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| 16. Abstract <p>The current Pavement Evaluation System used in Texas rates the condition of pavements in terms of visual distress and present serviceability index. This report discusses the addition of another dimension to the rating system; that of a Structural Strength Index. The Falling Weight Deflectometer is to be used for this purpose. In this report, an overview is given of the FWD and data analysis techniques, a discussion on sample size is presented and two possible structural strength schemes are proposed. The first is a simple statistically based scheme which ranks pavement strength in terms of key deflection bowl parameters, and includes weighting factors for traffic level and rainfall. The second is a mechanistic approach in which a remaining service life is calculated.</p> <p>These two approaches were pilot tested on data collected in several Texas districts. It was recommended that the statistically based scheme be implemented. Although the mechanistic scheme shows promise at the project level, several factors including incomplete layer information and insufficient traffic data, currently limit its applicability at the network level.</p> | | | |
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INCORPORATING A STRUCTURAL STRENGTH
INDEX INTO THE TEXAS PAVEMENT
EVALUATION SYSTEM

by

T. Scullion

Research Report 409-3F

Research Study 2-18-85-409

PES Improvements

Sponsored by

Texas State Department of Highways and Public Transportation
in cooperation with
U.S. Department of Transportation
Federal Highway Administration

TEXAS TRANSPORTATION INSTITUTE
The Texas A & M University System
College Station, Texas

April 1988

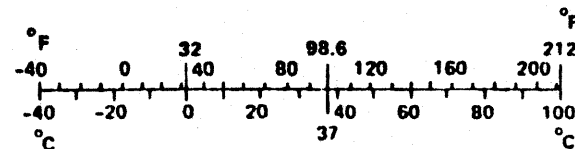
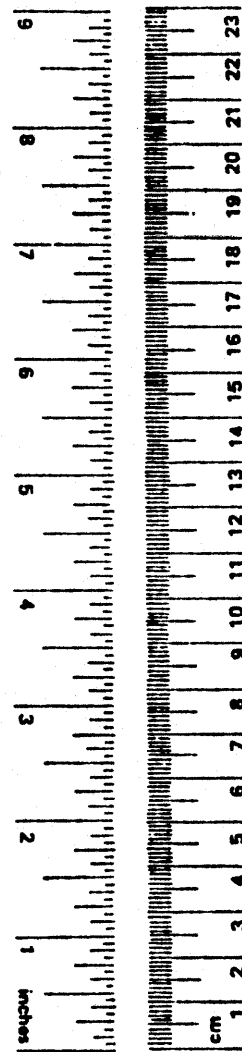
METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

| Symbol | When You Know | Multiply by | To Find | Symbol |
|----------------------------|-------------------------|----------------------------|---------------------|-----------------|
| LENGTH | | | | |
| in | inches | *2.5 | centimeters | cm |
| ft | feet | 30 | centimeters | cm |
| yd | yards | 0.9 | meters | m |
| mi | miles | 1.6 | kilometers | km |
| AREA | | | | |
| in ² | square inches | 6.5 | square centimeters | cm ² |
| ft ² | square feet | 0.09 | square meters | m ² |
| yd ² | square yards | 0.8 | square meters | m ² |
| mi ² | square miles | 2.6 | square kilometers | km ² |
| | acres | 0.4 | hectares | ha |
| MASS (weight) | | | | |
| oz | ounces | 28 | grams | g |
| lb | pounds | 0.45 | kilograms | kg |
| | short tons (2000 lb) | 0.9 | tonnes | t |
| VOLUME | | | | |
| tsp | teaspoons | 5 | milliliters | ml |
| Tbsp | tablespoons | 15 | milliliters | ml |
| fl oz | fluid ounces | 30 | milliliters | ml |
| c | cups | 0.24 | liters | l |
| pt | pints | 0.47 | liters | l |
| qt | quarts | 0.95 | liters | l |
| gal | gallons | 3.8 | liters | l |
| ft ³ | cubic feet | 0.03 | cubic meters | m ³ |
| yd ³ | cubic yards | 0.76 | cubic meters | m ³ |
| TEMPERATURE (exact) | | | | |
| °F | Fahrenheit temperature | 5/9 (after subtracting 32) | Celsius temperature | °C |

Approximate Conversions from Metric Measures

| Symbol | When You Know | Multiply by | To Find | Symbol |
|----------------------------|-----------------------------------|-------------------|------------------------|-----------------|
| LENGTH | | | | |
| mm | millimeters | 0.04 | inches | in |
| cm | centimeters | 0.4 | inches | in |
| m | meters | 3.3 | feet | ft |
| m | meters | 1.1 | yards | yd |
| km | kilometers | 0.6 | miles | mi |
| AREA | | | | |
| cm ² | square centimeters | 0.16 | square inches | in ² |
| m ² | square meters | 1.2 | square yards | yd ² |
| km ² | square kilometers | 0.4 | square miles | mi ² |
| ha | hectares (10,000 m ²) | 2.5 | acres | |
| MASS (weight) | | | | |
| g | grams | 0.035 | ounces | oz |
| kg | kilograms | 2.2 | pounds | lb |
| t | tonnes (1000 kg) | 1.1 | short tons | |
| VOLUME | | | | |
| ml | milliliters | 0.03 | fluid ounces | fl oz |
| l | liters | 2.1 | pints | pt |
| l | liters | 1.06 | quarts | qt |
| l | liters | 0.26 | gallons | gal |
| m ³ | cubic meters | 35 | cubic feet | ft ³ |
| m ³ | cubic meters | 1.3 | cubic yards | yd ³ |
| TEMPERATURE (exact) | | | | |
| °C | Celsius temperature | 9/5 (then add 32) | Fahrenheit temperature | °F |



* 1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price \$2.25, SD Catalog No. C13.10:286.

Preface

Research Report 409-3F, "Incorporating a Structural Strength Index into the Texas Pavement Evaluation System" is the third and final report for Research Project 409, "PES Improvements." This research study was conducted by the Texas Transportation Institute (TTI), Texas A&M University in College Station as part of the Cooperative Highway Research Program sponsored by the Texas State Department of Highways and Public Transportation (SDHPT) and the Federal Highway Administration (FHWA).

The purpose of this report is to present a proposal by which the Texas SDHPT can incorporate a structural strength index scheme into its network level pavement management system. The Falling Weight Deflectometer is used to measure in situ pavement structural strength.

This report was completed with the assistance of many people. Special appreciation is extended to Messrs. Bob Guinn and Bob Briggs of the Texas Department of Highways and Public Transportation for their encouragement and constructive criticism. Appreciation is also extended to the District personnel who assisted in deflection data collection.

T. Scullion

List of Reports

Report No. 409-1, "Estimating Flexible Pavement Maintenance and Rehabilitation Fund Requirements for a Transportation Network," by A. Stein and T. Scullion, presents two enhancements for the Pavement Evaluation System (PES). The first is the development of performance equations for each major distress type, built from regression analysis on condition trends from over 350 random pavements in Texas. The second enhancement is the development of maintenance decision trees based on experienced engineers' opinions, which relate distress levels to a recommended maintenance strategy.

Report No. 409-2, "Implementation of a Microcomputer-based Pavement Management System" by D.R. Smith, C. Cox and T. Scullion investigated the feasibility of building a microcomputer-based PMS for a single Texas District. The system is compatible with the existing state-wide PES but offers several distinct advantages such as access to historic maintenance and pavement condition history, graphical outputs of pavement condition and exception reporting.

Report No. 409-3, "Incorporating a Structural Strength Index into the Texas Pavement Evaluation System" by T. Scullion discusses the addition of a structural Strength Index based on FWD testing to the network-level Pavement Evaluation System.

Abstract

The current Pavement Evaluation System used in Texas rates the condition of pavements in terms of visual distress and present serviceability index. This report discusses the addition of another dimension to the rating system; that of a Structural Strength Index. The Falling Weight Deflectometer is to be used for this purpose. In this report, an overview is given of the FWD and data analysis techniques, a discussion on sample size is presented and two possible structural strength schemes are proposed. The first is a simple statistically based scheme which ranks pavement strength in terms of key deflection bowl parameters, and includes weighting factors for traffic level and rainfall. The second is a mechanistic approach in which a remaining service life is calculated.

These two approaches were pilot tested on data collected in several Texas districts. It was recommended that the statistically based scheme be implemented. Although the mechanistic scheme shows promise at the project level, several factors including; incomplete layer information and insufficient traffic data, currently limit its applicability at the network level.

Keywords: Pavement Management Systems, Deflection Testing, Sample Size Selection, Flexible Pavements, Falling Weight Deflectometer.

Implementation Statement

The statistical structure index scheme described in this report has been implemented within the Department's Pavement Evaluation System. Within this network-level system, pavements are now rated in terms of roughness, visual distress and structural strength. The structural strength is useful in identifying pavements which can be anticipated to deteriorate rapidly under existing traffic and environmental conditions. This information will be of interest to the pavement design and maintenance engineers in charge of selecting maintenance and rehabilitation strategies for distressed pavements.

Disclaimer

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

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1. INTRODUCTION

The Texas Department of Highways and Public Transportation implemented its Pavement Evaluation System in 1981. In this system the State's highway network is broken into sections approximately two miles in length. Pavement evaluation consists of a visual survey of surface distress and a Mays Ride survey of pavement roughness. These ratings are combined to produce a Pavement Score, which is a number between 0 and 100, with 100 being perfect. Pavements with scores below 70 are candidates for maintenance and scores below 40 are candidates for rehabilitation. For the past two years Texas has evaluated 100% of its Interstate, 50% of its US and State Highways and a random sample of 20% of its FM routes.

Information generated by this system has been used for a number of applications including project selection, network funding estimates, reports to the legislature on current network conditions and trend analysis of overall network conditions. Although useful at the network level for strategic planning purposes, this information has only been used on a limited basis by districts who are responsible for generating rehabilitation programs and allocating maintenance funds. A frequently heard complaint against the existing PES system is that pavements that are structurally very weak can be made to "look good" by diligent maintenance work. Seal coats and thin overlays are often applied as a stopgap measure to await adequate funding to perform the required pavement rehabilitation. However the pavement scores on these well maintained highways are frequently in the 80 to 100 range making them ineligible for consideration for rehabilitation.

To address this shortcoming of the PES system a research study was initiated in September 1985 to develop a procedure by which a Structural Index can be calculated to supplement the existing visual and roughness information. The generation of a Structural Index was one of the tasks of the Falling Weight Deflectometers purchased by the State to assist in Pavement Management activities. To adequately address the department's needs, the developed structural strength system must have the following capabilities:

- (a) It must be compatible with the existing PES system.
- (b) It must be capable of including all of the flexible pavement types found in the State ranging from surface treated pavements with 4 inch granular base to thick black base sections.
- (c) It must be capable of processing large volumes of data.
- (d) It must have the ability to process data collected continuously over a wide range of temperatures and seasons.
- (e) The Structural Index must be a number between 0 and 100 with a 100 being a pavement which is structurally strong and will not be susceptible to load associated damage in the near future. Low scores being indicators of pavements which are weak, in which under existing

traffic and environmental conditions rapid changes are expected in pavement condition.

- (f) The model should be available for implementation for the 1987 PES evaluation data collection.

In this report two procedures for generating Structural Indices from Falling Weight Data are evaluated. The first method is a statistical scheme in which distributions of pavement strengths were generated for each pavement type found in the state. The Structural Strength Index (SSI) is related to the percentile level. The second approach is a mechanistic based approach in which the available fatigue and deformation models are used to estimate a remaining life until failure. In this scheme, the SSI is defined as the number of months until structural failure is predicted to occur (with a maximum of 100). These procedures will be described in detail in later sections of this report.

The layout of this report is as follows. In Section 2, a description will be given of the Falling Weight Deflectometer equipment as well as summaries of published information on seasonal variation of deflection, empirical and mechanistic analysis procedures. Sample size selection will be discussed in section 3. Collecting deflection data at the network level is an extremely costly task, all efforts must be made to minimize the number of readings per section. In Section 4 the statistically based structural index scheme will be presented. The mechanistic procedure will be discussed in Section 5. Comparative runs and Conclusions will be given in Sections 6 and 7 respectively.

2. EVALUATION OF FALLING WEIGHT DEFLECTOMETER

The Texas State Department of Highways and Public Transportation has recently purchased seven Falling Weight Deflectometers (FWD) to assist with its overall Pavement Management at the network and project level. One of the tasks of these FWD will be to improve the State's current network-level Pavement Evaluation System by incorporating a Structural Strength Index into the system. In this section of the report the operational characteristics of the Falling Weight Deflectometer will be discussed along with typical pavement response patterns and analysis procedures.

2.1 Description of Falling Weight Deflectometer

The FWD testing system, shown in Figure 1, is trailer mounted and weighs between 1323 and 1875 lbs depending on the weight of the falling mass being used. In Texas the entire unit is towed by a standard passenger van.

The impulse is created by dropping masses from different heights. The system is equipped with different mass levels. By varying the masses and drop heights, force ranges can be achieved of 1500 to 27,000 lbs. The masses are raised hydraulically and released on an electronic signal. The system is equipped with electronic triggers to allow different drop heights without changing trigger locations. These heights are selected to give the desired range of loads.

The masses drop onto a rubber buffer system (different for each mass configuration) to provide a load pulse in approximately a half-sine wave form with 25 to 30 millisecond duration. The load is transmitted to the pavement through an 11.8 in. diameter loading plate. The impulse load is measured using a strain gauge-type load transducer (load cell).

The deflection is measured using up to seven velocity transducers mounted on a bar which is lowered automatically with the loading plate. The bar places six transducers at locations up to 6 ft from the center of the load plate. The other is located at the center of the plate. The velocity transducers (5 Hz, 485 ohms, 0.65 damping coefficient with 1.82 ohm load) are specially designed to insure a linear response within the 25 to 30 millisecond response period. The recommended working range of the sensors is 0 to 80 mils, although readings of up to 100 mils may still be used.

The information from the transducers and load cell are fed into a microprocessor based control unit. The signals from the velocity transducers are processed through an integrating preamplifier, then through an amplifier, and finally through a rectifier to produce a direct current signal which is directly proportional to displacement. The signal from the load transducer is processed the same way except it is not fed through the integrating preamplifier. The results are then fed into a Compaq computer which records them to diskettes, hard copy is also available. The Compaq also controls the complete operation including lowering the loading plate and deflection sensors to the pavement, recording the results, raising the loading plate and sensors, and signalling the operator that the system is ready to be moved to another

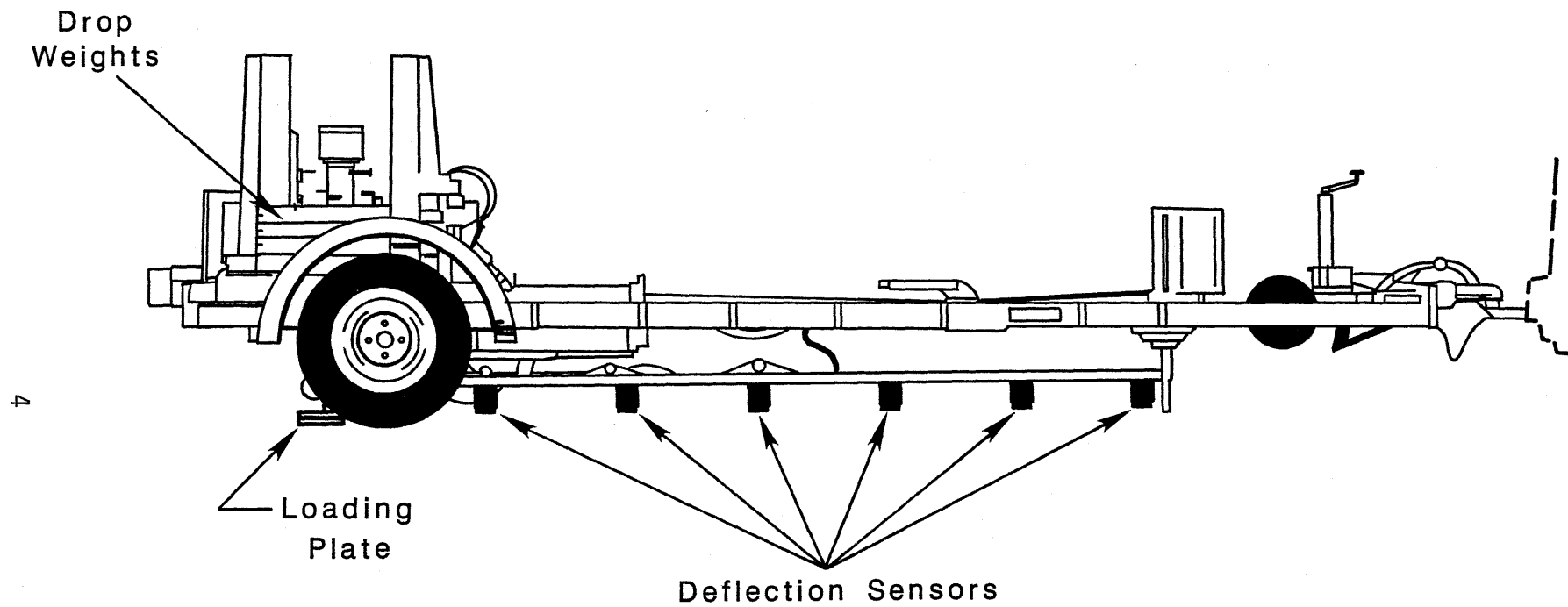


Figure 1. The Falling Weight Deflectometer

site. The force is converted to pressure and processed in units of kilopascals (1 kPa = 0.145 psi). The deflections are processed as micrometers (1 micrometer = 1 micron = 10). However, the system expresses these in pounds and mils so that the display and printed results are in English units.

The controls fit between the front seats of the tow vehicle and provide for an efficient operation with a one person crew. The normal sequence of operation is to move the device to the test point and hydraulically lower the loading plate and transducers to the pavement using the remote control capabilities of the computer. A test sequence is then completed using the number of drops at each height and number of drop heights selected. The loading plate and sensors are then hydraulically lifted, and the device is ready to move to the next site.

The information collected by the FWD is displayed in three different formats. The standard format is shown below in Table 1.

02:43 870504

File: B:\168s0471.fwd
 Road: 08168 sh0471s
 Subsection: 000+0.0 000+0.0

| Stn: | 60 | Lane:R | Temp: 70 | | Cnd: 00 | Air: 81 | | | | | 02:44 |
|---------|-------|--------|----------|-------|---------|---------|------|------|------|--|-------|
| Sto Hgt | psi | lbf | Df1 | Df2 | Df3 | Df4 | Df5 | Df6 | Df7 | | |
| 1 | 53.6 | 5880 | 21.58 | 8.33 | 2.67 | 1.62 | 1.13 | 0.85 | 0.72 | | |
| 2 | 71.4 | 7832 | 27.63 | 10.85 | 3.64 | 2.27 | 1.57 | 1.21 | 1.00 | | |
| 3 | 94.5 | 10360 | 34.55 | 14.03 | 5.01 | 3.12 | 2.18 | 1.66 | 1.36 | | |
| * 4 | 141.4 | 15504 | 44.44 | 18.91 | 7.35 | 4.66 | 3.23 | 2.47 | 2.04 | | |

| Stn: | 70 | Lane:R | Temp: 70 | | Cnd: 00 | Air: 79 | | | | | 02:45 |
|---------|-------|--------|----------|-------|---------|---------|------|------|------|--|-------|
| Sto Hgt | psi | lbf | Df1 | Df2 | Df3 | Df4 | Df5 | Df6 | Df7 | | |
| 1 | 51.3 | 5632 | 24.34 | 7.43 | 2.54 | 1.62 | 1.17 | 0.89 | 0.72 | | |
| 2 | 68.6 | 7512 | 30.23 | 9.87 | 3.64 | 2.27 | 1.61 | 1.21 | 0.96 | | |
| 3 | 93.2 | 10216 | 37.28 | 13.13 | 5.09 | 3.20 | 2.22 | 1.66 | 1.36 | | |
| * 4 | 141.4 | 15512 | 48.09 | 18.08 | 7.43 | 4.70 | 3.23 | 2.43 | 2.00 | | |

Table 1. Typical Output from FWD testing

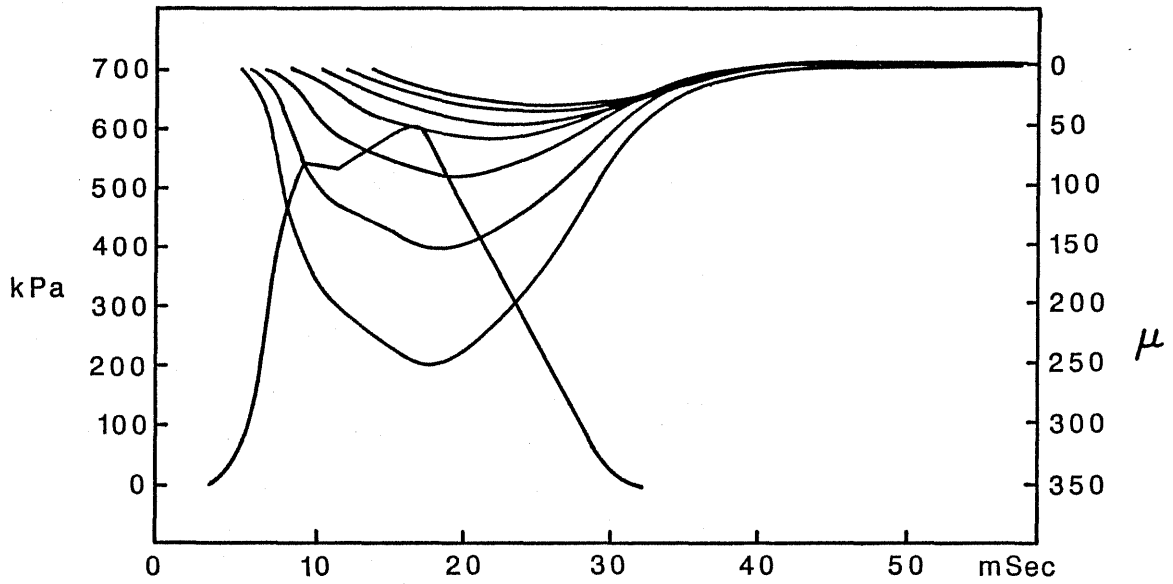
Table 1 shows results from four drops of the FWD with recorded loads of 5880, 7832, 10360, and 15504 lbs; the 7 deflections are given in mils. The air temperature is automatically logged and printed in the header information. Pavement surface temperatures are also routinely measured by the operator during project level evaluations. This information may be printed on paper, stored on floppy disk or both.

A second output which is available is the graphical display of a typical test sequence. This is shown in Figure 2 where the entire load and deflection history of each sensor is plotted. In Figure 2, the load and deflection values are in metric units. Figure 2a presents the maximum pressure and deflection at R=0, 1,2,3,4,5 and 6 ft. from the center of load as a function of time. Figure 2c is a perspective, three-dimensional plot of the surface deflection versus time and distance. The data exhibits dynamic effects. If

| | Peak @ mSec | |
|-----|-------------|-------|
| kPa | 604 | 17.20 |
| Df1 | 251 | 18.43 |
| Df2 | 152 | 19.45 |
| Df3 | 90 | 20.68 |
| Df4 | 60 | 22.30 |
| Df5 | 47 | 24.33 |
| Df6 | 37 | 25.55 |
| Df7 | 32 | 26.38 |

a)

b)



c)

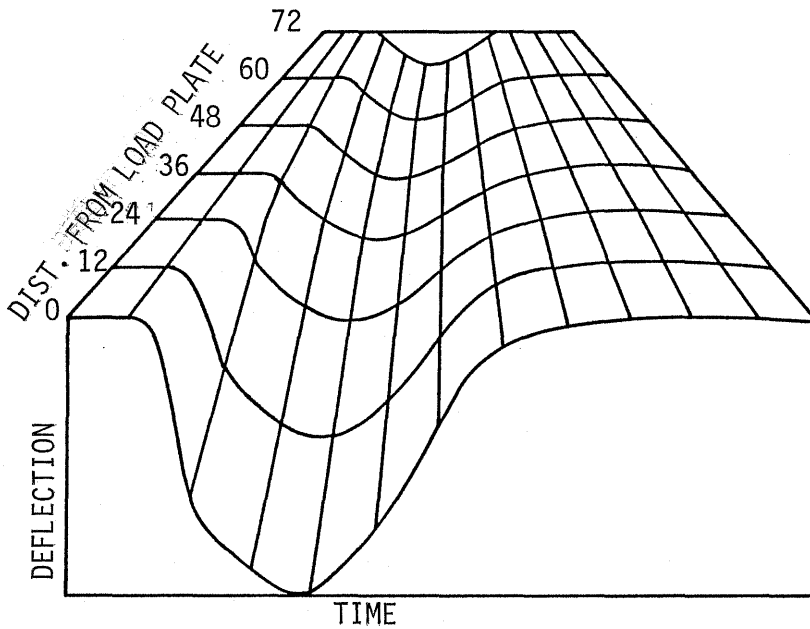


Figure 2 Detailed output from FWD
 2a) Timing and magnitude of peak deflection
 2b) Load and Deflection (7 sensors) v Time
 2c) Perspective view of Deflection v Time

| Section | Pavement Thickness | | | | Times When Peaks Occurred (mSec) | | | | | | |
|---------|--------------------|--------|---------|------|----------------------------------|-------|-------|------|-------|-------|-------|
| | AC | BASE* | SUBB | SUBG | Distance in feet | | | | | | |
| | | | | | 0 | 1 | 2 | 3 | 4 | 5 | 6 |
| 1 | 1 | 4(G) | -- | CLAY | 20.03 | 21.05 | 23.28 | 25.7 | 28.53 | 30.55 | 33.18 |
| 2 | 1 | 4(CL) | 12(CLC) | CLAY | 20.43 | 21.05 | 21.88 | 22.3 | 23.75 | 24.15 | 25.58 |
| 3 | 5 | 12(CL) | 12(CLC) | CLAY | 19.23 | 21.85 | 23.68 | 24.3 | 24.73 | 27.15 | 28.58 |
| 4 | 5 | 4(CL) | 4(CL) | SAND | 19.03 | 19.45 | 20.88 | 23.3 | 23.53 | 26.55 | 26.58 |

Table 2. FWD peak deflection data on various pavement types.

code G = Gravel

CL = Crushed Limestone

CLC = Crushed Limestone + 4% Cement

| | Peak@mSec | |
|--------------|-----------|-------|
| kPa | 593 | 17.00 |
| Df1(μ) | 172 | 21.23 |
| Df2(μ) | 173 | 20.85 |
| Df3(μ) | 177 | 21.88 |
| Df4(μ) | 71 | 27.70 |
| Df5(μ) | 61 | 29.33 |
| Df6(μ) | 48 | 30.55 |
| Df7(μ) | 40 | 31.98 |

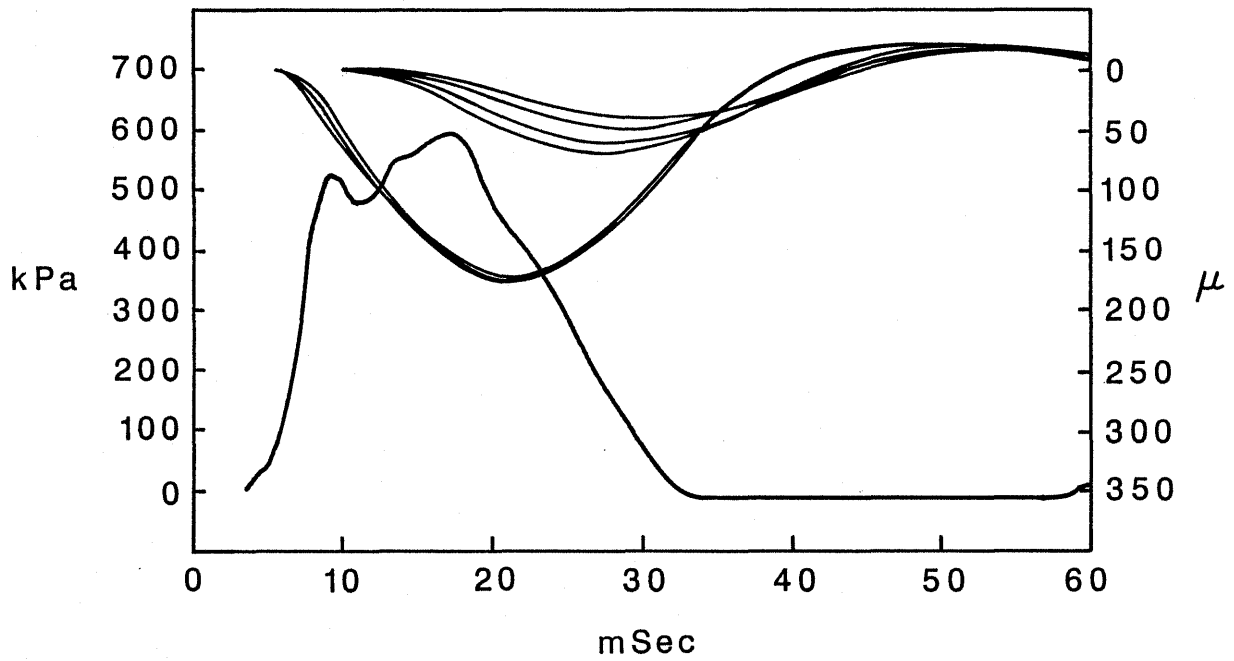


Figure 3. Detailed FWD output on a pavement section with a transverse crack between sensors 3 and 4.

the surface was responding strictly as a static deflection basis, the deflection pulses (as a function of time) would have the same shape as the load pulse. Inspection of Figures 2b and 2c shows that there is some distortion. A time delay or lag is also present. This effect is seen quantitatively in the peak data in Figure 2a, where the wave takes 8 milli seconds to travel 6 feet which corresponds to a surface wave of 750 ft./sec.

The data shown in Figure 2 was recorded on section 9 at the TTI research annex. This being a section with a 5 inch thick asphalt surface on top of a thick crushed limestone base. Shown in Table 2 are the results obtained from different pavement types. The speed of the maximum deflection wave varies from pavement type to pavement type. The data in Table 2 represents wave speeds from 450 ft./sec. to 1165 ft./sec. Further analysis of this data will be discussed later in this section.

The graphical outputs provide a wealth of information which is currently not used in pavement analysis techniques. Figure 3 illustrates the effect a severe transverse crack has on the deflection response. This pavement had a cement stabilized base, and the crack was located between sensors 3 and 4. The crack's effect on wavespeed and deflection are clearly demonstrated.

The third output which is available from the FWD is a digitalized file of load and deflection data. This is in fact the raw data used to generate the graphic output in Figures 2 and 3. The availability of this digitalized file means that there is potential to perform some dynamic type analysis. This topic will be discussed later in this chapter.

2.2 Use of Deflection Measurements to Determine Pavement Structural Properties

One of the major applications of deflection data is to calculate or infer structural information about the individual pavement layers. Several techniques are available, and an overview of each will be given in this section, a more detailed discussion can be found elsewhere (1). The available techniques include:

- 1) Empirical Structural Analysis Methods
- 2) Layered Elastic Methods
- 3) Dynamic Analysis Methods
- 4) Other Methods

Empirical Structural Analysis Methods to determine pavement properties are derived from statements of experience and observations of the strengths of pavements. An example is the procedure used by the Utah Department of Transportation (2). The measured Dynaflect maximum deflection (DMD), Surface Curvature Index (SCI) and Base Curvature Index (BCI), illustrated in Figure 4, are compared to acceptable values for different traffic levels. The relative comparison of these three values can indicate the relative or qualitative strengths of the pavement layers, as indicated in Figure 5. The acceptable

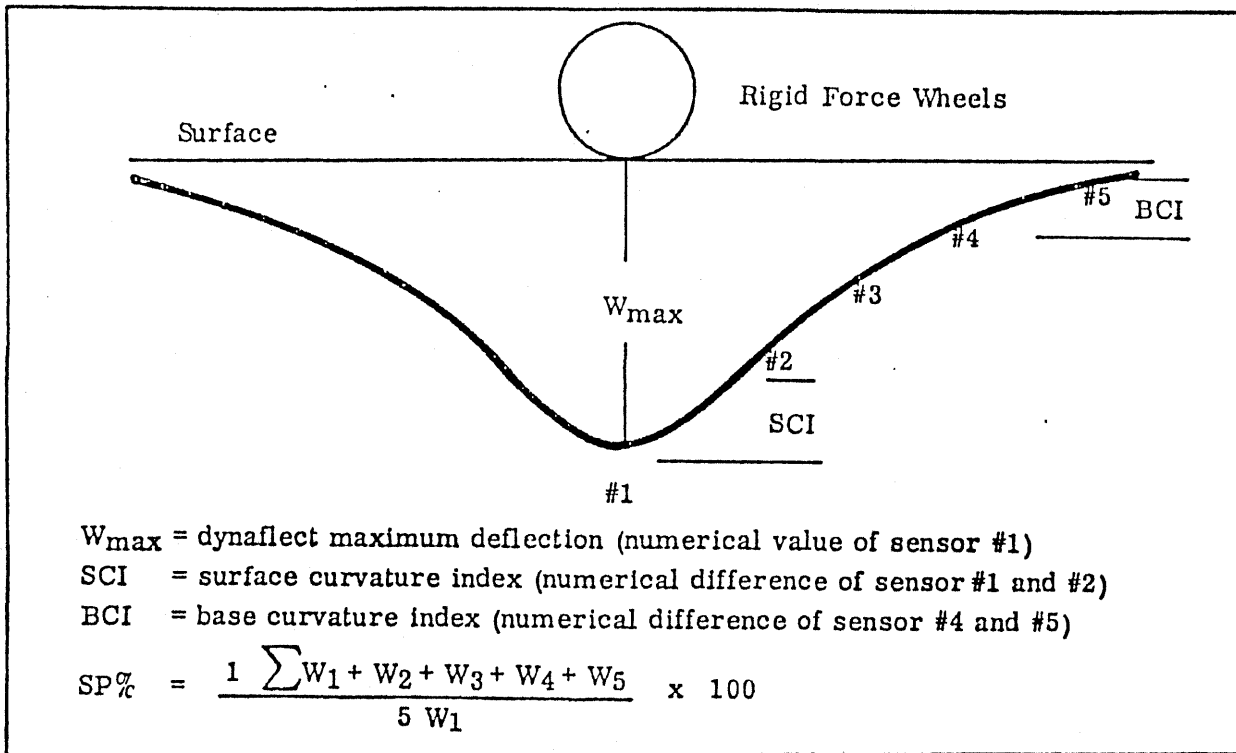


Figure 4. Deflection basin parameters as presented by Utah.

| MAXIMUM (DMD) DEFLECTION (mlis) | SURFACE CURVATURE INDEX (mlis) | BASE CURVATURE INDEX (mlis) | CONDITION OF PAVEMENT STRUCTURE |
|---------------------------------|--------------------------------|-----------------------------|----------------------------------|
| GT 1.25 | GT 0.48 | GT 0.11 | PAVEMENT AND SUBGRADE WEAK |
| | | LE 0.11 | SUBGRADE STRONG, PAVEMENT WEAK |
| LE 1.25 | GT 0.48 | GT 0.11 | SUBGRADE WEAK, PAVEMENT MARGINAL |
| | | LE 0.11 | DMD HIGH, STRUCTURE OK |
| LE 1.25 | GT 0.48 | GT 0.11 | STRUCTURE MARGINAL, DMD OK |
| | | LE 0.11 | PAVEMENT WEAK, DMD OK |
| LE 1.25 | LE 0.48 | GT 0.11 | SUBGRADE WEAK, DMD OK |
| | | LE 0.11 | PAVEMENT AND SUBGRADE STRONG |

GT= GREATER THAN
 LE= LESS THAN OR EQUAL TO

Figure 5. Use of deflection basin parameters to analyze pavement structural layers, (2)

values were selected by comparing performance of in-service pavement with different values for these parameters.

Layered Elastic Methods. All of the layered elastic analyses use assumed elastic moduli of the layers to match the measured deflection basin. The major differences between specific methods are as follows:

1. The layered elastic computer programs used to calculate surface deflections differ from one method to the next.
2. The method used to assume the initial "starting" values of the elastic moduli varies from one method to the next.
3. Some methods attempt to make some correction for the stress dependency of the layer moduli and others do not. In any case, this attempt is at best an approximation.
4. Some methods are capable of handling two layers, others three, and others still more layers. Computation time increases exponentially with an increasing number of layers.
5. The method of searching for new values of the layer moduli differs between analysis methods.

A great number of programs which backcalculate moduli by iterating around an elastic-layered program have been developed. As the number of layers increases, the method used for finding the next set of layer moduli must become more accurate and efficient in order to decrease computer running time. A discussion of the mathematical methods used in searching for the next set of moduli is beyond the scope of this report. It is sufficient to remark on this subject that it is crucial to the efficient operation of the computerized search for layer moduli.

Many layered elastic computer codes are available, and have the capability to compute surface deflections. However, it has been pointed out (3) that in some computer codes, serious anomalies exist when computing surface deflections near the point of load application. The anomalies are due to the numerical methods employed by the computer code. Since this is a critical point at which to calculate deflection, the anomalies can have very serious effects on calculated deflection basin shapes.

The advantage of using a layered elastic method to determine layer moduli from surface deflections is that it is mechanistic, that is, it satisfies the laws of statics and mechanics, and is thus capable of making consistent calculations for two or more layers. The recently published AASHTO Design Guide (4) now advocates the use of backcalculated modulus values for pavement rehabilitation design studies.

Dynamic Analysis Methods. It is well-known that pavements are made up of materials which have properties that are dependent on strain-rate, and which exhibit hysteresis under repeated loading. When an NDT device applies a load to a pavement surface, it excites viscous, friction, and geometric damping and

is resisted by the inertia of the layers. Geometric damping is the loss of energy due to the propagation of waves away from the point of impact. None of these effects are considered by the static analyses that are used in the methods of analysis that are discussed above.

The data presented in Figures 2 and 3 clearly demonstrate that there are some dynamic effects present in FWD testing. Up to the present time, the approach taken with the impulse load deflection data has been to ignore the dynamic effects. There are several levels of sophistication that can be applied to incorporate dynamic effects in the pavement analysis.

If the pavement layers are relatively thick and the ratio of moduli in adjacent layers is small (less than 5), an empirical rule-of-thumb may be used to derive the moduli of the layers. One method that has been used assumes that the average depth at which the wave travels is one third of the wavelength. The modulus of the material at that depth is calculated from the formula for the velocity of a Rayleigh wave:

$$E = 2 (1 + \mu) \frac{\gamma}{g} \left(\frac{v_r}{\alpha} \right)^2$$

where

- v_r = the Rayleigh wave velocity
- γ = the unit weight of the material
- g = the acceleration due to gravity
- μ = Poisson's ratio
- α = a correction factor relating the velocity of a shear wave to that of a Rayleigh wave (usually taken as 0.95)

The simple equation can be applied to the FWD velocity data presented in Table 2, the results are shown below in Table 3. These results are obtained using the peak time data from Section 1 of Table 2. The time the peak arrived at each sensor is subtracted from the time at which the first deflection sensor recorded the peak deflection.

| Sensors | Distance | Average Wave | E |
|---------|----------|----------------|--------|
| | (feet) | Speed (ft/sec) | (psi)- |
| 1 - 2 | 1 | 980 | 66100 |
| 1 - 3 | 2 | 615 | 26000 |
| 1 - 4 | 3 | 529 | 19200 |
| 1 - 5 | 4 | 470 | 15200 |
| 1 - 6 | 5 | 475 | 15500 |
| 1 - 7 | 6 | 456 | 14300 |

Table 3. E - values calculated for Section 1 of Table 2.

The moduli values in Table 3 appear reasonable for the thin pavement under analysis. However, it is difficult to define the value for any particular layer in the structure. This approach requires special analysis if there is a large difference in modulus from one layer to the next or if the layers are thin. Also, it has been noted (1) that a major difficulty with the approach is that the stress levels imposed on the layer materials by the traveling waves are much smaller than those imparted by moving traffic.

An alternative approach has recently been attempted at TTI to apply dynamic analysis techniques for processing FWD data (5). The technique makes use of the PUNCH program developed at MIT (6) and modified at Texas A & M. This is a VISCO-ELASTIC DYNAMIC PROGRAM which computes displacement due to sinusoidal loads on the surface of a layered medium. The fundamental technique which permits the use of such a program is that of Fast Fourier Transform (FFT) which assumes that any waveshape can be broken into its component sine waves. The FFT of a typical FWD load pulse is illustrated in Figure 6.

The PUNCH program inputs the Relative Amplitude of each frequency, as well as pavement layer properties (Elastic Modulus, Poisson Ratio, Damping Factor B) and predicts the resulting dynamic response.

The analysis techniques can be expanded by using the PUNCH program in a backcalculation mode in which the FFT of the loading and response signal can be input and the program will backcalculate in place material properties, such as moduli and damping factors. By performing such an analysis, the information generated has great potential for improving pavement analysis techniques. For example, the Damping Factor of the surfacing layer could indicate the presence of a brittle surface, low damping factors approaching zero would indicate an aged surface whereas high values would indicate a surface with good flexibility. Furthermore, it has been proposed (7) that the slope of the Log Frequency versus Log Deflection Curve is a measure of the creep properties of the material in place, and this could form the basis of an acceptance test for asphalt materials.

The possibility of routinely applying these advanced analysis techniques to FWD deflection data is the subject of a current research project (8).

Other Methods. Other methods that are used to determine layer properties from FWD deflection data include regression equations that have been developed from multiple runs of an elastic layered or finite element program. An example of this approach is the set of equations developed by Hoffman and Thompson (9).

Equations and nomographs were developed for the test data for three types of pavement: (1) conventional flexible pavement on granular base course, (2) asphalt concrete on stabilized base course, and (3) full depth asphalt. Numerous computations with the ILLI-PAVE finite element computer program were used to establish the equations for maximum deflection (D_0) and the normalized cross-sectional area (AREA) of the deflection basin out to the sensor at a 3 ft (91cm) distance from the center of the loading plate. Non-linear stress strain characteristics of the base course and subgrade were incorporated into

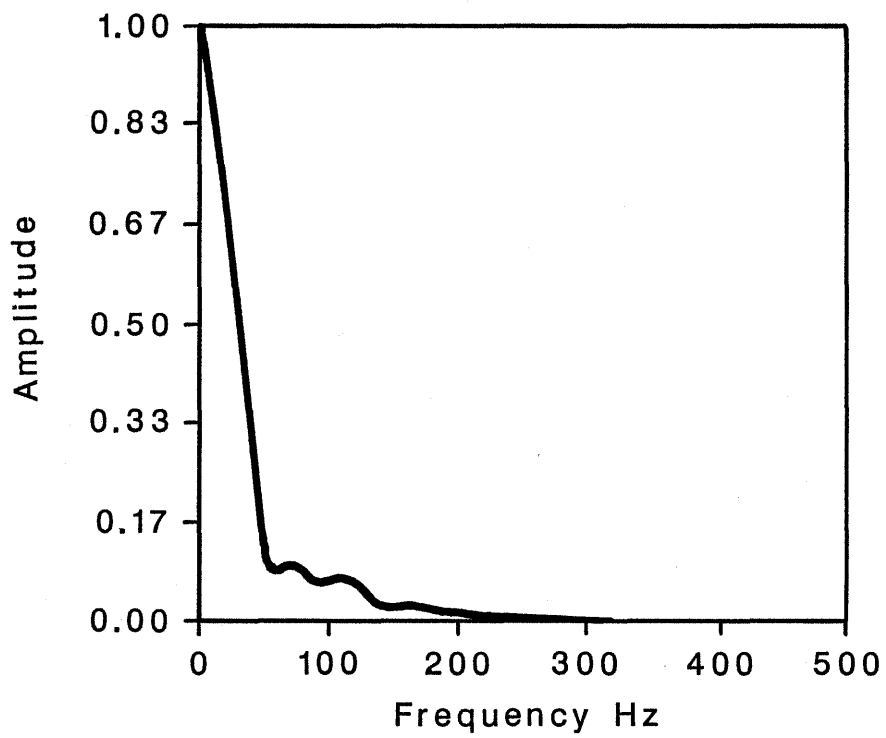


Figure 6. Fast Fourier Transform plots of FWD load pulse illustrating the relative amplitudes of the frequency components of the load pulse.

the analysis and into the equations for Do and AREA, as well. Both equations are of the form,

$$\log Do \text{ or AREA} = a + b \times Eri + c \times EAC + d \times TAC + e \times Tgr$$

where

Eri = the "break point subgrade resilient modulus, a non-linear material property of the subgrade.

EAC = the modulus of the asphalt concrete

TAC = the thickness of the asphalt concrete

Tgr = the thickness of the granular base

a,b,c,d,e = constants determined by regression analysis

With the two thicknesses known, the two equations can be used to solve for the two unknown moduli. With full depth asphalt, the thickness of the granular base is zero, and again the two equations can be used to solve for the moduli of the asphaltic concrete and subgrade. In developing these two simultaneous equations, the properties of the base course were assumed to be known. In this case, the modulus was assumed to be dependent exponentially on the mean principle stress, and the same relation was used in all of the finite element analyses.

This initial approach has recently been simplified by Thompson (10) and some simple regression equations have been developed which relate AC Modulus (EAC) and Subgrade Soil resilient modulus (Eri) to basic NDT parameters (Do, Di, D2, D3, Area) AC thickness and granular base thickness. Typical equations are shown in Table 4, these being for Full Depth Asphalt Concrete Pavements.

The simplicity of these equations is appealing. They are suitable for use in the field as the data is being collected possibly to identify area where additional deflection data needs to be collected.

2.3 Remaining Life Estimates

The purpose of this report is to identify a procedure by which the Falling Weight Deflectometer can be used at the network level to calculate a Structural Strength Index. This index is basically a remaining life index. A small number would indicate that the remaining life is low. In this section of the report, it is proposed to summarize some of the existing remaining life procedures. These procedures are either empirical or mechanistic based.

Empirical determinations of remaining life consist of direct empirical relationships between measured deflections and pavement life. Such relationships are included in the procedures of the Asphalt Institute (11) and the Transport and Road Research Laboratories of Great Britain (12). The Asphalt Institute design chart is shown in Figure 7, and the TRRL procedure in

Table 4. FULL DEPTH ASPHALT CONCRETE EQUATIONS (10)

SUBGRADE RESILIENT MODULUS - ERI

1. $ER_i = 24.7 - 5.41 D_3 + 0.310 D_3$
R = 0.98 SEE = 0.64
2. $ER_i = 26.3 + 1.67 D_3 - 42.28 \text{ LOG } D_3$
R = 0.98 SEE = 0.63
3. $\text{LOG } ER_i = 2.87 - 0.13 D_3 - 1.2 D_3 - 0.58 \text{ LOG } D_0$
R = 0.99 SEE = 0.04 D2
4. $\text{LOG } ER_i = 1.57 - 0.18 D_3 + 0.056 \text{ LOG } D_3$
R = 0.98 SEE = 0.063

ASPHALT RESILIENT MODULUS - EAC

5. $\text{LOG } EAC = 5.28 + 0.105 \text{ AREA} - 3.52 \text{ LOG AREA} + 0.30 \text{ AREA} - 0.98$
R = 0.98 SEE = 0.047
6. $\text{LOG } EAC = 11.012 + 0.322 \text{ AREA} - 7.704 \text{ LOG AREA} - 6.192 D_2 - 0.87$
R = 0.74 SEE = 0.191

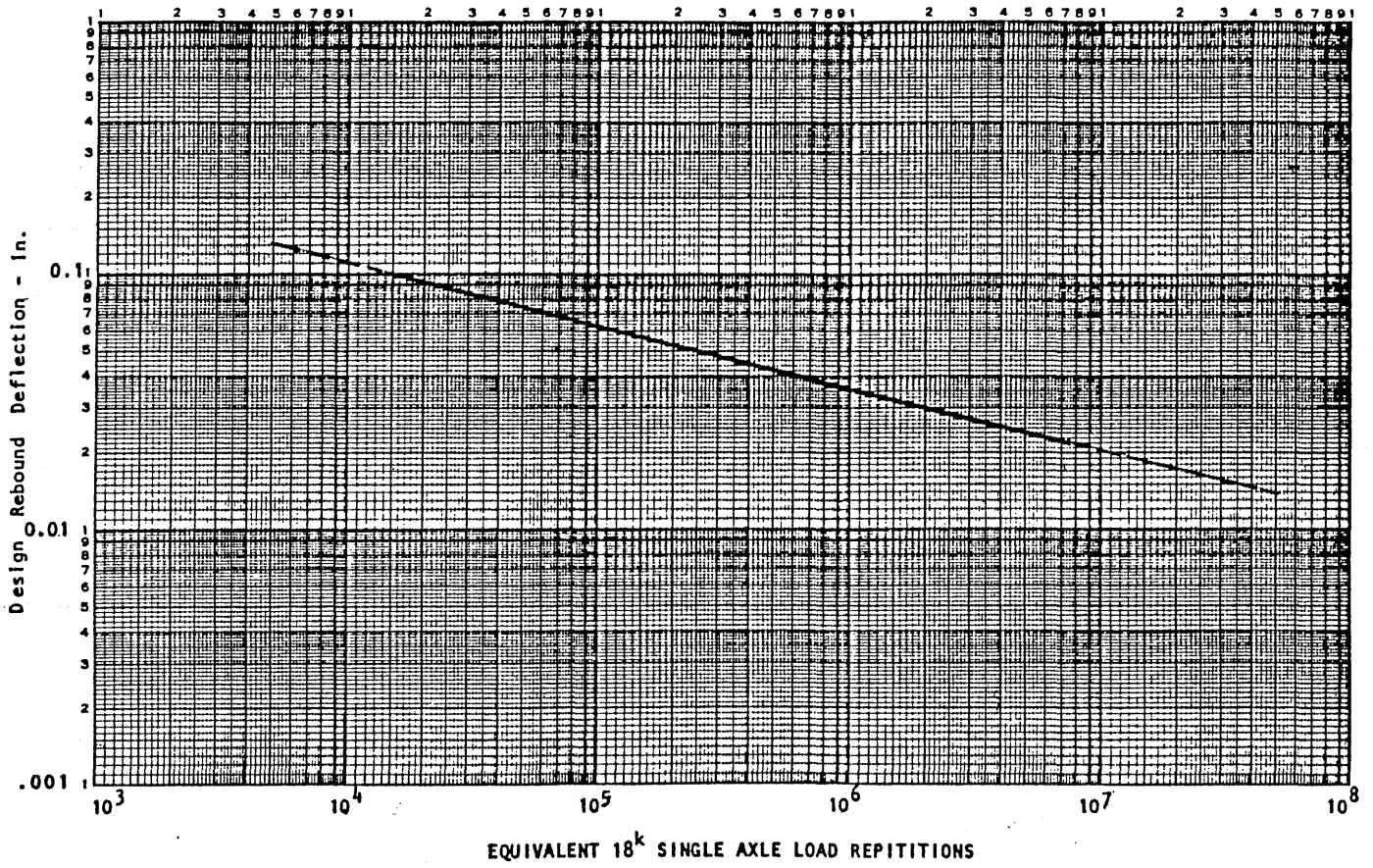


Figure 7. Design rebound deflection chart, The Asphalt Institute (11).

Figure 8. In the TRRL procedure, the measured deflections are adjusted to reflect the influence of temperature.

In mechanistic determination of remaining life, the layer moduli are first determined from analysis of nondestructive test data. Then, using a mechanistic analysis method such as elastic layered programs, the required strains can be computed, as illustrated in Figure 9. The pavement life is then estimated with regard to specific distress criteria, such as fatigue and rutting. These estimates require empirical field calibration. Pavement life can, for example, be related to the asphalt concrete maximum strain as follows:

$$N_f = K_1 \left(\frac{1}{\epsilon_1} \right)^{K_2}$$

where

N_f = the number of repetitions of the standard design load to failure

ϵ_1 = the mechanistically determined maximum asphalt concrete strain

K_1, K_2 = regression constants obtained from an analysis of fatigue data

Similarly, failure by rutting can be related to the maximum compressive strain in the subgrade.

The remaining life can then be calculated using the cumulative damage hypothesis as:

$$N_r = N_f \left(1 - \frac{N_p}{N_f} \right)$$

where

N_r = the number of repetitions of the standard design load still allowable (remaining life)

N_f = the number of repetitions of the standard design load to failure

N_p = the number of repetitions of the standard design load to date

For the consideration of a traffic mix, this can be written as

$$N_r = N_f \left[1 - \sum \left(\frac{N_i}{N_{f_i}} \right) \right]$$

where

N_i = actual number of applications at strain level i , and
 N_{f_i} = number of applications to failure at strain level i .

2.4 Daily and Seasonal Temperature Corrections

In a network-level analysis it is anticipated that deflection data will be collected eight hours a day and at several different times throughout the year. Therefore if the aim is to be able to compare the structural strength of pavements on a common basis then two corrections are needed. These being

- (a) a daily correction factor to convert to a standard monthly average temperature (Large swings in surface temperature have been observed, in February 1986 morning surface readings were 32°F by mid-afternoon the surface temperature was 115°F).
- (b) a seasonal correction factor so that for example data collected in the spring can be compared with that collected in the fall.

Daily Correction Factors - FWD data was collected in February and August 1986 at the TTI Research Annex to monitor variations in deflection with temperature. Four FWD deflections were taken on each section at the 5000, 9000, 12000 and 15000 lb load levels. Typical results from three sections are tabulated below, these being for:

Section 0 A surface treated pavement with 6 inch granular base
 Section 11 A 1 inch HMAC surface on a thick granular base
 Section 9 A 5 inch HMAC surface on a thick granular base.

| DATE | TIME | SURFACE TEMP °F | LOAD | W1 | W2 | Deflection (Mils) | | | | |
|-------|-------|--------------------|------|------|------|-------------------|-----|-----|-----|-----|
| | | | | | | W3 | W4 | W5 | W6 | W7 |
| 27/02 | 9:34 | 55 | 9840 | 58.2 | 27.1 | 11.3 | 6.4 | 4.7 | 3.5 | 2.8 |
| 27/02 | 15:14 | 67 | 9192 | 64.3 | 28.3 | 11.5 | 6.3 | 4.4 | 3.8 | 2.8 |
| 13/08 | 7:21 | 88 | 9032 | 62.3 | 22.9 | 10.8 | 6.5 | 4.4 | 3.4 | 2.8 |
| 13/08 | 10:37 | 97 | 8880 | 55.4 | 23.8 | 11.3 | 6.5 | 4.6 | 3.5 | 2.8 |
| 13/08 | 14:02 | 112 | 9120 | 49.5 | 26.7 | 10.1 | 6.0 | 4.2 | 3.2 | 2.6 |

Table 5. Typical Temp. v. deflection data on a surface treated Pavement (Section 0) at the TTI Research Annex.

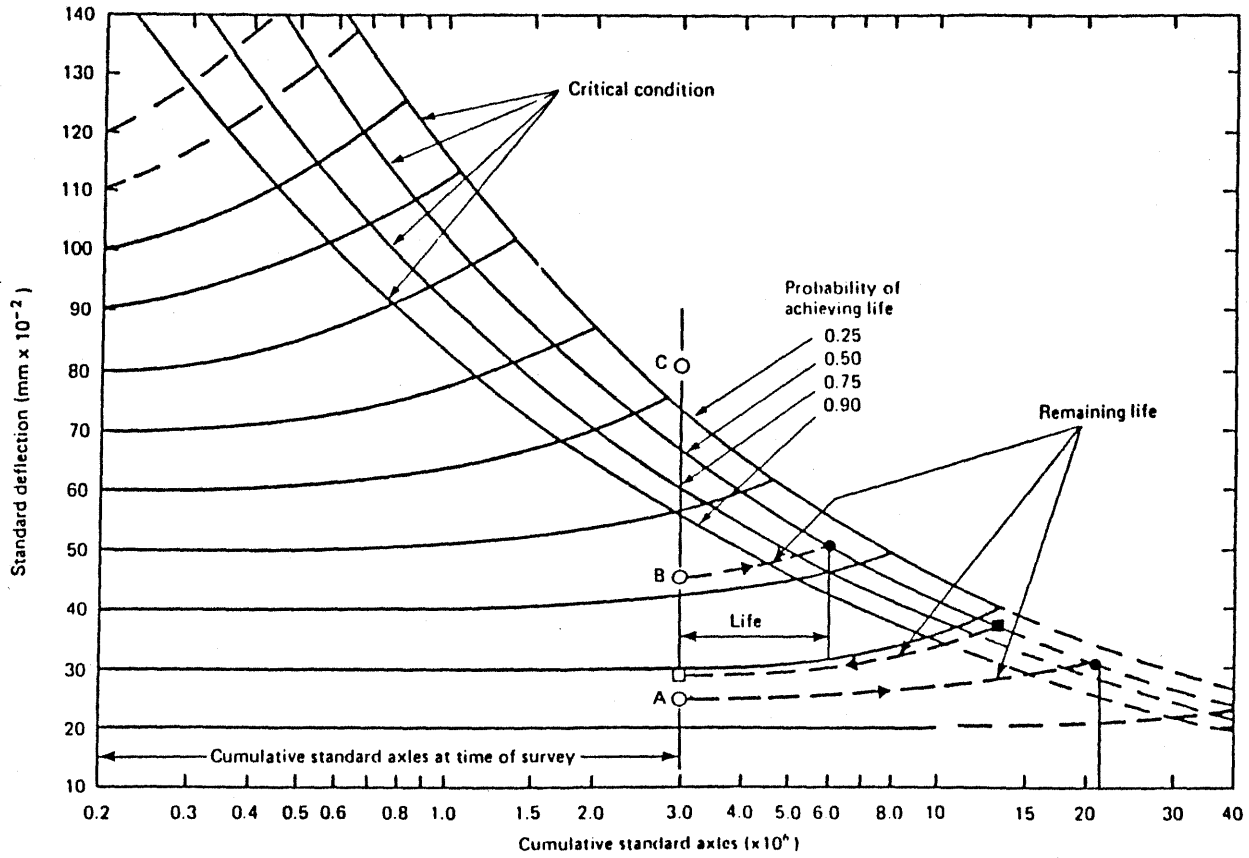


Figure 8. Relation between standard deflection and life for pavements with granular road bases whose aggregates exhibit a natural cementing, section-design example, TRRL, (12).

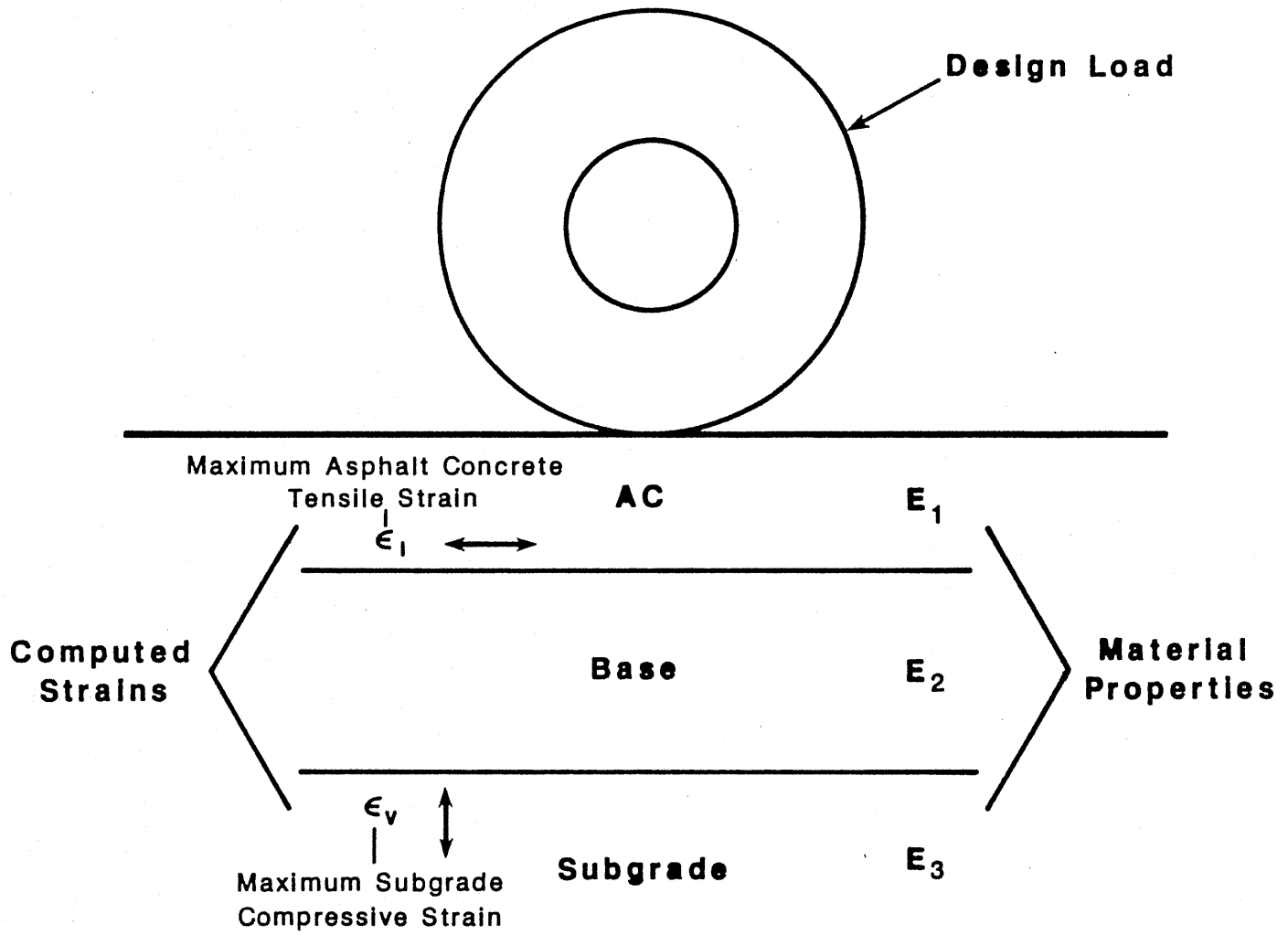


Figure 9. Use of layer moduli to determine critical strains for distress prediction.

| DATE | TIME | SURFACE TEMP °F | LOAD | W1 | W2 | Deflection (Mils) | | | | |
|-------|-------|--------------------|------|------|-----|-------------------|-----|-----|-----|-----|
| | | | | | | W3 | W4 | W5 | W6 | W7 |
| 27/02 | 9:45 | 56 | 9864 | 23.3 | 8.7 | 4.0 | 2.8 | 2.1 | 1.7 | 1.3 |
| 27/02 | 15:24 | 68 | 9312 | 24.8 | 8.7 | 4.1 | 2.6 | 2.0 | 1.6 | 1.2 |
| 13/08 | 7:35 | 86 | 9176 | 18.7 | 6.8 | 3.3 | 2.2 | 1.8 | 1.6 | 1.2 |
| 13/08 | 10:56 | 95 | 9096 | 18.4 | 7.0 | 3.4 | 2.4 | 1.9 | 1.6 | 1.3 |
| 13/08 | 14:20 | 114 | 8946 | 17.9 | 6.7 | 3.3 | 2.3 | 1.9 | 1.5 | 1.3 |

Table 6. Temperature v Deflection data on a thin asphalt pavement (Section 11) at TTI Research Annex.

| DATE | TIME | SURFACE TEMP °F | LOAD | W1 | W2 | Deflection (Mils) | | | | |
|-------|-------|--------------------|-------|------|-----|-------------------|-----|-----|-----|-----|
| | | | | | | W3 | W4 | W5 | W6 | W7 |
| 27/02 | 9:49 | 55 | 10008 | 8.8 | 6.2 | 3.9 | 2.6 | 1.9 | 1.5 | 1.2 |
| 27/02 | 15:27 | 67 | 9496 | 9.7 | 6.4 | 3.8 | 2.5 | 1.8 | 1.4 | 1.2 |
| 13/08 | 7:32 | 88 | 9288 | 9.9 | 5.9 | 3.3 | 2.2 | 1.7 | 1.4 | 1.0 |
| 13/08 | 10:53 | 97 | 9128 | 11.1 | 6.2 | 3.5 | 2.4 | 1.8 | 1.4 | 1.2 |
| 13/08 | 14:31 | 112 | 8984 | 12.1 | 6.2 | 3.0 | 2.1 | 1.6 | 1.3 | 1.0 |

Table 7. Temperature v Deflection data on a thick asphalt pavement (Section 9) at TTI Research Annex.

The data are shown graphically in Figure 10. In this figure, the maximum deflections have been normalized to 9000 lbs using linear interpolation. From the limited information presented in the above tables and figure 10, the following conclusions can be drawn.

1. The sensors closest to the load are those most affected by changing temperature and seasons. The W6 and W7 sensors show little variation. This would imply that the subgrade support is remaining relatively constant on these sections. This is reasonable as these experimental sections have an impermeable asphaltic membrane at a depth of 55 inches below the surface.
2. The thicker asphalt pavements behave in an expected manner, i.e. that as the surface temperature rises the maximum deflection increases. The temperature correction factors generated for

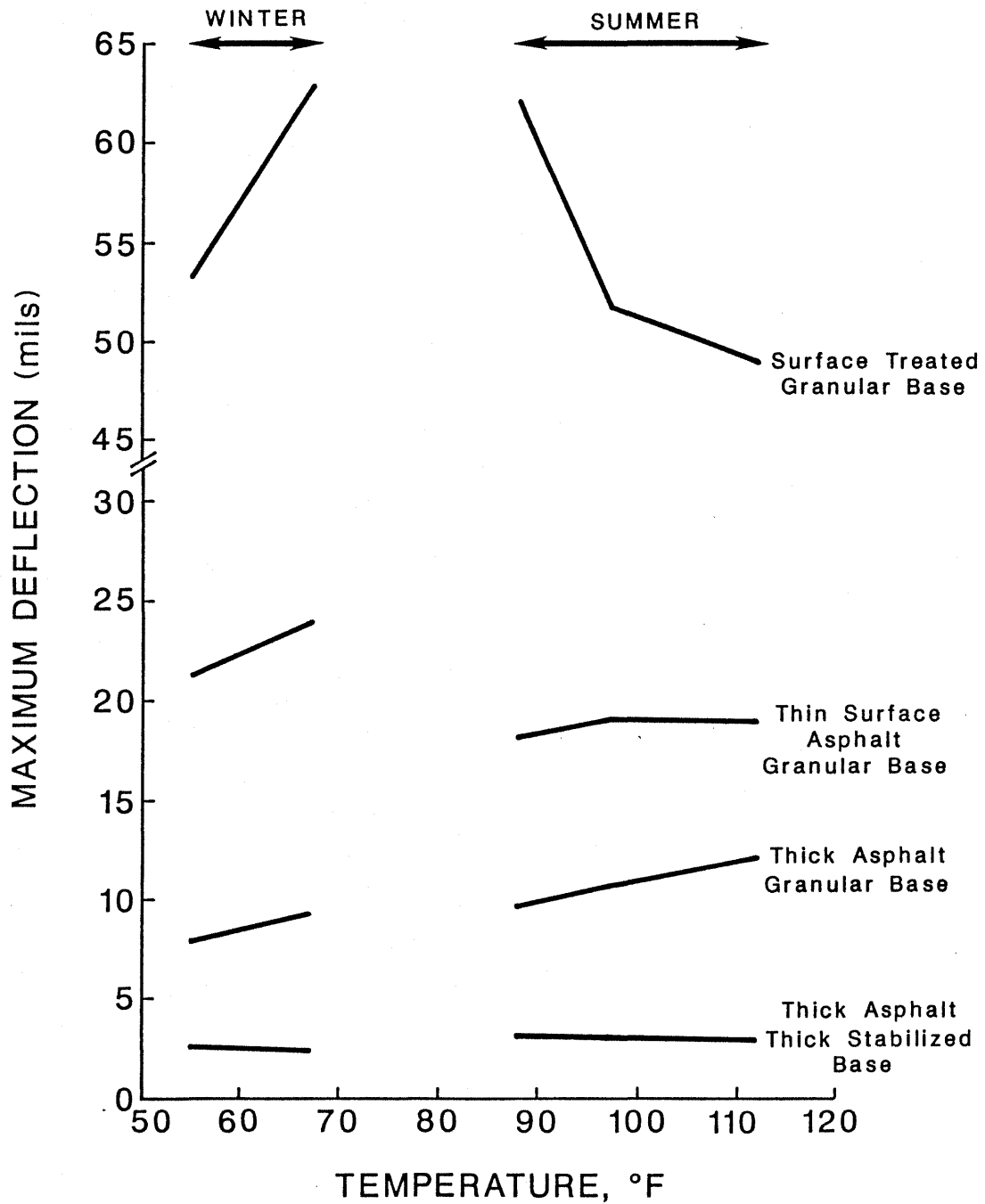


Figure 10. Relationship between Maximum Deflection at a FWD loading of 9000 lbs. versus Surface Temperature. Measured on several experimental pavements at the TTI Research Annex.

these thicker pavements were similar to those recommended in Appendix L of the new AASHTO Design Guide (4).

3. The surface treated pavements show a different deflection versus temperature behavior. At the higher temperatures as the temperature rises, the maximum deflection decreases. In Texas, the Dynaflect Surface Curvature Index (W1-W2) has been found to be a good indicator of upper pavement strength. Applying this concept to the data shown in Table 5, the following FWD Surface Curvature Indices are calculated.

| Temperature °F | FWD SCI |
|-------------------|------------|
| 88 | 39.4 |
| 97 | 31.6 |
| 112 | 22.8 |

Table 8. SCI v Temperature for Surface Treated Pavements

This would imply that the upper layers of the pavement are becoming stiffer as the temperature rises. As these pavements are basically a surface treatment on top of a granular base. It would indicate that the expansion of the granular base is causing it to "lock up."

The aim of this work was to be able to develop temperature correction factors so that deflections collected throughout the day can be normalized to a fixed temperature (say 70°F the mean monthly temperature). The information collected in this initial work is limited in that it only considered experimental pavements for a short period of time over a small temperature range. Follow up work is underway on instrumental test sites (moisture and temperature) in order to obtain a better understanding of temperature correction factors. However, if the existing data were to be used, the following corrections would be appropriate.

| Pavement Type | Temp. Correction | |
|-----------------|------------------|-----------------------|
| | Winter | FWD mils/°F Summer |
| Surface Treated | +0.82 | -0.50 |
| Thin/Medium AC | +0.05 | +0.05 |
| Thick AC | +0.12 | +0.12 |

Table 9. Maximum Deflection (W1) Correction Factors

The temperature corrections for surface treated pavements are extremely large and their use is not recommended. Large swings in surface temperatures, up to 70°F, have been recorded in a single day. In these instances, the correction factor would be overwhelming. The temperature corrections for thicker pavements are reasonable and match those given in the new AASHTO Design Guide (4).

Seasonal Correction Factors - The most comprehensive study of seasonal deflection patterns in Texas is that conducted by Poehl and Scrivner in the early 1970s (13). These researchers took monthly Dynaflect readings on over 30 sections located around the state of Texas. One of the major findings of their study is shown in Figure 11. This identifies the months which there is a high probability that above-average deflections would be measured. It is noted that high deflection could be anticipated in the summer months in East Texas.

As this is the only seasonal deflection data available for Texas, additional processing of the data was conducted for this study. The aim of this analysis was to generate monthly deflection correction factors.

The first step in the analysis was to divide the state into 5 zones which exhibited similar deflection patterns. The five zones chosen are shown in Figure 12. For each zone, the near monthly surface temperatures were calculated and are shown in Table 10. The deflection data was then grouped by zone and pavement type. For this analysis, two pavement types were selected: thin and surfacings less than 2.5 inches thick (Pavement Types 6 or 10 in the Texas PES system) and other pavement types having greater than 2.5 inches of asphalt. The average maximum dynaflect deflection for each of these groups is shown in Table 4. This table predicts how the maximum deflection on any pavement will vary from month to month. The use of this table will be demonstrated in section 5 of this report when the mechanistic structural strength index is presented. Its basic use is to take any monthly deflection measurement and predict what deflection could be anticipated for each month of the year.

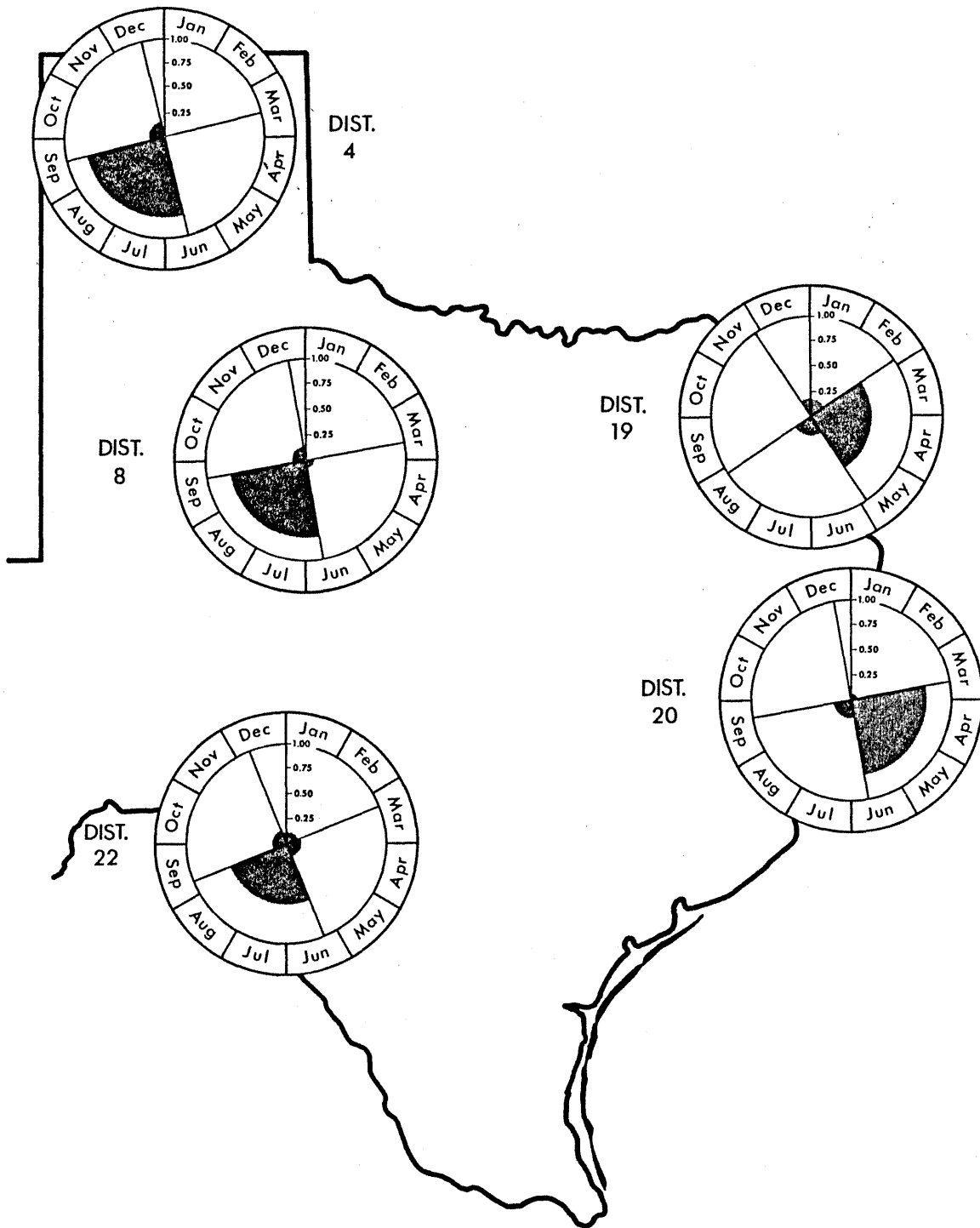


Figure 11. Annual clocks identifying the probability of an above average deflection. (13)

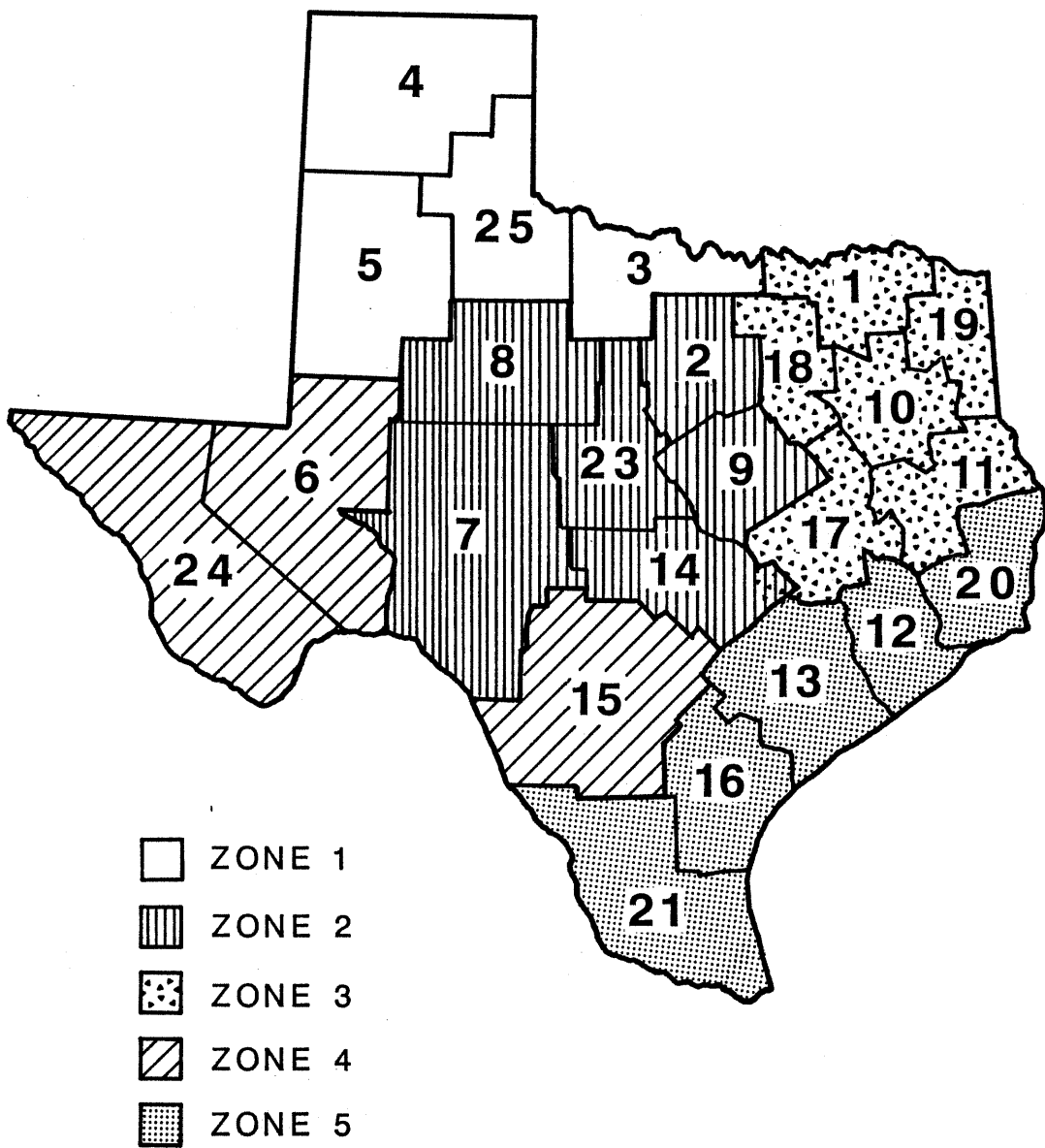


Figure 12. Environmental zones assumed to have similar deflection patterns from data presented in (13).

Table 10. Mean Temperature Data for 12-Month Test Period (13).

| Zone | Dec. | Jan. | Feb. | Mar. | Apr. | June | June | Aug. | Sept | Oct. | Nov. |
|------|------|------|------|------|------|------|------|------|------|------|------|
| 1 | 48 | 40 | 39 | 66 | 87 | 103 | 116 | 104 | 82 | 49 | 60 |
| 2 | 65 | 72 | 53 | 76 | 100 | 86 | 118 | 112 | 93 | 61 | 61 |
| 3 | 56 | 46 | 66 | 78 | 89 | 100 | 114 | 89 | 90 | 70 | 70 |
| 4 | 55 | 76 | 82 | 96 | 103 | 114 | 112 | 108 | 106 | 98 | 67 |
| 5 | 66 | 74 | 71 | 77 | 95 | 93 | 110 | 118 | 99 | 93 | 63 |

Table 11. Average maximum dynaflect deflection readings for each month, by environmental zone and pavement type. (13)

| Zone | P.T. | D | J | F | M | A | M | J | J | A | S | O | N |
|------|--------|------|------|------|------|------|------|------|------|------|------|------|------|
| 1 | 6,10 | 1.0 | 0.98 | 0.96 | 1.00 | 1.09 | 1.13 | 1.14 | 1.16 | 1.12 | 1.10 | 1.03 | 1.02 |
| 1 | Others | 1.0 | 0.91 | 0.95 | 1.01 | 1.16 | 1.23 | 1.20 | 1.29 | 1.21 | 0.93 | 0.95 | 0.93 |
| 2 | 6,10 | 1.06 | 1.04 | 1.02 | 1.02 | 1.11 | 1.05 | 1.14 | 1.21 | 1.17 | 1.15 | 1.10 | 1.08 |
| 2 | Others | 1.03 | 1.09 | 0.96 | 1.13 | 1.28 | 1.42 | 1.44 | 1.52 | 1.30 | 1.02 | 0.97 | 1.00 |
| 3 | 6,10 | 1.15 | 1.07 | 1.18 | 1.19 | 1.26 | 1.22 | 1.11 | 1.08 | 1.06 | 1.02 | 1.01 | 1.08 |
| 3 | Others | 0.79 | 0.78 | 0.92 | 0.87 | 0.84 | 0.78 | 0.68 | 0.59 | 0.66 | 0.68 | 0.72 | 0.75 |
| 4 | 6,10 | 0.63 | 0.64 | 0.61 | 0.61 | 0.67 | 0.75 | 0.72 | 0.67 | 0.60 | 0.60 | 0.63 | 0.58 |
| 4 | Others | 0.63 | 0.64 | 0.61 | 0.61 | 0.67 | 0.75 | 0.72 | 0.67 | 0.60 | 0.60 | 0.63 | 0.58 |
| 5 | 6,10 | 0.96 | 1.19 | 1.30 | 1.28 | 1.22 | 1.18 | 0.97 | 0.87 | 0.81 | 0.78 | 0.87 | 0.91 |
| 5 | Others | 1.04 | 1.13 | 1.14 | 1.22 | 1.21 | 1.35 | 1.45 | 1.29 | 1.26 | 1.22 | 1.05 | 1.05 |

3. SAMPLE SIZE SELECTION

Performing network-level deflection surveys is extremely expensive. In Texas, the highway network has been broken into sections each approximately two miles long. A key question is "How many deflection readings should be taken to adequately quantify the structural strength of these two mile sections?" The approach taken to answer this question is described in this section of the report.

A distinction needs to be made between network and project level NDT evaluations. At the project level, NDT is used to identify weak sections within a project and to design the appropriate rehabilitation strategy. At the network level, within the Texas Pavement Evaluation System, the objective is to generate a structural strength index with a range 0 to 100. Low scores would indicate a pavement section which appears inadequate for the anticipated traffic loadings. Therefore, the goal of the network analysis is simply to rank sections as strong or weak when compared with other sections of the same pavement type. For network level evaluation, the problem is therefore defined as "how many readings per section should be taken to effectively rank the section?"

3.1 Approach Taken

To establish a sample size, the following approach was taken.

- (1) Eight Farm-to-Market road sections in Texas were tested in this study. A minimum of 40 Falling Weight Deflectometer (FWD) readings were taken at 150 feet intervals.
- (2) The means and standard deviations of W1, the maximum FWD deflection, W1-W2, W7, and other statistics were calculated for each section.
- (3) Repeated procedure (2), assuming 20 readings per section were taken instead of the original 40 by selecting every other sample. Do the same for 10, 7, 5, 4 and 2 readings per sections.
- (4) Tabulated the mean values obtained above and rank the sections, from lowest to highest in order of mean deflection value. Based on 40 readings, the section which had the smallest mean value should have had a rank = 1 (strongest), the section which has the largest value should have had a rank = 8 (weakest). See Table 12 and Table 13 for the Maximum Deflection W1 ranking results.
- (5) The rankings obtained in step (4) by using 40 readings per section were assumed as the correct rankings. They were used to compare the rankings based on 20, 10, ..., 2 readings per section by applying rank correlation technique (the Spearman rank correlation coefficient [Rs]) as follows;

In the Spearman rank correlation technique, the ranking obtained with the 40 readings is denoted as X_i , and the ranking denote with the smaller sample is denoted as Y_i . The hypothesis is

- Ho: The two rankings are independent
 H1: There is a direct relationship between the two rankings

The Spearman coefficient (R_s) is defined as follows

$$R_s = 1 - \frac{6 \sum d_i^2}{n(n^2-1)}$$

where

$$\sum d_i^2 = \sum_{i=1}^N (X_i - Y_i)^2$$

The value of $R_s = 1$ when the X and Y rankings are identical. $R_s = -1$ when the rankings are inverse. The statistical decision rules for accepting or rejecting the null hypothesis H_0 , involves establishing a level of confidence α . A critical value of R_s is obtained by lookup, see Table 14. The null hypothesis will be rejected (two rankings are independent) if the calculated values of R_s is greater than the critical value.

Table 14. Critical Values of R_s for $n=8$

| | | | | | | |
|----------|---------|-------|-------|-------|-------|-------|
| α | : .001 | .005 | .01 | .025 | .05 | .10 |
| R_s | : .9286 | .8571 | .8095 | .7143 | .6190 | .5000 |

3.2 Recommendations

The data from these sections was analyzed in several ways. Rankings were produced in terms of Maximum Deflection, Surface Curvature Index (W1-W2), W5-W7, and others. Several tables similar to those shown in Tables 12 and 13 were generated. On review of all of these tables it was concluded that five deflection readings per section would be optimum for ranking purposes. For each case, below five readings per section erratic changes in rank were possible. Above five readings the rankings were, in general, more consistent; however, this was not always the case as shown in Table 13, erratic changes occurred when moving from 10 to 7 to 5 readings per section. However, when all the strength criteria were considered it was concluded that 5 readings would optimize the conflicting requirements of accuracy and cost of data collection.

Table 12. Mean FWD Maximum Deflections from Different Sample Sizes.

| Sample Sizes | Road Sections | | | | | | | |
|-----------------|---------------|-------|-------|-------|-------|---------|--------|---------|
| | FM785 | FM251 | FM249 | FM323 | FM974 | FM3058B | FM1362 | FM3058A |
| 40 | 41.36 | 44.06 | 42.95 | 35.84 | 48.21 | 44.45 | 65.08 | 51.27 |
| 20 | 41.57 | 42.33 | 44.52 | 34.29 | 46.42 | 45.17 | 69.95 | 51.72 |
| 10 | 39.18 | 44.32 | 47.05 | 37.14 | 48.54 | 56.30 | 69.63 | 52.51 |
| 7 | 48.64 | 40.34 | 37.74 | 36.08 | 38.22 | 46.27 | 76.71 | 48.62 |
| 5 | 41.45 | 48.27 | 41.79 | 36.50 | 53.99 | 47.55 | 67.72 | 45.82 |
| 4 | 47.88 | 55.62 | 24.75 | 36.86 | 50.68 | 45.66 | 51.36 | 46.58 |
| 2 | 44.41 | 60.58 | 21.67 | 43.66 | 47.52 | 45.23 | 55.30 | 35.56 |

Table 13. Rankings of Sections Based on Maximum FWD Deflections and the Computed Spearman Rank Correlation Coefficient.

| | FM785 | FM251 | FM249 | FM323 | FM974 | FM3058B | FM1362 | FM3058A | R_s |
|----|-------|-------|-------|-------|-------|---------|--------|---------|-------|
| 40 | 2 | 4 | 3 | 1 | 6 | 5 | 8 | 7 | 1.00 |
| 20 | 2 | 3 | 4 | 1 | 6 | 5 | 8 | 7 | .976 |
| 10 | 2 | 3 | 4 | 1 | 5 | 7 | 8 | 6 | .905 |
| 7 | 7 | 4 | 2 | 1 | 3 | 5 | 8 | 6 | .571 |
| 5 | 2 | 6 | 3 | 1 | 7 | 5 | 8 | 4 | .833 |
| 4 | 5 | 8 | 1 | 2 | 6 | 3 | 7 | 4 | .476 |
| 2 | 4 | 8 | 1 | 3 | 6 | 5 | 7 | 2 | .357 |

4. STATISTICALLY BASED STRUCTURAL INDEX

In this section, the development of a simple statistically based structural strength index scheme will be described. The approach is basically an extension of the Utah decision tree approach shown in Figure 5. In the Utah approach, the pavement and subgrade are each classified from strong to weak depending upon the dynaflect maximum deflection, surface curvature index and base curvature index. In the approach described in this section, the pavement is classified in terms of two parameters; the FWD Surface Curvature Index (W1-W2), and the W7 sensor reading (6 foot from the applied load) at the 9000 lb load level with the sensors 1 foot apart. As described below in Section 4.1, these parameters were found to be adequate in diagnosing surface treated pavement strengths. Each combination of SCI and W7 was then assigned a structural strength value from 0.10 to 1.00. Weighing factors were assigned for traffic level and rainfall. The approach initially developed for thin surface-treated pavements was subsequently expanded to cover all the flexible pavement types in Texas.

4.1 Layer Strength Analysis of Thin Pavement

Those pavements with less than two inches of asphalt are difficult to analyze using the traditional modulus backcalculation technique (14). Surface treated pavements are frequently modelled as two layer systems of base and subgrade. At the network level, analysis of these pavements is often difficult due to inadequate layer thickness information and marked environmental effects (see Figure 10). In an attempt to develop simple analysis tools for use with FWD deflection data, twenty surface treated pavements were selected for study. Each pavement is essentially a surface treatment on top of a normal 6 to 10 inch untreated granular base on top of the natural subgrade. On each pavement, dynamic cone penetrometer tests (15) were conducted along with Falling Weight Deflectometer testing. The dynamic cone penetrometer (DCP) is shown schematically in Figure 13, and a typical analysis of results is shown in Figure 14. This is an excellent tool for evaluating the properties of thin pavements with unstabilized layers, some agencies (16) use it as the basis of design for thin pavements. Correlations have been made relating penetration rates to laboratory CBR values. The DCP gives the following information about the pavement under test;

- 1) The Effective Base Thickness
- 2) The Base Penetration Rate
- 3) The Subgrade Penetration Rate

The results of the deflection and cone penetration testing are shown in Table 15. The data in this table were used to develop a tentative interpretation schedule scheme for FWD on thin pavements. This scheme, based on SCI (W1-W2) and W7, is shown in Table 16. In order to determine how well this scheme evaluated the structural condition of each of the twenty test pavements, a section by section comparison was undertaken and the results are shown in Table 17. In this table, the strength of the base course and subgrade were estimated in terms of the penetration rates and

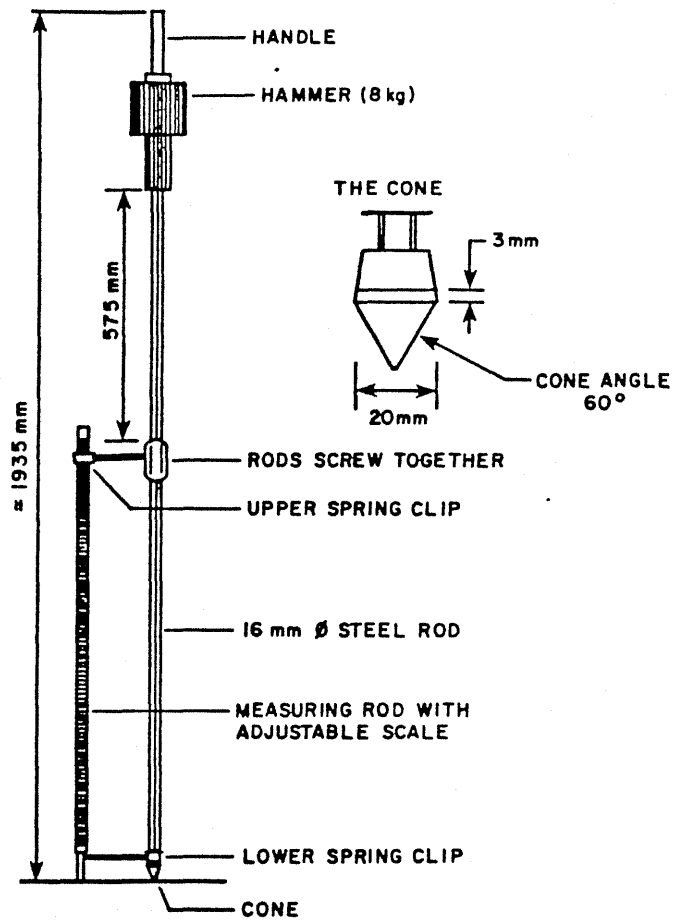


Figure 13. The Dynamic Cone Penetrometer

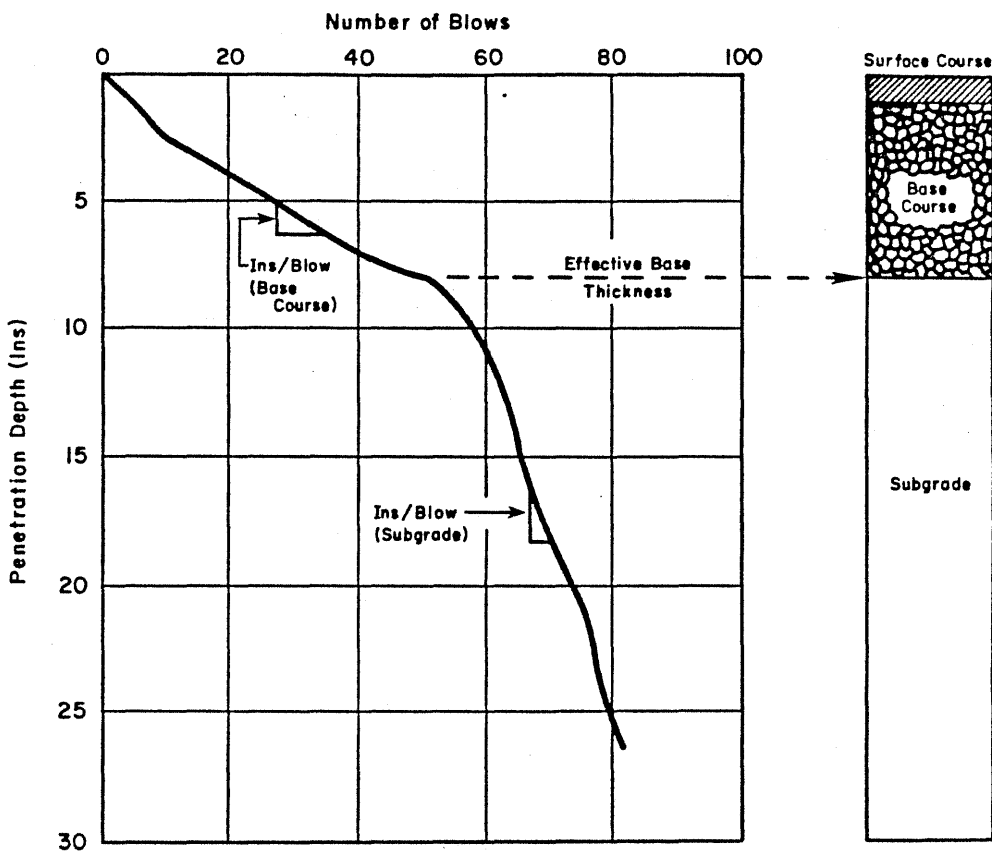


Figure 14. Typical penetration results from the DCP.

| OBS | W1 | W2 | W3 | W4 | W5 | W6 | W7 | P1 | P2 | P3 | P4 | T |
|-----|-------|------|------|-----|-----|-----|-----|------|------|------|------|----|
| 1 | 61.2 | 20.8 | 7.4 | 4.6 | 3.2 | 2.6 | 2.0 | 0.15 | 1.19 | 1.95 | 1.38 | 4 |
| 2 | 89.2 | 33.5 | 11.5 | 7.0 | 4.8 | 3.5 | 2.8 | 0.17 | 1.30 | 2.44 | 2.44 | 4 |
| 3 | 103.9 | 44.1 | 13.1 | 6.4 | 4.2 | 2.9 | 2.5 | 0.17 | 1.32 | 1.12 | 1.12 | 4 |
| 4 | 80.2 | 33.6 | 12.3 | 6.2 | 3.9 | 2.8 | 2.5 | 0.33 | 0.89 | 1.26 | 1.73 | 5 |
| 5 | 81.2 | 35.8 | 13.6 | 6.5 | 4.1 | 2.5 | 1.8 | 0.23 | 0.47 | 1.55 | 1.55 | 6 |
| 6 | 35.7 | 14.8 | 5.8 | 3.5 | 2.2 | 1.9 | 1.5 | 0.22 | 0.63 | 1.18 | 1.10 | 6 |
| 7 | 41.7 | 20.0 | 8.0 | 4.4 | 2.8 | 2.1 | 1.6 | 0.18 | 0.27 | 0.55 | 0.77 | 7 |
| 8 | 29.0 | 11.8 | 3.8 | 2.5 | 1.9 | 1.4 | 1.0 | 0.12 | 0.20 | 0.21 | 0.58 | 11 |
| 9 | 99.6 | 41.7 | 12.1 | 5.8 | 3.7 | 2.7 | 2.3 | 0.18 | 0.27 | 1.05 | 0.78 | 7 |
| 10 | 26.2 | 8.1 | 2.0 | 1.1 | 0.8 | 0.6 | 0.4 | 0.13 | 0.23 | 0.24 | 0.11 | 7 |
| 11 | 21.7 | 9.5 | 4.4 | 2.2 | 1.5 | 1.2 | 0.9 | 0.08 | 0.08 | 0.08 | 0.45 | 18 |
| 12 | 71.2 | 25.7 | 8.0 | 3.1 | 2.2 | 1.8 | 1.3 | 0.16 | 0.24 | 0.40 | 1.30 | 12 |
| 13 | 100.9 | 31.0 | 4.7 | 2.6 | 2.3 | 2.0 | 1.9 | 0.16 | 0.40 | 0.65 | 0.70 | 5 |
| 14 | 35.6 | 13.3 | 4.6 | 2.7 | 1.7 | 1.3 | 1.0 | 0.16 | 0.17 | 0.20 | 0.78 | 13 |
| 15 | 79.9 | 12.3 | 1.3 | 0.7 | 0.5 | 0.5 | 0.5 | 0.09 | 0.72 | 0.37 | 0.72 | 4 |
| 16 | 44.2 | 13.9 | 2.6 | 1.3 | 0.9 | 0.7 | 0.6 | 0.15 | 0.52 | 0.35 | 0.18 | 4 |
| 17 | 33.3 | 12.2 | 4.7 | 2.6 | 1.8 | 1.4 | 1.2 | 0.17 | 0.23 | 0.35 | 0.20 | 9 |
| 18 | 35.6 | 13.5 | 5.8 | 3.7 | 2.6 | 2.3 | 1.6 | 0.19 | 0.19 | 0.35 | 0.20 | 9 |
| 19 | 87.0 | 25.6 | 5.1 | 2.7 | 1.9 | 1.5 | 1.2 | 0.16 | 0.52 | 1.05 | 1.02 | 6 |
| 20 | 26.3 | 9.7 | 4.0 | 2.0 | 1.3 | 1.0 | 0.8 | 0.07 | 0.37 | 0.57 | 0.31 | 8 |

Table 15. Results of Cone penetrometer v FWD test (at 9000 lbs.) on 20 Surface Treated Pavements.

Key

W1-W7 FWD sensor readings

P1, P2, P3, P4 Penetration Ratio in inches/blow for depths 0-6", 6-12", 12-18" and 18 to 24" respectively.

T Effective Base Thickness

Table 16. Tentative FWD Interpretation scheme (9000 lb. load level) for Surface Treated Pavements.

| W7 | SCI | Pavement Diagnosis |
|------------|-----------------|---|
| ≤ 1.2 | $SCI \leq 20$ | Good Base, Stiff Subgrade |
| | $20 < SCI < 40$ | Marginal Base, Stiff Subgrade |
| | $SCI \geq 40$ | Thin and/or Soft Base, Stiff Subgrade |
| 1.3-1.9 | $SCI \leq 20$ | Good Base, Marginal Subgrade |
| | $20 < SCI < 40$ | Marginal Base, Marginal Subgrade |
| | $SCI \geq 40$ | Thin and/or Soft Base, Marginal Subgrade |
| ≥ 2.0 | $SCI \leq 20$ | Good Base, Soft or Wet Subgrade |
| | $20 < SCI < 40$ | Marginal Base, Soft or Wet Subgrade |
| | $SCI \geq 40$ | Thin and/or Soft Base, Soft or Wet Subgrade |

| Section No. | Cone Penetrometer Results | | | FWD | | |
|-------------|---------------------------------------|-------------------------------|-------|------|---------------------------------|----------|
| | Effective Base Thickness ¹ | Strength Ranking ² | | | Pavement Diagnosis ³ | |
| | | 0-6" | 6-18" | >18" | Base | Subgrade |
| 1 | 4 | 3 | 5 | 5 | Thin/Soft | Soft/Wet |
| 2 | 4 | 3 | 5 | 5 | Thin/Soft | Soft/Wet |
| 3 | 4 | 3 | 5 | 4 | Thin/Soft | Soft/Wet |
| 4 | 5 | 5 | 4 | 5 | Thin/Soft | Soft/Wet |
| 5 | 6 | 4 | 4 | 5 | Thin/Soft | Marginal |
| 6 | 6 | 4 | 3 | 4 | Marginal | Marginal |
| 7 | 7 | 3 | 2 | 3 | Marginal | Marginal |
| 8 | 11 | 2 | 1 | 2 | Good | Stiff |
| 9 | 7 | 3 | 3 | 3 | Thin/Soft | Soft/Wet |
| 10 | 7 | 2 | 1 | 1 | Good | Stiff |
| 11 | 18 | 1 | 1 | 2 | Good | Stiff |
| 12 | 12 | 3 | 2 | 5 | Thin/Soft | Marginal |
| 13 | 5 | 3 | 2 | 3 | Thin/Soft | Marginal |
| 14 | 13 | 3 | 1 | 3 | Marginal | Stiff |
| 15 | 4 | 1 | 2 | 3 | Thin/Soft | Stiff |
| 16 | 4 | 3 | 2 | 1 | Marginal | Stiff |
| 17 | 9 | 3 | 1 | 1 | Marginal | Stiff |
| 18 | 9 | 3 | 1 | 1 | Marginal | Marginal |
| 19 | 6 | 3 | 3 | 4 | Thin/Soft | Marginal |
| 20 | 8 | 1 | 2 | 2 | Good | Stiff |

1. Effective base thickness was obtained from cone penetration data, see Table 15.
2. Strength ranking is a code based on penetration per blow (1=strong to 5=weak).

0-6 inches (Base)

| Class | ins/blow |
|-------|-----------|
| 1 | < .10 |
| 2 | .11 - .14 |
| 3 | .15 - .20 |
| 4 | .21 - .25 |
| 5 | ≥ .26 |

>6 inches (Subbase/subgrade)

| Class | ins/blow |
|-------|------------|
| 1 | < .30 |
| 2 | .31 - .60 |
| 3 | .61 - .90 |
| 4 | .91 - 1.20 |
| 5 | ≥ 1.20 |

3. From Table 16.

Table 17. Comparing the Penetration data with the FWD Interpretation scheme in Table 16.

converted to a 1 to 5 scale. These can be compared with the diagnosis obtained using the simple scheme given in Table 16.

There is a reasonable comparison between the two procedures, sections 8, 10, 11 and 20 have stiff bases and subgrades, whereas sections 1, 2, 3, and 4 have poor bases and subgrades.

The scheme in Table 16 for interpreting the quality of the base course relies on the Surface Curvature Index value. If the SCI is less than 20 mils at the 9000 lb load level, the base is judged to be "Good and Stiff," if the SCI is greater than 40 mils, the base is assigned "Thin and/or Soft."

To determine what percentage of Texas thin pavements fall into these categories, a statistical analysis was performed on thin pavement deflection analysis results collected in two districts (District 11 and 21). District 11 is in East Texas with mostly marginal sandy/clay subgrades, District 21 is in South Texas, and the area tested had poor sandy subgrades with high water table problems. In total FWD data was available on 172 thin pavement sections. A histogram of SCI values is shown below in Figure 15.

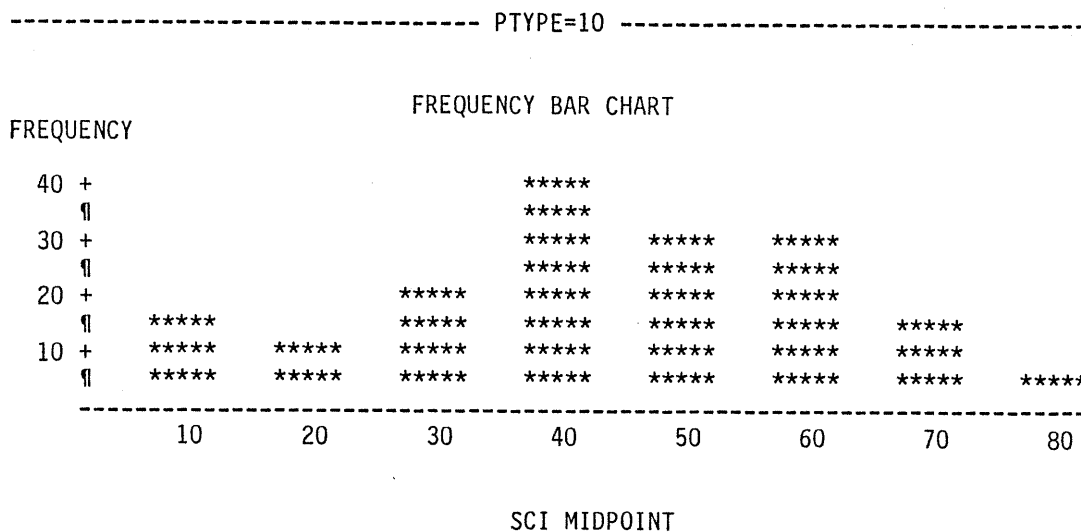


Figure 15. SCI (W1-W2) value for their surface treated pavements in District 11 and 21 (FWD loads of 9000 lbs, sensor spacing 1 foot).

The mean value was 44 mils with a standard deviation of 18.8 mils. Using the good base (<20 mils) versus poor base (>40 mils) definition it is estimated that only approximately 10% of the thin pavements in these districts would be judged as having a "Good and Stiff" base, whereas 58% would be judged as having a "Weak and/or Thin" base. A similar analysis of the W7 sensor

readings indicated that over 90% of the sections were located on "Soft or Wet" subgrades and less than 1% on "Stiff" Subgrade. These results are acceptable given the districts chosen to do the analysis. There are numerous other districts, particularly in West Texas, with markedly different deflection patterns.

4.2 Proposed Structural Index Scheme

The analysis described above indicated that, to a first approximation, that the SCI and W7 values were reasonable indicators of the structural strength of surface treated pavements. In this section a structural strength index scheme based on SCI and W7 is proposed. Table 18 shows the proposed scheme for surface treated pavements.

| Pavement Type - 10 | | |
|--------------------|-----------|------|
| W7 | SCI | SSI |
| < 1.2 | < 20 | 1.00 |
| | 20 - 25.9 | .80 |
| | 26 - 30.9 | .60 |
| | 31 - 35.9 | .40 |
| | 35 - 40 | .30 |
| | > 40 | .20 |
| 1.3 - 1.9 | < 20 | .90 |
| | 20 - 25.9 | .70 |
| | 26 - 30.9 | .50 |
| | 31 - 35.9 | .35 |
| | 36 - 40 | .25 |
| | > 40 | .15 |
| > 2.0 | < 20 | .80 |
| | 20 - 25.9 | .55 |
| | 26 - 30.9 | .40 |
| | 31 - 35.9 | .30 |
| | 36 - 40 | .20 |
| | > 40 | .10 |

Table 18. Defining the Structural Index in terms of SCI and W7 for Surface Treated Pavements.

The SSI in Table 18 is the structural strength index value, it is a number between 0 and 1 indicating the overall structural strength of the pavement. As will be described later in this section, the SSI value will be weighted with traffic and environmental factors to arrive at a final structural strength index. By analyzing field deflection data, similar tables have been constructed for the other flexible pavement types and these are shown in Tables 19 and 20. Table 19 are values appropriate for thin surfaced asphalt pavements (surfacing less than 2.5 inches thick). Table 20 is for intermediate and thick surfaced pavements with HMAC greater than 2.5 inches.

Pavement Type 6 & 9

| W7 | SCI | SSI |
|-----------|-----------|------|
| < 1.2 | < 15 | 1.00 |
| | 15 - 20.9 | .80 |
| | 21 - 25.9 | .60 |
| | 26 - 30.9 | .40 |
| | 31 - 35 | .30 |
| | > 35 | .20 |
| 1.3 - 1.9 | < 15 | .90 |
| | 15 - 20.9 | .70 |
| | 21 - 25.9 | .50 |
| | 26 - 30.9 | .35 |
| | 31 - 35 | .25 |
| | > 35 | .15 |
| > 2.0 | < 15 | .80 |
| | 15 - 20.9 | .55 |
| | 21 - 25.9 | .40 |
| | 26 - 30.9 | .30 |
| | 31 - 35 | .20 |
| | > 35 | .10 |

Table 19. Structural Strength Index in terms of SCI and W7 for thin asphalt surface pavements.

Pavement Type 4 & 5

| W7 | SCI | SSI |
|-----------|-----------|------|
| < 1.2 | < 10 | 1.00 |
| | 10 - 15.9 | .80 |
| | 16 - 20.9 | .60 |
| | 21 - 25.9 | .40 |
| | 26 - 30 | .30 |
| | > 30 | .20 |
| 1.3 - 1.9 | < 10 | .90 |
| | 10 - 15.9 | .70 |
| | 16 - 20.9 | .50 |
| | 21 - 25.9 | .35 |
| | 26 - 30 | .25 |
| | > 30 | .15 |
| > 2.0 | < 10 | .80 |
| | 10 - 15.9 | .55 |
| | 16 - 20.9 | .40 |
| | 21 - 25.9 | .30 |
| | 26 - 30 | .20 |
| | > 30 | .10 |

Table 20. Structural Strength Index in terms of SCI and W7 for intermediate and thick asphalt pavements.

To calculate the final Structural Strength Index (SSIF), the following equation is used;

$$SSIF = 100 (SSI)^{1/(RF*TF)}$$

where SSI are obtained from Tables 18,19 or 20
 RF is the rainfall factor (Table 21)
 TF is the traffic factor (Table 22)

The rainfall factors are shown in Table 21.

| Inches/Year | RF |
|-------------|------|
| < 20 | 1.0 |
| 21-40 | 0.97 |
| > 40 | 0.94 |

Table 21. Rainfall Factors

The traffic factors are shown in Table 22. The numbers in the body of the table are the 20 year projected 18 Kip equivalent Single Axles in millions broken into percentile levels 0-20%, 20-40%, etc., by pavement type. To use this table, a pavement type and estimate 20 year Kip ESAL are input and a traffic factor is output. For example, if the section had a pavement type = 4 and a projected 18 kip ESAL of 25 million, this would generate a Traffic Factor of 0.85 from Table 22.

| Pavement Type | Traffic Factor TF | | | | |
|---------------|-------------------|-------|-------|-------|-------|
| | 1.30 | 1.15 | 1.0 | .85 | .70 |
| 1 | < 17 | <27 | <40 | <54 | > 54 |
| 2 | <4.1 | <8.3 | <22 | <43 | >43 |
| 3 | <0.6 | <7.3 | <26 | <27 | >27 |
| 4 | <6 | <11 | <18 | <26 | >26 |
| 5 | <1.5 | <3.1 | <6.5 | <21 | >21 |
| 6 | <0.5 | <1.4 | <2.7 | <7.5 | >7.5 |
| 7 | <1.7 | <6.3 | <22 | <33 | >33 |
| 8 | >1.7 | <3.8 | <12.5 | <34.6 | >34.6 |
| 9 | <.26 | <1.4 | <3.2 | <6.4 | >6.4 |
| 10 | <0.09 | <0.24 | <0.79 | <3.4 | >3.4 |

Table 22. Traffic Factors

An example of the calculation process is shown below:

| | | | |
|--------------|----------------------|---|----------------|
| <u>Input</u> | Pavement Type | = | 5 |
| | Estimate 18 kip ESAL | = | 4.5 million |
| | SCI (W1 - W2) | = | 12.2 mils |
| | W7 | = | 0.8 mils |
| | Rainfall (County) | = | 30 inches/year |

| | | | |
|--------------------|---------|---|----------------------|
| <u>Calculation</u> | SSI | = | 0.8 (from Table 20) |
| | RF | = | 0.97 (from Table 21) |
| | TF | = | 0.85 (from Table 22) |
| | 1/RF*TF | = | 1.21 |

| | | |
|---------------------------|---|----------------|
| Structural Strength Index | = | 100 (SSI) 1.21 |
| | = | 76 |

The structural strength scheme described in this chapter is referred to in the remainder of this report as the statistically based Structural Index scheme. In Section 6, comparative runs are made contrasting the output of this scheme with those produced using the mechanistic scheme described in Section 5.

5. DEVELOPMENT OF A MECHANISTIC STRUCTURAL INDEX SCHEME

In the mechanistic approach, a remaining life until structural failure is calculated. This section describes a mechanistic approach by which the number of months until an unacceptable level of rutting or alligator cracking would be anticipated. The number of months until failure, with a maximum value of 100, is defined as the structural strength index of the pavement.

The approach described below was designed to be both comprehensive and modular. Comprehensive in that it includes many necessary features of a mechanistic procedure including daily, seasonal correction factors, distress models, failure levels based on pavement type and the possibility of including current distress condition in calculating remaining life. Modular in that if better correction factors or distress models become available, than they can easily replace the existing models.

In order to describe the model, a step by step description of the data processing steps required to generate the mechanistic structural strength index is given below.

5.1 Steps in Analysis Procedure

Step 1 Data Input

The data items listed below are obtained from the PES master file. This is the network level file maintained by Texas SDHPT containing information on the entire Texas highway network. Highways within the system are broken into sections approximately two miles in length.

1. Highway Identification
2. Texas District Number
3. FWD deflection data. For each PES Section 5 FWD deflection bowls are measured at the 9000 lb load level. For the purpose of this analysis the 80th percentile is used.
4. The month when the deflection testing was performed.
5. The surface temperature at the time of testing.
6. Current pavement visual distress information (level of cracking and rutting)
7. The pavement type code describing the surfacing thickness. The codes of interest to this study are:

| Code | Description |
|------|---------------------------|
| 4 | HMAC > 5.5 inches |
| 5 | 2.5 ins < HMAC < 5.5 ins. |
| 6 | HMAC < 2.5 ins. |
| 10 | Surface Treated |

8. The estimated 18 Kip Equivalent Single Axle application projected for this section for the next 20 years.

Step 2 Select Environmental Zone.

Figure 12 is used to identify the appropriate environmental zone for this district. The zone is one of the inputs to Tables 11 and 12 which give the near monthly surface temperature data and monthly deflection values.

Step 3 Correct Maximum Deflections readings to mean monthly temperature

Pavement surface temperatures can vary widely during a single test day. Data collected in February in the Lufkin district found an early morning temperature of 35°F and by mid-afternoon the surface temperature was 155°F. A study of deflection versus temperature was conducted at the Texas A & M Research Annex. The determined temperature correction factors are tabulated below in Table 9. The annex testing demonstrated that the W1 sensor was most markedly affected by daily temperature change, whereas the outer sensors 5, 6 and 7 showed very little change.

The correction factors presented in Table 9 were used to correct any given maximum deflection value to the deflection value anticipated at the near monthly temperature (Table 10). Correction factors were only applied to the W1 (maximum deflection) values. For example, if a +10 degree correction was required for a thick asphalt pavement, then a 10 x 0.12 correction would be added to the W1 sensor reading.

The correction factors for the surface treated pavements were found to be excessively large. These correction factors sometimes became as large as the actual measured deflection value. Because of this, the surface treated deflection corrections were eventually discontinued. Additional work is currently underway at the Texas Transportation Institute (17) to investigate daily and seasonal deflection patterns on pavements around the state of Texas. Problems were also encountered with applying maximum deflection correction factors to very stiff pavements, it was found possible to generate negative values of the Surface Curvature Index (W1-W2). Maximum deflection correction techniques similar to the above are recommended in the new AASHTO design guide. These corrections can run into problems if the design system, as is in Texas with its Flexible Pavement Design System, is based on surface curvature index. More work is needed in this area.

Step 4 Predict Annual Deflection Pattern.

Once the temperature correction in step 4 is complete for the month in which the data was collected, the annual variation in deflection is predicted using the monthly deflection values given in Table 11. Using Table 11, normalized correction factors are generated for the pavement type and zone of interest. These correction factors are applied to each of the seven FWD sensor readings. At the end of this step, an FWD deflection bowl is generated for each month of the year.

Step 5 Calculation of Strains within the Pavement Structure

The strains of interest are the tensile strains at the bottom of the asphalt (E_t) and the compressive strain at the top of the subgrade (E_v). The classical procedure for defining these is first to determine layer moduli by undertaking a backcalculation analysis. Then, to do forward analysis with the design load of interest to predict the strain levels. However, several factors weigh against using this approach, namely;

- 1) In this network level system large volumes of data are collected and analyzed. It is projected that more than 20,000 deflection bowls will be recorded each year.
- 2) Backcalculation techniques require considerable computer time.
- 3) The layer type and thickness information is extremely limited at this network level.

In order to eliminate this problem, several simplifying assumptions were made and simple regression equations were developed which directly linked bowl parameters such as W_1 (maximum deflection), SCI (W_1-W_2) and W_7 measured under the 9000 lb drop load to the critical strain values. The first step in this analysis was to use the iterative modulus backcalculation program CHEVDEF (14) to determine realistic layer moduli and predict the tensile and compressive strains for twenty in-service pavement sections of each pavement type. Regression models were then built to relate the input bowl parameters to the calculated strains.

The developed regression equations are as follows:

Surface Treated Pavement

(assumed structure 1 inch HMA, 6 inch granular base)

$$E_v = -130.78 + 22.87 W_1 + 1075.17W_7 \quad R^2 = 0.93$$

E_t = not calculated for thin pavements

Thin Asphalt Pavements

(assumed structure 2 inch HMA, 8 inch granular base)

$$E_v = -49.23 + 24.36 W_1 + 239.40 W_7 \quad R^2 = 0.89$$

$$E_t = -38.91 + 28.66 SCI \quad R^2 = 0.97$$

Medium Asphalt Pavements

(assumed structure 4 inch HMA, 10 inch granular base)

$$E_v = -103.5 + 15.75 W_1 + 223.08 W_7 \quad R^2 = 0.93$$

$$E_t = 98.6 + 30.67 SCI \quad R^2 = 0.86$$

At the time this analysis was performed, very few deflection bowls were available for thick asphalt pavement. It was therefore decided to use the medium asphalt pavement equations for all pavements with surfacings greater than 3 inches.

By combining the annual deflection bowl information (Step 4) with these simple regression equations, it is possible to calculate the strains within the pavement for each month of the year.

Step 6 Performance Prediction

To convert the monthly strains predicted in step 6 into a pavement life, a review of the literature was made and the Shell rutting model (18) and the Finn cracking model (19) were adopted. These models are shown below:

Rutting Model

$$N_{Rut} = \left(\frac{2/8 \times 10^{-2}}{E_v \times 10^{-6}} \right)^4$$

where

N_{Rut} is the number of 18 Kip ESAL to cause a 3/4 inch surface rut.

Cracking Model

$$\log_{10} N_{cR} = 15.988 - 3.291 \log_{10} (Et/10^{-6}) - 0.854 \log_{10} (Sm/1000)$$

where

N_{cR} is the number of 18 Kip ESAL to cause alligator cracking over 30% of surface area.

Sm is the asphalt mixture stiffness in psi

The additional term in the cracking model is the asphalt stiffness term. In Texas, two aggregate types predominate in mix design; these are crushed limestone and river gravel. As a generalization, river gravel is judged to be inferior, and its usage is usually confined to low to intermediate volume roads in East Texas. Typical laboratory determined stiffness values (20) (21) for similar mix types are tabulated in Table 23.

| Temperature °F | LAB STIFFNESS (PSI) | |
|-------------------|---------------------|--------------|
| | Limestone | River Gravel |
| 20 | 3,000,000 | 2,200,000 |
| 40 | 2,000,000 | 1,650,000 |
| 60 | 1,200,000 | 1,200,000 |
| 80 | 510,000 | 520,000 |
| 100 | 250,000 | 130,000 |
| 110 | 190,000 | 58,000 |

Table 23. Typical Mix Stiffness for mixes with different aggregate types.

The principal differences in performance are observed in the high temperature range. Regression analysis of these data produced the following equations.

River Gravel Mixes

$$SM = 10^{**}(6.377 - 0.001619 T + 9.15 \times 10^{-6} T^2 - 1.17 \times 10^6 T^3)$$

Limestone Mixes

$$SM = 10^{**}(6.429 - 0.007909 T + 0.0003295 T^2 - 1.47 \times 10^6 T^3)$$

where T is the test temperature in °F.

Step 7 Failure Conditions

In step 6, equations were presented which on a month by month basis permit the calculation of increases in rutting or surface cracking. What remains is to define what levels of each constitute pavement failure. The failure levels used in this study are shown in Table 24.

| Pavement Type | Cracking | Rut Depth (ins) |
|---------------|----------|-----------------|
| 4 | 30 | 0.75 |
| 5 | 30 | 0.75 |
| 6 | 40 | 1 |
| 10 | 50 | 2 |

Table 24. Defined Failure conditions within the Mechanistic System

Step 8 Calculation of Months Until Failure

Using the performance equations in step 6, it is possible to calculate NRVT and NCR for each month in service. These values can be compared with the actual traffic on the section to determine what percentage of the life will be used each month. These percentages are then accumulated until a value of 100% used is attained for either rutting or cracking. This gives the number of months until structural failure occurs.

Within the program, three different lives to failure are calculated, these being:

Case 1 Assume that project specific traffic data is not available and assume that the section will carry the average 18 Kip ESAL values shown in Table 25.

| Functional Class | 18 Kip Estimates (20 year in Millions) |
|------------------|--|
| 1 | 15.06 |
| 2 | 12.57 |
| 3 | 6.83 |
| 4 | 2.36 |
| 5 | 0.69 |
| 6 | 0.19 |
| 7 | 0.19 |

Table 25. Mean values of 20 year 18-Kip Estimates for each Functional Class

Case 2 Using the actual project specific 18 Kip estimates for the section, but ignoring the current levels of cracking or rutting in the section

Case 3 Using the actual project specific 18 Kip estimates for the section and taking into consideration the current levels of distress. This will estimate the remaining life until failure.

5.2 Output Description

The program to perform this analysis has been written on microcomputer and a listing is supplied in Appendix A. For this study, a detailed section output is produced and an example is shown in Figure 16. Each section of Figure 16 is numbered, the explanation of each is as follows:

1. This is the input condition and deflection data.
2. Month data taken, W1 correction factor and corrected W1 value in month data taken
3. Projected deflection pattern for each month of the year
4. Calculated monthly estimated strain at the top of the subgrade for a 9000 lb load
5. Calculated monthly estimated strain at the bottom of the asphalt for a 9000 lb load
6. Calculated monthly asphalt stiffness
7. The number of 18 Kip ESAL which would cause either rutting or cracking failure in that month.
8. The percentage of rutting and cracking life used each month.
9. The computed structural index values.

Index 1 - Months to failure under average traffic
(average for that functional class)

Index 2 - Months to failure under actual traffic levels

Index 3 - Months to failure under actual traffic adjusting (where required) for preexisting rutting or cracking.

In the next section of this report, the results of runs comparing the statistical (Chapter 4) versus mechanistic (Chapter 5) will be presented.

INPUT DATA

| | DIST | HIGHWAY | BMP | DSP | EMP | RUT | PAT | FLR | BCK | ALG | LNG | TRN | SRV | PVT | HFC | EALT | ADT |
|---|-----------|-----------|-----------|----------|-----------|----------|----------|-----------|----------|----------|----------|----------|-----------|-----|-----|------|-----|
| 1 | 11 | FM0942 | 10 | 0.0 | 12 | 100 | 0 | 0 | 0 | 0 | 0 | 0 | 2.2 | 6 | 5 | 513. | 750 |
| | W1 | W2 | W3 | W4 | W5 | W6 | W7 | TEMP | MTH | | | | | | | | |
| | 37.2 | 22.6 | 10.6 | 4.8 | 3.8 | 2.9 | 2.3 | 71. | 1 | | | | | | | | |
| 2 | WMTH, | TCORW1, | W(1) | | 1 | | 0.05 | | 35.93 | | | | | | | | |
| 3 | WCOR | 35.93 | 39.63 | 39.97 | 42.32 | 40.97 | 37.28 | 36.27 | 35.60 | 34.26 | 33.92 | 36.27 | 38.62 | | | | |
| | WCOR | 22.60 | 24.92 | 25.13 | 26.61 | 25.77 | 23.44 | 22.81 | 22.39 | 21.54 | 21.33 | 22.81 | 24.29 | | | | |
| | WCOR | 10.60 | 11.69 | 11.79 | 12.48 | 12.09 | 11.00 | 10.70 | 10.50 | 10.10 | 10.01 | 10.70 | 11.39 | | | | |
| | WCOR | 4.80 | 5.29 | 5.34 | 5.65 | 5.47 | 4.98 | 4.84 | 4.76 | 4.58 | 4.53 | 4.84 | 5.16 | | | | |
| | WCOR | 3.80 | 4.19 | 4.23 | 4.47 | 4.33 | 3.94 | 3.84 | 3.76 | 3.62 | 3.59 | 3.84 | 4.08 | | | | |
| | WCOR | 2.90 | 3.20 | 3.23 | 3.41 | 3.31 | 3.01 | 2.93 | 2.87 | 2.76 | 2.74 | 2.93 | 3.12 | | | | |
| | WCOR | 2.30 | 2.54 | 2.56 | 2.71 | 2.62 | 2.39 | 2.32 | 2.28 | 2.19 | 2.17 | 2.32 | 2.47 | | | | |
| 4 | EPSV | 1377.012 | 1523.637 | 1536.965 | 1630.272 | 1576.954 | 1430.330 | 1390.342 | 1363.684 | 1310.367 | 1297.037 | 1390.342 | 1483.647 | | | | |
| 5 | EPST | 343.244 | 382.531 | 386.102 | 411.103 | 396.817 | 357.530 | 346.815 | 339.673 | 325.386 | 321.815 | 346.815 | 371.816 | | | | |
| 6 | SM | 1614057.0 | 940698.9 | 563506.9 | 302150.7 | 200559.0 | 136819.6 | 37786.5 | 302150.7 | 283216.1 | 807212.9 | 807212.9 | 1286563.0 | | | | |
| | RUTN | 170954.6 | 114053.1 | 110148.0 | 87014.5 | 99393.1 | 146854.8 | 164492.3 | 177736.9 | 208478.5 | 217181.9 | 164492.3 | 126855.4 | | | | |
| | CRKN | 80125.4 | 88944.4 | 133628.3 | 185087.8 | 295071.2 | 576473.5 | 1912031.0 | 346873.1 | 422267.3 | 179020.6 | 139949.8 | 74747.7 | | | | |
| 7 | RTNC | 227939.3 | 152070.7 | 146863.9 | 116019.3 | 132524.0 | 195806.3 | 219323.0 | 236982.5 | 277971.3 | 289575.8 | 219323.0 | 169140.4 | | | | |
| | CKNC | 106833.9 | 118592.4 | 178170.9 | 246783.7 | 393428.1 | 768631.1 | 2549374.0 | 462497.4 | 563022.8 | 238694.0 | 186599.6 | 99663.5 | | | | |
| | PCTRUT | 0.013 | 0.019 | 0.020 | 0.025 | 0.022 | 0.015 | 0.013 | 0.012 | 0.010 | 0.010 | 0.013 | 0.017 | | | | |
| | PCTCRK | 0.027 | 0.024 | 0.016 | 0.012 | 0.007 | 0.004 | 0.001 | 0.006 | 0.005 | 0.012 | 0.016 | 0.029 | | | | |
| 8 | PCTRUT2 | 0.009 | 0.014 | 0.015 | 0.018 | 0.016 | 0.011 | 0.010 | 0.009 | 0.008 | 0.007 | 0.010 | 0.013 | | | | |
| | PCTCRK2 | 0.020 | 0.018 | 0.012 | 0.009 | 0.005 | 0.003 | 0.001 | 0.005 | 0.004 | 0.009 | 0.011 | 0.021 | | | | |
| 9 | INDEX 1 = | 64 | INDEX 2 = | 86 | INDEX 3 = | 86 | | | | | | | | | | | |

Figure 16. Detailed output from the mechanistic structural index program.

6. COMPARATIVE RUNS

To evaluate the two structural strength index schemes, FWD data was collected in two Texas districts. These being District 11 in East Texas and District 21 in South Texas. The average length of section was two miles, and within each section five FWD readings were taken. For this analysis, the bowl with the second highest maximum deflection reading was taken for analysis purposes. It was reasoned that the second highest bowl would be representative of the weakest 20% of the section. Surface temperature readings were also recorded during the deflection testing.

The selected deflection bowls were processed by both the statistical (section 4) and mechanistic (section 5) analysis schemes. Typical results of this analysis are shown in Table 26. To evaluate ability of each scheme to adequately estimate structural inadequacy two approaches were taken. First, the results were presented to experienced highway department personnel responsible for the pavements under test.

Secondly, the structural adequacies were compared with the actual pavement performance recorded in the Texas Pavement Evaluation System in terms of changes in pavement score.

The columns in Table 26 are defined as follows:

a) Pavement Type (Flexible Pavements)

- 4 = Thick surfaced (> 5 ins)
- 5 = Intermediate Surfaced (> 2.5 in)
- 6 = Thin Surfaced (< 2.5 ins)
- 10 = Surfaced Treated

b) W1, W7

The Falling Weight Deflectometer sensor 1 and 7 readings in mils under a 9000 lb load. This being the second highest W1 reading on the section.

c) ADT

The 1986 Average Daily Traffic

d) Pavement Score (1983-1986)

In Texas, a composite pavement score combining Ride and Visual condition is used to represent the overall pavement condition, with values of 100 being perfect. This represents the recorded changes in condition from 1983 to 1986.

e) Structural Index

This index is planned to represent the structural adequacy of the pavement section under the existing traffic and environmental conditions. The deflection data on these sections was collected in January 1986. Values of Structural Index greater than 80 would indicate a pavement with adequate strength to carry existing loads. Values less than 20 would indicate weak pavements which would be anticipated to show rapid deterioration.

In reviewing these results, the following conclusions were reached:

- 1) The mechanistic procedure greatly underestimates the lives of intermediate thickness asphalt pavements. The engineers in District 11 upon review of the structural evaluations thought the estimates for US 190 were poor. This was a relatively new pavement with little distress, the models predicted a life from 2 to 10 months. The State engineers estimated the remaining life of this pavement to be between 7 to 9 years.
- 2) Problems were encountered with the quality of certain key data items, particularly the 20 year estimate 18 Kip ESAL and the pavement type description. The traffic projection showed wide varieties from year to year. The pavement type is thought to be inadequate because it is independent of base type. For example, FM942 has a low deflection due to its lime stabilized base, yet the pavement experienced rapid deterioration. It is not anticipated that the quality of these data items will improve in the near future.
- 3) In this limited analysis, the statistical scheme results were considered superior to the mechanistic scheme. Neither was adequate on pavements with stabilized bases.

| Highway ID | Pavement Type | W1 mils | W7 mils | ADT | PAVEMENT SCORE 1983 - 1986 | Structural Index | |
|--------------------|---------------|---------|---------|------|-------------------------------|------------------|-------------|
| | | | | | | Statistical | Mechanistic |
| FM 0062 | 10 | 28.0 | 1.7 | 950 | 97 - 78 | 90 | 17 |
| FM 942 | 6 | 19.1 | 3.1 | 1300 | 85 - 25 | 80 | 100 |
| FM 2500 | 6 | 37.7 | 1.2 | 880 | 80 - 63 | 82 | 100 |
| FM 352 | 10 | 71.6 | 2.8 | 1100 | 83 - 11 | 6 | 2 |
| FM 0357 | 6 | 17.0 | 1.7 | 650 | 69 - 69 | 92 | 100 |
| FM 3277 | 10 | 94.0 | 2.1 | 990 | 64 - 31 | 9 | 2 |
| US 190 MP 10-12 | 5 | 25.6 | 1.6 | 6200 | 100 - 94 | 65 | 5 |
| US 190 MP 26-28 | 5 | 26.8 | 1.2 | 3700 | 93 - 93 | 79 | 2 |
| US 190 MP 34-36 | 5 | 33.6 | 1.1 | 1950 | 100 - 76 | 63 | 2 |
| US 190 MP 36-38 | 6 | 42.4 | 1.8 | 1950 | 92 - 65 | 49 | 9 |

Table 26. Comparison of Computed Structural Indices with actual pavement degradations observed over a 3 year period in District 11.

7. CONCLUSIONS

The aim of this research has been to recommend a network-level structural strength index scheme which would:

- a) be based on FWD testing
- b) be compatible with existing PES system
- c) limit the number of deflection readings required per section

The results of this research are summarized below:

- 1) Five equally-spaced FWD deflection bowls are required in each PES section.
- 2) Deflection testing should be conducted at the 9000 lb load level with sensors at one foot spacings
- 3) For analysis purposes it is recommended that the deflection bowl with the second highest maximum deflection bowl be used.
- 4) It is recommended that the statistical scheme be implemented within the Pavement Evaluation System.
- 5) It was thought that the mechanistic scheme was inappropriate at the network level, primarily because of the modelling problems with thicker pavements and the variability of traffic data.
- 6) A mechanistic scheme shows considerable promise at the project level for use in estimating remaining life. It exhibits many desirable features, including monthly distress analysis. However, the PES database needs to be upgraded to allow for storage of more detailed pavement layer information prior to implementing a mechanistic procedure.
- 7) The Falling Weight Deflectometer data recording system records considerable information about pavement response. Only a small portion (maximum values) of this is being used. There is potential for including dynamic analysis techniques in the existing system software.

References

- 1) Lytton, R.L., Roberts F.L. and Stoffels S., "Determination of Asphaltic Concrete Pavement Structural Properties by Nondestructive Testing." NCHRP 10-27, April 1986.
- 2) Peterson, G. and Shepard L.W., "Deflection Analysis of Flexible Pavements," Materials and Test Division Report, Utah DOT, January 1972.
- 3) Taute, A., et al.. "Improvements to the Materials Characterization and Fatigue Life Prediction Models of the Texas Rigid Pavement Overlay Design Procedure," Research Report 249-1, Center for Transportation Research, University of Texas at Austin, Nov. 1981.
- 4) AASHTO "Guide for Design of Powered Structures," published by AASHTO, 1986.
- 5) Scullion, T., Magnuson, A.H. and Lytton, R.L., "Dynamic Analysis for Falling Weight Deflectometer," a study proposal to Texas SDHPT, Study 1175, January 1987.
- 6) Kausel, E. and Peek, R., "Dynamic Loads in the Interior of a Layered Stratum: An Explicit Solution," Bulletin of the Seismological Society of America, Vol. 72, No.5, pp. 1459-1481, Oct. 1982.
- 7) R.L. Lytton of Texas Transportation Institute (Private communication).
- 8) "Dynamic Analysis for Falling Weight Deflectometer," Cooperative Study 1175 between Texas SDHPT, The Center for Transportation Research and the Texas Transportation Institute.
- 9) Hoffman, M.S. and Thompson, M.R., "Backcalculating Nonlinear Resilient Moduli from Deflection Data," Record 852, TRB, 1982.
- 10) Reference 1, Appendix I.
- 11) "Asphalt Overlays for Highways and Streets Rehabilitation," The Asphalt Institute, Manual Series No. 17, 1983.
- 12) Kennedy, C.K. and Lister, N.W., "Prediction of Pavement Performance and the Design of Overlays," TRRL Laboratory Report No. 833, Great Britain, 1978.
- 13) Poehl, R. and Scrivner, F.H., "Seasonal Variations of Pavement Deflections in Texas," TTI Research Report 136-1, January 1971.
- 14) Bush, A.J., "Nondestructive Testing for Light Aircraft Pavements," US Army Waterways Experiment Station, Report FAA-RD-80-9-11, Nov 1980.

- 15) Klein, E.G., Maree J.H. and Savage, P.F., "The Application of a Portable Pavement Dynamic Core Parameter to Determine In Situ Bearing Properties of Road Pavement Layers and Subgrades in South Africa," Proc.of 2nd European Symposium on Penetration Testing, Amsterdam, 24-27 May, 1982, pp 277-283.
- 16) "Structural Design of Pavements," TRH4, National Institute of Transportation and Road Research, South Africa, 1985.
- 17) "Non Destructive Test Procedures For Analyzing the Structural Condition of Pavements," Cooperative Research Study 1123, Texas Transportation Institute.
- 18) Shell International Petroleum Company Ltd., Shell Pavement Design Manual, 1978.
- 19) Finn, F.N., et al., "The Use of Distress Prediction Subsystems for the Design of Pavement Structures," Proc, Vol I, Fourth International Conference on the Structural Design of Pavements, University of Michigan, Ann Arbor, Aug. 1977, pp 3-38.
- 20) Witczak, M.W., "Development of Regression Model for Asphalt Concrete Modulus for use in MS-1 Study," The Asphalt Institute, January 1978.
- 21) Button, J.W. et al., "Predicted performance of Asphalt Pouring Mixtures containing Hercules Fibers," TTI Report, Sept. 1984.

Appendix A

Program Listings

The following microcomputer program computes the mechanistic structural index described in Section 5 of this report. The inputs to this program are as follows:

| | |
|-----------------|---|
| File FWD11C.DAT | provides the 80th percentile FWD deflection data together with month and surface temperature of test. |
| INSP11.DAT | provides for each section the PES data; location, pavement type, current distress levels and traffic information. |
| DEVICES | provides monthly surface temperatures and deflection correction factors from Table 11 and 12. |

The output from this program is the detailed section listing shown in Figure 16.

```
C
C
C MECHANISTIC STRUCTURAL INDEX PROGRAM
```

```
C AIM: TO COMPUTE AN INDEX, 0-100, FROM FALLING WEIGHT DEFLECTOMETER
C INPUT TO REPRESENT PAVEMENT STRENGTH
```

```
C INPUT: FWD11C.DAT FWD DEFLECTION DATA, MONTH, AND TEMPERATURE
```

```
C INSP11.DAT PES DATA
```

```
C DIST = DISTRICT NO.
C HWY = HIGHWAY
C IVIS(1) = RUTTING
C IVIS(5) = ALLIGATOR CRACKING
C IP = PAVEMENT TYPE
C EALT = 20 YR. 18 KIPS
```

```
C REAL*4 MTEMP
```

```
C INTEGER*4 ADT, AVU, BMP, CNTY, DIST, EMP, HFC, PES, SIUC, WMTH, WVU
```

```
C CHARACTER*1 BSIGN, LANE
```

```
C CHARACTER*7 HWY
```

```
C DIMENSION MTEMP(5,12), DEFCF(2,5,12), EPSV(12), EPST(12), RUTN(12)
+ , CRKN(12), W(7), WCOR(7,12), AVGADT(7), SM(12),
& PCTCRK(12), PCTRUT(12), CKNC(12), RTNC(12)
C DIMENSION IVIS(7)
```

```
C AVERAGE 20 YEAR 18 KIPS BY FUNCTIONAL CLASS
```

```
C DATA AVGADT/ 15.060, 12.578, 6.838, 2.363, 0.695, 0.192, 0.192 /
```

```
C OPEN THE INPUT AND OUTPUT FILES
```

```
C OPEN(UNIT=1,FILE='FWD11C.DAT',STATUS='UNKNOWN')
```

```
C OPEN(UNIT=2,FILE='INSP11.DAT',STATUS='UNKNOWN')
```

```
C OPEN(UNIT=3,FILE='SIVPES.DAT',STATUS='UNKNOWN')
```

```
C WRITE(6,250)
```

```
C THE MONTHLY SURFACE TEMPERATURE (MTEMP) AND DEFLECTION
C CORRECTION FACTORS (DEFCF) READ FROM UNIT 5
```

```
C READ(5,310) ((MTEMP(I,J), J = 1, 12), I = 1, 5)
310 FORMAT( 12F5.0)
```

```
C WRITE(6,606) ((MTEMP(I,J),J=1,12), I=1,5)
606 FORMAT( /2X, 'MTEMP ', 12F7.0 )
```

```
C DO 6 I = 1, 2
```

```
DO 6 J = 1, 5
```

```

        READ(5,310) (DEFCF(I,J,K), K = 1, 12)
        6 WRITE(6,608) I, (DEFCF(I,J,K), K = 1, 12)
608 FORMAT( /2X, 'DEFCF ', I3, 12F7.2 )
C
C   READ PAVEMENT CONDITION DATA
C
        10 READ(2,300,END=500) DIST, CNTY, HWY, BMP, BSIGN, BDISP, EMP,
          + (IVIS(I),I=1, 7), SRVC, IP, HFC, ADT, EALT, AVU, WVU,PES, SIUC
300 FORMAT(I2,I3,A7,I3,A1,F2.1,I3,T27,7I3,F2.1,T54,I2,I1,
          + T62,I6,F5.0,T78,3I3,3X,I3 )
C
        WRITE(6,260) DIST, HWY, BMP, BSIGN, BDISP, EMP, (IVIS(I),I=1,7),
          + SRVC, IP, HFC, EALT, ADT
C
        260 FORMAT( '1', T3, 'INPUT DATA' /// T3, 'DIST HIGHWAY BMP ',
          + 'DSP EMP RUT PAT FLR BCK ALG LNG TRN SRV PVT HFC EALT ADT'/
          * I5,3X,A7,I5, 1X, A1, F4.1, I4, 1X, 7I4, F4.1, I4, I3,F9.0,2X,I6/)
C
C   READ FWD DATA (80 TH PERCENTILE)
C
        READ(1,305) LANE, (W(I), I = 1, 7), WMTH, WTEMP
305 FORMAT(T39, A1, 1X,7F6.1, T85, I2, T92, F5.1 )
C
        WRITE(6,262) (W(I), I = 1, 7), WTEMP, WMTH
262 FORMAT( T5, 'W1 W2 W3 W4 W5 W6 W7 TEMP MTH' /
          + 2X, 7F5.1, F6.0, I5 )
C
        WRITE ( 3, 350) CNTY, HWY, BMP, BSIGN, BDISP, EMP, LANE,
          + (W(I), I=1,7), IP, HFC, ADT, EALT
350 FORMAT ( I3,A7,I3,A1,F3.0,I3,A1,7F5.1,I2,I1,I6,F6.0)
C
C
C
        DO 5 I = 1, 12
        EPSV(I) = 0.0
        EPST(I) = 0.0
        RUTN(I) = 0.0
        CRKN(I) = 0.0
        5 SM(I) = 0.0
C
C   GET ZONE FROM DISTRICT NUMBER
C
        CALL ZONE( DIST, IZ )
C
C   CORRECT W(1) ONLY FOR TEMPERATURE
C
        NOTE : THE PAVEMENT TYPE 10 CORRECTION FACTORS COULD NOT BE USED
              BECAUSE THE LARGE TEMP VARIATIONS MEASURED
C
C
C
        TCORW1 = 0.82
        TCORW1 = 0.05
        IF( IP .GE. 5 .AND. IP .LE. 9 ) TCORW1 = 0.05

```

```

C      IF( IP .EQ. 4 )          TCORW1 = 0.12
C      IF( IP .EQ. 10 .AND. WMTH .GE. 4 .AND. WMTH .LE. 9 ) TCORW1 = -0.5
C
C      W(1) = W(1) + TCORW1 * (MTEMP(IZ,WMTH) - WTEMP )
C
C      WRITE(6,630) WMTH, TCORW1, W(1)
630  FORMAT( /2X, 'WMTH, TCORW1, W(1) ', I5, 2F10.2 )
C
C      IT = 2 FOR PVMT 4 & 5    IT = 1 FOR PVMT 6 & 10
C
C      CORRECT W(1) - W(7) FOR EACH MONTH
C
C      IT = 2
C      IF( IP .EQ. 6 .OR. IP .EQ. 10 ) IT = 1
C
C      DO 20 J = 1, 12
C      DO 15 I = 1, 7
15  WCOR(I,J) = W(I) * DEFCF(IT,IZ,J)/DEFCF(IT,IZ,WMTH)
20  CONTINUE
C
C      DO 21 I = 1, 7
21  WRITE(6,600) (WCOR(I,J), J = 1, 12)
600  FORMAT( / 2X, 'WCOR ', 12F8.2 )
C
C      WCOR(7,12) HAS THE MONTHLY CORRECTED FWD DATA
C
C      CALCULATE EPSV & EPST  EPSV = STRAIN AT TOP OF SUBGRADE
C                             EPST = STRAIN AT BOTTOM OF ASPHALT
C
C      GO TO ( 25, 25, 25, 25, 30, 35, 35, 35, 35, 40 ), IP
C
C      PAVEMENT TYPE 4
C
C      25 DO 27 I = 1, 12
C      EPSV(I) = -103.5 + 15.75*WCOR(1,I) + 223.08*WCOR(7,I)
27  EPST(I) = 98.60 + 30.67 * (WCOR(1,I) - WCOR(2,I) )
C      GO TO 45
C
C      PAVEMENT TYPE 5
C
C      30 DO 32 I = 1, 12
C      EPSV(I) = -103.5 + 15.75*WCOR(1,I) + 223.08*WCOR(7,I)
32  EPST(I) = 98.60 + 30.67 * (WCOR(1,I) - WCOR(2,I) )
C      GO TO 45
C
C      PAVEMENT TYPE 6
C
C      35 DO 37 I = 1, 12
C      EPSV(I) = -49.23 + 24.367*WCOR(1,I) + 239.398*WCOR(7,I)
37  EPST(I) = -38.91 + 28.658 * (WCOR(1,I) - WCOR(2,I) )
C      GO TO 45
C
C      PAVEMENT TYPE 10

```



```

C
40 DO 42 I = 1, 12
42 EPSV(I) = -130.78 + 22.87 * WCOR(1,I) + 1075.17 * WCOR(7,I)
C
45 CONTINUE
C
WRITE(6,601) (EPSV(I), I = 1, 12)
601 FORMAT( / 2X, 'EPSV ', 12F10.3 )
WRITE(6,604) (EPST(I), I = 1, 12)
604 FORMAT( /2X, 'EPST ', 12F10.3 )
C
CALCULATE SM(I) STIFFNESS OF MIX IN MONTH I
C
IF( IZ .EQ. 1 .OR. IZ .EQ. 2 .OR. IZ .EQ. 4 ) GO TO 50
IF( HFC .GE. 5 ) GO TO 55
C
LIMESTONE
C
50 DO 52 I = 1, 12
EXP = 6.429 + 0.007909*MTEMP(IZ,I) - 0.0003295*MTEMP(IZ,I)**2 +
= 0.000001473*MTEMP(IZ,I)**3
52 SM(I) = 10.0 ** EXP
GO TO 70
C
RIVER GRAVEL
C
55 DO 57 I = 1, 12
EXP = 6.377 - 0.001619*MTEMP(IZ,I) + 0.000009115*MTEMP(IZ,I)**2
= -0.0000011701*MTEMP(IZ,I)**3
57 SM(I) = 10.0 ** EXP
C
70 CONTINUE
C
WRITE(6,605) (SM(I), I = 1, 12)
605 FORMAT( / 2X, 'SM ', 12F10.1 )
C
CALCULATE RUTN & CRKN RUTN = 18 KIPS TO CAUSE RUTTING FAILURE
CRKN = 18 KIPS TO CAUSE CRACKING FAILURE
C
DO 75 I = 1, 12
75 RUTN(I) = (0.028/(EPSV(I) * (10.0**(-6)))) ** 4
C
IF( IP .EQ. 10 ) GO TO 85
C
DO 80 I = 1, 12
EXP = 15.988 - 3.291*ALOG10(EPST(I))-0.854*ALOG10(SM(I)
+ / (10.0**3))
80 CRKN(I) = 10.0 ** EXP
C
85 CONTINUE
C
WRITE(6,602) (RUTN(I), I = 1, 12)
602 FORMAT( / 2X, 'RUTN ', 12F10.1 )
C

```

```

        WRITE(6,603) (CRKN(I), I = 1, 12)
603 FORMAT( / 2X, 'CRKN ', 12F10.1 )
C
C   ADJUST CRKN & RUTN FOR INDEX 1 & 2 CALC.
C
        TRAF = (AVGADT(HFC)/240.0) * 1000000.0
C
        GO TO ( 90,90,90,90,90, 95, 95,95,95,100 ), IP
C
C   PAVEMENT TYPES 4 & 5, NO CORRECTION FOR RUT OR CRACK
C
90 DO 92 I = 1, 12
    RTNC(I) = RUTN(I)
    CKNC(I) = CRKN(I)
    PCTRUT(I) = TRAF/RTNC(I)
92 PCTCRK(I) = TRAF/CKNC(I)
    GO TO 105
C
C   PAVEMENT TYPE 6
C
95 DO 97 I = 1, 12
    CKNC(I) = CRKN(I) * (4.0/3.0)
    RTNC(I) = RUTN(I) * (1.0/0.75)
    PCTRUT(I) = TRAF/RTNC(I)
97 PCTCRK(I) = TRAF/CKNC(I)
    GO TO 105
C
C   PAVEMENT TYPE 10, CORRECTION FOR RUTTING ONLY
C
100 DO 102 I = 1, 12
    RTNC(I) = RUTN(I) * (2.0/0.75)
    CKNC(I) = 0.0
    PCTRUT(I) = TRAF/RTNC(I)
102 PCTCRK(I) = 0.0
C
105 CONTINUE
C
        WRITE(6,612) (RTNC(I), I = 1, 12 )
612 FORMAT( /2X, 'RTNC ', 12F10.1 )
        WRITE(6,614) (CKNC(I), I = 1, 12)
614 FORMAT( /2X, 'CKNC ', 12F10.1 )
        WRITE(6,616) (PCTRUT(I), I = 1, 12)
616 FORMAT( /2X, 'PCTRUT', 12F10.3 )
        WRITE(6,618) (PCTCRK(I), I = 1, 12)
618 FORMAT( /2X, 'PCTCRK', 12F10.3 )
C
C   CALCULATE NUMBER OF MONTHS TO FAILURE USING AVERAGE MONTHLY
C   TRAFFIC BY FUNCTIONAL CLASS
C
        IND1 = 100
        CSUM = 0.0
        RSUM = 0.0
        DO 106 I = 1, 100, 12
        DO 106 J = 1, 12

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    CSUM = CSUM + PCTCRK(J)
    RSUM = RSUM + PTRUT(J)
    IF( CSUM .GE. 1.0 ) GO TO 108
    IF( RSUM .GE. 1.0 ) GO TO 109
106 CONTINUE
    GO TO 110
108 IND1 = (I - 1) + J
    GO TO 110
109 IND1 = (I - 1) + J
110 CONTINUE
C
C   CALCULATE INDEX 2 USING THE ACTUAL TRAFFIC FOR SECTION INPUT EALT
C
    EALT = (EALT/240.0) * 1000.0
C
    DO 125 I = 1, 12
    IF( IP .EQ. 10 ) GO TO 125
    PCTCRK(I) = EALT/CKNC(I)
125 PTRUT(I) = EALT/RTNC(I)
C
    WRITE(6,620) (PTRUT(I), I = 1, 12)
620 FORMAT( /2X, 'PTRUT2', 12F10.3 )
    WRITE(6,622) (PCTCRK(I), I = 1, 12)
622 FORMAT( /2X, 'PCTCRK2', 12F10.3 )
C
C   CALCULATE THE NUMBER OF MONTHS TILL FAILURE (ACTUAL TRAFFIC)
C
    IND2 = 100
    CSUM = 0.0
    RSUM = 0.0
    DO 130 I = 1, 100, 12
    DO 130 J = 1, 12
    CSUM = CSUM + PCTCRK(J)
    RSUM = RSUM + PTRUT(J)
    IF( CSUM .GE. 1.0 ) GO TO 132
    IF( RSUM .GE. 1.0 ) GO TO 133
130 CONTINUE
    GO TO 135
132 IND2 = (I - 1) + J
    GO TO 135
133 IND2 = (I - 1) + J
135 CONTINUE
C
C   ADJUST THE RTNC & CKNC VALUES FOR EXISTING DISTRESS
C   CALCULATE NO. OF MONTHS TO FAILURE ADJUSTING FOR EXISTING
C   DISTRESS LEVELS
C   CHECK FOR FAILED PAVEMENTS, SET INDEX 3 TO ZERO AND PRINT
C
    IND3 = 0
    IF( IVIS(1) .EQ. 2 .OR. IVIS(1) .EQ. 20 ) GO TO 190
    IF(IVIS(2) .EQ. 1 .OR. IVIS(3) .EQ. 1 .OR. IVIS(5) .EQ. 1) GOT0190
C
C   GET FACTOR FOR RUTTING DISTRESS
C

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    LC = 0
    IF( IVIS(1) .NE. 10 ) GO TO 155
    GO TO ( 140,140,140, 140, 142, 145, 145,145,145, 147 ), IP
C
C   PAVEMENT TYPE 4
140 RFAC = 0.33
    GO TO 150
C
C   PAVEMENT TYPE 5
142 RFAC = 0.33
    GO TO 150
C
C   PAVEMENT TYPE 6
145 RFAC = 0.50
    GOTO 150
C
C   PAVEMENT TYPE 10
147 RFAC = 0.75
C
150 DO 152 I = 1, 12
152 RTNC(I) = RTNC(I) * RFAC
    LC = 1
C
C   CHECK FOR CRACKING AND GET CRACKING FACTOR
C
155 IF ( IP .EQ. 10 ) GOTO 159
    IF( IVIS(2) .NE. 10 .AND. IVIS(3) .NE. 10 .AND. IVIS(5) .NE. 10)
    + GO TO 159
C
    IF( IP .EQ. 4 ) CFAC = 0.33
    IF( IP .EQ. 5 ) CFAC = 0.33
    IF( IP .EQ. 6 ) CFAC = 0.50
    IF( IP .EQ.10 ) CFAC = 0.0
C
    DO 157 I = 1, 12
157 CKNC(I) = CKNC(I) * CFAC
C
    LC = 1
159 IF ( LC .EQ. 0) GOTO 180
C
C   CALCULATE INDEX 3 USING THE AVERAGE MONTHLY TRAFFIC
C
160 DO 161 I = 1, 12
    IF( IP .EQ. 10 ) GO TO 161
    PCTCRK(I) = EALT/CKNC(I)
161 PCTRUT(I) = EALT/RTNC(I)
C
C   SUM THE RTNC AND CKNC TO GET 1 OR UNTIL I = 100
C
    IND3 = 100
    CSUM = 0.0
    RSUM = 0.0
    DO 162 I = 1, 100, 12
    DO 162 J = 1, 12

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    RSUM = RSUM + PCTRUT(J)
    CSUM = CSUM + PCTCRK(J)
    IF( CSUM .GE. 1.0 ) GO TO 163
    IF( RSUM .GE. 1.0 ) GO TO 164
162 CONTINUE
    GO TO 165
163 IND3 = (I - 1) + J
    GO TO 165
164 IND3 = (I - 1) + J
165 CONTINUE
C
    WRITE(6,624) (RTNC(I), I = 1, 12)
624 FORMAT( /2X, 'RTNC3 ', 12F10.1 )
    WRITE(6,625) (CKNC(I), I = 1, 12)
625 FORMAT( /2X, 'CKNC3 ', 12F10.1 )
    WRITE(6,626) IZ, IVIS(1), EALT, TRAF
626 FORMAT( /2X, 'ZONE, RUT, EALT, TRAF ', 2I5, F10.3, F12.0 )
C
    GO TO 190
C
C    NOD EXISTING DISTRESS, SET INDEX 3 TO INDEX 2 VALUE
C
180 IND3 = IND2
C
C    PRINT OUT RESULTS, INDEX 1 2 & 3, ETC
C
    IF( IND1 .GT. 100 ) IND1 = 100
    IF( IND2 .GT. 100 ) IND2 = 100
    IF( IND3 .GT. 100 ) IND3 = 100
C
190 WRITE(6,200) IND1, IND2, IND3
    WRITE ( 3, 201) IND1, IND2, IND3
200 FORMAT( / 2X, 'INDEX 1 = ', I6, 5X, 'INDEX 2 = ', I6,
+ 5X, 'INDEX 3 = ', I6 )
201 FORMAT(1H+, 82X, 3I3)
C
C
    III = INT(BDISP)
    II2 = INT(EALT)
C
    GO TO 10
C
500 WRITE(6,250)
250 FORMAT( '1' )
C
C    CLOSE THE FILES
C
    CLOSE(UNIT=1,STATUS='KEEP')
    CLOSE(UNIT=2,STATUS='KEEP')
    CLOSE(UNIT=3,STATUS='KEEP')
C
C
    STOP
    END

```

