVE model is an axisymmetric solid of revolution based on the finite-element method. The model incorporates nonlinear stress-dependent material models and failure criteria for granular materials and fine grained soils. The principal stresses in the granular and subgrade layers are modified at the end of each iteration so that they do not exceed the strength of the materials as defined by the Mohr-Coulomb theory of failure. Raad and Figueroa (1980) in their study showed that measured and ILLI-PAVE predicted load deformation responses yielded favorable results.

Material characterizations for the ILLI-PAVE model are shown in Table A.2. The asphalt concrete (AC) material is assumed to be linear elastic with a modulus ranging from 100 to 1400 ksi (690 to 9650 MPa). Two material models are used to characterize the granular base materials. The general model is of the form:

$$E_r = k\theta^n \qquad \text{(A.11)}$$

where:

E_r = resilient modulus (psi),

θ = first stress invariant or bulk stress (psi), and

k,n = material constants determined in repetitive triaxial tests.

Four different fine-grained subgrade soil models were used. These models are given in Figure A.7. The "breaking point" of the curves at a

Table A.2. Material Characterization for ILLI-PAVE (Hoffman & Thompson, 1982).

a) Summary of Material Properties

	Asphalt Concrete			Crushed		Subgrade			
	40°F	70°F	100°F	Stone	Gravel	Stiff	Medium	Soft	V. Soft
Unit Weight (psf)	145.00	145.00	145.00	135.00	135.00	125.00	120.00	115.00	110.0
Lateral Pressure Coeff. at Rest	0.37	0.67	0.85	0.60	0.60	0.82	0.82	0.82	0.82
Poisson's Ratio	0.27	0.40	0.46	0.38	0.38	0.45	0.45	0.45	0.45
Unconfined Compress. Strength (psi)	-	-	-	-	-	32.80	22.85	12.90	6.21
Deviator Stress									
Upper limit (psi)	-	-	-	-	-	32.80	22.85	12.90	6.21
Lower limit (psi)	-	-	-	-	-	2.00	2.00	2.00	2.00
σ_{di} (psi)	-	-	-	-	-	6.20	6.20	6.20	6.20
E_{ri} (ksi)	-	-	-	-	-	12.34	7.68	3.02	1.00
$E_{failure}$ (ksi)	-	-	-	4.00	4.00	7.605	4.716	1.827	1.00
$E_{const.\ mod.}$ (ksi)	1400.00	500.00	100.00	-	-	-	-	-	-
$E_{r\text{-}model}$ (psi)	-	-	-	$9000\theta^{0.33}$	$6500\theta^{0.30}$	-	-	-	-
Friction angle (°)	-	-	-	40.0	40.0	0.0	0.0	0.0	0.0
Cohesion (psi)	-	-	-	0.0	0.0	16.4	11.425	6.45	3.105

1 psi = 6.89 kPa
1 in. = 2.54 cm

b) Layer Thickness (inches)

Asphalt Concrete Layer	Granular Base
0.0	4.0
1.5	6.0
3.0	9.0
	12.0

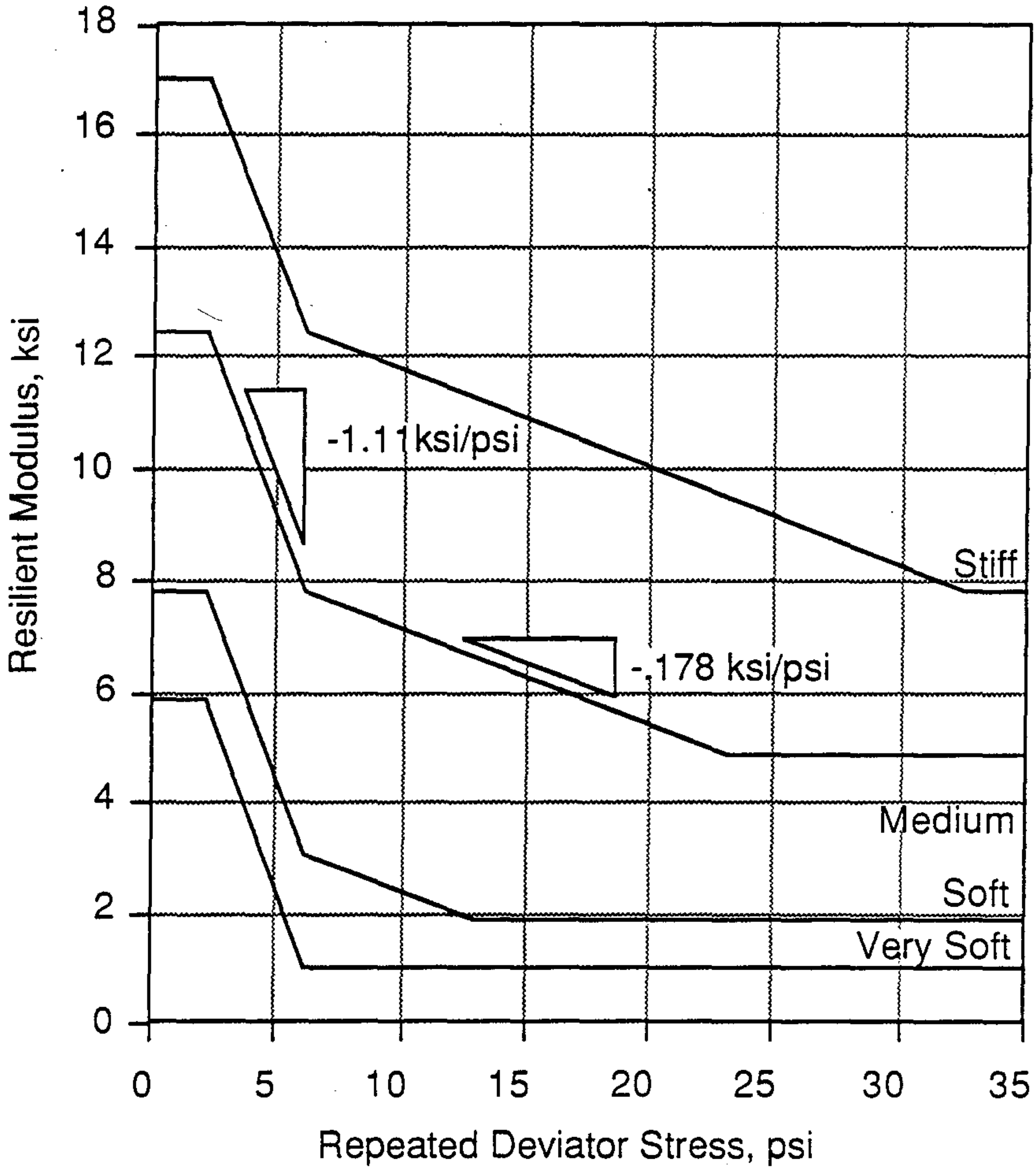

Figure A.7. Subgrade Soil Material Models for ILLI-PAVE Analysis (Hoffman & Thompson, 1981).

deviator stress of 6 psi (41 kPa) corresponds to a resilient modulus denoted E_{ri}. For each of the subgrades chosen, E_{ri} is the main parameter characterizing the nonlinear subgrade soil.

By using the material properties and cross-sections summarized in Table A.2, ILLI-PAVE deflection basin data were generated for a total of 144 combinations. Using multiple-regression techniques deflection-basin predictive equations were developed as a function of the four ILLI-PAVE inputs (Eac, Eri, Tac, Tgr) for conventional flexible pavements, where:

E_{ac} = Modulus of asphalt concrete layer,

E_{ri} = Breaking point subgrade moduli (Figure A.7),

T_{ac} = Thickness of asphalt concrete layer, and

T_{gr} = Thickness of granular layer.

The crushed stone material model is kept constant. The regression equations show that it is possible to predict ILLI-PAVE deflection-basin parameters with reasonable accuracy. (R^2 ranges from 0.90 to 0.95 at the 1% level.)

The backcalculation procedure, given that T_{gr} and T_{ac} are known is as follows:

1. Determine the mean RR (Road Rater) maximum deflection D0.
2. Determine mean RR area (in).
3. Determine mean RR shape factors F1 and F2.
4. Determine the predicted FWD values for steps 1-3.
5. Determine D0 for ILLI-PAVE interpretation.
6. Using nomographs with T_{ac} and T_{gr}, determine E_{ri} and E_{ac}.
7. Check the ratio of measured and computed F1 and F2.

The advantages of this method are:

1. The deflection-basin predictive models can be used in lieu of expensive and frequently unavailable computer runs.
2. The model used to generate the equations takes into account the nonlinear behavior of base and subgrade material.

The limitations of this method are:

1. The method lacks universality in that it requires the use of specific testing devices, one of which is owned by the Illinois DOT and the other (FWD) which is still to be used on a large scale in the United States.
2. The method assumes a subgrade material relationship which might not be typical of subgrade soils in other areas.
3. The method assumes one relationship for the unbound aggregate layer, which might not apply to all aggregate materials.
4. The model used is only capable of one loading configuration i.e a single load.
5. Because of its reliance on regression equations, this method cannot be transferred to another area without having to go through the development of new regression models.

4.0 EVALUATION OF BACKCALCULATION METHODS

Few studies have been carried out to evaluate the different methods used in the determination of moduli from deflection basin data. The following discussion will therefore deal with the basic principles of the various methods in a very general manner.

4.1 Accuracy of the Deflection Matching Process

Any method that uses an iterative procedure to match measured to predicted deflection basins will result in some error. The magnitude of this error depends on many different factors including the accuracy of measuring device. Other factors include:

1. Combining different layers into one structural layer, and
2. The number of deflection points and limitations in the number of layers used in the analysis.

The number of deflection points and the variation of materials both vertically and laterally probably account for the majority of the errors in any deflection matching process.

Another factor known to affect accuracy is the spacing of the deflection sensors. Most procedures suggest that the number of unknown layers for backcalculation of layer moduli should be equal to or less than the number of deflection points measured at the surface. It has been shown that the stiffness of the surfacing layer is highly affected by the measured basin or spacing between sensors especially near the load. Overall, it is an accepted fact that sensor spacing and number of deflection points to define the basin are important to backcalculate the layer moduli.

Another critical factor that has to do with the accuracy of matching deflection basins is the combination of two structural layers, or having two significantly different materials in the subgrade simulated by one composite subgrade stiffness. This usually ignores the stress sensitivity of different subgrade materials, as well as the variation of modulus in the horizontal and vertical direction.

Also to be noted is the fact that the solution for elastic moduli from surface-deflection basin for a three or four-layer system can produce more than one set of modulus values. To obtain a unique solution, several of the methods assign deflections values to different layers i.e. the furthest deflection is assigned to the subgrade in MODCOMP2. Alternatively the range of modular ratios for the various layer materials can be initially assumed from laboratory results.

4.2 Suitability of Analysis Methods

One other factor that affects the use of any analysis procedure is the suitability of the NDT device used. For example, some NDT devices can be used only for detailed analysis such as the La Croix deflectograph and the Benkelman Beam because of the need to take into consideration the location of the supports. Some methods, particularly the equivalent thickness method and the ones based on regression equations are inherently either field checks or production analysis methods which are incapable of handling unusual loading or basin characteristics.

Lytton et al. (1985) have summarized the suitability of different methods and equipment for different levels of analysis. The summary is shown in Table A.3. It is recommended that the selection of a method for analyzing deflec-

Table A.3. Suitability of Analysis Methods for NDT Devices (Lytton, Roberts & Stoeffels, 1985).

NDT Device	Equivalent Thickness	Layered Elastic Theory	Finite Element
Static Deflection			
Benkelman Beam	N	3	3
Plate Bearing Test	3	3	3
Curvature Meter	3	3	3
La Croix Deflectograph	N	3	3
Steady State Dynamic			
Dynaflect	1,2	1,2,3	1,2,3
Road Rater 400B	1,2	1,2,3	1,2,3
Road Rater 2000	1,2	1,2,3	1,2,3
Road Rater 2008	1,2	1,2,3	1,2,3
WES 16-kip Vibrator	1,2	1,2,3	1,2,3
FHWA Cox Van (Thumper)	N	3	3
Impulse Deflection			
Dynatest	1,2	1,2,3	1,2,3
KUAB FWD	1,2	1,2,3	1,2,3
Phoenix FWD	1,2	1,2,3	1,2,3

Key to Levels of Appropriate Analysis Method

1. Field check of deflection data.

2. Production level analysis.

3. Detailed analysis.

N. Inappropriate.

tion test data to determine layer moduli should be compatible with the analytical procedure that is used in designing pavements.

4.3 Problems and Limitations

The use of backcalculation methods do not preclude the absence of problems associated with these procedures. At present, they are unfamiliar to most designers and they also require the use of new equipment, particularly computers. It is necessary to utilize computers because of the extensive time required for the iterative calculations. Because they are fairly recent developments, there is also a limited amount of experience to date.

A major problem is that these procedures do not produce unique moduli values. Several layer combinations can exist to produce the same deflection basin. Variations in the layer thicknesses also affect the layer moduli significantly. Therefore, accurate pavement layer properties are needed and this implies the need for coring. Care must also be taken to ensure that the input values used are within the calibrated values used within the development of the model.

There has been no work found in the literature to verify that the use of analysis methods produce accurate values of strain or deflection in pavements. Also, no work was found that verified whether the determined material properties were correct or if they were consistent with corresponding design methods. Finally, of all the procedures discussed in this section, only one program attempted to include the nonlinear properties of pavement layer materials. Pavement materials are often nonhomogeneous, anisotropic and exhibit nonlinear stress-strain relationships. They may be particulate, i.e. consisting of discrete particles. Discontinuities such as cracks are often present and the condition at the interfaces (whether rough or smooth) are not well known.

4.4 Applicability for Cold Regions

In general, the principle of the backcalculation procedures should be able to include the conditions encountered in climates such as that found in Alaska. However, care should be taken to ensure that the input variables used are within the calibrated values of the selected method. In particular, the low deflection values and the effects of frozen pavement layers should be kept in mind.

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APPENDIX B

PROGRAM LISTING OF THE ASPHALT INSTITUTE'S PROCEDURE

PROGRAM LISTING OF THE ASPHALT INSTITUTE'S PROCEDURE

B.1 Introduction

This appendix documents the computer program that was written by Kingham and Jester (1983)* for designing an asphalt overlay using the Asphalt Institute procedure as developed in MS-17. Input guides are also included to assist the user, together with examples. Since both programs were written in FORTRAN IV, care must be taken to input values in the appropriate columns.

There are two versions of the program listed, Program A and Program B. Program B is more versatile in that multiple values of the representative rebound deflections (RRD) and traffic volume in EALs can be generated with the use of DO loops. This version uses a subroutine to perform the iteration ans includes options for using the default values of tire contact pressure (P), radius of the loading plate (A) and the pavement modulus (EP).

B.2 Input Guides

The input guide for Programs A and B are relatively simple and take no more than 5 lines for one problem. For users unfamiliar with FORTRAN IV, note that real numbers need not be right justified as long as they are contained in the specified field. However, for real numbers, they must be right justified.

B.2.1 Input Data For Program A

Line 1: NAME (20A4):

Up to 80 alphanumerics can be used to briefly describe the problem. Included in here would be the problem name, design parameters and run date. In the output, this will be printed as the heading line.

---

*Kingham, R.I. & Jester, R.N., "Deflection Method for Designing Asphalt Concrete Overlays for Asphalt Pavements," *Research Report No. 83-1*, The Asphalt Institute, August 1983.

Line 2: F1, F2 (2F10.5):

These are the specified constants for the equation relating design deflection and design traffic. For the Asphalt Institute's MS-17, they are assumed to be 1.036 and -0.2438 respectively.

Line 3: NOPT (I5):

This selects one of the following options below:

0 = End of run.
1 = New RRD, EAL (MS-17 default values are used for P, A, and EP).
2 = New RRD, EAL, P, A, EP.

Line 4: RRD, EAL, P, A, EP (F5.3, E10.5, 2F5.1, E10.5):

RRD = Representative Rebound Deflections in inches.
EAL = Design traffic in terms of 18 kip (80 kN) single axle loads.
P = Tire contact pressure in psi. Default value is 70 psi.
A = Radius of single plate in inches. Default value is 6.4 in.
EP = Pavement modulus in psi. Default value is 500,000psi.

Note: If NOPT = 1, the default values for P, A and E_p are assigned and only values for RRD and EAL are to be input.

Lines 3 and 4 are repeated for additional runs with new values for NOPT and input variables. This is not required if the previous values of NOPT is 0.

B.2.2 Input Data For Program B

Line 1: NAME (20A4):

Up to 80 alphanumerics can be used to briefly describe the problem. Included in here would be the problem name, design parameters and run date. In the output, this will be printed as the heading line.

Line 2: F1,F2 (2F10.5):

These are the specified constants for the equation relating design deflection and design traffic. For the Asphalt Institute's MS-17, they are assumed to be 1.036 and -0.2438 respectively.

Line 3: NOPT (I5):

This selects one of the following options below:

NOPT = 0 indicates end of run.
NOPT = 1 sets P, A and EP to the default values for MS-17. An initial and maximum (EAL, MAXEAL) value for EAL are selected as well as the increment (INCR) by which the EAL is to be increased. The representative rebound deflection (RRD) is generated for each EAL value within a preselected range (MIN, MAX) for RRD at specified intervals (STEP).
NOPT = 2 is the same as NOPT = 1 except that default values are supplied for P, A and EP. If the user wishes to run a subsequent set of data using NOPT = 1, the default values for P, A and EP must be reset or a new run started.
NOPT = 3 is similar to NOPT = 1 because it uses the default values for P, A and EP. However, RRD and EAL are input variables and are not generated through the use of a loop.
NOPT = 4 is the same as NOPT = 3, except that P, A and EP are input variables. The same steps for resetting the defaults should be followed when changing from NOPT = 4 to NOPT = 3 as those described in NOPT = 2.

Line 4: These are the specified input variables needed depending on the choice of NOPT. If NOPT equals:

1 : Input MIN, MAX, STEP, EAL, EALMAX, INCR (2I5, F10.5, E10.5, 2I10, 2F5.1, E10.5)

2 : Input MIN, MAX, STEP, EAL, EALMAX, INCR, P, A,EP (2I5, F10.5, E10.5, 2I10, 2F5.1, E10.5)

3 : Input RRD, EAL (F5.3, E10.5, 2F5.1, E10.5)

4 : Input RRD, EAL, P, A, EP (F5.3, E10.5, 2F5.1, E10.5)

where:

RRD = Representative Rebound Deflection, in.
P = Tire contact pressure, psi. Default is 70 psi.
A = Radius of single plate, in. Default is 6.4 in.
MIN = Minimum value of RRD (x 100) if overlay thicknesses are to be computed over a range of values for RRD.
MAX = Maximum value of RRD (x 100) if overlay thicknesses are to be computed over a range of values for RRD.
STEP = Incrementation for RRD if overlay thicknesses are to be computed over a range of values for RRD.
EAL = Design traffic in terms of 18-kip (80 kN) single axle loads. Also, the minimum value for design traffic if overlay thicknesses are to be computed over a range of values for EAL.
EALMAX = Maximum EAL value if overlay thicknesses are to be computed over a range of values for EAL.

INCR = Incrementation for EAL if overlay thicknesses are to be computed over a range of values for EAL.

Lines 3 and 4 are repeated for additional runs with new values of NOPT and input variables. This is not required if the previous value of NOPT = 0.

B.3 Examples

Figures B.1, B.2 and B.3 illustrate example inputs and outputs for Programs A and B. Figures B.4 and B.5 contain the program listings for programs A and B, respectively.

Program A

```
TEST RUN OF MS-17 (PROGRAM A)                          8/12/86
1.03630    -0.24380
    1
0.10 0.75000E06
    1
0.10 0.90000E05
    2
0.10 0.15000E07 60.0  6.00.20000E06
    2
0.01 0.75000E06 70.0  6.40.50000E06
    0
```

Program B

```
TEST RUN OF MS-17 (PROGRAM B)                          8/13/86
1.03600    -0.24380
    1
   10    20    0.010000.15000E07    1850000      50000
    0
```

Figure B.1. Input Data File For Programs A and B.

1 TEST RUN OF MS-17 (PROGRAM A) 8/12/86

EAL	RRD	OVERLAY THICKNESS
.75000E+06	.1000	4.620
.90000E+05	.1000	2.275
.15000E+07	.1000	6.772
.75000E+06	.0100	NO OVERLAY REQUIRED

Figure B.2. Output File for Program A.

1 TEST RUN OF MS-17 (PROGRAM B) 8/13/86

EAL	RRD	OVERLAY THICKNESS
.15000E+07	.1000	5.625
.15000E+07	.1100	6.060
.15000E+07	.1200	6.475
.15000E+07	.1300	6.875
.15000E+07	.1400	7.261
.15000E+07	.1500	7.636
.15000E+07	.1600	8.001
.15000E+07	.1700	8.356
.15000E+07	.1800	8.704
.15000E+07	.1900	9.044
.15000E+07	.2000	9.377
.15500E+07	.1000	5.677
.15500E+07	.1100	6.114
.15500E+07	.1200	6.533
.15500E+07	.1300	6.935
.15500E+07	.1400	7.324
.15500E+07	.1500	7.702
.15500E+07	.1600	8.069
.15500E+07	.1700	8.428
.15500E+07	.1800	8.778
.15500E+07	.1900	9.121
.15500E+07	.2000	9.457
.16000E+07	.1000	5.727
.16000E+07	.1100	6.167
.16000E+07	.1200	6.589
.16000E+07	.1300	6.994
.16000E+07	.1400	7.386
.16000E+07	.1500	7.767
.16000E+07	.1600	8.137
.16000E+07	.1700	8.498
.16000E+07	.1800	8.851
.16000E+07	.1900	9.196
.16000E+07	.2000	9.535
.16500E+07	.1000	5.776
.16500E+07	.1100	6.219
.16500E+07	.1200	6.643
.16500E+07	.1300	7.052
.16500E+07	.1400	7.447
.16500E+07	.1500	7.830
.16500E+07	.1600	8.203
.16500E+07	.1700	8.566
.16500E+07	.1800	8.922
.16500E+07	.1900	9.270

Figure B.3. Output File For Program B.

EAL	RRD	OVERLAY THICKNESS
.16500E+07	.2000	9.611
.17000E+07	.1000	5.824
.17000E+07	.1100	6.270
.17000E+07	.1200	6.697
.17000E+07	.1300	7.108
.17000E+07	.1400	7.506
.17000E+07	.1500	7.891
.17000E+07	.1600	8.267
.17000E+07	.1700	8.633
.17000E+07	.1800	8.991
.17000E+07	.1900	9.342
.17000E+07	.2000	9.685
.17500E+07	.1000	5.871
.17500E+07	.1100	6.320
.17500E+07	.1200	6.749
.17500E+07	.1300	7.163
.17500E+07	.1400	7.563
.17500E+07	.1500	7.952
.17500E+07	.1600	8.330
.17500E+07	.1700	8.699
.17500E+07	.1800	9.059
.17500E+07	.1900	9.412
.17500E+07	.2000	9.758
.18000E+07	.1000	5.916
.18000E+07	.1100	6.368
.18000E+07	.1200	6.800
.18000E+07	.1300	7.217
.18000E+07	.1400	7.620
.18000E+07	.1500	8.011
.18000E+07	.1600	8.391
.18000E+07	.1700	8.763
.18000E+07	.1800	9.126
.18000E+07	.1900	9.481
.18000E+07	.2000	9.830
.18500E+07	.1000	5.961
.18500E+07	.1100	6.416
.18500E+07	.1200	6.851
.18500E+07	.1300	7.270
.18500E+07	.1400	7.675
.18500E+07	.1500	8.068
.18500E+07	.1600	8.452
.18500E+07	.1700	8.825
.18500E+07	.1800	9.191
.18500E+07	.1900	9.549
.18500E+07	.2000	9.900

Figure B.3. Output File For Program B (cont'd).

```
C                             PROGRAM A
C     THIS PROGRAM CALCULATES OVERLAY THICKNESS USING THE DEFLECTION
C     PROCEDURE IN TAI MANUAL (MS-17).
C
C                            INPUT VARIABLES
C     A    - RADIUS OF SINGLE PLATE, INCHES (DEFAULT - 6.4)
C     EP   - PAVEMENT MODULUS, PSI (DEFAULT - 500,000)
C     P    - CONTACT PRESSURE, PSI (DEFAULT - 70)
C     EAL - EQUIVALENT 80 KN SINGLE AXLE LOAD
C     F1   - CONSTANT FOR DESIGN DEFLECTION PROCEDURE
C     F2   - EXPONENT FOR DESIGN DEFLECTION PROCEDURE
C     NAME - DESCRIPTION OF RUN
C     NOPT - INDICATES VARIABLES TO BE INPUT
C          0 - END OF RUN
C          1 - NEW: RRD, EAL(MS-17 DEFAULT VALUES USED FOR P,A & EP)
C          2 - NEW: RRD, EAL, P, A, EP
C     RRD - REPRESENTATIVE REBOUND DEFLECTION
C     STEP - INCREMENTATION FOR RRD
C     T    - ASSUMED OVERLAY THICKNESS, INCHES (SET AT 1.00)
C
      DIMENSION NAME (20)
      DATA P,A,EP/70.,6.4,500000./
      NIN-5
      NOUT-6
      READ(NIN,10) NAME
      WRITE(NOUT,20) NAME
   10 FORMAT(20A4)
   20 FORMAT(1H1,11X,20A4)
      WRITE(NOUT,30)
   30 FORMAT(10X,3HEAL,11X,3HRRD,8X,17HOVERLAY THICKNESS)
      WRITE(NOUT,40)
   40 FORMAT(1X,52(1H-),/)
      READ(NIN,50) F1,F2
   50 FORMAT(2F10.5)
   60 READ(NIN,65) NOPT
   65 FORMAT(I5)
      IF (NOPT .EQ. 0) GO TO 999
      IF (NOPT .EQ. 2) GO TO 80
      READ (NIN,70) RRD,EAL
   70 FORMAT(F5.3,E10.5,2F5.1,E10.5)
      GO TO 90
   80 READ(NIN,70) RRD,EAL,P,A,EP
   90 CONTINUE
      N- 1
      T- 1.0
  100 FAC1- (1.5*P*A/EP)*(0.8*T**2.0/A**2.0+1)**(-0.5)
      FAC2- (((0.488*((EP*RRD/P)**(2.0/3.0))
     1 /A**(8.0/3.0)*T**2.0)+1.0)**(-0.5)*RRD)
      FAC3- F1*EAL**F2
      FAC4- (1.2*P/EP/A)*((0.8*((T/A)**2.0)+1.0)**(-3.0/2.0))*T
```

Figure B.4. Program Listing for Program A

```
      FAC5= (0.488*RRD**(5.0/3.0)/A**(8.0/3.0)*((EP/P)**
     1 (2.0/3.0)))*((0.488*T**2.0/A**(8.0/3.)*
     2 ((RRD*EP/P)**(2.0/3.0)))+1.0)**(-3.0/2.0)*T
      TNEW= T-(((1.5*P*A/EP)-FAC1+FAC2-FAC3)/(FAC4-FAC5))
      IF (TNEW .LT. 0.0) GO TO 110
      IF (ABS(T-TNEW) .LT. 0.001 .OR. N .GT. 20) GO TO 115
      T= TNEW
      N=N+1
      GO TO 100
  115 WRITE(NOUT,130) EAL,RRD,TNEW
      GO TO 60
  110 WRITE(NOUT,120) EAL, RRD
  120 FORMAT(1X,E15.5,5X,F6.4,5X,'   NO OVERLAY REQUIRED')
  130 FORMAT(1X,E15.5,5X,F6.4,5X,F12.3)
      GO TO 60
  999 STOP
      END
```

Figure B.4. Program Listing for Program A (cont'd).

```
C                           PROGRAM B
C     THIS PROGRAM CALCULATES OVERLAY THICKNESS USING THE DEFLECTION
C     PROCEDURE IN TAI MANUAL (MS-17).
C
C     THIS PROGRAM IS MORE VERSATILE THAN PROGRAM A IN THAT
C     MULTIPLE VALUES OF RRD AND EAL CAN BE GENERATED THROUGH
C     THE USE OF DO LOOPS.
C
C                          INPUT VARIABLES
C     A    - RADIUS OF SINGLE PLATE, INCHES (DEFAULT - 6.4)
C     EP   - PAVEMENT MODULUS, PSI (DEFAULT - 500,000)
C     P    - CONTACT PRESSURE, PSI (DEFAULT - 70)
C     EAL - EQUIVALENT 80 KN SINGLE AXLE LOAD
C     F1   - CONSTANT FOR DESIGN DEFLECTION PROCEDURE
C     F2   - EXPONENT FOR DESIGN DEFLECTION PROCEDURE
C     INCR -  INCREMENTATION FOR EAL DO LOOP
C     MAX   - MAXIMUM DEFLECTION X 100
C     MIN   - MINIMUM DEFLECTION X 100
C     NAME - DESCRIPTION OF RUN
C     NOPT - INDICATES VARIABLES TO BE INPUT
C       0 - END OF RUN
C       1 - NEW:MIN,MAX,STEP,EAL,MAXEAL,INCR,(DEFAULT VALUES USED FOR P,A & EP)
C       2 - NEW: MIN,MAX, STEP,EAL,MAXEAL,INCR,P,A,EP
C  ** USE NOPT- 1 OR 2 WHEN RRD & EAL ARE TO BE COMPUTED **
C       3 - NEW:RRD, EAL (MS-17 DEFAULT VALUES USED FOR P,A,EP)
C       4 - NEW:RRD,EAL,P,A,EP
C  ** USE NOPT- 3 OR 4 WHEN RRD AND EAL ARE INPUT VALUES **
C     RRD - REPRESENTATIVE REBOUND DEFLECTION
C     STEP - INCREMENTATION FOR RRD
C     T    - ASSUMED OVERLAY THICKNESS, INCHES (SET AT 1.00)
C
      DIMENSION NAME (20)
      INTEGER EALMAX
      DATA P,A,EP/70.,6.4,500000./
      NIN=5
      NOUT=6
      READ(NIN,10) NAME
      WRITE(NOUT,20) NAME
   10 FORMAT(20A4)
   20 FORMAT(1H1,11X,20A4)
      WRITE(NOUT,30)
   30 FORMAT(10X,3HEAL,11X,3HRRD,8X,17HOVERLAY THICKNESS)
      WRITE(NOUT,40)
   40 FORMAT(1X,52(1H-),/)
      READ(NIN,50) F1,F2
   50 FORMAT(2F10.5)
   60 READ(NIN,65) NOPT
   65 FORMAT(I5)
      IF (NOPT .EQ. 0) GO TO 999
```

Figure B.5. Program Listing for Program B.

```
      GO TO (70,90,130,150), NOPT
 70   READ(NIN,80) MIN,MAX,STEP,EAL,EALMAX,INCR
 80   FORMAT(2I5,F10.5,E10.5,2I10,2F5.1,E10.5)
      GO TO 100
 90   READ(NIN,80) MIN,MAX,STEP,EAL,EALMAX,INCR,P,A,EP
100   IEAL= EAL
      DO 120 I=IEAL,EALMAX,INCR
      DO 115 J=MIN,MAX
      RRD= J*STEP
      CALL OVER(F1,F2,RRD,EAL,P,A,EP,TNEW)
      IF (TNEW .LT. 0.0) GO TO 110
      WRITE(NOUT,190) EAL,RRD,TNEW
      GO TO 115
110   WRITE(NOUT,180) EAL,RRD
115   CONTINUE
      EAL= EAL + INCR
120   CONTINUE
      GO TO 60
130   READ(NIN,140) RRD,EAL
140   FORMAT(F5.3,E10.5,2F5.1,E10.5)
      GO TO 160
150   READ(NIN,140) RRD,EAL,P,A,EP
160   CALL OVER (F1,F2,RRD,EAL,P,A,EP,TNEW)
      IF (TNEW .LT. 0.0) GO TO 170
      WRITE(NOUT,190) EAL,RRD,TNEW
      GO TO 60
170   WRITE(NOUT,180) EAL,RRD
180   FORMAT(1X,E15.5,5X,F6.4,5X,'   NO OVERLAY REQUIRED')
190   FORMAT(1X,E15.5,5X,F6.4,5X,F12.3)
      GO TO 60
999   STOP
      END

      SUBROUTINE OVER(F1,F2,RRD,EAL,P,A,EP,TNEW)
      T= 1.0
      N= 1
 10   FAC1= (1.5*P*A/EP)*((((T/A)**2.0)*0.8)+1.0)**(-0.5)
      FAC2= (((0.488*((EP*RRD/P)**(2.0/3.0))/A**
     1 (8.0/3.0)*T**2.0)+1.0)**(-0.5)*RRD)
      FAC3= F1*EAL**F2
      FAC4= (1.2*P/EP/A)*((0.8*((T/A)**2.0)+1.0)**(-3.0/2.0))*T
      FAC5= (0.488*RRD**(5.0/3.0)/A**(8.0/3.0)*((EP/P)**
     1 (2.0/3.0)))*((0.488*T**2.0/A**(8.0/3.0)*
     2 ((RRD*EP/P)**(2.0/3.0)))+1.0)**(-3.0/2.0)*T
      TNEW= T-(((1.5*P*A/EP)-FAC1+FAC2-FAC3)/(FAC4-FAC5))
      IF (TNEW .LT. 0.0) GO TO 20
      IF (ABS(T-TNEW) .LT. 0.001 .OR. N .GT. 20) GO TO 30
      T= TNEW
      N= N+1
      GO TO 10
```

Figure B.5. Program Listing for Program B (cont'd).

```
20 TNEW= -1.0
30 CONTINUE
   RETURN
   END
```

Figure B.5. Program Listing for Program B (cont'd).

APPENDIX C

ANALYSIS OF PROJECT DATA

ANALYSIS OF PROJECT DATA

This appendix includes all the data used for analyzing the three projects, Sterling and Seward Highways in Anchorage, and Parks Highway in Fairbanks. It lists the assumptions and details the calculations that form the basis of the results that are summarized in Chapter 4.

C.1 Sterling Highway

This project has been divided into five sections for analysis by the regional engineer. The first section of this project, from MP 117 to MP 130 (Clam Gulch to 2 miles (3.2 km) north of Ninilchik) was selected for analysis in this report. Falling Weight Deflectometer deflection readings from 1986 were available for a period extending from March 19th to July 23rd. (See Table C.1).

To determine the critical time of the year, a sample location was selected that represented a range of temperatures and loads. In this case, MP 120 was chosen. As can be seen from Table C.1, six sets of deflection basins were available. ELSYM5 was then used to determine the critical (i.e. maximum) tensile strain at the bottom of the asphalt concrete layer. The tensile strain was selected because it was determined that fatigue was the predominant failure mode. This critical strain is then assumed to represent the time of the year when the pavement is weakest and the entire section will be designed for this condition. It was assumed that the pavement was partially thawed if the pavement surface temperatures were 50°F (10°C) and below. This value was provided by ADOT&PF as a reasonable assumption.

Pavement thicknesses for the unfrozen case were 1.5 in. (3.8 cm) for the AC layer and 34 in. (86 cm) for the base and subbase combined. When the pavement is partially frozen, it was assumed that the thaw depth lies approximately 15 in. (38 cm) below the pavement surface. From conversations with ADOT&PF, it was indicated that the thaw depth typically varied between 12 to 24 in. (30 to 61 cm) deep. Using the modified Berggren equation, the thaw depth was 30 in. (76 cm). However, the inputs to this equation were probably inaccurate estimations as more accurate climate and pavement soil data were not available. Since varying the thaw depth between this range had little or no effect on the results, 15 in. (38 cm) was selected to represent the partially thawed condition.

The base resilient modulus was assumed to be 100 ksi (689 MPa) and the subgrade was 30 ksi (207 MPa). These figures represent the average values from a report by Johnson and Hicks (1986) on aggregate bases in Anchorage. The frozen materials were assumed to have a modulus of 500 ksi (3445 MPa). To determine the AC modulus, a computer program, AMOD, was used. AMOD is based on relationships developed by the Asphalt Institute and determines the AC modulus as a function of the physical properties of the asphalt cement, temperature and frequency of loading. Table C.2 indicate the parameters used in AMOD with temperature being the variable. Figure D.2, which is based on laboratory data from Fairbanks, may also be used.

The tensile strain is then calculated by ELSYM5 under a load of 9000 lb. (40 kN) and with a loading radius of 5.91 in. (150 mm). As can be seen from Table C.1, it was found that the strains from March 25th were critical. Notice that this is true despite the fact that the center deflection readings are not the maximums.

Once the critical time of the year has been determined, the March 25th deflection basins for other locations are used to design the overlay. In

Table C.2. Parameters Used in AMOD to Determine E_1.

Parameter	Value
Void Ratio	3
Penetration of asphalt cement	200
Absolute viscosity at 70°F, Poises	0.264062
Percent asphalt by weight	6
Percent passing No. 200 sieve	5
Velocity of vehicle, mph	3
Temperature, °F	variable

other words, it is assumed that the critical time of the year for MP 120 is also the critical time of the year for the rest of the road section.

The selection of the critical section for analysis is a little qualitative. As discussed in previous sections of this report, there are at present no clear guidelines for this procedure. For this report, the "area" factor (Equation 3.1) as defined by Hoffman and Thompson (1981) was calculated. The basins with area factors ranging from 8 to 13 were selected for consideration. Also, the center-of-load deflections for basins measured at the critical time of the year was adjusted to 70°F and normalized to a 9000 lb load. Sections with the highest and lowest deflections were also then considered for analysis. This was done in an effort to obtain as representative a sample of the project as was possible.

However, for the Sterling and Seward Highway projects, this was not done, primarily because the format of the deflection basin data provided did not lend themselves to this procedure very easily. Instead, deflection basins were chosen at every half-mile (0.8 km), beginning at MP 117.5 and ending at MP 129.5. In this way, it is possible to obtain a profile of the overlay thickness along the project instead. MP 117 and 130 were not used because of the uncertainty in determining the exact boundaries between the sections. Table C.3 presents the deflection basin data that were used. Four methods of overlay design are presented in the following sections.

C.1.1 The Asphalt Institute

The Asphalt Institute procedure, as outlined in MS-17, is also the official ADOT&PF procedure. Using the center deflections (corrected to 70°F (21°C) and normalized to 9000 lb. (40 kN)), the Representative Rebound Deflection (RRD) is determined.

Table C.3. FWD Deflection Basins for Sterling Highway.

Location	Temp °F	Cf[1]	Load lb.	FWD Deflection Basins (mils) S1[2]	S1	S2	S3	S4	S5	S6	S7
117.50	57	1.07	10248	8.45	9	5.4	2.8	1	0.3	0.3	0.4
118.00	37	1.83	1012	24.5	15.3	9.7	5.5	2.2	0.4	0.2	0.4
118.50	60.1	1.05	9968	13.8	15	8.6	4.2	1.6	0.4	0.3	0.4
119.00	37	1.83	9928	21.6	13.5	7.8	3.6	1.1	0.1	0.2	0
119.50	63.9	1.03	9896	13.0	14.4	8.2	4	1.2	0.1	0.2	0.4
120.00	37	1.83	9880	24.9	15.5	10.2	6.3	2.8	0.5	0	0.1
120.50	60.1	1.05	9792	16.3	17.7	9.1	4.1	1.2	0	0.1	0.3
121.00	37	1.83	9920	26.5	16.5	9.4	4.4	1.1	0	0.1	0.2
121.50	61	1.04	9664	10.4	11.4	5	1.3	0.1	0.2	0.3	0.3
122.00	37	1.83	10032	33.4	20.8	12.1	6	1.4	0	0.1	0.5
122.50	53.1	1.3	9528	12.5	11	5.1	1.5	0	0.1	0.1	0.1
123.00	37	1.83	10024	43.2	26.9	18.4	10.7	4.9	0.8	0.1	0.5
123.50	57	1.07	9544	13.4	14.3	8	4.2	1.4	0	0.3	0.3
124.00	37	1.83	10272	27.9	17.4	10.3	5.4	1.8	0	0	0.2
124.50	57	1.07	9744	15.5	16.5	8.5	3.2	0.2	0	0.3	0.2
125.00	37	1.83	10552	24.7	15.4	8.6	4	0.7	0	0.2	0.4
125.50	61	1.04	9600	16.6	18.2	10.2	4.8	1.3	0	0.2	0.4
126.00	37.9	1.80	9896	16.9	10.7	7	4.4	2.3	0.7	0.2	0.2
126.50	57	1.07	9592	14.6	15.6	7.5	2.6	0	0	0.3	0.3
127.00	37.9	1.80	10120	38.4	24.2	16.4	10.2	4.8	0.8	0	0.1
127.50	53.1	1.3	10640	19.5	17.1	9.6	4.6	1.3	0	0.1	0.3
128.00	37.9	1.80	10048	23.8	15	8.7	4.3	1.4	0.1	0	0.2
128.50	57	1.07	9936	8.08	8.5	4.2	1.4	0.2	0.3	0.4	0.3
129.00	37.9	1.80	10096	30.4	19.2	13.6	5.2	1.5	0.1	0.2	0.3
129.50	63	1.04	10560	15.9	17.5	8	1.6	0	0	0.2	0.2

[1]The temperature correction factor is developed in Appendix D.
[2]This deflection reading has been corrected to 70°F and normalized to a 9000 lb. load. The mean deflection is 21.3 mils, with a standard deviation of 9.46 mils.

$$RRD = \text{mean} + 2 \text{ standard deviations} \tag{C.1}$$

$$= 21.3 + 2*(9.46)$$

$$= 40.2 \text{ mils}$$

With a predicted 20 year EAL of 1,800,000, Figure C.1 is used to obtain the overlay thickness, and this was found to be 3.0 in. (7.6 cm) from MP 117.5 to 129.5.

C.1.2 Newcomb's Equations

Newcomb's regression equations was used to calculate the layer moduli. Chapter 2 discusses this method in greater detail. Using MP 117.5 as our example, the subgrade modulus is first calculated:

$$E_s = - 111 + 0.00577(P/S7) \tag{C.2}$$

$$= -111 + 0.00577(10248 \text{ lb})/(0.0004 \text{ in})$$

$$= 147{,}716 \text{ psi}$$

Then an "area" factor is calculated:

$$A_1 = 4S_1 + 6S_2 + 8S_3 + 12S_5 + 6S_6 \tag{C.3}$$

$$= 4(9 \text{ mils}) + 6(5.4) + 8(2.8) + 12(0.3) + 6(0.3)$$

$$= 96.2 \text{ mils.}$$

The base and AC modulus can then be found:

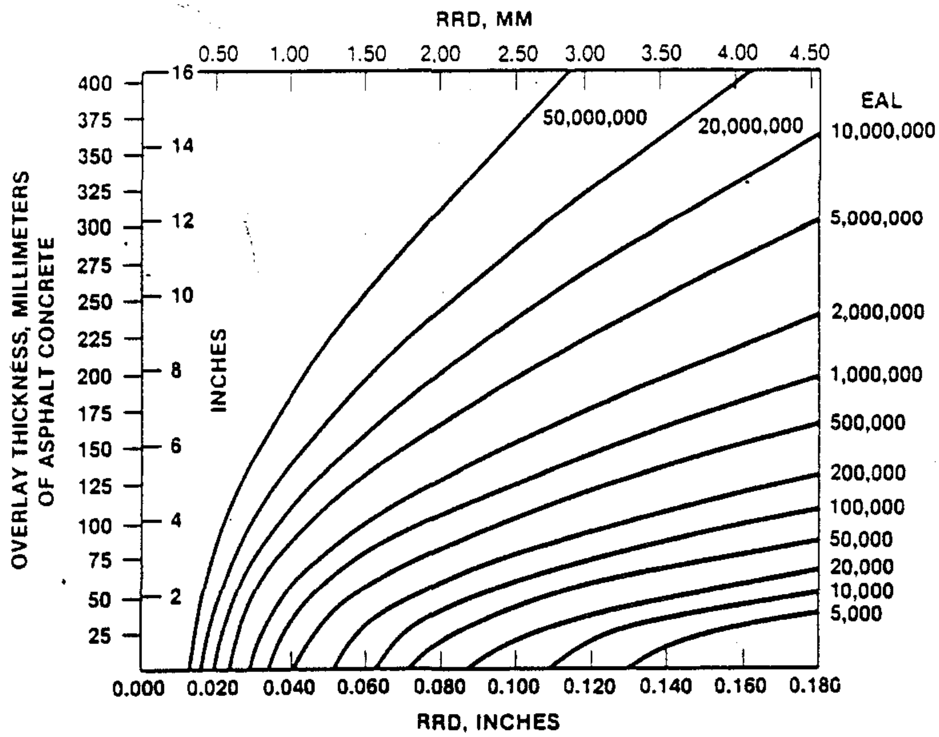

Figure C.1 Asphalt Concrete Overlay Thickness Required to Reduce Pavement Deflection From a Measured to a Design Deflection Value (Rebound Test), Asphalt Institite, 1983

$$\log E_{AC} = -4.13464 + 0.25726*(5.9/h_{AC}) + 0.92874*(5.9/h_B)^{0.5} \quad (C.4)$$

$$- 0.69727*(h_{AC}/h_B)^{0.5} - 0.96687*\log E_s$$

$$+ 1.88298*\log(PA_1/D_o^2)$$

$$= - 4.13462 + 0.25726(5.9/1.5 \text{ in.})$$

$$+ 0.92874(5.9/34 \text{ in.})^{0.5} - 0.69727(1.5/34)^{0.5}$$

$$- 0.96687 \log (147{,}716 \text{ psi})$$

$$+ 1.88298 \log [(10248 \text{ lb.})(96.2 \text{ mils})(1000)/(81)]$$

$$= 5.461066$$

$$E_{AC} = 289{,}112 \text{ psi.}$$

$$\log E_B = 0.50634 + 0.03474*(5.9/h_{AC}) + 0.12541*(5.9/h_B)^{0.5} \quad (C.5)$$

$$- 0.09416*(h_{AC}/h_B)^{0.5} + 0.51386*\log E_s$$

$$+ 0.25424*\log (PA_1/D_o^2)$$

$$= 0.50634 + 0.03474*(5.9/1.5) + 0.12541*(5.9/34)^{0.5}$$

$$- 0.09416*(1.5/34)^{0.5} + 0.51386*\log (147{,}716 \text{ psi})$$

$$+ 0.25424*\log [(10248)(96.2)(1000)/81]$$

$$= 5.133184$$

$$E_s = 135{,}889 \text{ psi.}$$

Then using ELSYM5 again, the tensile strain is calculated and this is found to be 29.5 microstrain. Next, using the fatigue relationship developed by the Asphalt Institute, the number of applications to failure with this strain may be calculated assuming a mix of 6% asphalt cement by volume and 3% air voids.

$$N_f = 18.4*C*(4.325 \times 10^{-3})*\epsilon_t^{-3.291}*E_{AC}^{-0.854} \quad \text{(C.6)}$$

$$M = 4.84[V_b/(V_v+V_b) - 0.69] \quad \text{(C.7)}$$

$$= - 0.1129$$

$$C = 10^M \quad \text{(C.8)}$$

$$= 0.7710$$

Then,

$$N_f = 1{,}078{,}756{,}917$$

With an actual EAL of 130,000 to date, the remaining life can be calculated using Miner's Hypothesis:

$$R_f = 1 - N_{actual}/N_f \quad \text{(C.9)}$$

$$= 1 - 130{,}000/1{,}078{,}756{,}917$$

$$= 99.9879\%$$

From this, we can say that the life remaining to the pavement (which is 99.9879%) must be able to withstand the predicted 20 year EAL of 1,800,000. The number of applications the pavement must withstand for a 50% reliability level is therefore:

$N_r = 1,800,000/0.999879 = 1,800,217$

For a 90% reliability level, $F_R = 4.25$ (see Appendix E), and the number of applications is:

$N_r = 1,800,000*4.25/0.999879 = 7,650,926$

For this to occur, the pavement tensile strain must not exceed a tolerable tensile strain (determined using Equation C.6 rearranged):

For 50%:

$$\log \epsilon_t = \log [N_r/(18.4*C*E_{AC}^{-0.854})]/(-3/291)$$

$$\epsilon_t = 1076\ \mu\text{strain}.$$

For 90%:

$$\epsilon_t = 693\ \mu\text{strain}.$$

In both cases, the tensile strain in the existing pavement does not exceed the tolerable strains calculated above, indicating that the pavement does not require an overlay. Table C.4 summarizes the results of this method for all the sections.

C.1.3 Fernando's Equations

The equations developed by Fernando et al. (1986) are discussed in detail in Chapter 2. MP 117.5 illustrates the calculations below. To obtain the tensile strain in the existing pavement, the following equation is used:

$$\begin{aligned} \log [\epsilon_t/\log (H_1+1)] &= -2.261 - 0.944 \log (S1-S3) \\ &\quad + 1.947 \log[(S1-S5)/S3] + 0.175\ (S1*H_2) \\ &\quad + 0.926 \log (S1*S3) \\ &= -2.261 - 0.944 \log (9-2.8/1000) \\ &\quad + 1.947 \log[(9-0.3)/2.8] + 0.175\ (9*34)/1000 \\ &\quad + 0.926 \log (9*2.8/1000000) \\ &= -\ 3.4231 \end{aligned} \qquad \text{(C.10)}$$

where: ϵ_t = 150.2 μstrain.

The performance equation developed by Austin Research Engineers is used to compute the tolerable strain for a predicted 20 year EAL of 1,800,000. Tensile strain was used rather than the compressive subgrade strain because fatigue was the predominant failure mode.

$$W_{18} = 9.73 \times 10^{-15} (\epsilon_t)^{-5.16} \tag{C.11}$$

Rearranging and solving for the tolerable tensile strain, we obtain:

$$\log \epsilon_t = \log [(1{,}800{,}000/9.73 \times 10^{-15})]/(-5.16)$$

$$= -3.927744$$

where: ϵ_t = 118 μstrain.

Since the tolerable strain of 118 is less than the existing strain of 150.2, an overlay is required. Assuming an overlay thickness of 2.5 in. (6.4 cm), the strain with the overlay is:

$$\log (\epsilon_t)_{ov} = -\ 0.689 + 0.793 \log \epsilon_t$$

$$-\ 0.041\ (H_{ov}+H_1)^{0.5} - 0.057H_{ov} \tag{C.12}$$

$$= -\ 0.689 + 0.793 \log (150.2/1000000)$$

$$-\ 0.041\ (2.5 + 1.5 \text{ in.})^{0.5} - 0.057(2.5 \text{ in.})$$

$$= -\ 3.945401$$

$$= 113.2\ \mu\text{strain}$$

This is less than the tolerable strain of 118, so the 2.5 in. (6.4 cm) overlay satisfies the requirements. If the strain had still exceeded the tolerable strain, continue iterating with additional thicknesses of overlay (to the nearest half inch) until the tolerable strain is met. Table C.5 summarizes the results of these equations for all sections.

C.1.4 <u>Mechanistic</u>

This is a similar procedure to that of Newcomb's method except that the layer moduli are derived from backcalculation rather than from regression equations. Two backcalculation programs, ELSDEF and BISDEF were used to obtain the layer moduli. As discussed in Chapter 4, there were problems with BISDEF when the program stopped calculating. Therefore, ELSDEF was used instead. Figures C.2 and C.3 are the inputs and outputs for MP 117.5, respectively.

From Figure C.3, the layer moduli are 399, 71 and 120 ksi for the AC, base and subgrade layers, respectively. With these values, ELSYM5 is used to obtain an existing pavement tensile strain of 106 microstrain. As before, the procedure in Section C.1.2 is followed, and the results are:

N_f = 12,200,000

R_f = 98.934426%

N_r = 1,819,387 (50% reliability level)

N_r = 7,732,394 (90% reliability level)

The tolerable strains calculated from ELSYM5 are:

ϵ_t = 188.9 μst (50% reliability level)

ϵ_t = 121.7 μst (90% reliability level)

Again, the existing pavement strain of 106 is less than the tolerable strains calculated above, indicating that an overlay is not needed. If the tolerable

Table C.5. Summary of Results Using Fernando's Equations for Sterling Highway.

Location	ϵ_t	Overlay Thickness*	ϵ_{ov}
117.50	150	2.5	113.2
118.00	249	5.0	115.8
118.50	281	5.5	113.4
119.00	268	5.5	107.6
119.50	280	5.5	112.9
120.00	236	5.0	110.9
120.50	407	8.0	109.7
121.00	340	7.0	110.1
121.50	436	8.0	115.8
122.00	423	8.0	113.1
122.50	365	7.0	116.5
123.00	462	>8.0	-
123.50	272	5.5	109.4
124.00	328	6.5	115.2
125.00	320	6.5	113.0
125.50	384	7.5	112.7
126.00	147	2.5	111.5
126.50	457	>8.0	-
127.00	392	7.5	114.5
127.50	352	7.0	113.2
128.00	288	5.5	116.5
128.50	230	5.0	117.2
129.00	397	7.5	115.7
129.50	886	>8.0	-

*Overlay thicknesses are in inches (1 in. = 2.54 cm).
Strains are in microstrain.

```
010 1
Sterling Highway Station 117.5
030 7,9,5.4,2.8,1,.3,.3,.4
040 3,10,5
050 1,80000,2000000
060 2,5000,150000
070 3,1000,1000000
PROBLEM
LOADS     1    10248. 93.4
   0.   0.
NLAYER    3
LAYER     1   400312.  1.5  .35
LAYER     2    20000. 13.5  .35
LAYER     3    147716  75.   .4FF
XYOUT     7
   0.   0.   0. 7.89   0. 11.8   0. 17.7   0. 25.6   0. 35.4   0. 47.2
ZOUT      1
0.
END
```

Figure C.2. ELSDEF Input for MP 117.5, Sterling Highway.

THE NUMBER OF PROBLEMS TO BE SOLVED IS 1

PROBLEM NUMBER 1

Sterling Highway Station 117.5

NUMBER OF VARIABLE LAYERS AND TARGET DEFLECTIONS = 3

DEFLECTION READINGS IN MILS

POSITION NO:	1	2	3	4	5	6	7
DEFLECTIONS:	9.000	5.400	2.800	1.000	.300	.300	.400
WEIGHTING FACTOR:	.111	.185	.357	1.000	3.333	3.333	2.500

VARIABLE LAYER NO	SYSTEM LAYER NO	VALUE OF MAXMUM MODULUS	VALUE OF MINIMUM MODULUS
1	1	2000000.0	80000.0
2	2	150000.0	5000.0
3	3	1000000.0	1000.0

ELSYM5 - FIVE LAYERED ELASTIC SYSTEM - VERSION 4.5
LATEST REVISION: 81/02/07 - P. R. JORDAHL BRENT RAUHUT ENGINEERING, INC.

SYSTEM NUMBER 0

NUMBER OF ELASTIC LAYERS = 3
NUMBER OF LOAD LOCATIONS = 1
NUMBER OF OUTPUT LOCATIONS= 7
NUMBER OF OUTPUT DEPTHS = 1

LAYER	ELASTIC MODULUS	POISSONS RATIO	THICKNESS (IN.)
1	400312.	.350	1.500
2	20000.	.350	13.500
3	147716.	.400	75.000

LOAD DESCRIPTION:

LOAD FORCE = 10248.
TIRE PRESSURE = 93.
LOAD RADIUS = 5.91

LOADS LOCATED AT:

LOAD	X	Y
1	.000	.000

Figure C.3. ELSDEF Output for MP 117.5, Sterling Highway.

RESULTS REQUESTED FOR SYSTEM LOCATION(S)

X-Y POINT(S)

X	Y
.00	.00
.00	7.89
.00	11.80
.00	17.70
.00	25.60
.00	35.40
.00	47.20

DEPTHS = .00

POSITION	DEFLECTION	MEASURED	DIFFERENCE	% DIFF.
1	28.8912	9.0000	-19.8912	-221.0
2	10.6668	5.4000	-5.2668	-97.5
3	3.9277	2.8000	-1.1277	-40.3
4	.8962	1.0000	.1038	10.4
5	.2147	.3000	.0853	28.4
6	.1347	.3000	.1653	55.1
7	.0932	.4000	.3068	76.7
		ABSOLUTE SUM:	26.9469	529.4259
		ARITHMETIC SUM:		-188.2197

DATA FOR DEVELOPING EQUATIONS FOR ITERATION NO. 1

LAYER NO.	INITIAL MODULUS	CHANGED MODULUS	OFFSET DISC.	DEFLECTIONS INITIAL	DEFLECTIONS CHANGED	READINGS
1	400312.	2000000.	.00	28.891	21.441	9.000
			7.89	10.667	10.609	5.400
			11.80	3.928	5.061	2.800
			17.70	.896	1.298	1.000
			25.60	.215	.178	.300
			35.40	.135	.100	.300
			47.20	.093	.089	.400
2	20000.	150000.	.00	28.891	5.680	9.000
			7.89	10.667	2.362	5.400
			11.80	3.928	1.374	2.800
			17.70	.896	.788	1.000
			25.60	.215	.446	.300
			35.40	.135	.242	.300
			47.20	.093	.122	.400
3	147716.	1000000.	.00	28.891	27.506	9.000
			7.89	10.667	9.451	5.400
			11.80	3.928	2.894	2.800
			17.70	.896	.167	1.000
			25.60	.215	-.198	.300
			35.40	.135	-.065	.300
			47.20	.093	.002	.400

Figure C.3. ELSDEF Output for MP 117.5, Sterling Highway (cont'd).

PREDICTED E DISREGARDING BOUNDARY CONDITIONS

351515. 106274. 79146.

POSITION	DEFLECTION	MEASURED	DIFFERENCE	% DIFF.
1	8.4331	9.0000	.5669	6.3
2	3.8137	5.4000	1.5863	29.4
3	2.3610	2.8000	.4390	15.7
4	1.4380	1.0000	-.4380	-43.8
5	.8471	.3000	-.5471	-182.4
6	.4678	.3000	-.1678	-55.9
7	.2352	.4000	.1648	41.2
		ABSOLUTE SUM:	3.9099	374.6419
ARITHMETIC SUM:		-189.5145		
		AVERAGE:	53.5203	53.5203

DATA FOR DEVELOPING EQUATIONS FOR ITERATION NO. 2

LAYER NO.	INITIAL MODULUS	CHANGED MODULUS	OFFSET DISC.	DEFLECTIONS INITIAL	DEFLECTIONS CHANGED	READINGS
1	351515.	363126.	.00	8.433	8.418	9.000
			7.89	3.814	3.814	5.400
			11.80	2.361	2.360	2.800
			17.70	1.438	1.437	1.000
			25.60	.847	.847	.300
			35.40	.468	.468	.300
			47.20	.235	.235	.400
2	106274.	115836.	.00	8.433	7.940	9.000
			7.89	3.814	3.652	5.400
			11.80	2.361	2.307	2.800
			17.70	1.438	1.429	1.000
			25.60	.847	.851	.300
			35.40	.468	.472	.300
			47.20	.235	.237	.400
3	79146.	92508.	.00	8.433	8.134	9.000
			7.89	3.814	3.537	5.400
			11.80	2.361	2.113	2.800
			17.70	1.438	1.246	1.000
			25.60	.847	.718	.300
			35.40	.468	.393	.300
			47.20	.235	.197	.400

PREDICTED E DISREGARDING BOUNDARY CONDITIONS

395818. 76171. 110465.

Figure C.3. ELSDEF Output for MP 117.5, Sterling Highway (cont'd).

POSITION	DEFLECTION	MEASURED	DIFFERENCE	% DIFF.
1	10.0155	9.0000	-1.0155	-11.3
2	3.9891	5.4000	1.4109	26.1
3	2.0859	2.8000	.7141	25.5
4	1.0776	1.0000	-.0776	-7.8
5	.5748	.3000	-.2748	-91.6
6	.3084	.3000	-.0084	-2.8
7	.1563	.4000	.2437	60.9
		ABSOLUTE SUM:	3.7449	225.9668
		ARITHMETIC SUM:		-.8707
		AVERAGE:	32.2810	32.2810

DATA FOR DEVELOPING EQUATIONS FOR ITERATION NO. 3

LAYER NO.	INITIAL MODULUS	CHANGED MODULUS	OFFSET DISC.	DEFLECTIONS INITIAL	DEFLECTIONS CHANGED	READINGS
1	395818.	593443.	.00	10.016	9.781	9.000
			7.89	3.989	4.028	5.400
			11.80	2.086	2.100	2.800
			17.70	1.078	1.077	1.000
			25.60	.575	.574	.300
			35.40	.308	.309	.300
			47.20	.156	.156	.400
2	76171.	82784.	.00	10.016	9.393	9.000
			7.89	3.989	3.786	5.400
			11.80	2.086	2.025	2.800
			17.70	1.078	1.072	1.000
			25.60	.575	.580	.300
			35.40	.308	.312	.300
			47.20	.156	.158	.400
3	110465.	191611.	.00	10.016	9.255	9.000
			7.89	3.989	3.307	5.400
			11.80	2.086	1.492	2.800
			17.70	1.078	.639	1.000
			25.60	.575	.303	.300
			35.40	.308	.162	.300
			47.20	.156	.085	.400

Figure C.3. ELSDEF Output for MP 117.5, Sterling Highway (cont'd).

PREDICTED E DISREGARDING BOUNDARY CONDITIONS
398610. 70620. 119918.

POSITION	DEFLECTION	MEASURED	DIFFERENCE	% DIFF.
1	10.4812	9.0000	-1.4812	-16.5
2	4.0656	5.4000	1.3344	24.7
3	2.0373	2.8000	.7627	27.2
4	1.0019	1.0000	-.0019	-.2
5	.5190	.3000	-.2190	-73.0
6	.2774	.3000	.0226	7.5
7	.1415	.4000	.2585	64.6
		ABSOLUTE SUM:	4.0803	213.7532
		ARITHMETIC SUM:		34.4836
		AVERAGE:	30.5362	30.5362

THE FINAL MODULUS VALUES ARE

398610. 70620. 119918.
CHANGE IN MODULUS VALUES ARE IN TOLERANCE
***** END OF PROGRAM *****

Figure C.3. ELSDEF Output for MP 117.5, Sterling Highway (cont'd).

strains had been exceeded, then iterations with additional thicknesses of overlay would proceed until the tolerable strains are met. Again, the trial overlays selected should be rounded up to the nearest half inch. Table C.6 summarizes the results of this procedure for all sections.

C.2 Seward Highway

A similar process to that for Sterling Highway was undertaken here as well. For Seward Highway, the critical time of the year was May 3rd (Table C.7). FWD deflections from April to May 1986 were used. The section from 36th Avenue to Benson Blvd. was selected for analysis. This section had a variety of AC thicknesses ranging from 2.25 to 3.5 in. (5.7 to 8.9 cm). It was assumed that there existed a base and subbase of 34 in. (86 cm) similar to that found in Sterling Highway since no other data were available. Again, thaw depths were assumed to be found 15 in. (38 cm) below the surface. The modified Berggren equation yielded the same thaw depth as that for Sterling Highway since the same climate and pavement conditions were assumed. This was 30 in. (74 cm).

This highway was constructed in 1969 and has had approximately 4.4 million EALs since then. This is a conservative figure according to ADOT&PF. More accurate values are not available due to the lack of historical traffic data. The predicted 10 year EAL is 5,083,000. Table C.8 present the deflection basins that were used in the following sections.

C.2.1 Asphalt Institute

The RRD for this section was found to be 25.74 mils (654 μm) from Table C.8. From Figure C.1, and with an EAL of 5,083,000, the 50% reliability level

Table C.6. Summary of Results Using ELSDEF for Sterling Highway.

Location	ϵ_t	N_f	R_f	Allowable N_r		Overlay Thickness		Tolerable Strain	
				50%[1]	90%	50%	90%	50%	90%
117.5	106	1.22E+07	0.99	1.82E+06	7.73E+06	0	0	188.90	121.70
118.0	265	2.93E+05	0.56	3.24E+06	1.38E+07	4	6.5	127.64	82.23
118.5	231	1.18E+06	0.89	2.02E+06	8.60E+06	5.5	8	196.08	126.33
119.0	220	5.15E+05	0.75	2.41E+06	1.02E+07	-	-	137.70	88.72
119.5	231	1.29E+06	0.90	2.00E+06	8.51E+06	5.5	8	202.22	130.28
120.0	270	2.42E+05	0.46	3.90E+06	1.66E+07	4.5	6.5	116.01	74.74
120.5	344	2.96E+05	0.56	3.21E+06	1.37E+07	6.5	>8	166.60	107.33
121.0	371	9.24E+04	-0.41	-4.43E+06	-1.88E+07	-	-	-	-
121.5	179	2.79E+06	0.95	1.89E+06	8.02E+06	4.5	7	201.58	129.87
122.0	394	7.67E+04	-0.69	-2.59E+06	-1.10E+07	-	-	-	-
122.5	38.8	3.48E+08	1.00	1.80E+06	7.65E+06	1	1	192.01	123.70
123.0	434	4.65E+04	-1.80	-1.00E+06	-4.26E+06	-	-	-	-
123.5	294	4.67E+05	0.72	2.49E+06	1.06E+07	5.5	8	176.72	113.86
124.0	385	7.97E+04	-0.63	-2.85E+06	-1.21E+07	-	-	-	-
124.5	308	3.95E+05	0.67	2.68E+06	1.14E+07	5.5	8	172.03	110.83
125.0	295	2.09E+05	0.38	4.74E+06	2.02E+07	4.5	7	114.31	73.64
125.5	365	2.48E+05	0.48	3.78E+06	1.61E+07	6.5	>8	159.64	102.85
126.0	148	2.09E+06	0.94	1.92E+06	8.16E+06	2.5	5	151.98	97.92
127.0	395	6.00E+04	-1.17	-1.54E+06	-6.55E+06	-	-	-	-
127.5	327	2.64E+05	0.51	3.54E+06	1.51E+07	5.5	8	148.61	95.74
128.0	281	2.19E+05	0.41	4.43E+06	1.88E+07	4.5	6.5	112.69	72.60
128.5	114	1.12E+07	0.99	1.82E+06	7.74E+06	2	5.5	198.12	127.64
129.0	388	8.12E+04	-0.60	-2.99E+06	-1.27E+07	-	-	-	-
129.5	246	1.43E+06	0.91	1.98E+06	8.41E+06	0	0	223.05	143.70

[1]These are the reliability levels as defined by AASHTO.
Strain is measured in microstrain.
Overlay thicknesses are in inches (1 in. = 2.54 cm)

Table C.7. Deflection Basins For TH1 To Determine Critical Period, Seward Highway.

				FWD Deflection Readings (mils)[2]								Layer Moduli (ksi)			
Date	Temp. °F	Cf[1]	Load lb	S1	S1[3]	S2	S3	S4	S5	S6	S7	E1[4]	E2	E3	ϵ_t[5]
4/16	63	1.055	10720	12	12.66	8.7	6.1	3.7	1.6	0.6	0.4	300	100	30	17.2
4/24	54	1.13	10200	12.5	14.12	9	6.1	3.9	2.1	1	0.6	461	100	30	24.2
5/3	46.9	1.73	9456	11.5	19.90	8.6	5.9	3.7	2.1	1.2	0.7	640	100	30	39.0
5/10	45	1.75	5816	9.2	16.10	7	4.8	3	1.6	1	0.6	697	100	30	39.0

[1]The temperature correction factor is developed in Appendix D.
[2]FWD sensor locations are at 0, 7.87, 11.8, 17.7, 25.6, 35.4 and 47.2 inches.
[3]The S1 deflection reading has been corrected to 70 F and normalized for a 9000 lb. load.
[4]E_1 is derived from AMOD at the measured temperatures.
[5]ϵ_t is measured in microstrain.

Table C.8. FWD Deflection Basins for Seward Highway.

Location	Temp. °F	Cf[1]	Load	FWD Deflection Basins (mils) S1	S1[2]	S2	S3	S4	S5	S6	S7
TH1	46.9	1.73	9456	11.5	19.90	8.6	5.9	3.7	2.1	1.2	0.7
TH2	55	1.083	9304	11.5	12.45	8.5	6.2	4.1	2.5	1.8	1
TH3	53.1	1.3	9016	13.7	17.81	11.6	9.3	5.9	4.2	2.2	0.7
TH4	55	1.083	9056	15.5	16.79	12.1	7.7	4.9	2.8	2	1.3
TH45	53.1	1.3	8952	19.3	25.09	14.5	10.4	6.8	4.1	2.6	1.5
TH34	53.1	1.3	9072	14	18.20	11.3	8.5	6.1	3.6	2.1	1.2
TH35	53.1	1.3	9080	12.9	16.77	9.1	6.2	3.8	2.2	1.4	0.8

[1]The temperature correction factor is developed in Appendix D.
[2]This deflection reading has been corrected to 70 F and normalized to a 9000 lb load. The mean deflection is 18.14 mils, with a standard deviation of 3.82 mils.

overlay was 2 in. (5 cm). For a 90% reliability level, the overlay is 3 in. (7.6 cm).

C.2.2 Newcomb's Equations

As before, the layer moduli were calculated from the regression equations. Using TH1 as an example, they were 247, 88, and 78 ksi, respectively. From this and using ELSYM5, the existing pavement tensile strain was 116 μstrain. Continuing with the calculations as before;

$N_f = 13,625,858.$

$R_f = 1 - 4,400,000/13,625,858 = 67.78\%$

$N_r = 5,083,000/0.6778 = 7,499,262$ (50%)

$N_r = 31,871,865$ (90%)

Next, the tolerable strains are calculated and for TH1, this is 190 μstrain for the 50% reliability level. In this case, no overlay is needed. However, for the 90% level, the tolerable strain is 90, and an overlay of 8 in. (20 cm) is needed to reduce the existing strains to this level. Table C.9 summarizes the results of this procedure for all sections. Note that for sections TH3, Th4, TH45 and TH34, the remaining life is negative. This indicates that the traffic to date (4.4 million) has already exceeded the fatigue life of the pavement and no amount of overlays are sufficient. Instead, reconstruction is probably needed for these sections.

C.2.3 Fernando's Equations

For TH1, the existing pavement tensile strain was found to be 130 μstrain using the same equations as before. The tolerable strain for the given EAL of 5,083,000 is 97. Through a process of iteration, it was found that an overlay

Table C.9. Summary of Results Using Newcomb's Equations, Seward Highway.

Location	h_{AC} in.	A_1 mils	E_{AC} ksi	E_B ksi	E_s ksi	ϵ_t	EALs	R_f	Allowable N_r 50%	Allowable N_r 90%	Tolerable Strain 50%	Tolerable Strain 90%	Overlay (in.) 50%	Overlay (in.) 90%
TH1	2.25	177.2	247	88	78	116	1.36E+07	0.68	7.48E+06	3.19E+07	139	89.6	0	8
TH2	2	187.4	307	76	54	127	8.40E+06	0.48	1.07E+07	4.54E+07	118	84.7	6	>8
TH3	3.5	262.4	198	79	74	175	4.25E+06	-0.03	-1.47E+08	-6.24E+08	-	-	-	-
TH4	2.25	241.8	252	58	40	190	2.64E+06	-0.67	-7.63E+06	-3.24E+07	-	-	-	-
TH45	3.5	312.2	131	46	34	298	1.05E+06	-3.19	-1.59E+06	-6.77E+06	-	-	-	-
TH34	2.25	247.6	359	64	44	177	2.46E+06	-0.79	-6.47E+06	-2.75E+07	-	-	-	-
TH35	2.5	190.6	169	75	65	145	9.04E+06	0.51	9.90E+06	4.21E+07	141	98.8	6	>8

Note: The negative remaining life indicates that an overlay is inappropriate. The traffic has exceeded the pavement's fatigue life.

of 3.5 in. (8.9 cm) was sufficient to reduce the strains to below this level. Table C.10 summarizes the results of this method.

C.2.4 Mechanistic

Again, the moduli from BISDEF did not converge and were not used. ELSDEF results were used instead. For TH1, the backcalculated layer moduli were 1038, 52,24 ksi for the AC, base and subgrade layers, respectively. Existing pavement strain was 196 μstrain.

$N_f = 712{,}000$

$R_f = 1 - 4{,}400{,}000/712{,}000 = -5.18$

The negative remaining life for both the 50% and 90% reliability levels indicate that the fatigue life of the pavement has been exceeded. This is true of all the sections. Therefore, it is recommended that a complete reconstruction be performed instead of an overlay. Table C.11 summarizes the results of this procedure for all sections analyzed.

C.3 Parks Highway

There are two sections in the Parks Highway project. The North section begins 10 miles (16 km) west of Fairbanks near Ester and extends 24.4 miles (39 km) to the southwest. The South section begins 83 miles (134 km) further at Dragonfly Creek and ends 11.2 miles (18 km) later at McKinley Village.

Since no data were available on the pavement structure, it was assumed that there existed a 2 in. (5 cm) asphalt concrete surface, with a 36 in. (91 cm) combined base, subbase and select material. These dimensions were provided by ADOT&PF as "typical" sections for roads in the Fairbanks region. For the partially frozen case, the thaw depth was again assumed to be found 15 in. (38 cm) below the surface. Using the modified Berggren equation, the thaw

Table C.10. Summary of Results Using Fernando's Equations for Seward Highway.

Location	ϵ_t	Overlay[1]	ϵ_{ov}
TH1	130	3.5	85.4
TH2	111	2	95.3
TH3	158	4	90.4
TH4	172	5	85.2
TH45	250	6	96.9
TH34	133	4.5	69.5
TH35	159	4	92.5

[1]Overlay thicknesses are in in. (1 in.=25.4 cm).
Strains are measured in microstrain.

Table C.11. Summary of Results Using ELSDEF for Seward Highway.

Location[1]	ϵ_t	Nf	Rf[2]	Allowable Nr[3] 50%	90%	Tolerable Strain[4]
TH1	196	7.12E+05	-5.18	-9.81E+05	-4.17E+06	-
TH2	172	1.67E+06	-1.64	-3.10E+06	-1.32E+07	-
TH4	278	4.71E+05	-8.34	-6.09E+05	-2.59E+06	-
TH45	354	1.95E+05	-21.59	-2.35E+05	-1.00E+06	-
TH35	241	5.60E+05	-6.86	-7.41E+05	-3.15E+06	-

[1] Two sections, TH3 and TH34 are not included because moduli did not reach tolerance after 9 iterations.
[2] The remaining life is negative indicating that past traffic has exceeded the fatigue life of the pavement.
[3] Since the remaining life is negative, the allowable traffic is also negative.
[4] The tolerable strain cannot be calculated because of the negative values.

depth was approximately 20 in. (51 cm). Again, the estimates made of the climate and soils may have been inaccurate, so a 15 in. (38 cm) thaw depth was used.

The format of the deflection basin data available for this project lent itself to the easy determination of the area factors and the corrected center-of-load deflections. Therefore, it was possible to use these criteria to determine critical sections for analysis for Parks Highway. ELSYM5 was used to calculate the tensile strains at the bottom of the asphalt concrete layer for deflection basins measured at different times of the year, from April 28th to June 24th. Table C.12 show the results of this analysis. For the South section, two sections were chosen, CDS 206.8 and 198. They represented the highest and lowest mean center deflection respectively. It was found that the critical time of the year was May 6th and April 28th, respectively. For the North section, two locations were also chosen, CDS 293.8 and 304.2. For the latter pair, the critical times of the year were April 23rd and April 19th, respectively.

Once the critical times of the year had been determined, the analysis continued in the same manner as before. Table C.13 presents the deflection basins that were selected for analysis. 1982 deflections were used because this was the data that was supplied by ADOT&PF.

Historical traffic data were not available for this section, but through a procedure of extrapolation, it is assumed that the South section has had a cumulative EALs of 76,069 since its paving in 1974. The predicted 20 year EALs are 345,526. For the North section, which was paved in 1975, the EALs are 78,723 and 390,521, respectively.

Table C.12. Deflection Basins to Determine Critical Period, Parks Highway.

					FWD Deflection Readings (mils)[2]									
Date	CDS	Temp. °F	Cf[1]	Load lb	S1[3]	S1	S2	S3	S4	S5	S6	S7	E1[4] ksi	ϵ_t[5]
4/28	206.8	65	1.029	9349	62.63	60.87	34.92	16.10	1.73	0.08	0.04	0.24	272	80.8
5/6	206.8	60	1.059	9221	21.18	20.00	11.26	6.14	2.13	0.51	0.24	0.24	347	88.9
5/25	206.8	64	1.03	9508	17.03	16.54	9.06	5.04	2.52	1.26	0.83	0.67	286	82.6
6/1	206.8	79	0.925	8982	27.09	29.29	8.62	5.12	2.68	1.38	0.83	0.71	193	67.7
6/9	206.8	68	1.02	9173	14.58	14.29	8.27	5.08	2.76	1.65	1.10	0.87	235	75.4
6/24	206.8	95	0.83	8966	12.48	15.04	8.23	5.16	3.11	2.13	1.46	1.02	116	46.4
4/28	198	67	1.02	9970	16.95	16.61	8.43	4.02	1.42	0.39	0.12	0.08	247	77.3
5/6	198	68	1.02	9603	17.39	17.05	9.72	5.47	2.83	1.18	0.35	0.51	235	75.4
5/25	198	70	1	9524	16.77	16.77	9.92	6.06	3.46	2.05	1.22	0.75	213	71.6
6/1	198	72	0.975	9635	16.70	17.13	9.13	5.63	3.23	1.89	1.18	0.79	193	67.7
6/9	198	68	1.02	9253	16.18	15.87	9.37	5.98	3.31	2.05	1.26	0.91	235	75.4
6/24	198	82	0.91	9030	13.47	14.80	9.02	5.51	3.07	1.93	1.18	0.83	116	46.6
4/23	293.8	48	1.72	9460	44.56	25.91	16.42	9.49	3.78	1.02	0.35	0.39	608	111
4/26	293.8	66	1.02	8887	28.11	27.56	16.46	9.53	4.29	1.73	0.83	0.51	259	79.1
5/3	293.8	62	1.05	9269	29.89	28.46	17.99	11.22	5.75	2.72	1.34	0.83	315	85.9
5/19	293.8	73	0.97	8934	27.04	27.87	17.91	11.65	6.69	3.54	2.20	1.46	183	65.6
5/27	293.8	75	0.953	8600	80.67	84.65	21.61	13.70	7.44	3.82	2.36	1.50	165	61.3
6/8	293.8	76	0.95	8616	31.49	33.15	21.50	14.21	7.72	4.33	2.60	1.77	157	59.2
6/23	293.8	82	0.91	8536	30.45	33.46	21.57	14.25	7.60	4.25	2.76	1.93	116	46.6
4/19	293.8	36	1.835	8584	46.96	25.59	15.94	10.00	6.30	2.36	0.94	0.59	1035	109
6/22	293.8	90	0.87	8696	29.05	33.39	21.02	12.95	7.64	4.96	2.87	2.13	76	29.6
4/23	304.2	51	1.54	10240	7.52	4.88	2.01	0.63	0.04	0.12	0.08	0.12	530	110
4/26	304.2	64	1.03	9731	12.41	12.05	6.34	3.31	1.26	0.35	0.12	0.16	286	82.6
5/3	304.2	59	1.063	9635	23.27	21.89	13.82	8.98	5.16	2.80	1.22	0.47	364	90.4
5/10	304.2	73	0.97	9189	20.32	20.94	13.82	9.02	5.51	3.19	1.77	0.87	183	65.6
5/19	304.2	66	1.02	9142	17.71	17.36	11.54	7.76	5.00	3.07	1.77	1.02	259	79.1
5/27	304.2	69	1.01	9205	22.63	22.40	10.94	7.48	4.88	3.11	1.89	1.14	223	73.4
6/8	304.2	75	0.953	9078	14.71	15.43	9.88	6.97	4.72	3.23	2.05	1.30	165	61.3
6/23	304.2	80	0.923	8998	12.50	13.54	9.09	6.65	4.69	3.23	2.09	1.38	128	50.7
4/19	304.2	35	1.843	8966	16.04	8.70	5.39	3.39	2.24	0.98	0.51	0.31	1080	109
6/22	304.2	90	0.87	8839	10.79	12.40	8.43	6.26	4.65	3.46	2.09	1.30	76	29.6

[1]The temperature correction factor is developed in Appendix D.
[2]FWD sensors are located at 0, 7.87, 11.8, 17.7, 25.6, 35.4 and 47.2 inches.
[3]The S1 deflection reading has been corrected to 70F.
[4]E_1 is derived from AMOD at the measured temperature. E_2 & E_3 assumed to be 100 & 30 ksi, respectively.
[5]ϵ_t is measured in microstrain.
1 in. = 25.4 cm; 1 lb. = 4.45 N; 1 ksi = 6.89 MPa

Table C.13. FWD Deflection[1] Basins for Parks Highway.

CDS	Load lb	S1[2]	S1	S2	S3	S4	S5	S6	S7	Mean Deflection	Standard Deviation
206	9810	24.10	24.80	16.50	11.18	5.67	2.48	0.87	0.24	23.51	1.47
206.2	9333	23.96	23.46	13.90	8.23	3.86	1.54	0.59	0.08		
206.4	9221	23.89	23.11	12.99	7.40	3.90	1.93	0.87	0.24		
206.6	9540	24.94	25.08	14.76	9.21	4.57	1.85	0.67	0.24		
206.8	9221	20.67	20.00	11.26	6.14	2.13	0.51	0.24	0.24		
198	9970	15.30	16.61	8.43	4.02	1.42	0.39	0.12	0.08	16.55	3.13
198.2	9492	13.25	13.70	7.87	4.53	1.26	0.51	0.08	0.08		
198.4	10017	14.43	15.75	10.75	7.20	4.06	1.81	0.59	0.31		
198.6	9572	17.63	18.39	11.61	7.48	3.62	1.42	0.94	0.63		
198.8	9396	22.12	22.64	13.31	5.87	2.28	0.87	0.71	0.39		
293	9810	25.59	16.22	9.02	4.53	1.46	0.55	0.24	0.24	35.50	8.20
293.2	9842	23.10	14.69	8.27	3.94	1.14	0.39	0.31	0.35		
293.4	8473	42.21	21.65	12.83	7.64	4.41	1.26	0.59	0.35		
293.5	8743	42.98	22.76	14.29	9.13	5.79	2.17	0.63	0.31		
293.6	9253	37.08	22.17	14.61	8.46	3.90	1.69	0.83	0.55		
293.8	9460	42.39	25.91	16.42	9.49	3.78	1.02	0.35	0.39		
304	8919	50.89	27.36	17.83	12.09	8.39	3.54	1.22	0.51	29.81	23.97
304.2	8966	16.10	8.70	5.39	3.39	2.24	0.98	0.51	0.31		
304.4	10432	10.51	7.91	3.82	1.46	0.31	0.51	0.12	0.12		
304.5	8966	73.56	39.76	23.98	7.28	2.24	0.08	0.08	0.16		
304.6	10017	10.68	7.72	2.83	0.55	0.12	0.08	0.08	0.39		
304.8	10272	17.10	12.68	4.80	1.34	0.16	0.16	0.24	0.20		

[1] All deflection readings are in mils.
[2] S1 has been corrected to 70 F and normalized to 9000 lb.
1 mil = 0.0254 mm
1 lb. = 4.45 N

C.3.1 Asphalt Institute

For the South section, the RRD (from Table C.13) is 26.5 mils (0.67 mm) for CDS 206 to 206.8 and 22.8 mils (0.58 mm) for CDS 198 to 198.8. With the predicted EALs of 345,526, Figure C.1 indicates that no overlay is needed. For the North section, with a RRD of 52 mils (1.3 mm) for CDS 293 to 293.6 and 77.8 mils (2.0 mm) for CDS 304.2 to 304.8, the overlay required is 2 and 3 in. (5 and 7.6 cm), respectively.

C.3.2 Newcomb's Equations

Table C.14 present the results obtained with Newcomb's regression equations. Note that the moduli for the AC and base layers are low. This has caused problems in that additional thicknesses of AC actually INCREASE the tensile strains rather than decrease them. Therefore, it was concluded that the layer moduli are inaccurate and that these regression equations are not applicable to the conditions described by the deflection basins. In particular, this would indicate strongly that the effects of frozen layers in the pavement structure are not considered in the model.

In any case, Table C.14 shows that there is either no need for overlays or when overlays are needed, the layer moduli are such that additional thicknesses of AC increase the tensile strains.

C.3.3 Fernando's Equations

As before for Sterling and Seward Highways, the existing pavement strains are first calculated. Using CDS 293 as an example, the existing tensile strain is 330 μstrain. The performance equation indicates that the tolerable strain is 159 μstrain for an EAL of 345,526. An overlay of 4.5 in. (11.4 cm)

Table C.14. Summary of Results Using Newcomb's Equations for Parks Highway.

CDS	A_1 mils	E_{AC} ksi	E_B ksi	E_s ksi	ϵ_t	N_f EALs	R_f	Allowable N_r 50%	Allowable N_r 90%	Tolerable Strain 50%	Tolerable Strain 90%
206	322.60	18	22	240	205	1.96E+07	1.00	3.47E+05	1.47E+06	698.3	449.9
206.2	265.04	5	37	684	10.4	1.07E+12	1.00	3.46E+05	1.47E+06	974.8	628.0
206.4	257.95	15	21	225	191	2.89E+07	1.00	3.46E+05	1.47E+06	732.4	471.8
206.6	288.82	14	21	233	180	3.72E+07	1.00	3.46E+05	1.47E+06	745.8	480.5
206.8	204.25	23	21	225	362	2.44E+06	0.97	3.57E+05	1.52E+06	649.7	418.6
198	154.57	7	40	730	10.4	8.00E+11	1.00	3.46E+05	1.47E+06	893.3	575.5
198.2	144.88	13	42	695	36.7	7.43E+09	1.00	3.46E+05	1.47E+06	760.7	490.1
198.4	210.39	62	22	183	413	6.79E+05	0.89	3.89E+05	1.65E+06	489.2	315.2
198.6	225.75	75	14	88	756	7.90E+04	0.04	9.47E+06	4.02E+07	176.5	113.7
198.8	231.97	31	16	138	572	4.20E+05	0.82	4.22E+05	1.79E+06	571.4	368.1
293	163.23	35	23	240	371	1.58E+06	0.95	4.11E+05	1.75E+06	558.1	359.5
293.2	146.46	62	19	160	547	2.69E+05	0.71	5.52E+05	2.34E+06	440.0	283.4
293.4	243.39	33	16	138	583	3.74E+05	0.79	4.95E+05	2.10E+06	535.7	345.1
293.5	279.61	32	17	160	536	5.07E+05	0.84	4.62E+05	1.96E+06	551.1	355.1
293.6	269.29	61	14	97	774	8.72E+04	0.10	4.03E+06	1.71E+07	241.4	155.5
293.8	292.44	29	15	139							
304	362.99	43	13	100							
304.2	109.13	211	23	164	481	1.45E+05	0.46	8.58E+05	3.64E+06	280.0	180.4
304.4	73.07	63	40	509	217	5.57E+06	0.99	3.96E+05	1.68E+06	484.5	312.2
304.5	362.60	3	20	328							
304.6	53.70	120	19	147	587	1.22E+05	0.35	1.11E+06	4.71E+06	299.8	193.2
304.8	93.54	28	25	301	308	3.52E+06	0.98	3.99E+05	1.70E+06	596.5	384.3

Note: With this procedure, the overlay thicknesses are either zero or cannot be found because of inaccurate layer moduli.

1 mil = 0.0254 mm

(at 50% reliability level) was needed to reduce the strains to the tolerable level. Table C.15 summarizes the results of Fernando's equations.

C.3.4 Mechanistic

Again, the backcalculation program ELSDEF provided the most reliable estimates of layer moduli. Using the example of CDS 293, the backcalculated moduli were 587, 22 and 223 ksi (4044, 152 and 1536 MPa). With these properties, the existing tensile strain in the pavement was 419 μstrain. Remaining life was 17%, which lead to overlays of 4.5 in. (11.4 cm) and 6.5 in. (16.5 cm) for reliability levels of 50% and 90%, respectively. Table C.16 summarizes the results of this analysis.

C.4 Summary

This appendix details the calculations that were performed to obtain the results summarized in Chapter 4. The tables in here attempt to show as many of the intermediate steps as possible, and explain the assumptions made during the computations.

Table C.15. Summary of Results Using Fernando's Equations for Parks Highway.

CDS	ϵ_t	Overlay	ϵ_{ov}
206	484	6.5	155.6
206.2	508	6.5	161.7
206.4	511	6.5	150.9
206.6	538	7	157.2
206.8	386	5	162.4
198	440	6	155.3
198.2	290	3.5	162.2
198.4	277	3.5	156.4
198.6	336	4.5	145.5
198.8	470	6.5	152
293	330	4.5	154.6
293.2	303	4	150.3
293.4	375	5	158.8
293.5	372	5	157.8
293.6	363	5	154.7
293.8	456	6.5	148.4
304	423	6	150.6
304.2	137	1	135.7
304.4	223	2.5	153.4
304.5	1185	>8	-
304.6	512	7	151.2
304.8	565	7.5	151.9

Overlays are in inches (1 in. = 2.54 cm).
Strains are in microstrain.

Table C.16. Summary of Results Using ELSDEF for Parks Highway.

Location CDS	ϵ_t	N_f	R_f	Allowable N_r 50%	Allowable N_r 90%	Tolerable Strain 50%	Tolerable Strain 90%	Overlay 50%	Overlay 90%
206	416	1.29E+05	0.41	8.44E+05	3.59E+06	234.9	151.4	4	6.5
206.2	430	1.26E+05	0.39	8.76E+05	3.72E+06	238.4	153.6	4.5	6.5
206.4	405	1.57E+05	0.52	6.71E+05	2.85E+06	260.5	167.8	3.5	6.5
206.6	440	1.17E+05	0.35	9.87E+05	4.20E+06	230.2	148.3	4.5	7
206.8	502	8.11E+04	0.06	5.52E+06	2.35E+07	139.2	89.7	7	>8
198	502	1.11E+05	0.32	1.09E+06	4.65E+06	250.6	161.5	6	8.5
198.2	246	1.12E+06	0.93	3.71E+05	1.58E+06	344.3	221.8	1	4.5
198.4	239	9.35E+05	0.92	3.76E+05	1.60E+06	315.2	203.1	1	3.5
198.6	472	1.18E+05	0.36	9.65E+05	4.10E+06	249.6	160.8	4	6.5
198.8	481	1.30E+05	0.41	8.33E+05	3.54E+06	273.6	176.2	5	8
293	419	9.50E+04	0.17	2.28E+06	9.69E+06	159.5	102.8	4.5	6.5
293.2	327	2.38E+05	0.67	5.83E+05	2.48E+06	249.0	160.4	2	4.5
293.4	364	7.94E+04	0.01	4.29E+07	1.82E+08	53.8	34.7	8	8.5
293.5	332	9.44E+04	0.17	2.36E+06	1.00E+07	124.9	80.4	3.5	6
293.6	421	8.63E+04	0.09	4.45E+06	1.89E+07	127.0	81.8	6	8.5
293.8	490	4.69E+04	-0.68	-5.76E+05	-2.45E+06	-	-	-	-
304	571	2.28E+04	-2.46	-1.59E+05	-6.75E+05	-	-	-	-
304.2	160	2.06E+06	0.96	4.06E+05	1.73E+06	262.1	168.9	1	2
304.4	150	3.09E+06	0.97	4.01E+05	1.70E+06	278.9	179.7	1	1
304.5	482	3.08E+04	-1.56	-2.51E+05	-1.07E+06	-	-	-	-
304.6	150	3.50E+06	0.98	4.00E+05	1.70E+06	289.9	186.8	1	1
304.8	255	5.99E+05	0.87	4.50E+05	1.91E+06	278.2	179.3	2	4

Notes: Overlay thicknesses are in inches. (1 in. = 2.54 cm)
A negative remaining life indicates that the fatigue life of the pavement has been exceeded.

APPENDIX D

DEVELOPMENT OF TEMPERATURE CORRECTION CHARTS

DEVELOPMENT OF TEMPERATURE CORRECTION CHARTS

D.1 Introduction

Before the pavement surface deflections obtained from the FWD may be used for the design of an overlay, they must first be corrected for temperature so as to remove the variance in deflections. In low temperatures, typically below 50°F (10°C) the pavement layers remain frozen and therefore have a much higher stiffness than when thawed. Consequently, any surface deflections measured at this time will be smaller than those from a thawed pavement. If corrections are not made, it may be erroneously concluded that the pavement is actually performing better than it really is.

Typically, deflections are corrected to a standard 70°F (21°C). The Asphalt Institute (1983) provides correction factors for their procedure (Figure D.1). However, these correction factors were developed from data that were not typical of Alaskan conditions (Kingham, 1969).*

D.2 Development of Correction Factors

The temperature correction charts used for the evaluation of the three projects in Chapter 4 were developed from laboratory data obtained from ADOT&PF in Fairbanks. Asphalt concrete cores were obtained from the Old Richardson Highway, Elliot Highway and Tryphs for a total of ten samples in 1981. The diametral resilient moduli were then obtained at temperatures of 70°F (21°C), 40°F (4.5°C) and 25°F (-4°C). One sample was also tested at 32°F (0°C). Table D.1 summarizes the resilient moduli results from the diametral tests.

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*Kingham, R.I., "A New Temperature-Correction Procedure for Benkelman-Beam Rebound Deflections," *Research Report 69-1*, The Asphalt Institute, Feb. 1969.

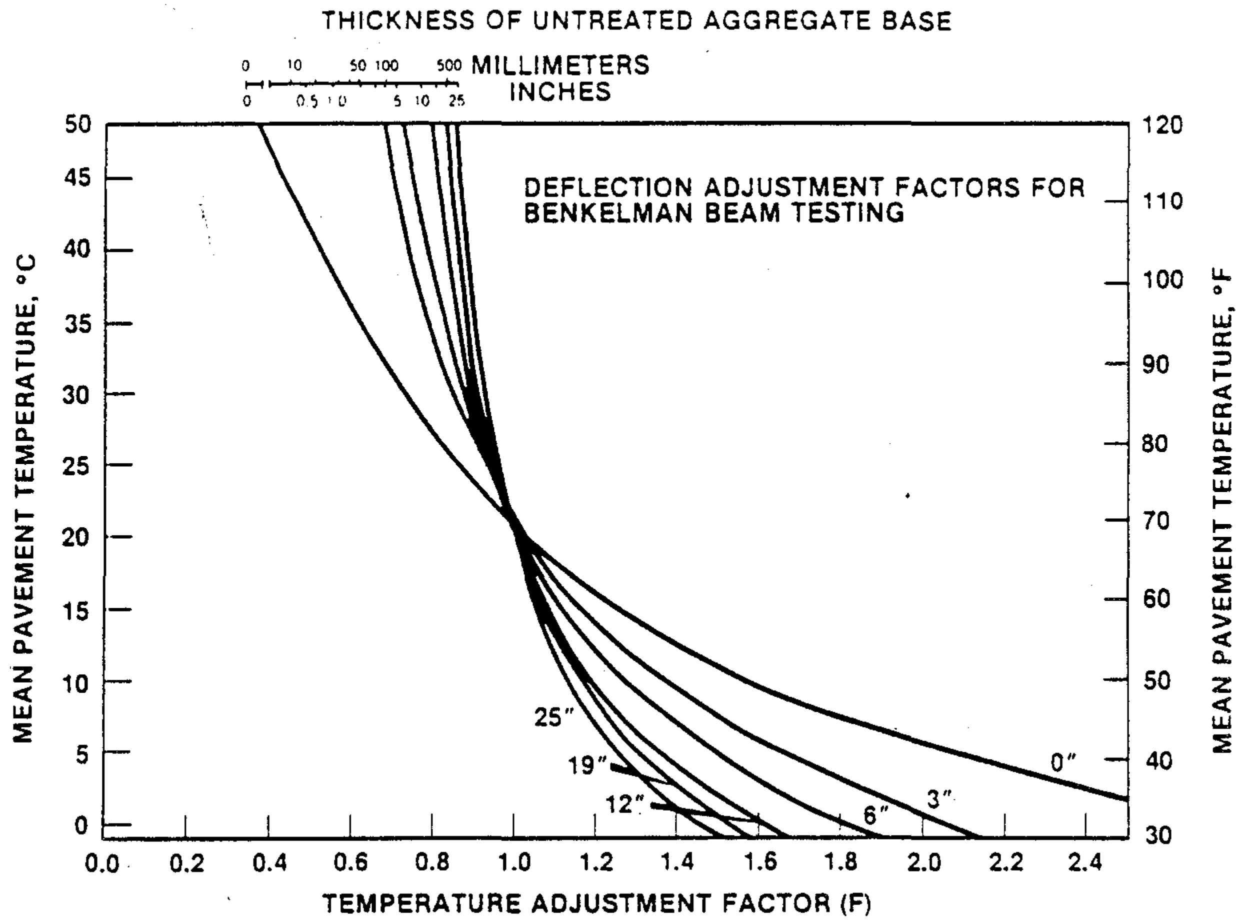


Figure D.1 Average Pavement Temperature Versus Benkelman Beam Deflection Adjustment Factors For Full-Depth And Three-Layered Asphalt Concrete Pavements (Asphalt Institute, 1983).

Table D.1. Resilient Modulus and Temperature Relationships from Fairbanks.

Sample No.	Modulus at Different Temperatures (psi) @ 70°F	@ 40°F	@ 25°F
Old Richardson Hwy			
B-1	1.95×10^5	8.84×10^5	1.89×10^6
B-2	4.09×10^5	1.33×10^6	1.96×10^6
Elliott Hwy			
C-1	3.86×10^4	3.95×10^5	8.73×10^5
C-2	4.35×10^4	3.88×10^5	7.98×10^5
E-1	4.87×10^4	7.15×10^5	1.69×10^6
E-2	5.78×10^4	4.87×10^5	1.06×10^6
Tryphs			
D-1	1.59×10^5	4.88×10^5	7.76×10^5
D-2	1.44×10^5	7.15×10^5	1.69×10^6
F-1	1.73×10^5	5.87×10^5	1.07×10^6
F-2	1.51×10^5	5.94×10^5	1.07×10^6

NB: Sample E-1 was also tested at 32°F and found to have a modulus of 7.36×10^5 psi.

°C = (°F-32)*5/9
1 psi = 6.89 kPa

Figure D.2 illustrates the spread of moduli that the data encompasses. It can be seen that the outer and inner limits of the data describe a band that is approximately 400,000 psi (4130 MPa) wide. The midpoint of this band was selected and used for analysis. Once the relationship between AC resilient modulus and temperature has been established, it is possible to compute temperature correction factors with the help of the computer program ELSYM5.

As an additional note, Figure D.3 compares the relationships between temperature and the AC modulus between AMOD and the laboratory data. There are four graphs illustrated, of which three come from the AMOD program, and the fourth from the midpoint of the laboratory data. The AMOD graphs were calculated at different vehicle speeds of 1 mph (1.6 km/h) and 50 mph (80 km/h). The third AMOD graph was calculated at 50 mph (80km/h) and then the modulus halved. This is the value that ADOT&PF uses as its estimation of the asphalt concrete modulus. From this, it can be seen that at 50 mph (80 km/h), the modulus obtained is significantly greater than those calculated at the lower speeds and that from the laboratory data. Therefore, for an initial estimation of the asphalt concrete, the values obtained from the laboratory tests or that from AMOD at lower speeds is recommended.

The variable inputs used in ELSYM5 are summarized in Figure D.4. Two pavement conditions exist, an unfrozen pavement and a partially frozen pavement. Since the thaw depth varies with pavement location and environmental conditions, it was assumed that for the critical condition, it lies 15 in. (38 cm) below the surface. ADOT&PF indicate that the typical thaw depth would lie between 12 to 24 in. (30.5 to 61 cm) below the surface and that variations of this order of magnitude in the thaw depth do not greatly affect the results. It is also assumed that above 50°F (10°C), an unfrozen condition

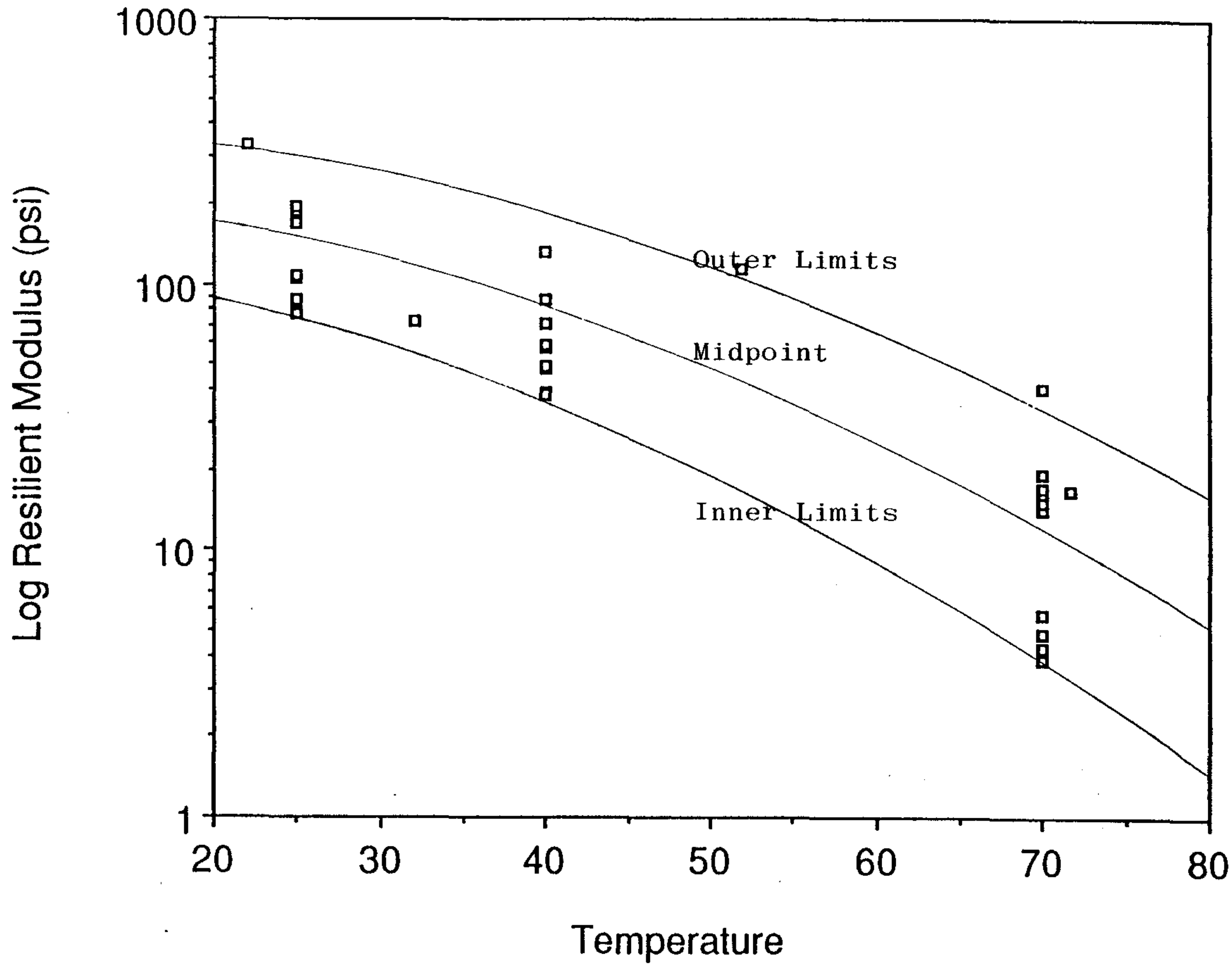


Figure D.2 Modulus versus Temperature from Fairbanks Laboratory Data.

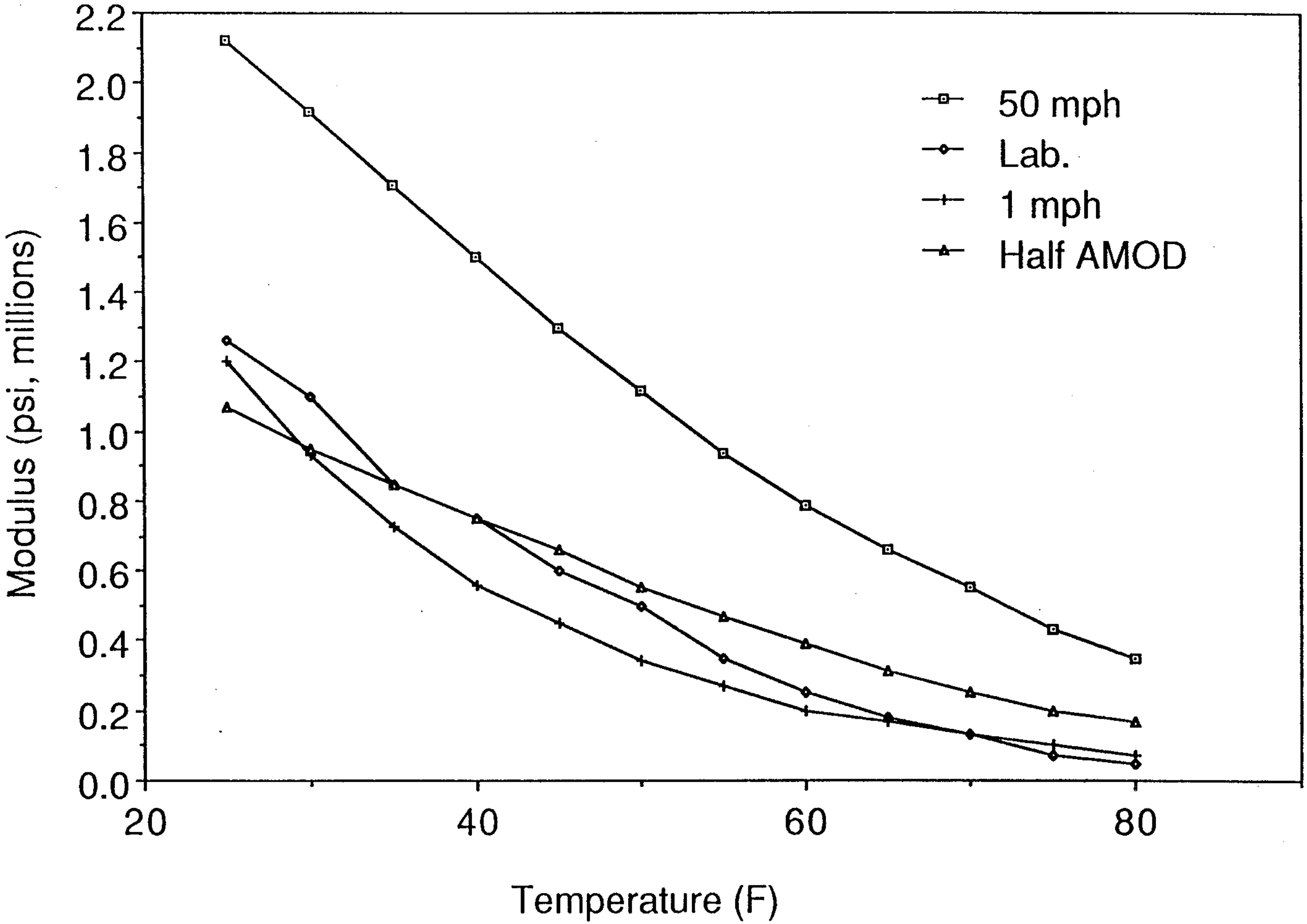


Figure D.3 Comparison between Fairbanks Laboratory Data and AMOD for Modulus.

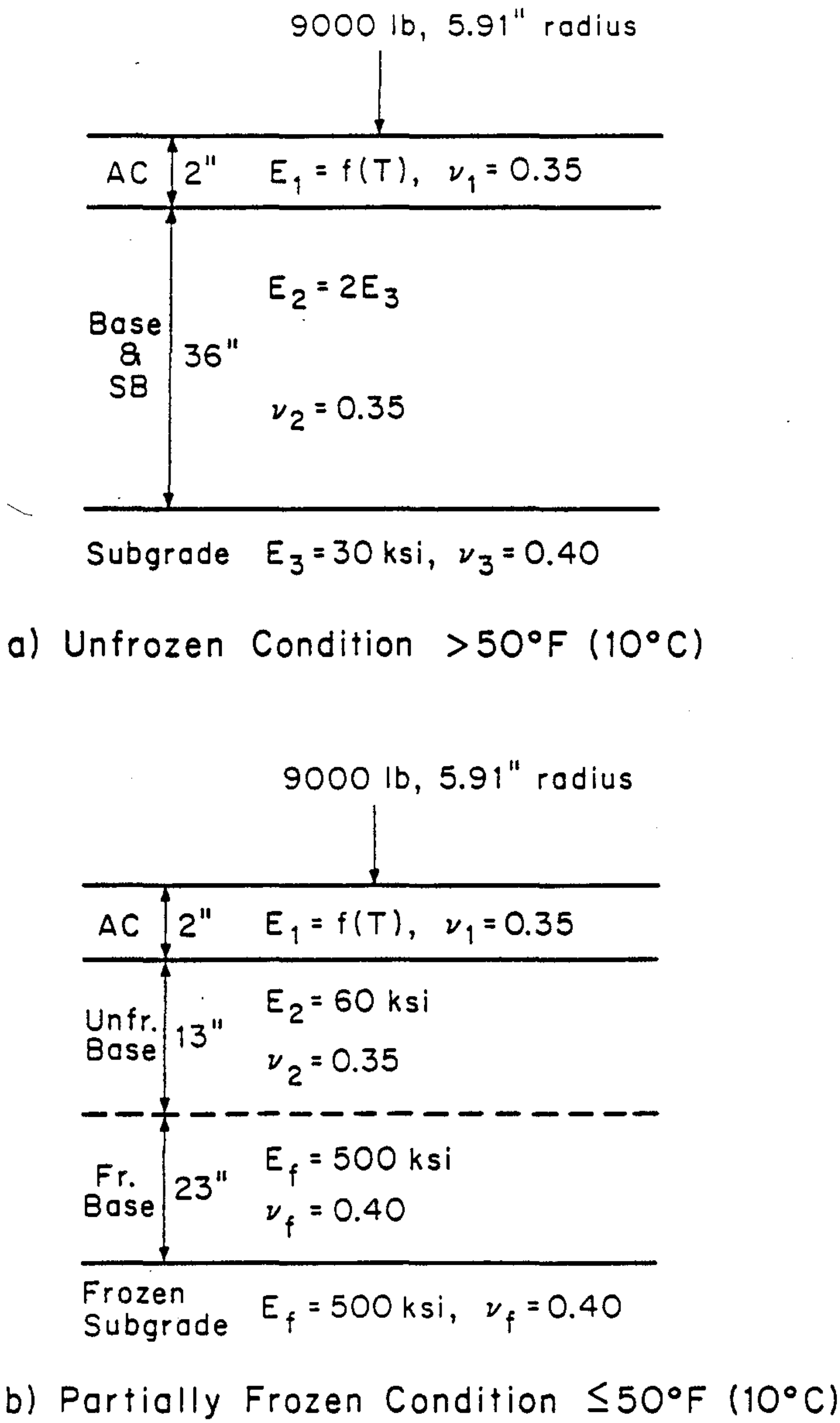


Figure D.4 Input Variables Used in ELSYM5 Analysis.

would exist, and below that, a partially to completely frozen condition would exist.

Other assumptions include a 2 in. (5 cm) asphalt concrete layer with an 36 in. (66 cm) granular base and subbase. A 9000 lb. (2022 N) force is applied to a loading plate 11.8 in. (30 cm) in diameter. The base modulus is twice the subgrade modulus, and the values shown in Figure D.4 are typical for Fairbanks. Frozen granular materials are assumed to have a modulus of 500,000 psi (3445 MPa).

With the above data, the center-of-load surface deflection for a variety of different asphalt concrete moduli may be calculated. Since a relationship exists between the moduli and temperature and another between moduli and deflection, it is possible to derive a relationship between deflection and temperature. Using 70°F (21°C) as the standard, other center deflections at different temperatures may be adjusted accordingly. The adjustment factors are then the temperature correction factors. Table D.2 presents the results of the ELSYM5 analysis and Figure D.5 illustrate a plot of the correction factors versus temperature.

The "kink" in Figure D.5 occurs at the 50°F (10°C) point and is due to the fact that this was assumed to be the boundary between the unfrozen and the partially frozen pavement layer conditions.

Table D.2. Results of ELSYM5 Analysis.

Temperature (°F)	Modulus (psi)	Deflection (mils)	Correction Factor C_f
25	1,260,000	7.24	1.975
30	1,100,000	7.42	1.927
35	850,000	7.76	1.843
40	750,000	7.91	1.808
45	600,000	8.17	1.750
50	500,000	8.37	1.709
55	350,000	13.20	1.083
60	260,000	13.50	1.059
65	180,000	13.90	1.029
70	126,000	14.30	1.000
75	79,400	15.00	0.953
80	59,000	15.50	0.923

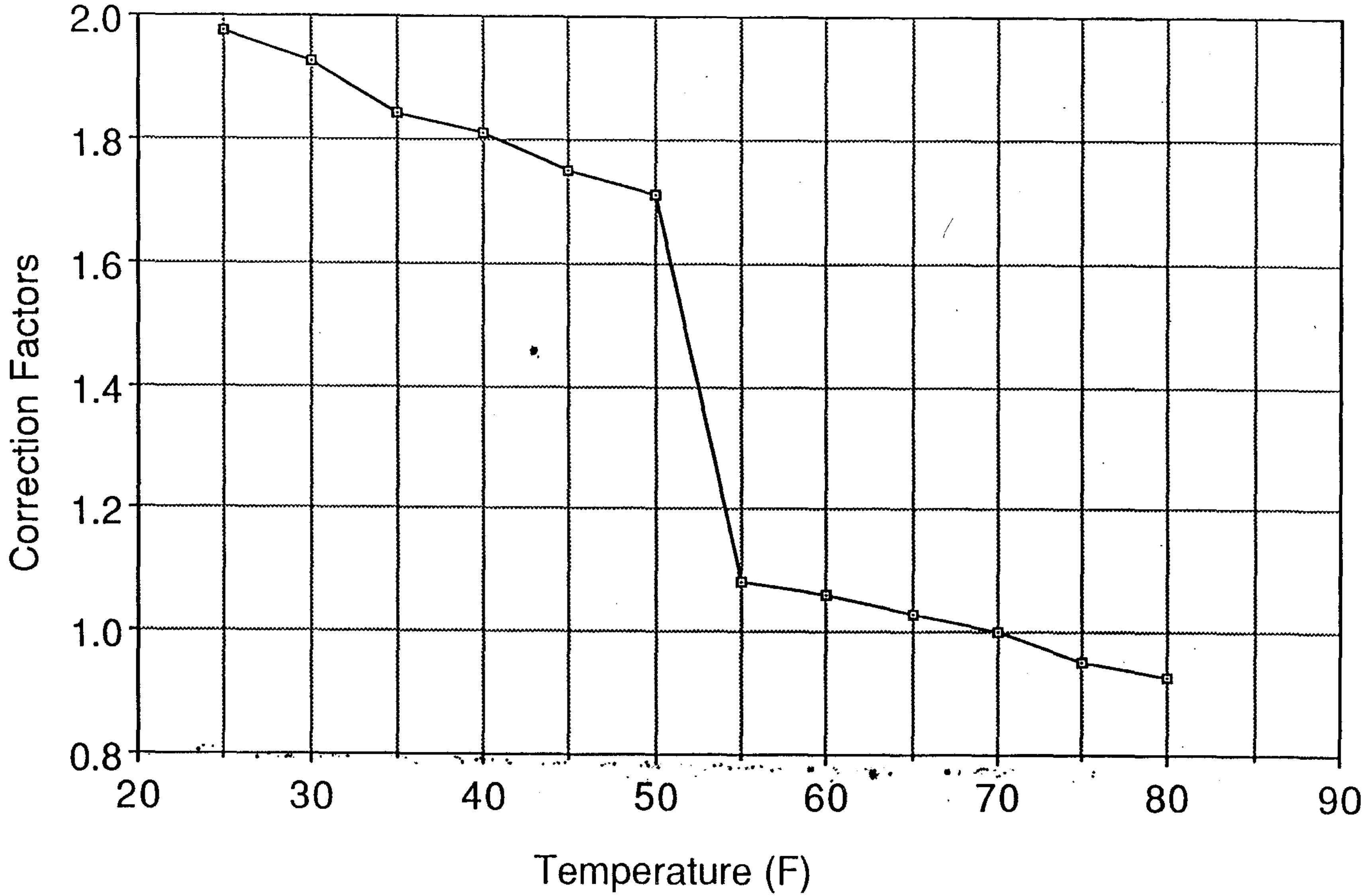


Figure D.5 Temperature Correction Factors Using Fairbanks Data.

APPENDIX E

PROCEDURE FOR RELIABILITY LEVELS AND DESIGN FACTORS

PROCEDURE FOR RELIABILITY LEVELS AND DESIGN FACTORS

Reliability is defined as "the probability that a pavement section designed ... will perform satisfactorily over the traffic and environmental conditions for the design period." For example, an engineer may decide Highway X is to be designed with a 95% probability that it will perform as expected over its design life, while a low-volume road, such as timber-haul spur road, may be designed such that it has only a 50% probability that it will perform satisfactorily.

To incorporate this concept of reliability into the design process, a reliability factor, F_R, is used to shift the design traffic upwards. This procedure is summarized below:

1. Obtain the predicted traffic of the analysis section over the design period, W_T. Assume this is 1 million EALs.
2. Select a reliability level, R. As an example, assume R = 90%.
3. Select a standard deviation, S_o. AASHTO recommends the following values for flexible and rigid pavements.

   Flexible pavements $S_o = 0.49$

   Rigid pavements $S_o = 0.39$

   Assume we have a flexible pavement and $S_o = 0.49$.
4. From Table E.1, obtain the reliability factor, F_R. For this example, $F_R = 4.25$.
5. Calculate the design traffic by multiplying the predicted traffic and the reliability factor together:

$$\text{Design traffic} = W_T * F_R$$

$$= 1{,}000{,}000 * 4.25$$

$$= 4{,}250{,}000 \text{ EALs.}$$

Note that the absence of a reliability factor in design simply means that $F_R = 1$, i.e. a 50% reliability level. This means that there is a 50% chance that such a designed section will not survive the design period.

Table E.1. Reliability Factors for Specified Reliability Levels and Overall Variance Levels* (AASHTO, 1985).

Estimated Std. Dev.	Reliability Levels (%)							
	50	60	70	80	90	95	99	99.9
0.39	1.00	1.26	1.60	2.13	3.16	4.38	8.08	16.00
0.49	1.00	1.33	1.81	2.58	4.25	6.40	13.80	32.70

*This table only includes reliability factors for two standard deviations (typical values for rigid and flexible pavements, respectively). A complete table is available in Appendix EE of the AASHTO Guide.

APPENDIX F

DETERMINATION OF SENSOR LOCATIONS

DETERMINATION OF SENSOR LOCATIONS

This appendix determines the deflection basins under a wheel load for "typical" unfrozen and partially frozen pavements in Alaska. The following discussion describes the procedure used to determine a "typical" deflection basin and the appropriate sensor locations of the FWD that would be necessary to obtain a complete profile.

Frozen vs. Unfrozen Pavements

Figure F.1 illustrates the typical unfrozen and partially frozen pavement cross-sections that were used. A 9000 lb (40 kN) load is applied over a loading plate of radius 5.91 in. (15 cm). The thaw depth for the partially frozen pavement is assumed to be 15 in. (38 cm) below the pavement surface. The layer properties (resilient moduli, Poisson's ratio and dimensions) are as indicated in Figure F.1.

Using the computer ELSYM5, the surface deflections at different radial distances from the applied load may be calculated. Table F.1 summarizes the surface deflections that were obtained. Figure F.2 illustrate the deflection basins that were obtained for both the frozen and unfrozen pavements, respectively. As expected, the basin for a frozen pavement is shallower than that for the unfrozen case.

The current configuration of the FWD sensors used in Alaska include an outermost sensor located 47.2 in. (1200 mm) form the load center. However, from the results shown in Table F.1 and Figure F.2, it is obvious that there are significant deflections beyond this point. There are deflections of up to 1 mil at a distance 72 in. from the load center for the unfrozen pavement. For the frozen pavement, the current configuration is probably sufficient as

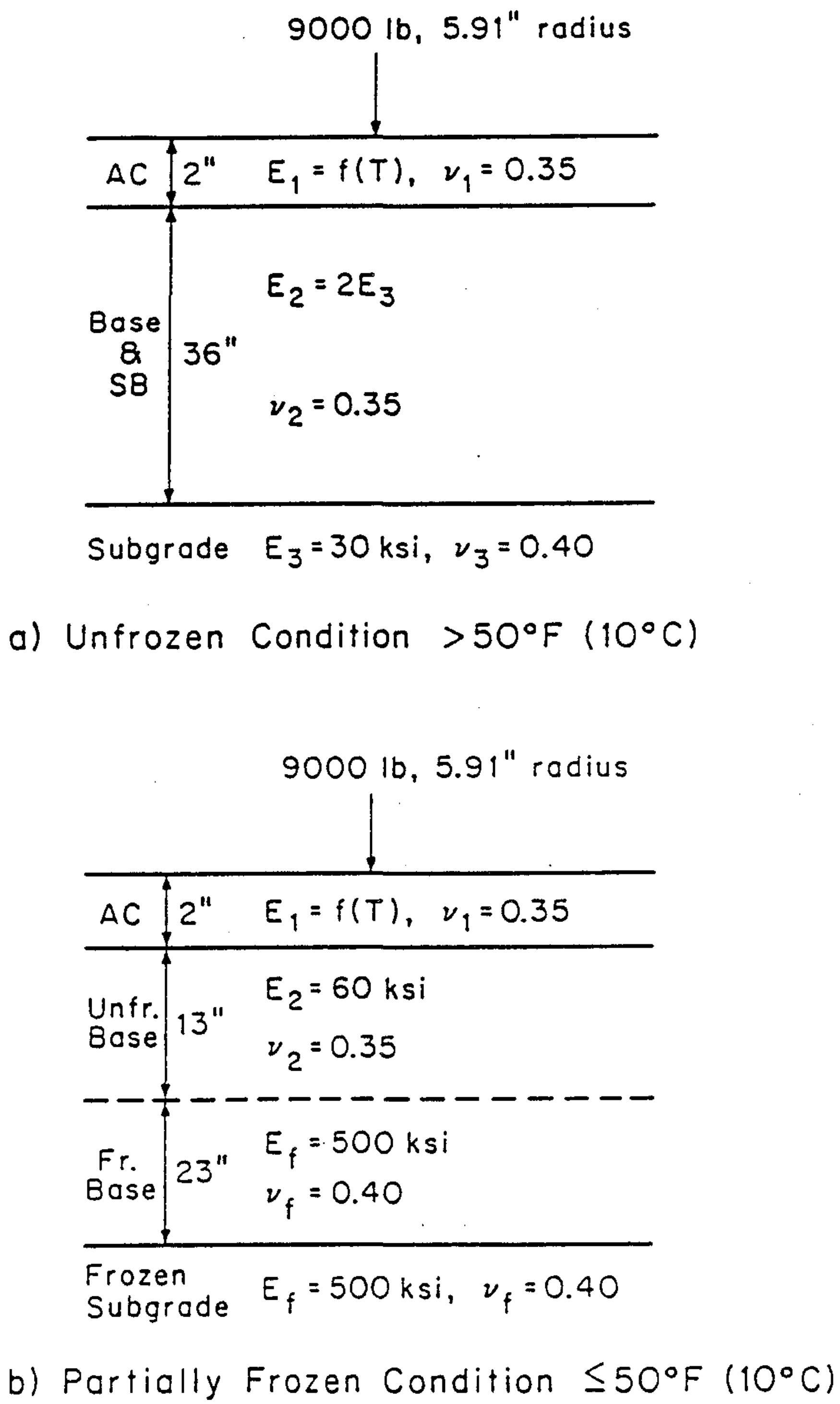


Figure F.1 Cross Sections and Material Properties for Frozen and Unfrozen Cases

Table F.1. Deflection Basins from ELSYM5 Analysis.

Sensor Location (in.)	Surface Deflections (mils)	
	Unfrozen Case	Partially Frozen
0	13.2	8.27
6	9.1	4.76
12	4.8	1.13
18	3.46	0.299
24	2.81	0.139
30	2.40	0.115
36	2.10	0.110
42	1.87	0.104
48	1.68	0.0962
54	1.52	0.0874
60	1.39	0.079
66	1.27	0.0716
72	1.17	0.0654
78	1.08	0.0602
84	1.00	0.0558
90	0.936	0.0522

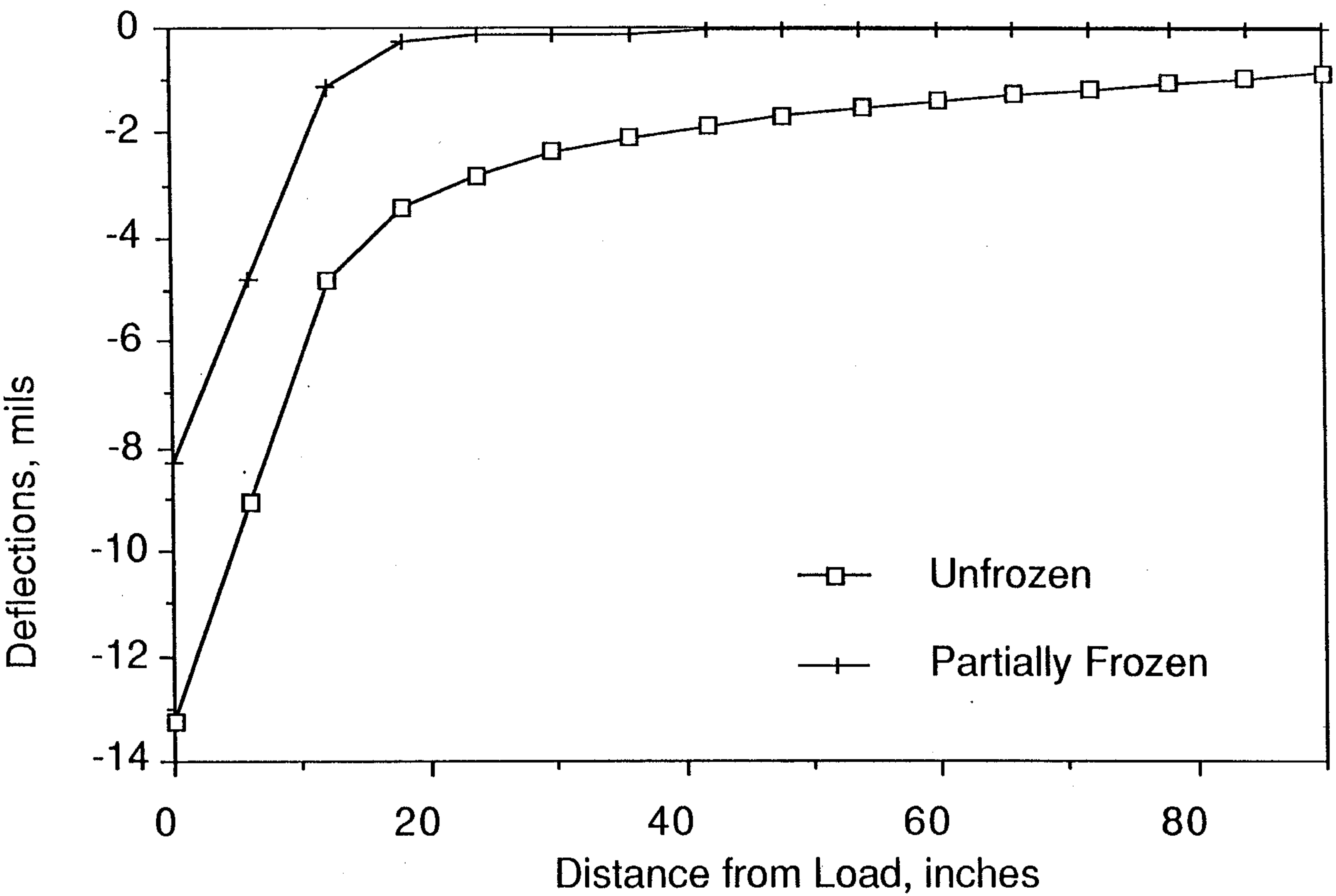


Figure F.2 Typical Deflection Basins - Unfrozen and Particially Frozen Pavements.

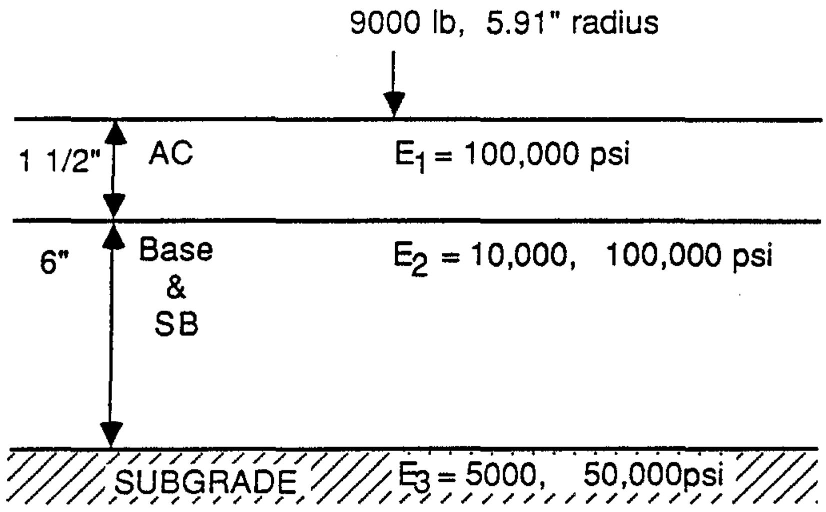
the deflection does not differ significantly from 48 in. to 72 in. Therefore, the estimates of subgrade modulus from both these values in the backcalculation procedure are probably not too dissimilar.

Unfrozen Pavement - Thin vs. Thick Pavement

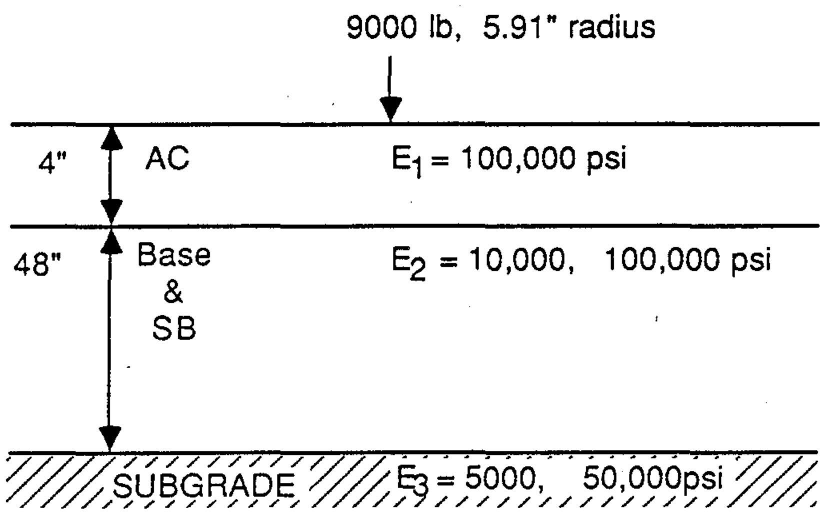
A similar analyses was performed on the cross sections shown on Figure F.3. In this case the modulus of the base and subgrade was varied. The weak subgrade was assumed to have a modulus of 5000 psi while strong subgrade had a modulus of 50,000 psi. Similarly, a weak base had a modulus of 10,000 psi while a strong had a modulus of 100,000 psi.

The results of the calculations are summarized in Figures F.4 and F.5 In general, they indicate:

1) For thin pavements (Figure F.4)
   a) Both the subgrade and base modulus effect the basin shape.
   b) The base modulus only effects the shape within 10 inches of the load (for a stiff subgrade) and about 30 inches of the load (for a soft subgrade).
   c) Significant deflections still are not beyond 72 inches for the weak subgrade.
2) For thick pavements (Figure F.5)
   a) The base modulus affects the basin shape for both the weak and strong subgrade.
   b) Significant deflections exist for the weak subgrade beyond 72 inches.



a) Thin Pavement



b) Thick Pavement

Figure F.3 Cross Sections for Unfrozen Cases

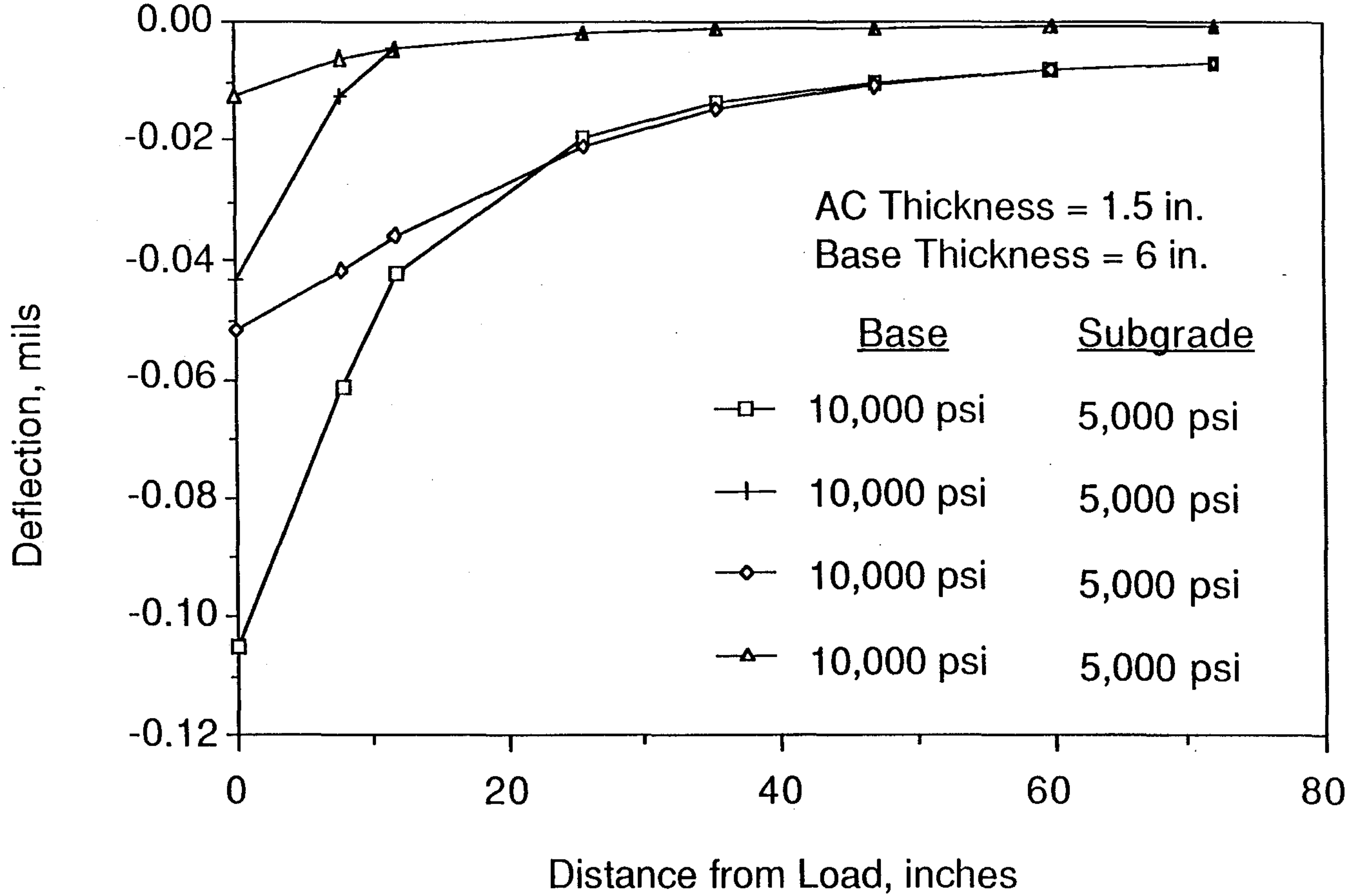


Figure F.4 Deflection Response for Thin Pavement.

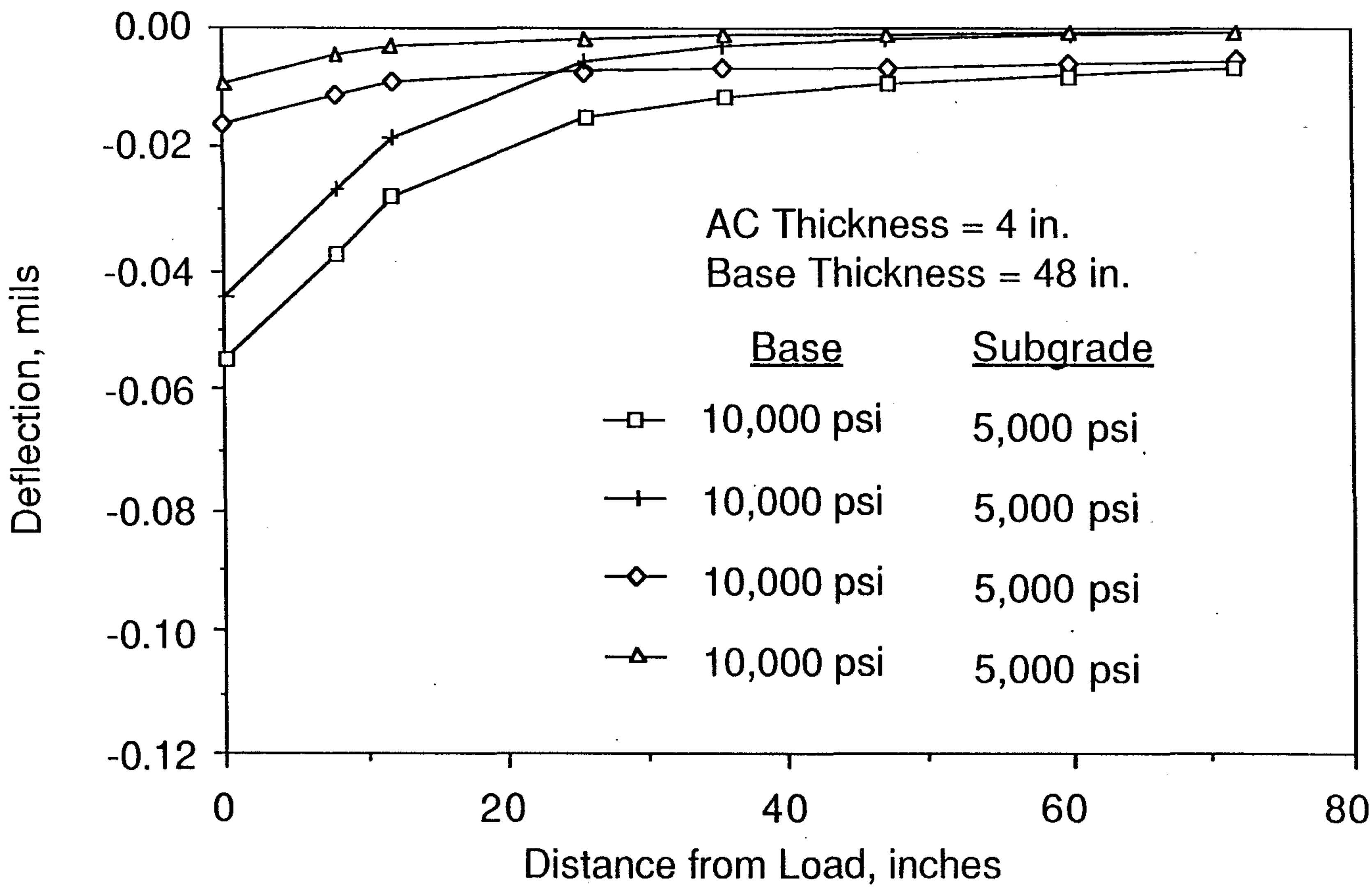


Figure F.4 Deflection Response for Thick Pavement.

Conclusion

To conclude, it might prove beneficial if the outermost sensor were extended beyond the current 47.2 in. in cases where the pavement may reasonably be expected to be thawed. This would reduce the errors that would result from inaccurate estimates in the backcalculation of layer moduli.

APPENDIX G

DETERMINATION OF OVERLAY THICKNESS BASED ON SUBGRADE STRAIN

DETERMINATION OF OVERLAY THICKNESS BASED ON SUBGRADE STRAIN

1. INTRODUCTION

The purpose of this appendix is to evaluate the significance of subgrade strain criteria in the determination of the overlay thickness. Calculations using tensile strain criteria were given in Chapter 4.

The typical cross section and the material properties is as shown in Figure G.1. The calculated moduli values from ELSDEF for each project are extracted from Tables 4.2, 4.5, and 4.7.

2. ANALYSIS

The ELSYM5 program was used to calculate the subgrade strain at the top of the subgrade layer for both the unfrozen and partially frozen situations. The Asphalt Institute (1982) vertical compression strain criterion was used to calculate the number of 18 kip (80 kN) equivalent single axle loads:

$$N = 1.365E\text{-}9 * \epsilon_c^{-4.477}$$

where:

ϵ_c = vertical compression strain at the subgrade surface.

N = Number of 18 kip (80 kN) equivalent single axle loads.

The remaining life of the pavement was determined using Miner's hypothesis:

$$N_r/N_{D1} = 1 - N_{A1}/N_{D1}$$

where

N_r/N_{D1} = remaining life

N_{A1} = number of applications of EALS to date.

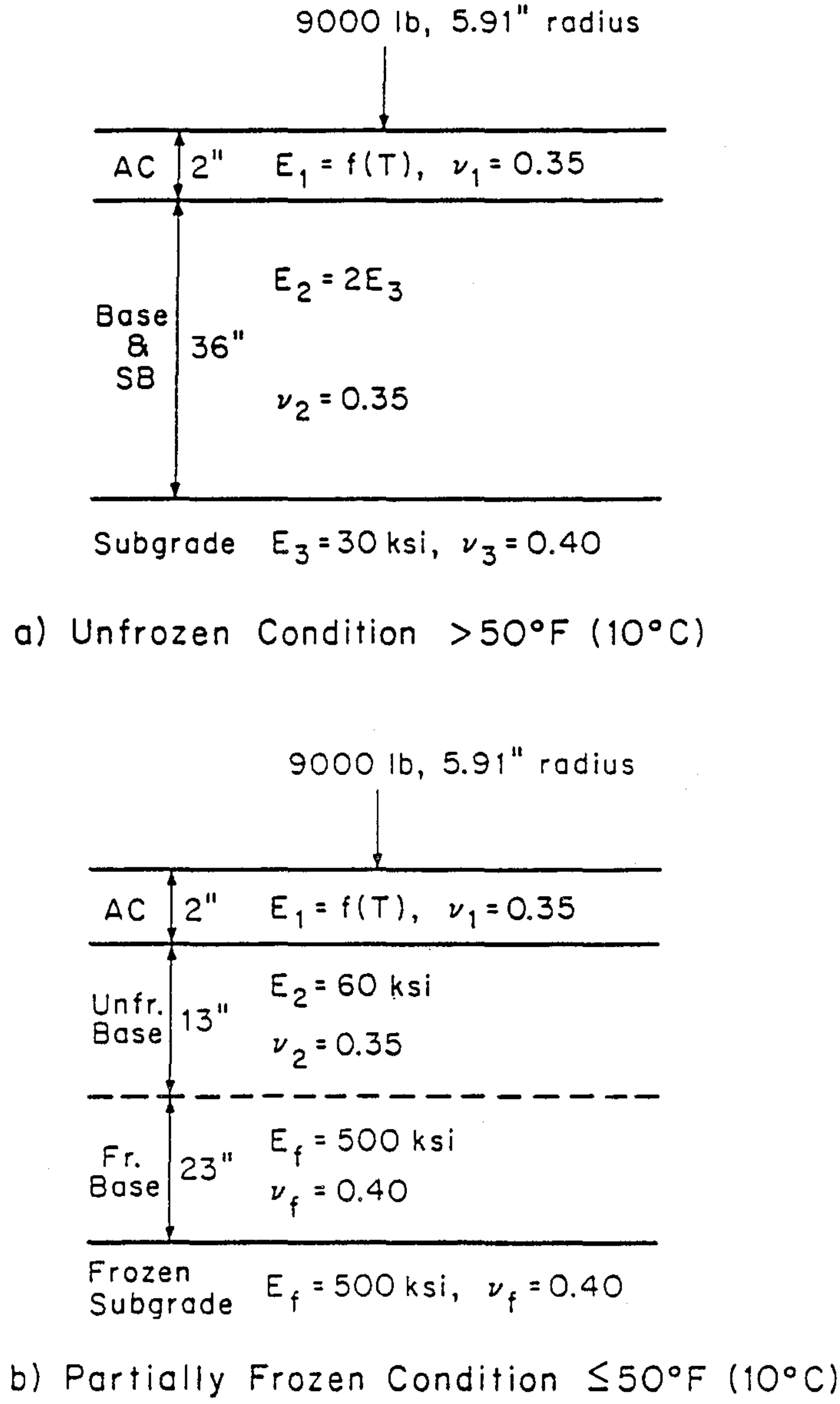


Figure G.1 Input Variables Used in ELSYMS Analysis.

N_{D1} = allowable number of applications of EALS according to fatigue relationship, and

N_{r1} = additional number of applications of EALS that can be applied to the existing pavement.

Using MP 117.5 as an example, the vertical compression strain calculated using ELSYM5 is found to be $4.3410*10^{-5}$. Using the relationship developed by Asphalt Institute, the number of applications to failure with this strain may be calculated:

$$N = 1.365E\text{-}9 * \epsilon_c^{-4.477}$$

Then,

$$N = 4.631*10^{10}$$

with an actual EAL of 130,000 to date. Hence, the remaining life can be calculated using Miner's hypothesis:

$$R_f = 1 - N_{actual}/N_f$$

$$= 1 - 130000/4.63066*10^{10}$$

$$= 99.99997\ \%$$

Therefore, the pavement has a remaining life of 99.99997%.

If the pavement must be able to withstand the predicted 20 year EAL of 1,800,000, the number of applications for a 50% reliability level is therefore:

$$N_r = 1800000/.9999997 = 1800000$$

For a 90% reliability level, $F_r = 4.25$ (see Appendix E), and the number of applications is:

$$N_r = 1800000 * 4.25/.9999997 = 7650002$$

For this to occur, the pavement compressive strain must not exceed a tolerable strain (determined using the Asphalt Institute vertical compressive strain criterion equation rearranged) of:

For 50%:

$$\epsilon_c = 419\ \mu\text{strain}$$

For 90%:

$$\epsilon_c = 303\ \mu\text{strain}.$$

These strain levels are 10 times greater than the calculated values, indicating rutting not to be a problem.

3. CONCLUSION

The calculated remaining life using subgrade strain criteria is tabulated in Tables G.1, G.2, and G.3 are greater than 99.8%. In addition, the values of calculated compressive strain in these tables varies considerably and in all cases are substantially less than the tolerable values. This shows that subgrade strain is insignificant in the determination of overlay thickness. Hence, subgrade rutting is not a problem.

Table G.1. Summary of Results Using Subgrade Strain Criteria for Sterling Highway.

Location	ϵ_c	Nf	Rf (%)	Allowable N		Tolerable ϵ_c	
				50%[1]	90%	50%	90%
117.5	4.341E-05	4.631E+10	100.00	1.800E+06	7.650E+06	4.195E-04	3.054E-04
118	8.806E-06	5.853E+13	100.00	1.800E+06	7.650E+06	4.195E-04	3.054E-04
118.5	8.016E-05	2.972E+09	100.00	1.800E+06	7.650E+06	4.195E-04	3.054E-04
119	8.728E-06	6.091E+13	100.00	1.800E+06	7.650E+06	4.195E-04	3.054E-04
119.5	7.872E-05	3.224E+09	100.00	1.800E+06	7.650E+06	4.195E-04	3.054E-04
120.5	1.052E-04	8.802E+08	99.99	1.800E+06	7.651E+06	4.195E-04	3.053E-04
121	8.895E-06	5.595E+13	100.00	1.800E+06	7.650E+06	4.195E-04	3.054E-04
121.5	6.147E-05	9.756E+09	100.00	1.800E+06	7.650E+06	4.195E-04	3.054E-04
122	8.907E-06	5.561E+13	100.00	1.800E+06	7.650E+06	4.195E-04	3.054E-04
123	8.869E-06	5.669E+13	100.00	1.800E+06	7.650E+06	4.195E-04	3.054E-04
123.5	9.178E-05	1.621E+09	99.99	1.800E+06	7.651E+06	4.195E-04	3.053E-04
124	8.895E-06	5.595E+13	100.00	1.800E+06	7.650E+06	4.195E-04	3.054E-04
124.5	9.472E-05	1.408E+09	99.99	1.800E+06	7.651E+06	4.195E-04	3.053E-04
125	8.843E-06	5.744E+13	100.00	1.800E+06	7.650E+06	4.195E-04	3.054E-04
125.5	1.135E-04	6.265E+08	99.98	1.800E+06	7.652E+06	4.195E-04	3.053E-04
126	8.561E-06	6.641E+13	100.00	1.800E+06	7.650E+06	4.195E-04	3.054E-04
127	8.857E-06	5.703E+13	100.00	1.800E+06	7.650E+06	4.195E-04	3.054E-04
127.5	1.008E-04	1.066E+09	99.99	1.800E+06	7.651E+06	4.195E-04	3.053E-04
128	8.822E-06	5.805E+13	100.00	1.800E+06	7.650E+06	4.195E-04	3.054E-04
129	8.903E-06	5.573E+13	100.00	1.800E+06	7.650E+06	4.195E-04	3.054E-04

[1]These are the reliability levels as defined by AASHTO

Table G.2. Summary of Results Using Subgrade Strain Criteria for Seward Highway.

				Allowable N		Tolerable ϵ_c	
Location	ϵ_c	Nf	Rf (%)	50%[1]	90%	50%	90%
TH1	8.217E-06	7.979E+13	100.00	5.083E+06	1.870E+07	3.327E-04	2.501E-04
TH2	6.921E-05	5.737E+09	99.99	5.083E+06	1.870E+07	3.327E-04	2.501E-04
TH4	9.639E-05	1.302E+09	99.97	5.085E+06	1.871E+07	3.326E-04	2.501E-04
TH45	1.361E-04	2.779E+08	99.84	5.091E+06	1.873E+07	3.325E-04	2.500E-04
TH35	7.872E-05	3.224E+09	99.99	5.084E+06	1.870E+07	3.327E-04	2.501E-04

[1]These are the reliability levels as defined by AASHTO

Table G.3. Summary of Results Using Subgrade Strain Criteria for Parks Highway.

Location	ϵ_c	Nf	Rf (%)	Allowable N		Tolerable ϵ_c	
				50%[1]	90%	50%	90%
206	7.484E-05	4.042E+09	100.00	3.455E+05	1.469E+06	6.065E-04	4.415E-04
206.2	1.189E-04	5.088E+08	99.99	3.456E+05	1.469E+06	6.065E-04	4.415E-04
206.4	1.027E-04	9.802E+08	99.99	3.456E+05	1.469E+06	6.065E-04	4.415E-04
206.6	1.149E-04	5.930E+08	99.99	3.456E+05	1.469E+06	6.065E-04	4.415E-04
206.8	1.241E-04	4.201E+08	99.98	3.456E+05	1.469E+06	6.065E-04	4.415E-04
198	1.130E-04	6.390E+08	99.99	3.456E+05	1.469E+06	6.065E-04	4.415E-04
198.2	5.768E-05	1.297E+10	100.00	3.455E+05	1.468E+06	6.065E-04	4.415E-04
198.4	7.296E-05	4.530E+09	100.00	3.455E+05	1.469E+06	6.065E-04	4.415E-04
198.8	1.168E-04	5.511E+08	99.99	3.456E+05	1.469E+06	6.065E-04	4.415E-04
293	1.103E-04	7.121E+08	99.99	3.906E+05	1.660E+06	5.901E-04	4.296E-04
293.2	8.026E-05	2.956E+09	100.00	3.905E+05	1.660E+06	5.901E-04	4.296E-04
293.4	1.490E-04	1.853E+08	99.96	3.907E+05	1.660E+06	5.901E-04	4.295E-04
293.5	1.440E-04	2.158E+08	99.96	3.907E+05	1.660E+06	5.901E-04	4.295E-04
293.6	1.271E-04	3.775E+08	99.98	3.906E+05	1.660E+06	5.901E-04	4.296E-04
304.2	4.261E-05	5.033E+10	100.00	3.905E+05	1.660E+06	5.901E-04	4.296E-04
304.4	3.460E-05	1.278E+11	100.00	3.905E+05	1.660E+06	5.901E-04	4.296E-04
304.6	3.374E-05	1.431E+11	100.00	3.905E+05	1.660E+06	5.901E-04	4.296E-04
304.8	5.789E-05	1.276E+10	100.00	3.905E+05	1.660E+06	5.901E-04	4.296E-04

[1]These are the reliability levels as defined by AASHTO