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16. Abstract

A structural evaluation was performed on 25 base recycling projects in the Bryan District. The recycled layers were stabilized with cement or lime, and each was 250 mm thick. On the higher volume roadways, an unstabilized flexible base was placed over the stabilized layer, followed by a two course surface treatment (2 CST). On the other pavements the 2 CST was placed directly on the stabilized layer. Testing involved the use of the Dynaflect, falling weight deflectometer, dynamic cone penetrometer, and ground-penetrating radar. A correlation was generated between the backcalculated layer moduli and the percentage of stabilizer used. Tentative moduli values, to be used in future thickness designs, were also proposed.

Two visual surveys were completed. In 1997, of the 25 sections evaluated, 23 were judged to be performing well, with little or no surface distress. However, after the severe Texas summer of 1998, only 17 were judged to be performing well. The major distress found was severe localized longitudinal cracking, which originated in the subgrade. The shrink/swell potential of the subgrade soil appears to be the major factor controlling pavement performance. Sections constructed on soils with a plasticity index (PI) of more than 35 did not perform well. The severity of the surface cracking was also related to the following secondary factors: a) the summer droughts of 1996 and 1998, b) the presence of trees near the edge of the pavement, c) the side slope conditions, and d) the strength of the stabilized layer.

In conclusion, the base recycling technique applied by the Bryan District appears to be working if the subgrade soils have low to moderate PIs. This technique is not recommended for sections constructed on high PI subgrades. Recommendations to modify the existing design approach are proposed for these problem areas.

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IN-PLACE ENGINEERING PROPERTIES OF RECYCLED AND STABILIZED PAVEMENT LAYERS

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DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the opinions, findings, and conclusions presented herein. The contents do not necessarily reflect the official view or policies of the Texas Department of Transportation. This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes. The engineer in charge of the project was Tom Scullion, P.E. #62683.

There is no invention or discovery conceived or reduced to practice in the course of or under this contract including any art, method, process, machine, manufacture, design, or composition of matter; or any new and useful improvement thereof; or any variety of plant which is or may be patentable under the patent law of the United States of America or any foreign country.

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Elias Rmeili, design engineer of the Brownwood District and Craig Hogan, construction engineer of the Bryan District of the Texas Department of Transportation are acknowledged for initiating this study. Karl Nelson supervised this study; his careful planning and coordination helped us in all phases of this work. Darlene Goehl, the pavement design engineer, and Al Farrell, the district lab manager, assisted with data collection and data analysis. Gary Mitchell of the Bryan District and the Dynaflect operators from the Waco District collected the deflection data used in this study.

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IMPLEMENTATION RECOMMENDATIONS

The following recommendations are for future projects where the subgrade materials have a low to moderate Plasticity Index (PI<35):

1. Use the current design approach shown in Chapter 6 for selecting stabilizer type and stabilizer content.
2. Use the design moduli values or stiffness coefficients presented in Chapter 4 in either FPS 11 or FPS 19 for thickness design.

The following recommendations consider modifications to the design process for future projects where the subgrade has a high PI value or where the soils are known to have a high shrink swell potential:

1. Until more research is completed, it is recommended that the maximum level of stabilization should be 5 percent, when high PI clays are encountered.
2. On a research basis, consider the seven action items presented in Chapter 6 to minimize problems in these areas.

One concern from the project results is the large variability in final in-situ layer stiffnesses for the stabilized layers. While a lot of this is related to the variability of both the initial pavement and subgrade conditions, the authors are not convinced that Tex 121-E provides the optimal stabilizer content in all circumstances. On a research basis, it is proposed that TxDOT investigate if other design procedures will yield different optimal stabilizer contents. Both the compressive strength test and pH test methods should be compared to the current method. It is also proposed that once an optimum stabilizer content has been selected, then the moisture susceptibility of the material should be investigated with the Tube Suction test.

CHAPTER 1
INTRODUCTION

GENERAL

Now that most of the new road construction in the United States is complete, the major emphasis has switched to maintaining those roads. The focus of many of the district offices in the Texas Department of Transportation (TxDOT) is the maintenance and rehabilitation of the existing highway network. The rural districts now have large portions of their networks that need major rehabilitation and reconstruction. Many districts are experimenting with innovative rehabilitation strategies for repairing the wide range of pavement types found around the state. One area of major interest is in rehabilitating thin flexible pavements which are near the end of their service lives. It has been estimated that over 70 percent of the mileage of the Texas network has a thin surface less than 50 mm (2 in) thick over an unstabilized granular base (*4*). Many kilometers of these thin pavement type highways are being subjected to heavy loads from both agricultural and oil field activities. Generally, these roads are not structurally adequate to carry the heavy loads, and many are exhibiting significant deterioration.

To address this problem in a cost-effective manner, several of the districts have actively recycled pavements and stabilized bases in an attempt to correct deterioration and increase the structural capacity. In a one pass operation, the existing surface and base are mixed and stabilized with lime or cement. Frequently, the reshaped and compacted base is simply sealed with a one or two course surface treatment. For higher volume roads, a new flexible base may be added followed by a thin asphalt surfacing.

The goal of this research is to investigate the performance of the existing recycled pavements within the Bryan District and to evaluate the impact of the different variables used during their construction. Some of the variables of interest include:

- variations in stabilizer types (most of the projects used lime only, but several have used cement);
- variations in the percentage of stabilizer used (this varies from project to project based on the plasticity index (PI), i.e., PI of the layer to be treated);

- thickness of the stabilized layer (thickness ranging from 150 to 250 mm (6 to 10 in) have been used);
- adding new base on top of stabilized layer; and
- experimental sections (in at least one case, experimental sections were built in the same project to permit side by side comparison of performance).

Determining the optimal amount of stabilizer to add to an existing base is not well understood. There are at least two principal reasons for stabilizing bases: first to reduce the PI, making the base more resistant to moisture-induced damage and to aid construction, and second to improve the long-term structural strength of the layer. Many districts are having problems with both under- and over-stabilizing of bases. Guidelines for selecting the optimum level of stabilization are urgently required. The practice of the stronger the better is not appropriate. Most districts have reported extremely poor performance in projects that were over-stabilized. These projects frequently have extensive block cracking caused by shrinkage of the base layer followed by secondary deterioration caused by water entering the structure. This can take the form of subgrade swells or base erosion.

The problem of determining the optimum stabilizer content is complex since it is material dependent. The Bryan District has developed its own policy in this regard, and it will be described in the next section of this report together with the practices used in other districts. The main goal of this work is to develop a field testing protocol by which the performance of stabilized bases can be evaluated in terms of their visual condition, resistance to moisture damage, and improvements in long-term structural strength. The performance data collected in this study will also be used to develop input values (elastic moduli) for future pavement designs.

Methods Used to Determine Stabilizer Contents for Base Materials

In Texas, there are a variety of approaches used by districts to determine the percentage of stabilizer to use when recycling flexible base materials. The most widely used approach is "district experience," in which a fixed percentage of stabilizer is used independent of the base type to be treated. This approach works well when the district has only a few base types and a track record of successful performance. However, several districts have encountered problems with this approach and have developed in-house field and laboratory test procedures for arriving at the optimum stabilizer content. A few examples of the approaches used around the state are as follows.

The Bryan District uses an extension of TxDOT method 121-E. Field samples are obtained for each layer to be stabilized. In this method, the PI and percentage of material passing the No. 40 sieve of each layer are used to obtain the percentage stabilizer. If the Bryan District is mixing 150 mm (6 in) of base with 50 mm (2 in) of subgrade, then an analysis will be performed for each. The final percentage will be the weighted average of the two. For materials with a PI of less than 10, cement would be considered; for all other cases, lime would be used.

The Atlanta District performs an extensive series of Texas Triaxial tests on both the raw base material and the material stabilized with low percentages of stabilizer. Using blends of lime (L) and fly ash (FA) as stabilizers, they evaluate in the laboratory the benefits of 1 percent L/2 percent FA and 1.5 percent L/3 percent FA, etc. After 10 days of moist curing, the unconfined compressive strength of the material is measured. The district selects the combination of lime/fly ash which produces a three-fold increase in compressive strength over the raw material strength. In recent years, however, the district has stopped using fly ash and reverted to using lime stabilization only.

The Childress District has experienced some pavement failures caused by over-stabilization of the base layer. They have reviewed their philosophy with regard to stabilizer content. The new approach calls for testing the raw and improved material with the Texas Triaxial procedure and selecting the stabilizer percentage that produces a Class 1 or slightly better material. With their recent series of tests, they plan to reduce their stabilizer content from around 5 percent to around 3 percent. This district is actively trying to relate the laboratory compressive strength to the field modulus as backcalculated from the falling weight deflectometer (FWD).

Limited guidance is given in TxDOT's specifications (Red Book 1995) on selecting optimum stabilizer contents. Item 276 on cement-treated bases specifies minimum compressive strengths of 5170 kpa (750 psi) or 3450 kpa (500 psi). These strength values are high and can only be achieved with relatively high cement contents, usually 5 percent or above. However, districts using these levels have reported problems with shrinkage cracking (*5*). Most districts do not require such high compressive strength values; a common value now used is 1380 kpa (200 psi).

Clearly, the work conducted in this project will be of substantial interest to other districts in Texas. More work is needed around the state to systematically relate laboratory strengths to layer moduli to pavement performance.

Stabilizer Contents for Subbase and Subgrade Stabilization

Subbase and subgrade stabilization is very popular in many areas of Texas. These subbases and subgrades will have a substantial impact on long-term performance of the unstabilized base and surfacing placed on top of them. With these subbase layers, the impact of layer cracking is not as significant. The major concern with these materials is permanency and whether the stabilization will disappear with time. For these layers, the recent research findings have determined that loss of stabilization does occur, and in some instances, TxDOT should increase the stabilizer content. This should be contrasted with the recommendations for base stabilization where, in some instances, the recommendations have been to reduce the percentages of stabilizer. The leaching or non-permanency of stabilization of the lower layers is attributed to the fact that these layers are more prone than bases to experience wetting and drying cycles.

OBJECTIVES OF STUDY

In-place stabilization is one of the pavement rehabilitation strategies used for low volume farm-to-market roads. This strategy consists of in-place stabilization of the top 250 mm (10 in) of an existing pavement with either lime or cement and sealing the surface with a surface treatment. The engineering properties for this stabilized layer generally have not been determined. Currently, engineering judgment is often used to select stabilizer quantities used for pavement design. The approach followed in this research project is to evaluate projects that have been constructed in order to determine the in-place engineering properties achieved by stabilization and to develop guidelines on stabilizer levels. The objectives of this research are listed below:

i. Develop a correlation between the stiffness coefficients obtained from the Dynaflect and layer moduli values backcalculated from the FWD.

ii. Dynamic Cone Penetromer (DCP) tests results will be compared with the subgrade modulus as estimated from MODULUS 5.1. Thus, a correlation between DCP values and the subgrade modulus will be developed.

iii. Guidelines for stabilizer content selection based on the field-measured stiffness of the stabilized pavement layers will be developed. A correlation between the stabilizer content and the layer moduli values will be established.

iv. This research will help in the evaluation of completed projects by checking if the construction of the pavement layers conform to the specifications. This shall be achieved by comparing the FWD deflection profile after construction with the design deflection profile. The design deflection profile will be determined from the average subgrade modulus and the upper layer moduli values obtained from (iii) above.

v. A field testing protocol for using ground penetrating radar (GPR) and FWD to evaluate the performance of stabilized bases in terms of resistance to moisture damage and improvements in long-term structural strength will be developed.

vi. Investigate and identify the cause of pavement cracking, if any. Also, to make recommendations about how to avoid it in future projects.

RESEARCH ORGANIZATION

This research has been documented in six chapters. The second chapter summarizes the existing knowledge and current methods of collecting and analyzing nondestructive deflection data. It describes the GPR and DCP that were used in this project.

Chapter 3 summarizes data analysis of FWD, Dynaflect, GPR, and the DCP for each of the test sections. The performance and strength profiles for each section are presented together with a comparison to the target deflection profile.

Chapter 4 describes the problems encountered, namely, longitudinal cracking in a few of the test sections. It describes the additional work undertaken to identify the causes of the problem and the observations made during site investigations.

Chapter 5 presents several correlations between backcalculated layer moduli values and stabilizer content. At any particular stabilizer content, a large variation in layer moduli was found; therefore, a conservative approach was taken to generate recommended moduli values for future designs. Also included in this section is a correlation between layer moduli backcalculated from the FWD and stiffness coefficients calculated from the Dynaflect.

Chapter 6 presents overall conclusions and recommendations for future implementation. All of the backcalculated strength data and observed performance data are summarized. Several action items for district consideration are given.

Appendix A presents a summary of the performance data collected on each section. Photographs of the observed distresses are included.

Appendix B describes a simple test procedure to detect organic matter in soils.

Appendix C describes a simple test procedure to detect high sulfate contents in soils. Organics and sulfates are both potential problems when lime is to be used for soil stabilization.

CHAPTER 2
TECHNIQUES OF PAVEMENT EVALUATION

GENERAL

Devices used to perform non-destructive testing (NDT) on pavements include deflection measuring devices and GPR. Deflection measurements are used to evaluate the structural capacity of in-situ pavements by backcalculating an elastic moduli of various pavement layers. GPR is used to check the consistency, thickness, and entrapped moisture in the pavement layers. Each type of NDT deflection testing device applies a different type of loading to the pavement. On this basis, NDT deflection testing can be divided into three categories: static or slowly moving loads, steady-state vibration, and impulse loads (*20*).

The deflection devices available to TxDOT are the impulse load FWD and the steady state vibration Dynaflect. Each will be described in the following sections of this report.

FALLING WEIGHT DEFLECTOMETER (FWD)

All devices that deliver a transient force impulse to the pavement surface, such as the various types of FWD, are included in this category (*20*). By varying the amount of weight and the height of drop, different impulse forces can be generated. The normal operation is to move the trailer-mounted device to the test location; lower the loading plate and transducers hydraulically to the pavement surface; complete the test sequence by dropping the weight at each height selected; lift the loading plate and sensors; and tow the device to the next site. The major advantages of the impulse loading device are the ability to generate a deflection bowl similar to that from a moving wheel load in both magnitude and duration and the use of a relatively small static load compared to the impulse loading (*20*).

The most widely used FWD in the United States is the Dynatest Model 8000 FWD system (Figure 1), which was also used in this project (*20*). The impulse force is created by dropping a weight of 491; 983; 1965; or 2948 N (110; 220; 440; or 660 lb) from a height of 20 to 381 mm (0.8 to 15 in). By varying the drop height and weight, a peak force ranging from 6.7 to 107 kN (1500 to 24000 lb) can be generated. The load is transmitted to the pavement through a loading plate,

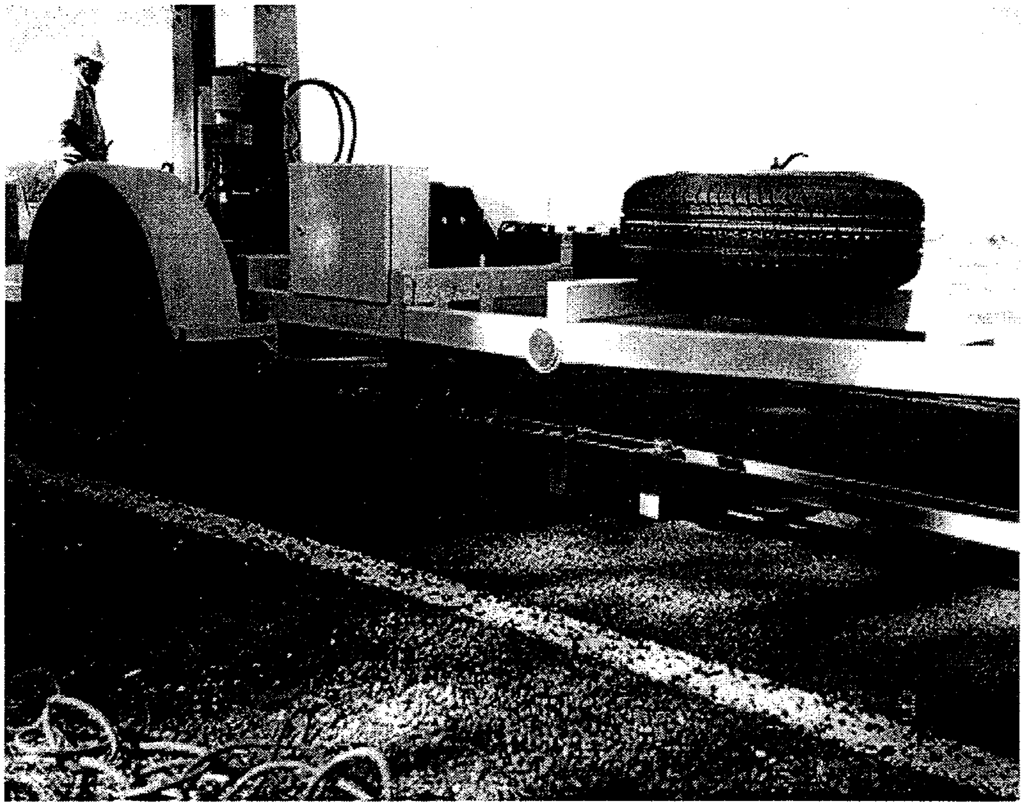


Figure 1. Falling Weight Deflectometer.

300 mm (11.8 in) in diameter, to provide a load pulse in the form of a half sine wave with a duration from 25 to 30 ms. The magnitude of load is measured by a load cell.

Deflections are measured by seven velocity transducers mounted on a bar that can be lowered automatically to the pavement surface with the loading plate. One of the transducers is located at the center of the plate, while the remaining six can be placed at locations up to 2.25 m (7.4 ft) from the center. The Dynatest FWD is also equipped with a microprocessor-based control console (*20*).

ANALYSIS OF FWD DEFLECTION DATA

Most techniques developed for the analysis of deflection data fall into two categories: deflection parameters and backcalculation of layer moduli. The deflection basin parameters are used directly to evaluate the pavement's structural integrity. These parameters are not a basic property of the pavement system, and at best they can be empirically related to pavement strength. Deflection basin parameters are device dependent, and relationships developed using one deflection device may not be applicable for use on deflection data obtained using a different device (*6,13*).

The second method of analysis, backcalculation of layer moduli, allows for the individual evaluation of each pavement layer and thus helps in identifying the causes of distress in the pavement layers. Moreover, since the elastic modulus of the pavement material is a fundamental property, it can be used to evaluate the performance of a pavement system (*13*).

The deflection data obtained from NDT are primarily used for evaluating the in-situ stiffness of individual pavement layers. The method chosen for the analysis of deflection data should in principle be compatible with the model that is used during pavement design (*6*). If a layered elastic approach is to be used in analyzing pavements during design, then a layered elastic technique should be used to analyze the deflection data. All assumptions made during backcalculation should be consistent with the pavement layer material behavior assumptions used in rehabilitation design (*6*).

Microcomputer-based backcalculation approaches generally use iterative techniques. The iteration process requires considerable computer time for each deflection data set, which makes it slow and expensive (*6*). A second approach stores many generated deflection basins and corresponding moduli values in a database for a given layer configuration. When a measured deflection basin is analyzed, the database is screened, and interpolations are used to find a deflection basin that best represents the measured basin. This procedure eliminates the iteration process and greatly reduces the computer time for a backcalculation when several deflection basins with the same layer configuration are being analyzed. This procedure is incorporated by Uzan et al. (*18*) in their backcalculation program MODULUS.

MODULUS generates a database for a specific cross section being analyzed and employs a Hooke-Jeeves (*3*) pattern search algorithm and a three-point LaGrange interpolation technique (*11*) to backcalculate layer moduli that minimize the error between the measured and the calculated deflections. The MODULUS program has been modified to estimate the depth to an apparent rigid layer (*12*).

In the linear elastic analysis, pavement materials are assumed to be homogenous, isotropic, and elastic. The discrepancies between the linear elastic model and actual pavement behavior are numerous. Pavement materials are generally heterogeneous, anisotropic, and granular. Some materials are highly stress dependent (non-linear), and some may become plastic or viscoelastic under elevated loads and temperatures. Furthermore, in the linear elastic approach, no failure criteria is considered. Therefore, calculated stress states can exceed the strengths of the materials.

The current version of MODULUS (version 5.1) was used to process the FWD data collected in this study. This procedure has been accepted by the pavement design division as part of the Flexible Pavement Design System. MODULUS 5.1 is now widely used in Texas districts to obtain layer moduli for input into FPS19.

DYNAFLECT

The Dynaflect is a steady-state vibration device. The deflections are generated by vibratory devices that impose a sinusoidal dynamic force over a static force. The magnitude of the peak-to-peak dynamic force is less than twice that of the static force, so the vibratory device always applies a compressive force of varying magnitude on the pavement. The deflections are measured by velocity transducers. These sensors are placed directly under the center of the load and at specified distances from the center, usually at 0.3 m (1 ft) intervals (*20*).

An inertial reference is used so the change in deflection can be compared to the magnitude of the dynamic force. A disadvantage of the Dynaflect is that the actual loads applied to pavements are not in the form of steady-state vibration and that the use of a relatively large static load may have some effect on the behavior of stress-sensitive materials.

The Dynaflect is trailer mounted and can be towed by a standard vehicle (Figure 2). A static weight of 8.9 to 9.3 kN (2000 to 2100 lb) is applied to the pavement through a pair of rigid steel wheels. A dynamic force generator is used to produce a 4.45-kN (1000-lb) peak-to-peak force at a frequency of eight cycles per second. The dynamic force is superimposed on the static force, and the deflections due to the dynamic force are measured by five velocity transducers (*20*).

Figure 2. Dynaflect.

The normal sequence of operations is to move the device to the test location and hydraulically lower the loading wheels and transducers to the pavement surface. A test is then conducted, and the data are recorded. If the next test site is nearby, the device can be moved on the loading wheels at speeds up to 9.6 km/h (6 mph). After the last test is completed, the loading wheels and transducers are hydraulically lifted and locked in a secure position. The relatively small fixed magnitude and frequency of the loading are major limitations of the device.

ANALYSIS OF DYNAFLECT DEFLECTION DATA

Scrivner et al. developed an equation for predicting the surface deflections of a pavement subjected to a known load (*14*). This deflection equation is the basis for several computer codes that compute stiffness coefficients of pavement materials used by TxDOT in designing pavements. The equations in the computer codes for computing elastic moduli of pavement materials are from Burmister's theory of elasticity in layered pavements (*10*). A deflection basin results from the Dynaflect loading. A computer program, STCOEF2, was used to calculate the stiffness coefficient of materials. The deflections at each radial distance are calculated from the geophone deflection readings and multipliers on the appropriate data cards. The Surface Curvature Index (SCI) is calculated as:

$$SCI = W_1 - W_2$$

where W_1 and W_2 are deflections of the respective sensors.

If any W (deflection) is equal to zero, or if any W is greater than its preceding W, the cases are flagged to denote data errors and are not used for further calculations. If the Ws are valid observations, they are passed to the subroutine along with the total pavement thickness for the stiffness coefficients and Root Mean Square Error (RMSE) calculations.

STCOEF2 returns to the main program the stiffness coefficients for both subgrade (AS2) and pavement (AP2) along with their corresponding RMSEs. The counter N (the number of sets of observations) is incremented, and the program reads the next data card and continues the process until all stations in a section are read. A loop is set up to print the station numbers, measured and predicted deflections, SCIs, stiffness coefficients for the subgrade and pavement, and RMSEs for all data observations. After all data and any error messages for a section are printed, the average deflections, SCIs, stiffness coefficients of the subgrade and pavement, standard deviations, and RMSE are calculated. These averages are then printed along with the number of points used in calculating each average.

The program then returns to its beginning to read data for another section or terminates execution normally when all data have been read (*10*).

GROUND PENETRATING RADAR (GPR)

Principles of GPR

GPR operates by transmitting short pulses of electromagnetic energy from an antenna into the pavement *(16)*. These pulses, as shown in Figure 3, are reflected back to a receiving antenna with the amplitude and arrival time that is related to the electrical properties of the pavement layers. The reflected energy is collected and displayed as a waveform; Figure 4 is a typical example showing amplitudes and arrival times of reflections. This wave is from a flexible pavement consisting of 178 mm (7 in) of hot mix over a 152 mm (6 in) granular base over a clay subgrade. The large peak (A) at 6 nanoseconds is the energy reflected from the surface; the peaks (B) and (C) represent reflections from the top of the base and subgrade, respectively. The time interval between peaks (A) and (B) is the travel time for the radar wave to travel from the surface to the top of the base and back (twice the asphalt thickness). The speed with which the electromagnetic radar wave travels in a particular layer is related to the dielectric constant of that layer. It is also the dielectric which determines what percentage of the energy is transmitted and reflected at each layer interface (*16*).

In pavements, the parameter that most influences the dielectric properties of materials is the moisture content. Table 1 shows dielectric constants for typical pavement materials. As can be seen from this table, the addition of moisture to any of these materials will have a significant influence on the dielectric properties of that layer. For example, a dry crushed limestone aggregate base course with 4 percent by weight of moisture will have a dielectric constant of around six; if the moisture content increases to 10 percent, then the dielectric of the layer would increase to around 11. The impact of a wet base on the trace shown in Figure 4 would be to increase the amplitude of peak B and to increase the travel time between peaks B and C.

The fact that GPR is sensitive only to changes in dielectric constants, which mostly equates to changes in layer moisture content, is of major significance. Without these differences in electrical properties, no energy will be reflected at interfaces. Several cases exist in pavements where the layers are so similar in dielectric constants that no significant reflections will be detected (*16*). Cases like this are common, such as granular base over sand subgrade, or concrete over cement-stabilized bases. In these cases, the difference in dielectric constants between layers may not be sufficient to permit layer thickness estimates.

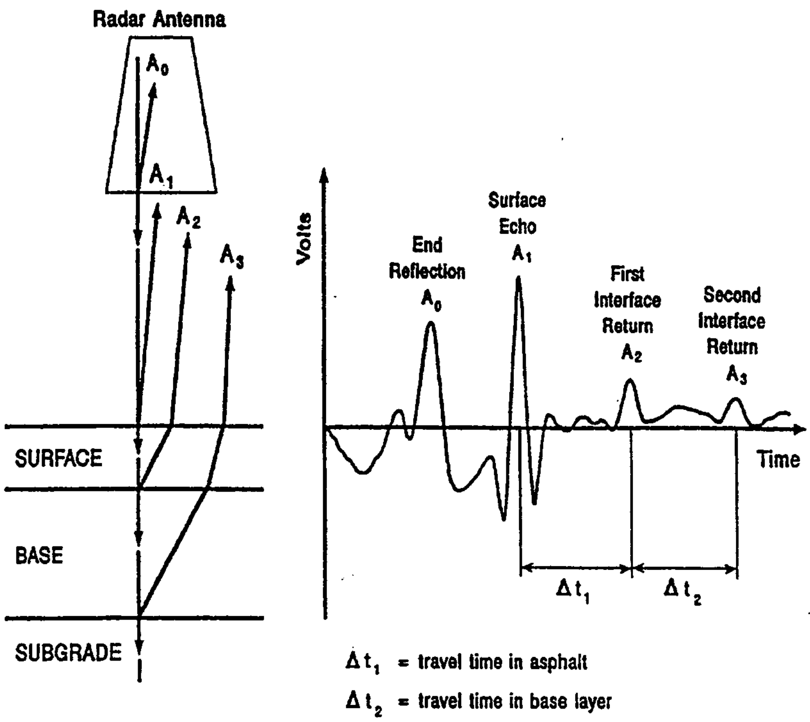


Figure 3. Principles of Ground Penetrating Radar. The Incident Wave is Reflected at Each Layer Interface and Plotted as Return Voltage Against Time of Arrival in Nanoseconds.

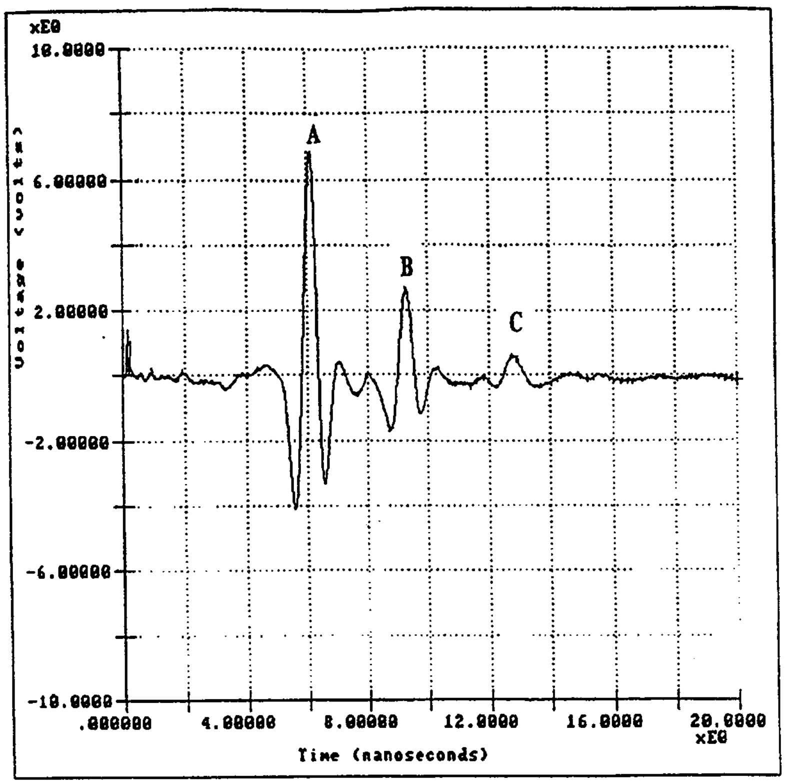


Figure 4. Typical GPR Waveform. Peaks A, B, and C are Reflections from the Surface, Top of the Base, and Top of the Subgrade, Respectively.

Table 1. Typical Dielectric Constant for Highway Materials.

Material	Dielectric Constant
Air	1
Water	81
Asphalt Concrete	3 - 6
Portland Cement Concrete	6 - 11
Limestone Aggregate	4 - 8
Clays	5 - 40
Dry Sand	3 - 5
Saturated Sand	20 - 30

Efforts have been made to develop signal processing techniques to interpret individual GPR waveforms. Without the ability to interpret a single trace, it may be difficult to process multiple traces. To use automated signal processing techniques, it is necessary to utilize a GPR system with clean, repeatable, transmitted pulses of GPR energy (*16*).

The software developed to analyze the signals automatically measures the amplitudes and time delays of each radar trace received and applies the signal processing described below. Figure 4 shows a single trace from a section of highway. The user can specify the frequency at which traces are to be collected. In some instances, such as void detection, one trace per 305 mm (1 ft) of pavement may be required. In others, such as layer thickness inventorying, one measurement (one trace) per 30 m (100 ft) may be adequate. In either case, a typical radar survey consists of collecting and processing multiple traces similar to the one shown in Figure 4.

The principles of GPR applied to highways have been given elsewhere (*8*). By automatically monitoring the amplitudes and time delays between peaks, it is possible to calculate layer dielectric constants and layer thicknesses. It is also possible to estimate the moisture content of a granular base material. These equations are summarized below (*8*).

$$\epsilon_a = \left[\frac{1 + A_o/A_m}{1 - A_o/A_m}\right]^2 \qquad (1)$$

where

ϵ_a	=	the dielectric of the asphalt or concrete surfacing layer
A_o	=	the amplitude of reflection from the surface in volts (peak A in Figure 4)
A_m	=	the amplitude of reflection from a large metal plate in volts (this represents the 100 percent reflection case)

$$h_1 = \frac{c \times \Delta t_1}{\sqrt{\epsilon_a}} \qquad (2)$$

where

h_1	=	the thickness of top layer
c	=	a constant obtained from the time calibration procedure described in section 3 of this report
Δt_1	=	the time delay between peaks A and B of Figure 4

$$\sqrt{\epsilon_b} = \sqrt{\epsilon_a}\left[\frac{1 - \left[\frac{A_o}{A_m}\right]^2 + \left[\frac{A_1}{A_m}\right]}{1 - \left[\frac{A_o}{A_m}\right]^2 - \left[\frac{A_1}{A_m}\right]}\right] \qquad (3)$$

where

ϵ_b	=	the dielectric of the base layer
A_1	=	the amplitude of reflection from the top of the base layer in volts (peak B in Figure 4)

$$M = \frac{\sqrt{\epsilon_b} - 1 - \gamma(\sqrt{\epsilon_s} - 1)}{\sqrt{\epsilon_b} - 1 - \gamma(\sqrt{\epsilon_s} - 22.2)} \tag{4}$$

where

M	=	the moisture content of base (percent of total weight)
ϵ_s	=	solids dielectric constant (varies from 4 to 8 depending on source material)
γ	=	dry density γ_d (lbs/ft^3) divided by density of solids γ_s (~165 lbs/ft^3)

Equation 4 assumes that the density of the material along a highway remains constant. This assumption is incorrect and will limit the accuracy of moisture content estimation. However, the moisture content is the major factor that influences the measured base dielectric constant ϵ_b. High base dielectric constants are almost certainly attributable to high moisture contents. The accuracy of equation 4 is yet to be determined (*16*).

The above equations serve as the basis for analysis of the data collected in this study. They are based on the assumption that the layer materials are non-conductive and homogenous. This assumption means that the imaginary component of the dielectric constant tends to be zero; therefore, the medium does not attenuate the radar signal. As a result, all of the energy is either reflected or transmitted, and none is lost in heating free water in the layer. The assumption of a very low imaginary dielectric from laboratory tests at the Texas Transportation Institute appears to be reasonable for hot mix asphalt concrete (*16*). However, it does not seem to be the case for either portland cement concrete or wet base course material. Because of the higher attenuation, it is thought that the accuracy of layer thickness estimates for both portland cement concrete layers and granular base layers may be less than for hot mix layers. The layer thickness estimates for hot mix asphalt concrete was found to be very good (*9*). The accuracy on granular base courses was reasonable, but this was also tied to the inability to physically measure the thickness of existing bases given the intrusion of subgrade materials (*16*). The accuracy of these equations for measuring concrete thicknesses is the subject of on-going research efforts.

DYNAMIC CONE PENETROMETER (DCP)

From an engineering viewpoint, one of the most important properties which a soil possesses is shearing resistance or shear strength (*19*). A soil's shearing resistance under given conditions is related to its ability to withstand load. The shearing resistance is especially important in its relation to the supporting strength or bearing capacity of a soil used as a base or subgrade beneath a road. For many pavement applications, the California Bearing Ratio (CBR) value of a soil is used as a measure of shear strength. DCP was originally designed and used for determining the strength profile of flexible pavements. It penetrates soil layers having CBR strengths in excess of 100 and also measures soil strengths less than a CBR of one (*1*). The DCP is a powerful, relatively compact, sturdy device that can be used by inexperienced personnel in pavement layer strength testing.

DCP Device

The DCP used in this project consists of a 16 mm (5/8 in.) diameter steel rod with a steel cone attached to one end which is driven into the pavement or subgrade by means of a sliding mass hammer, weighing 79 N (17.6 lb). The angle of the cone is 60°, and the diameter of the base of the cone is 4 mm (0.16 in) larger than that of the rod to ensure that the resistance to penetration is exerted on the cone. The DCP is driven into the soil by dropping a 79 N (17.6 lb) sliding hammer from a height of 574 mm (22.6 in). The depth of cone penetration is measured at selected penetration or hammer drop intervals, and the soil shear strength is reported in terms of DCP index. The DCP index is based on the average penetration depth resulting from one blow of the 79 N (17.6 lb) hammer. The DCP is designed to penetrate soils to depths of 914 mm (36 in). Individual DCP index values are reported for each test depth, resulting in a soil-strength-with-depth profile for each test location.

Extraction of the Cone

After the cone has been driven to the desired test depth, the cone or driving rod without disposable cone is extracted from the soil by driving the hammer against the top handle (*19*). Caution must be exercised during this operation in order not to damage the DCP device. The hammer must be raised in a vertical direction (rather than in an arching motion), or the rod may be bent or broken where it connects to the anvil. In soils where great difficulty is encountered in extracting the DCP device, the disposable cones should be used. Use of disposable cones will save wear and tear on both the device and operator. The DCP is kept clean, and all soil is removed from the penetration rod and cone before each test. A light application of spray lubricant or oil is applied to the hammer slide rod before each day's use. All joints are constantly monitored and kept tight. The lower penetration rod is kept clean and lubricated with oil when clay soils are tested.

Disposable Cone

The disposable cone was used in this project because, for the type of soils encountered, it was difficult to remove the standard cone. The disposable cone mounts on an adapter. At the conclusion of the test, the disposable cone easily slides off the cone adapter, allowing the operator to easily remove the DCP device from the soil. The disposable cone remains in the soil. Use of the disposable cone approximately doubles the number of tests per day that can be run by two operators.

Correlation of DCP Index with Subgrade Moduli Values

Correlation of the DCP index with Subgrade Moduli is necessary since the subgrade modulus is used for designing and evaluating flexible pavements (*19*).

CHAPTER 3
ENGINEERING ANALYSIS

GENERAL

A total of 25 projects were selected for testing as described in Chapter 1. Details of each of these projects are shown in Table 2. Fifteen projects were a four-layer system—surface treatment, aggregate base, stabilized sub-base, and subgrade. For four-layer pavements, the base was unbound granular aggregate, except for FM 1687, where 1 percent lime was used to stabilize the base as well. The remaining 10 projects were three-layer pavement systems—surface treatment, stabilized base, and subgrade. Lime was used as the stabilizer in all but two of the projects, with the percentage of lime varying between 3 percent and 10 percent. FM 3178 and FM 977 used 4 percent cement as the stabilizer.

CORRELATION OF DCP INDEX WITH SUBGRADE MODULI

DCP was attempted in all 25 projects. It was successfully completed in 19 projects. The original intent was to use the DCP to test both the stabilized layer and unstabilized subgrade. It was soon discovered that the DCP could not be used to test the stabilized layer. These layers were found to be extremely stiff and impossible to penetrate. The DCP was limited to testing the subgrade strengths. An access hole was drilled through the upper pavement layers. In some instances, the total upper layers were thicker than the drill available—500 mm (20 in). In these cases, no subgrade testing was possible. Where DCP testing was possible, the DCP tests were conducted following an FWD test. Generally, each project had one DCP, while a few had as many as three DCPs. Results from five projects were discarded because the values were not realistic. Review of geotechnical information provided by TxDOT indicated that the subgrade in these five projects, comprised of sand/silt, was probably the reason for unusual DCP behavior.

The subgrade soils were classified using the Unified System Classification of Soils (ASTM D-2487) based on the geotechnical data provided by TxDOT. Upon completion of the test, DCP penetration v/s number of blows was plotted for the DCP data, and subgrade moduli was backcalculated using MODULUS for the FWD data. As per the soil classification, the various

Table 2. Section Location, Construction Date, and Layer Thicknesses.

S. NO.	ROAD	COUNTY	C-S-J	LIMITS	LEN	DATE	STRUCTURE
1	FM 3090	Grimes	0643-05-026	From SH 6 to FM 3455	1.9	5-94	LSB (60-40) (3%), 8" FB, & 2 CST
2	FM 149	Grimes	0720-01-027	Montgomery C/L to 2.2 Miles West	2.2	6-94	LSB (60-40) (7%), 8" FB, & 2 CST
3	FM 1486	Grimes	1416-01-014	2.3 Miles S of FM 149 to 2.3 Miles South	2.3	7-94	LSB (60-40) (5%), 8" FB, & 2 CST
4	FM 542	Leon	0426-03-026	US 79 to 5 Miles South	5.0	6-94	LSB (60-40) (3%), 8" FB, & 2 CST
5	FM 542	Leon	0426-03-028	5 Miles South of Oakwood to Shiloh	2.8	4-95	LSB (60-40) (5%), 7" FB, & 2 CST
6	FM 3478	Walker	3550-01-006	1.8 Miles N of FM 980 to 1.1 Miles N	1.1	3-95	LSB (60-40) (6%), 11" FB, & 2 CST
7	FM 1362	Burleson	0833-12-015	SH 21 to FM 166	4.3	5-95	LSB (80-20) (4%), 7" FB, & 2 CST
8	FM 246	Freestone	0998-01-012	FM 27 to 3.3 Miles E	3.1	5-95	LSB (40-60) (6%), 9" FB, & 2 CST
9	FM 977	Leon	1147-03-008	SH 75 to 4.5 Miles E	4.5	7-95	LSB (60-40) (3%), 11" FB, & 2 CST
10	FM 27	Freestone	0456-01-031	Curb & Cutter in Wortham to FM 1366	5.0	9-95	LSB (60-40) (5%), 12" FB, & 2 CST
11	FM 3411	Walker	3394-01-005	SH 19 to FM 2929	2.3	9-95	LSB (50-50) (6%), 10" FB, & 2 CST
12	FM 111	Burleson	1922-01-012	FM 60 to 3.4 Miles E	3.4	12-95	LSB (50-50) (3%), & 2 CST
13	FM 1124	Freestone	2848-01-003	FM 488 to 1.8 Miles E	1.8	12-95	LSB (50-50) (4%), 5" FB, & 2 CST
14	FM 244	Grimes	0643-05-029	FM 3090 to SH 30	1.8	5-95	LSB (60-40) (4%), 12" FB, & 2 CST
15	FM 2223	Brazos	2130-01-007	OSR to FM 974	6.5	9-96	LSB (50-50) (10%), 12" FB, & 2 CST
16	FM 1687	Brazos	1560-01-021	FM 50 to OSR	3.92	5-96	LSB (60-40) (XX), XX", & 2 CST
17	FM 977	Leon	1147-03-010	3.2 Miles E of FM 1119 to FM 1119	3.2	2-96	LSB (30-60) (4%), & 2 CST
18	FM 978	Madison	0552-02-016	FM 39 to FM 2289	7.8	4-96	LSB (40-60) (5%), & 2 CST
19	FM 1373	Robertson	0540-06-013	Falls County Line to 6 Miles E	6.0		LSB (50-50) (XX), & 2 CST
20	FM 3178	Leon	1145-01-XX	FM 1511 to FM 542	5.0		CSB (50-50) (4%), & 2 CST
21	FM 977	Leon	114701020	FM 3 to 2 Miles E	2.0	5-96	CSB (50-50) (4%), & 2 CST
22	FM 1935	Washington	2619-01-XX	FM 390 to end	2.0		
23	FM 975	Burleson	1129-01-XX	SPRR to 5.5 Miles S	5.5	6-96	LSB (60-40) (3%), & 2 CST
24	FM 2446	Robertson	-------	Intersection of FM 46	1.0		---------
25	FM 2780	Washington	---(additional)--	FM 1697 to FM 1697	8.0		----------

sites were grouped together as CL (clays, low/medium plasticity) or CH (clays high plasticity) subgrade soils. Seven locations were tested in CH type soils. Figure 5a illustrates the correlation for highly plastic clay soils. Fifteen locations were tested in CL type soils. Figure 5b shows the correlation for low plasticity clay soils.

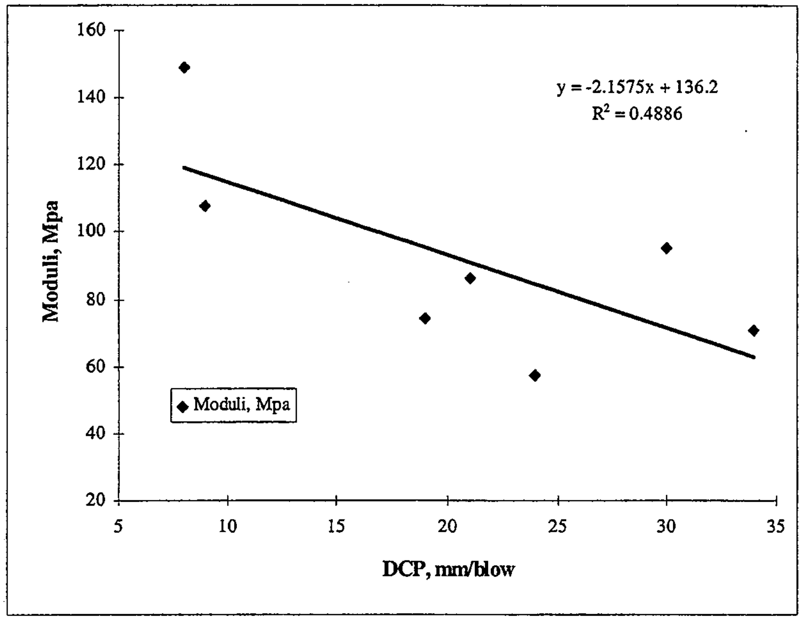


Figure 5a. DCP Index v/s Subgrade Moduli (CH Soils).

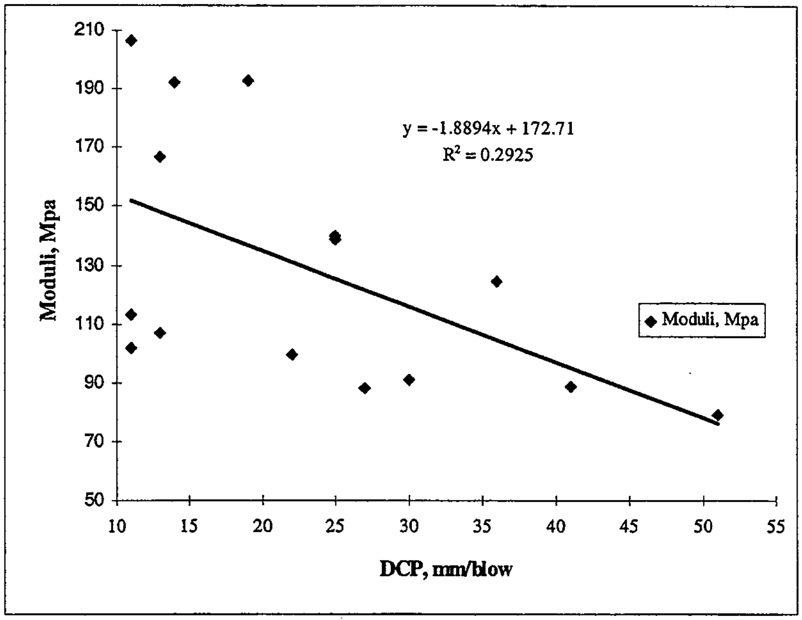


Figure 5b. DCP Index v/s Subgrade Moduli (CL Soils).

CORRELATION BETWEEN DYNAFLECT AND FWD

Figure 6 represents a correlation between the stiffness coefficients obtained from the Dynaflect and layer moduli values obtained from the FWD.

The solid line in the figure indicates the best fit line based on the field data. For the purpose of developing this correlation, only three-layer pavement systems—subgrade, base, and surface layers—were considered. Conducting four-layer analysis with Dynaflect doesn't yield realistic values because STCOEF2 software used in analyzing Dynaflect data, assumes a two-layer system. With the four-layer pavements, the stiffness coefficients would be a composite modulus for the stabilized subbase and unstabilized base.

It is important to note that the minimum stiffness coefficient calculated was 0.7. This is the value recommended by the Pavement Division for general pavement design. This data would indicate that the 0.7 value is conservative.

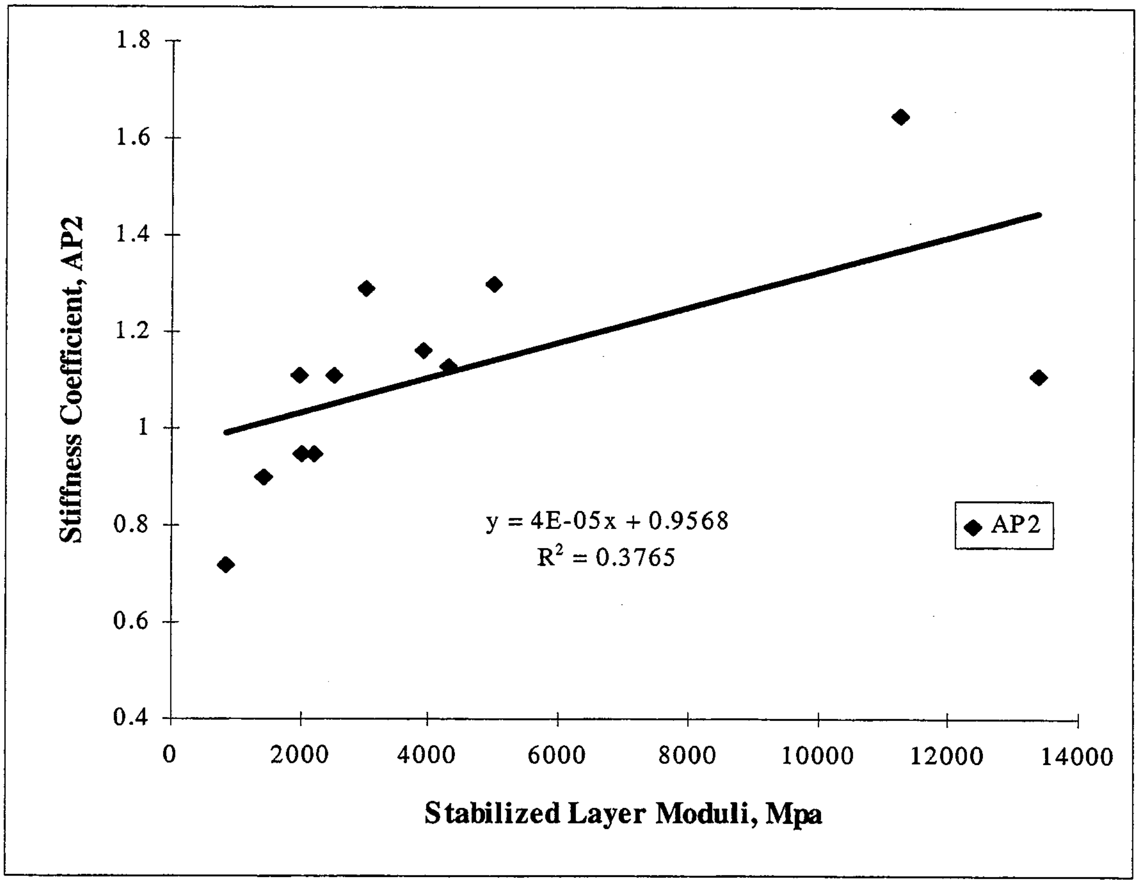


Figure 6. Correlation between Dynaflect and FWD.

CORRELATION BETWEEN STABILIZER CONTENT AND DYNAFLECT

The stabilizer content in each project was provided by TxDOT. All the projects used lime as stabilizer, except two where cement was used. Figure 7 illustrates the relationship between stabilizer content and Dynaflect stiffness values based on the field-measured stiffness of the stabilized pavement layers. Lime and cement-stabilized pavement layers seem to have nearly identical performance.

The solid line in Figure 7 indicates the best fit line based on the field data. For the purpose of developing this correlation, only three-layer pavement systems, i.e., subgrade, base, and surface layers, were considered. As mentioned earlier, conducting four-layer analysis with Dynaflect doesn't yield realistic values.

Although the trend is consistent, the variability of stiffness coefficients at any stabilizer content is very large. This will be discussed later when design recommendations will be given.

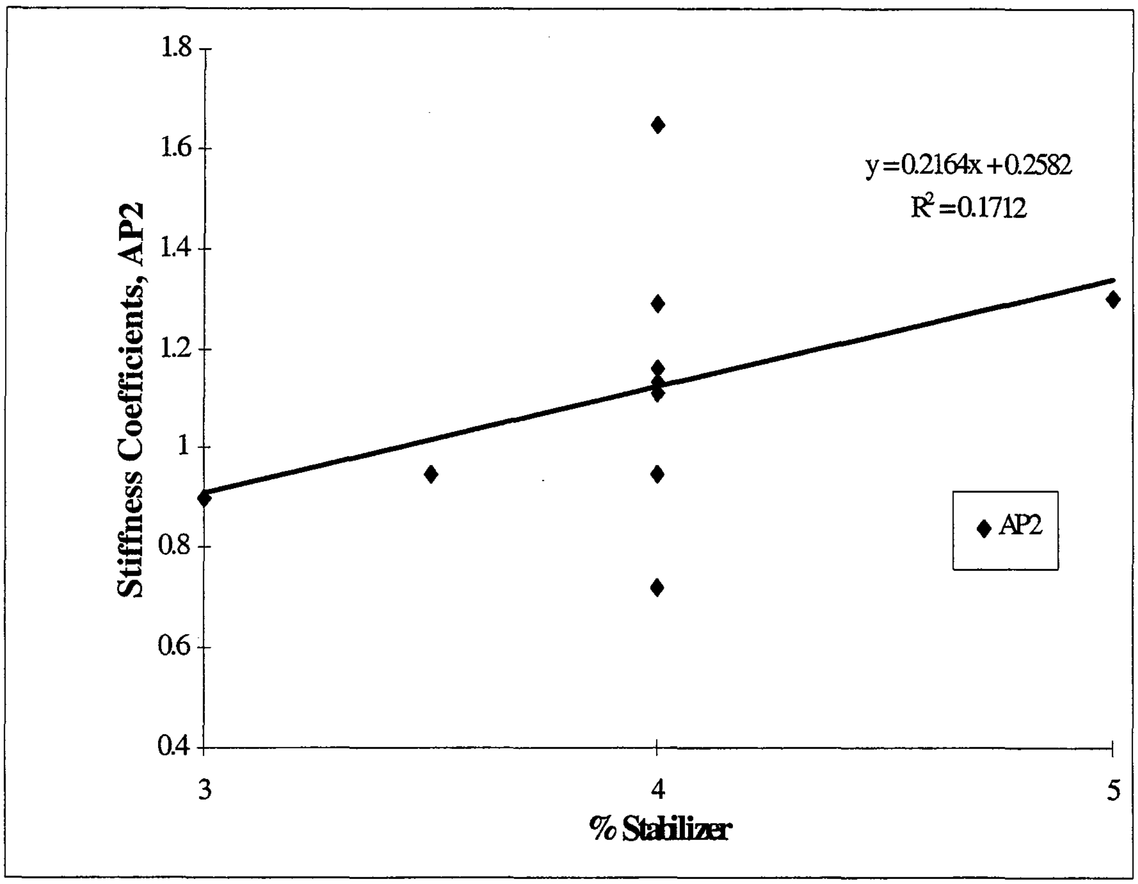


Figure 7. Correlation between Stabilizer Content and Dynaflect.

CORRELATION BETWEEN STABILIZER CONTENT AND FWD

As mentioned earlier, the stabilizer content in each project was provided by TxDOT. All the projects with four-layer pavement systems used lime as a stabilizer for the subbase; an unstabilized base layer was placed on top of the subbase. The one exception was FM 1687 where 1 percent lime was also used in the base layer. Figures 8 and 9 represent the relationship between the stabilizer content and the layer moduli values as calculated from FWD testing using MODULUS 5.1. Figure 8 indicates the correlation for a three-layer pavement system (2 CST, stabilized base, subgrade), while Figure 9 indicates the correlation for a four-layer pavement system (2 CST, unstabilized base, stabilized subbase, subgrade). The solid lines in the figures indicate the average lines based on the FWD testing data. For the purpose of developing this correlation, both three-layer and four-layer pavement systems—subgrade, subbase, base, and surface layers—were considered. This was possible because it is easier to carry out a four-layer analysis with an FWD than it is with a Dynaflect.

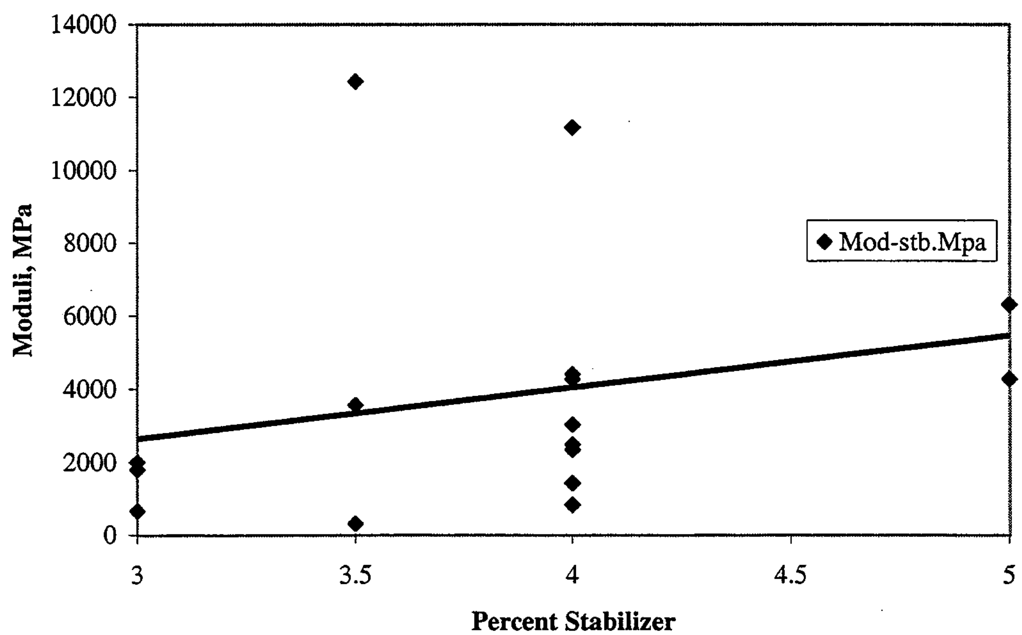


Figure 8. Correlation between Stabilizer Content and Moduli (Three-Layer).

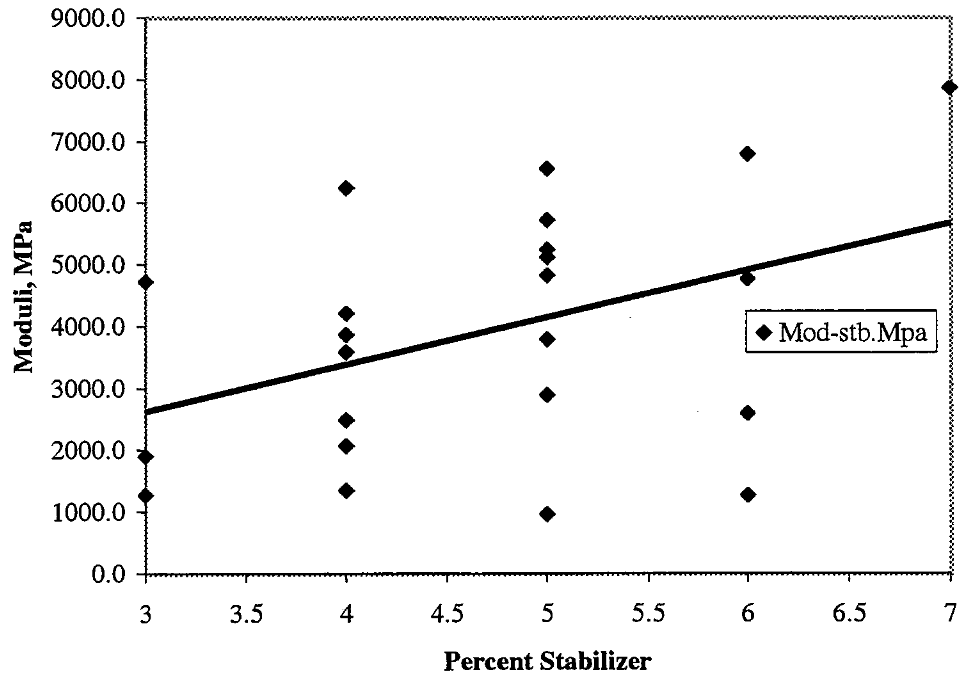


Figure 9. Correlation between Stabilizer Content and Moduli (Four-Layer).

SECTION BY SECTION ANALYSIS

This section presents a section by section analysis of each base recycling project included in the study. In order to evaluate the FWD data and its variation along the project, a **target deflection value** was computed for each project. This was achieved by assigning a target moduli value for each pavement layer using the average subgrade moduli for the section and the design layer thicknesses. Target deflection value is the acceptable deflection of the pavement based on the engineering properties of the pavement layer for the specified materials and the type of stabilizer. It was based on the level of stabilizer used and was obtained from the graphs of average layer moduli versus stabilizer content presented in the proceeding paragraphs.

Knowing the stabilizer content and the cross-section of the pavement, Figure 8 or 9 (as applicable) is used to calculate the anticipated moduli for the stabilized base and subbase layer. The modulus for the subgrade on each project was obtained from the FWD results. The average subgrade modulus was used in the analysis. Once the layer properties are known, maximum anticipated deflection can be calculated. This is called the **target deflection value**. If the pavement deflection is close to the **target deflection value**, the pavement generally has acceptable layer strengths. If the deflections are very much lower than the target value, then the pavement section is very stiff, and there is a chance that the pavement would crack. On the other hand, if the deflections are far in excess of the target value, then there are chances that the pavement section will deteriorate under traffic loads.

Pavement Rating

In order to rank both the pavement performance and in-place structural strength, an arbitrary ranking scheme was developed. A pavement performance indicator based primarily on the amount of cracking found in the sections was defined as follows:

A No Distress,

B Minor or Localized Cracking, and

C Major cracks in more than 25 percent of section.

A structural strength indicator based on FWD deflection data, its variability along the section, and the computed target deflection value discussed above was defined as follows:

A Deflection profile close to target deflection,

B Localized weak sections, >10 percent of readings with deflections greater than 50 percent above target deflection, and

C Several weak locations, >25 percent of readings with deflections greater than 50 percent above target deflection.

Most of the distress was found to be longitudinal cracking, with severe cracking and edge faulting found in many places. In a few projects—usually the lightly stabilized sections—some rutting or subgrade shear failures were found. Pavements with structural problems were given a B or a C performance rating depending upon the extent of the problem.

When reviewing this classification, it is critical to keep the following in mind:

1. With stabilized materials, there is no guarantee that a low FWD deflection/high layer moduli will translate to a good long-term pavement performance. A good structural rating (A) does not necessarily translate to a good performance rating. In study 1287 conducted by the Texas Transportation Institute (*5*), it was concluded that at the high end of the stabilization scale, the opposite was true; the stronger pavement provided poorer long-term performance. This loss in performance was attributed to shrinkage cracks from the stabilized layer. It is of considerable surprise to the research team that although several of the sections included in this study had very stiff bases with moduli in excess of 6900 Mpa (1000 ksi), no evidence of traditional shrinkage cracking was found on any of the sections. Regular shrinkage cracking is normally observed as regular transverse cracks.
2. Continued monitoring of these sections will be required to evaluate long-term performance, since the majority of the sections was less than four years old at the time of testing.
3. The real objective should be to identify sections that are performing well and then look at layer strengths, materials used, and stabilizer contents and types. The aim of this effort would be to evaluate the favorable conditions for pavement recycling and stabilization.
4. With newly constructed flexible base layers, the moduli value anticipated for the base is usually three to four times the subgrade value (*5*). This is of significance in the sections with unstabilized bases only. For example, with a subgrade of 69 Mpa (10 ksi), the anticipated unstabilized base moduli value would be 241.5 Mpa (35 ksi), and any increase above this would be attributed to the positive contribution of the stabilizer.
5. The subgrade moduli values obtained under the heavily stabilized subbase layers are inflated. This is because the moduli value obtained from backcalculation is dependent upon the stress level seen by the material under FWD loading. With heavily stabilized subbase layers, low subgrade stresses would be transmitted to the subgrade during testing, giving an apparent high subgrade modulus value (*5*). If the same subgrade was

used on a traditional flexible base pavement, substantially lower subgrade values would be backcalculated. In general, the subgrade values obtained in this study should not be used for everyday design work.

6. For several of the projects, clear breaks in subgrade soil type were found in either the USDA county soil series maps or from the District 17 drill logs. It became clear early in the study that when major changes in soil type occurred, then this usually translated to changes in pavement performance. In the section by section analysis that follows, a ranking is presented for each major soil type identified in each project.
7. In the remainder of this section, a year to year comparison is provided to compare the 1997 to 1998 visual inspection and FWD data. For example, the "Performance (97/98) **A/B**" means that the visual performance was rated as A in 1997 and B in 1998.

Of interest in the following pages is: a) the impact of soil type, as indicated by PI on pavement performance and b) the trends in pavement condition rating. The general trend was for the 1998 visual performance ratings to be significantly worse than the 1997 ratings. Three sections changed from an "A" rating (no distress) to a "C" rating (significant distress in over 25 percent of section). The summer of 1998 was hot and dry, with little rainfall for several months. This appears to have had an impact on the longitudinal cracking problem. It must also be remembered that several projects were only one or two years old at the time of the 1997 survey. The age of the sections is definitely a factor in both the strength and performance results; several sections have not apparently reached an equilibrium state (long term stable strength).

SECTION 1

Road	County	Limits	Date	Structure
FM 3090	Grimes	From SH 6 to FM 3455	5-94	LSB (60-40) (3%), 8"FB & 2 CST

RATING Performance (97/98) A/A Structural (97/98) B/B PI<20

FM 3090 had little or no structural surface distress in both the 97 and 98 inspections; however, some flushing was observed. Variability was observed in the surface deflections, although this was mostly attributed to weak subgrade areas. In 1997, about 25 percent of the section (five bowls out of 20) had deflections more than 50 percent higher than the target deflection of 272 microns (10.7 mils). Weak areas were encountered at 1220, 1830, and 1983 m (4000, 6000 and 6500 ft) from the beginning of a section. The GPR indicated some high dielectric constants in the stabilized subbase. These are also areas of higher deflection. The presence of moisture in stabilized layers could indicate potential future leaching problems. Figure 10 summarizes the deflection profiles for FM 3090 with the 1997 and 1998 maximum deflections from the FWD drop closest to 4140 Kg (9000 lbs) together with the target deflection line. Table 3 summarizes analysis of the 1997 FWD data for this project.

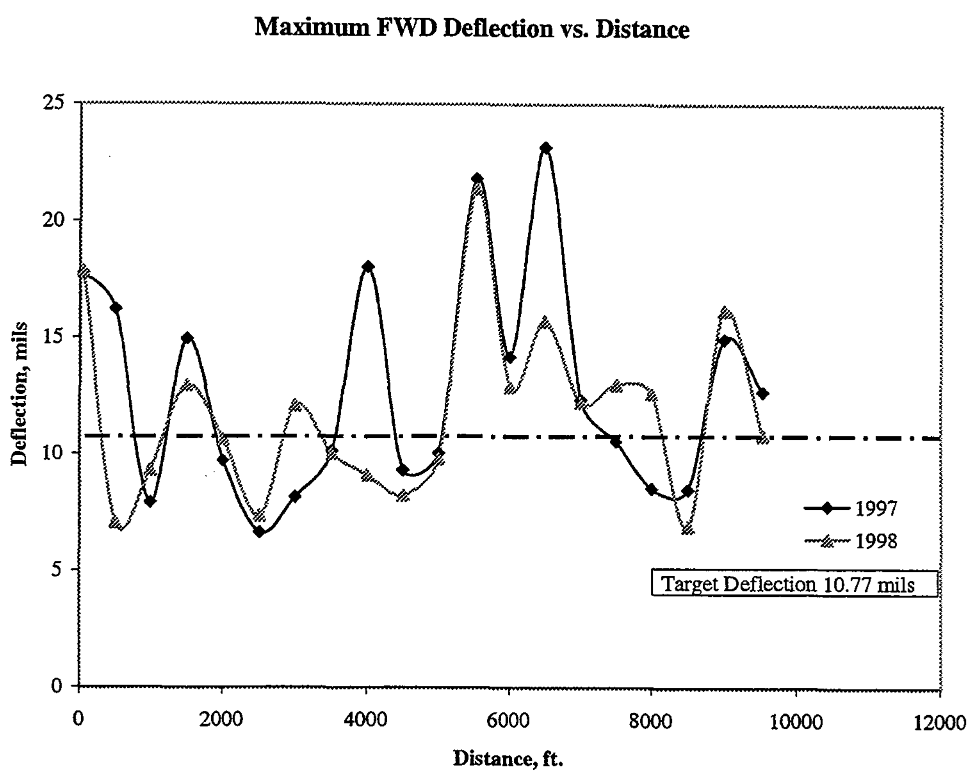


Figure 10. Target Deflection Profile for FM 3090.

Table 3. FM 3090, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 17			MODULI RANGE(psi)		
County: 94		Thickness(in)	Minimum	Maximum	Poisson Ratio Values
Highway/Road: fm3090	Pavement:	0.70	199,980	200,020	H1: % = 0.35
	Base:	8.00	25,000	500,000	H2: % = 0.35
	Subbase:	8.00	10,000	1,000,000	H3: % = 0.30
	Subgrade:	57.30		10,000	H4: % = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
52.000	10,340	17.77	3.20	2.55	1.97	1.53	1.17	0.91	200.	40.8	1000.0	24.8	22.08	294.19 *
506.000	10,367	16.20	4.56	3.62	2.68	1.92	1.48	1.18	200.	54.0	1000.0	14.7	11.40	300.00 *
1005.000	9,756	7.93	3.97	2.63	1.63	1.05	0.70	0.49	200.	210.0	172.2	26.3	2.20	36.00
1505.000	9,982	14.94	8.90	6.03	4.04	2.78	1.99	1.48	200.	115.1	220.2	7.4	2.66	230.32
2013.000	9,950	9.71	4.46	2.78	1.78	1.15	0.79	0.61	200.	150.1	155.5	25.3	1.47	155.34
2518.000	10,363	6.65	2.89	1.90	1.17	0.74	0.49	0.36	200.	230.7	268.8	39.3	6.86	36.00
3004.000	10,415	8.14	3.76	2.21	1.26	0.75	0.45	0.35	200.	205.8	121.0	36.2	2.58	36.00
3515.000	10,208	10.15	5.51	3.48	1.87	1.22	0.81	0.57	200.	228.0	87.8	19.6	2.43	98.74
4010.000	9,712	18.02	5.60	3.51	2.44	1.48	1.05	0.86	200.	41.8	759.6	13.7	11.82	117.35
4500.000	9,668	9.32	4.19	2.97	1.22	0.77	0.51	0.37	200.	202.6	65.4	29.8	8.74	36.00
5005.000	9,748	10.05	5.08	3.16	2.07	1.37	0.98	0.72	200.	156.3	151.0	20.2	1.14	181.14
5525.000	9,386	21.82	9.87	4.99	2.72	1.75	1.19	0.90	200.	58.3	47.9	11.7	9.25	107.55
6001.000	9,620	14.17	7.46	4.76	2.96	1.96	1.22	0.89	200.	99.8	166.1	10.8	2.65	126.03
6504.000	9,342	23.17	9.91	7.07	4.81	3.19	2.03	1.46	200.	42.2	286.8	6.1	2.51	136.71
7004.000	9,728	12.34	6.40	4.80	1.94	1.53	1.15	0.93	200.	145.7	84.0	15.6	11.88	59.93
7502.000	10,129	10.56	5.72	3.48	2.02	1.30	0.92	0.71	200.	196.3	88.7	19.4	1.07	148.65
8003.000	9,744	8.54	5.01	3.04	1.86	1.24	0.81	0.62	200.	230.0	152.2	19.2	2.45	141.63
8501.000	9,279	8.51	3.67	2.11	1.31	0.83	0.59	0.41	200.	159.9	131.1	31.1	3.16	36.00
9001.000	9,613	14.95	7.18	4.24	2.42	1.59	1.09	0.82	200.	97.6	80.4	15.0	4.42	135.25
9526.000	9,907	12.68	5.82	3.36	1.56	0.79	0.48	0.37	200.	191.4	27.6	24.5	11.37	36.00
Mean:		12.78	5.66	3.63	2.19	1.45	1.00	0.75	200.	142.8	253.3	20.5	6.11	73.98
Std. Dev:		4.73	2.08	1.33	0.93	0.65	0.45	0.34	0.	69.1	298.4	9.1	5.47	53.25
Var Coeff(%):		36.99	36.75	36.70	42.47	44.99	45.37	45.28	0.	48.4	100.0	44.4	89.58	71.98

SECTION 2

Road	County	Limits	Date	Structure
FM 149	Grimes	Montgomery C/L to 2.2 Miles West	6-94	LSB (60-40) (7%), 8"FB & 2 CST

RATING Performance (97/98) B/C Structural (97/98) A/A PI>35

In 1997, unusually high deflections were observed in the section from 1068 to 1251 m (3500 to 4100 ft), but this section was not part of the study. It is an existing bridge structure with 25 mm (1 in) HMAC over flexible base. The remainder of this pavement section was judged to be structurally strong, with only 5 percent (one bowl in 21) reporting more than 50 percent greater than target deflection. In 1997, at some locations, a severe longitudinal edge cracking problem was noticed; in 1998, the condition had deteriorated significantly, particularly in the last 1.6 km (mile) of the project (see condition report in Appendix A). GPR data did not indicate any apparent defects or wet areas. However, the section around these bridges is substantially wetter. Figure 11 summarizes the deflection profiles for FM 149; note that most of the deflections are below the target line. Table 4 summarizes analysis of the 1997 FWD data for this project.

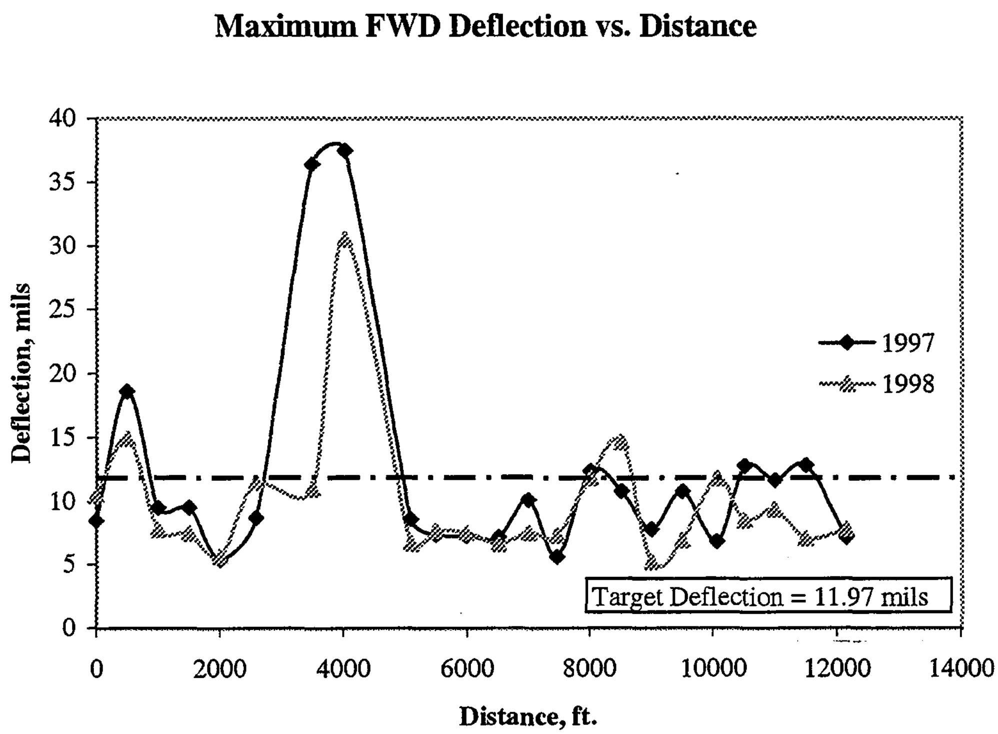


Figure 11. Target Deflection Profile for FM 149.

Table 4. FM 149, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 17
County: 94
Highway/Road: FM0149

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	0.70	199,980	200,020	H1: ¥ = 0.35
Base:	8.00	25,000	600,000	H2: ¥ = 0.35
Subbase:	8.00	10,000	2,000,000	H3: ¥ = 0.30
Subgrade:	255.60	10,000		H4: ¥ = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock	
0.000	9,620	8.48	7.01	5.37	4.01	2.96	2.21	1.75	200.	600.0	104.8	16.5	4.68	300.00	*
499.000	9,219	18.59	10.96	8.67	5.96	4.06	2.80	2.24	200.	117.5	65.1	10.7	5.16	193.90	
1005.000	10,161	9.47	5.43	4.33	3.31	2.61	2.04	1.57	200.	185.1	778.4	17.7	3.58	283.95	
1504.000	9,748	9.48	4.69	3.83	2.91	2.15	1.63	1.27	200.	122.5	1513.1	19.4	2.27	300.00	
2002.000	9,470	5.35	2.76	2.44	2.05	1.71	1.41	1.22	200.	376.0	2000.0	26.1	8.29	300.00	*
2598.000	9,501	8.65	5.86	4.42	3.33	2.60	2.02	1.63	200.	293.8	302.0	17.8	1.61	300.00	
3499.000	8,262	36.40	18.98	9.91	6.36	4.56	3.43	2.75	200.	40.8	12.0	8.5	3.48	300.00	
4028.000	8,671	37.48	17.72	7.59	4.98	3.69	2.88	2.23	200.	33.8	11.0	11.0	7.02	119.88	*
5088.000	9,370	8.58	4.26	3.89	3.11	2.50	1.97	1.63	200.	163.4	2000.0	15.8	6.08	300.00	*
5497.000	9,648	7.33	5.48	4.84	3.95	3.11	2.41	1.96	200.	533.5	573.3	14.9	2.46	300.00	
5997.000	9,561	7.26	4.40	3.95	2.86	2.28	1.69	1.35	200.	225.5	1574.3	18.3	1.90	281.16	
6511.000	9,593	7.13	4.20	3.60	2.79	2.19	1.70	1.42	200.	224.1	2000.0	18.3	1.94	300.00	*
7002.000	10,252	10.07	5.34	4.16	3.24	2.40	1.79	1.38	200.	131.2	1189.5	18.8	1.74	284.87	
7459.000	9,815	5.56	3.10	2.81	2.06	1.61	1.26	1.12	200.	323.4	2000.0	25.9	4.19	300.00	*
8005.000	8,977	12.29	7.71	7.12	5.60	4.24	2.92	1.93	200.	123.9	821.4	10.0	4.61	152.33	
8508.000	9,287	10.72	6.49	5.32	4.06	3.07	2.29	1.76	200.	146.4	607.5	13.9	0.64	289.50	
9000.000	9,521	7.73	4.00	3.65	3.19	2.75	1.52	1.32	200.	185.5	2000.0	17.2	7.67	97.19	*
9497.000	9,072	10.75	7.04	5.68	4.50	3.38	2.52	1.68	200.	175.4	428.1	12.8	1.24	153.09	
10059.000	9,815	6.78	4.17	3.67	3.16	2.65	2.17	1.82	200.	336.6	2000.0	17.1	4.80	300.00	*
10511.000	9,076	12.70	4.38	3.11	2.59	2.18	1.80	1.54	200.	59.1	2000.0	19.0	7.48	300.00	*
10995.000	9,168	11.57	5.51	5.31	4.44	3.70	3.02	2.03	200.	107.4	2000.0	11.1	6.53	154.09	*
11499.000	8,941	12.77	6.01	5.13	3.81	2.86	2.14	1.65	200.	76.1	1326.8	14.0	1.70	299.09	
12150.000	9,509	7.09	3.27	2.56	1.95	1.49	1.15	0.90	200.	151.4	2000.0	27.2	6.25	300.00	*
Mean:		11.84	6.47	4.84	3.66	2.82	2.12	1.66	200.	205.8	1187.3	16.6	4.14	272.35	
Std. Dev:		8.44	4.15	1.91	1.21	0.83	0.60	0.41	0.	146.6	791.7	5.0	2.34	133.46	
Var Coeff(%):		71.30	64.15	39.55	33.01	29.34	28.24	24.99	0.	71.3	66.7	30.2	56.53	49.00	

SECTION 3

Road	County	Limits	Date	Structure
FM 1486	Grimes	2.3 Mi S of FM 149 to 2.3 Mi N	7-94	LSB (60-40) (5%), 8"FB & 2 CST

RATING 0.0 to 1.5 mile Performance (97/98) A/A Structural (97/98) A/n.c. , PI<15

RATING 1.5 to 1.9 mile Performance (97/98) A/C Structural (97/98) C/n.c., PI>35

In 1997, the overall surface condition for FM 1486 looked good; no cracks were found. In 1998, a short section of longitudinal and transverse cracking was found. These cracks were severe in places, and crack sealing had been performed. The cracked section ties in with the area of high deflection observed in 1997. The **n.c.** indicates that the FWD data were not collected on this project in 1998. From the 1997 data, two locations with high deflections were observed. These are areas where the subgrade strength is only 50 percent of the average strength. GPR data did not indicate any apparent defects. Low subbase strengths were observed between 2898 and 3279 m (9500 and 10,750 ft). The average subbase moduli in this project is 3195 Mpa (463 ksi), but between 2898 and 3279 m (9500 and 10,750 ft), the moduli was only 414 Mpa (60 ksi). Continued monitoring of this section is recommended. Figure 12 summarizes the deflection profiles for FM 1486, and Table 5 summarizes analysis of the 1997 FWD data for this project.

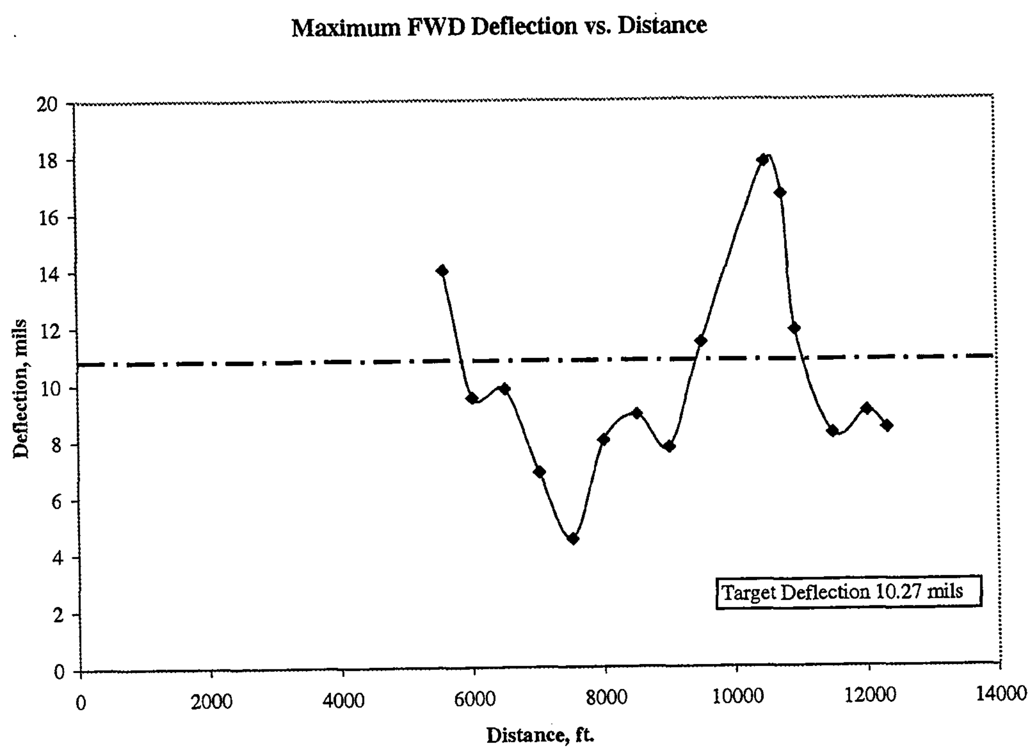


Figure 12. FM 1486, Target Deflection Analysis.

Table 5. FM 1486, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

				MODULI RANGE(psi)		
			Thickness(in)	Minimum	Maximum	Poisson Ratio Values
District:	17	Pavement:	0.70	199,980	200,020	H1: % = 0.35
County:	94	Base:	10.00	25,000	500,000	H2: % = 0.35
Highway/Road:	FM1486	Subbase:	10.00	25,000	1,000,000	H3: % = 0.30
		Subgrade:	166.60	10,000		H4: % = 0.40

	Load	Measured Deflection (mils):							Calculated Moduli values (ksi):				Absolute	Dpth to	
Station	(lbs)	R1	R2	R3	R4	R5	R6	R7	SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	ERR/Sens	Bedrock	
5590.000	9,525	13.97	5.01	3.69	2.71	1.93	1.33	0.98	200.	76.2	160.5	21.6	7.61	174.76	
6015.000	9,458	9.51	3.88	3.17	2.33	1.72	1.28	1.02	200.	113.3	672.2	20.6	4.34	284.36	
6518.000	10,117	9.82	3.76	3.07	2.41	1.22	1.06	0.86	200.	104.0	832.8	23.9	5.24	78.26	
7021.000	9,330	6.88	3.13	2.50	1.96	1.45	1.07	0.81	200.	177.1	1000.0	21.1	5.24	247.32	*
7525.000	9,470	4.52	1.86	1.38	0.97	0.67	0.48	0.36	200.	383.0	231.3	55.8	8.29	24.00	
8011.000	9,323	8.00	4.04	3.31	2.57	1.96	1.49	1.18	200.	155.2	1000.0	15.9	2.73	300.00	*
8513.000	9,477	8.91	4.45	3.92	3.16	2.45	1.89	1.54	200.	147.5	1000.0	13.9	2.29	300.00	*
9008.000	9,136	7.74	4.65	4.28	1.99	1.67	1.39	1.22	200.	296.9	87.6	22.7	10.40	69.36	
9515.000	8,897	11.45	6.35	4.77	3.48	2.48	1.51	1.22	200.	140.5	94.7	15.7	5.51	117.89	
10495.000	9,307	17.79	10.65	8.42	5.22	3.70	2.80	2.15	200.	133.1	29.2	10.8	3.49	300.00	
10756.000	9,124	16.65	8.21	5.96	4.19	2.86	2.06	1.54	200.	81.8	74.6	13.5	2.75	239.70	
10937.000	9,716	11.87	5.97	5.20	3.63	2.86	2.10	1.75	200.	110.8	417.5	13.7	3.30	300.00	
11504.000	9,084	8.22	4.87	3.75	3.26	2.59	2.12	1.67	200.	173.7	1000.0	13.0	2.99	300.00	*
12015.000	9,867	9.00	5.90	4.91	3.90	3.03	2.31	1.81	200.	256.0	282.3	14.2	1.30	300.00	
12321.000	9,044	8.40	5.02	3.16	2.26	1.62	1.22	0.98	200.	235.8	74.7	23.4	2.39	300.00	
Mean:		10.18	5.18	4.10	2.94	2.15	1.61	1.27	200.	172.3	463.8	20.0	4.53	187.32	
Std. Dev:		3.61	2.12	1.65	1.06	0.80	0.60	0.47	0.	86.1	403.6	10.8	2.56	234.99	
Var Coeff(%):		35.49	40.86	40.29	36.01	37.14	37.07	36.70	0.	50.0	87.0	54.3	56.46	125.45	

SECTION 4

Road	County	Limits	Date	Structure
FM 542-1	Leon	US 79 to 5 Miles South	6-94	LSB (80-20) (3%), 8"FB & 2 CST

RATING 0.0 to 1.2 mile Performance (97/98) A/A Structural (97/98) C/C PI>25

RATING 1.2 to 5.0 mile Performance (97/98) A/A Structural (97/98) A/A PI<10

This section was stabilized with 3 percent lime because the subgrade in this area is generally sandy. Most of the 1997 deflections were found to be below the target deflection; however, major increases were observed in the section from 1220 to 1830 m (4000 to 6000 ft). This weak section is clear in both the 1997 and 1998 data; from reference to the USDA soils series maps, this section does not have a sandy subgrade. The PI for this location was reported as greater than 25. The deflections increased significantly from 1997 to 1998 in this area.

The other high deflection spot was on a bridge approach, not a part of the reconstructed section. The remainder of the section looked good; no significant distress was found in either year. Although the deflection data are variable, this section is performing very well. It will be important to track the long-term performance of the first 1.9 km (1.2 mile) of this project. Figure 13 summarizes deflection profiles for FM 542-1, and Table 6 summarizes analysis of the 1997 FWD data for this project.

Maximum FWD Deflection vs. Distance

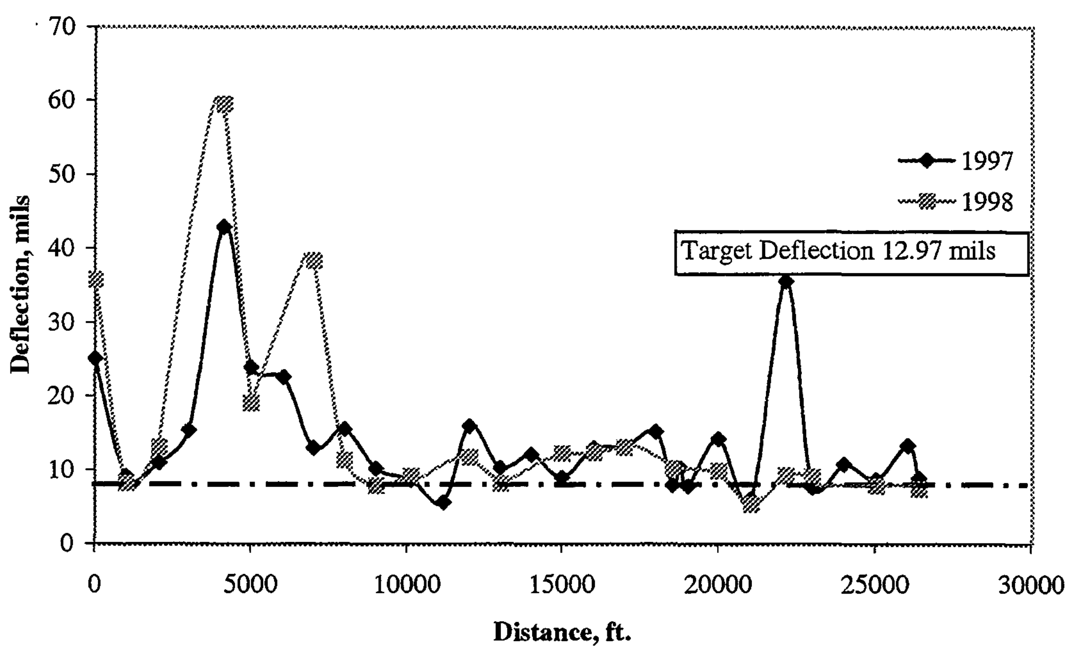


Figure 13. FM 1486, Target Deflection Analysis.

Table 6. FM 542-1, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District:	17			MODULI RANGE(psi)		
County:	145		Thickness(in)	Minimum	Maximum	Poisson Ratio Values
Highway/Road:	FM542-1	Pavement:	0.70	199,980	200,020	H1: % = 0.35
		Base:	8.00	25,000	500,000	H2: % = 0.35
		Subbase:	10.00	10,000	1,000,000	H3: % = 0.30
		Subgrade:	170.40	10,000		H4: % = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
0.000	8,743	25.08	15.47	7.07	3.36	1.91	1.35	1.14	200.	65.5	14.1	14.1	15.92	74.66 *
1002.000	9,501	9.11	6.30	4.42	3.15	2.42	1.90	1.56	200.	342.7	116.2	17.7	3.00	300.00
2086.000	10,506	10.95	4.49	3.72	2.84	2.32	1.94	1.67	200.	101.8	715.0	20.5	8.99	300.00
3036.000	9,950	15.36	7.35	6.09	4.66	2.67	2.20	1.82	200.	79.9	252.4	14.4	5.91	99.84
4146.000	8,905	42.89	24.17	9.54	4.78	3.26	2.70	2.25	200.	30.9	10.0	9.3	11.25	81.17 *
4999.000	9,970	23.86	12.74	7.97	5.05	3.54	2.43	1.95	200.	85.5	26.5	12.3	1.91	189.01
6029.000	9,617	22.57	12.50	10.33	7.49	5.43	3.80	2.60	200.	64.5	125.4	7.8	3.26	178.60
7008.000	10,169	12.96	5.96	4.90	3.72	2.87	2.31	1.81	200.	83.2	1000.0	14.0	3.35	300.00 *
8002.000	10,852	15.51	4.75	3.54	3.20	2.77	2.21	1.98	200.	57.1	904.1	19.1	11.25	300.00
9002.000	10,455	10.15	3.62	3.16	2.62	2.11	1.72	1.46	200.	101.1	930.5	23.0	12.47	300.00
10114.000	10,177	8.61	3.01	2.04	1.57	1.17	0.93	0.70	200.	105.3	1000.0	32.0	8.31	300.00 *
11151.000	9,732	5.59	3.52	2.96	2.38	1.83	1.37	1.05	200.	334.6	1000.0	22.0	1.13	267.56 *
12002.000	9,386	15.90	3.75	2.63	1.89	1.38	1.09	0.92	200.	41.6	730.0	27.7	4.77	300.00
12998.000	10,379	10.22	5.13	4.37	3.48	2.67	2.03	1.55	200.	130.8	1000.0	15.1	3.03	266.35 *
14006.000	10,312	12.08	3.56	4.11	1.53	1.18	1.04	0.90	200.	74.3	409.7	30.6	13.33	***
14996.000	10,165	8.91	5.05	3.90	3.22	2.47	1.78	1.34	200.	162.8	866.4	16.2	1.97	221.35
16029.000	9,891	12.94	6.07	5.27	4.50	3.77	3.12	2.61	200.	94.5	1000.0	11.3	6.26	300.00 *
17004.000	9,859	13.04	5.49	4.22	3.47	2.68	2.00	1.55	200.	73.8	1000.0	15.2	4.39	287.69 *
18006.000	9,815	15.13	8.65	6.48	4.84	3.56	2.55	1.77	200.	108.1	152.6	12.2	1.26	175.85
18534.000	10,737	7.83	5.16	4.29	3.72	3.13	2.62	2.31	200.	323.4	1000.0	14.7	3.33	300.00 *
18783.000	10,713	10.21	5.80	4.12	3.23	2.27	1.63	1.14	200.	175.9	248.8	20.0	1.21	173.18
19034.000	10,387	7.75	5.16	4.43	3.58	2.83	2.20	1.75	200.	279.4	825.6	15.6	1.03	300.00
20002.000	10,030	14.20	7.76	7.57	2.64	2.21	1.87	1.50	200.	178.6	43.0	17.9	13.87	57.65
21029.000	10,681	6.01	3.02	2.54	1.96	1.53	1.17	0.91	200.	312.8	1000.0	27.9	6.47	293.75 *
22152.000	9,628	35.59	17.59	8.43	5.46	4.02	3.18	2.55	200.	42.4	15.9	10.4	6.53	300.00
23003.000	9,962	7.72	3.36	3.48	3.19	1.15	0.93	0.81	200.	142.5	1000.0	24.3	16.82	*** *
23997.000	9,922	10.74	4.73	3.83	3.32	2.80	2.43	0.87	200.	115.4	1000.0	15.0	11.21	89.01 *
25011.000	9,565	8.67	4.97	3.92	2.80	2.13	1.58	1.20	200.	198.6	292.7	19.2	1.93	263.50
26019.000	9,970	13.28	6.68	4.90	3.52	1.80	1.42	1.14	200.	132.1	80.9	20.0	8.66	80.64
26375.000	10,101	8.87	4.15	2.81	1.73	1.14	0.83	0.70	200.	167.3	153.3	33.4	2.14	175.56
Mean:		14.06	7.00	4.90	3.43	2.50	1.94	1.52	200.	140.2	563.8	18.4	6.50	189.13
Std. Dev:		8.43	4.86	2.12	1.28	0.97	0.73	0.57	0.	91.9	418.9	6.7	4.81	131.04
Var Coeff(%):		59.93	69.46	43.20	37.43	38.81	37.34	37.66	0.	65.6	74.3	36.5	74.07	69.29

SECTION 5

Road	County	Limits	Date	Structure
FM 542-2	Leon	5 Mi S of Oakwood to Shiloh	4-95	LSB (80-20) (5%), 7"FB & 2 CST

RATING 0.0 to 2.0 mile Performance (97/98) B/B Structural (97/98) B/A PI>25

RATING 2.0 to 2.8 mile Performance (97/98) B/B Structural (97/98) B/A PI<25

In 1997, a few severe longitudinal cracks were found between two bridge structures (1680 m (5508 ft) from the start of section) on FM 542-2. The subgrade modulus was computed to be very low in this area—approximately 50 percent of the average section subgrade modulus. The DCP reading indicated very poor subgrade in cracked areas—noted as heavy clays. Deflections in this short section increased significantly from 1997 to 1998. Additional site investigations have been conducted in this area, with drilling and sampling down to 6.1 m (20 ft). The average moduli for the stabilized layer was found to be high for this section. This stiff layer could be related to severity of cracking over the heavy clay. The initial cracks observed in the 1997 survey were patched with hot mix and had not reoccurred in the 1998 survey.

Outside the cracked areas, the deflections were low, and no distresses were observed. Figure 14 summarizes the deflection profiles for FM 542-2, and Table 7 summarizes analysis of the 1997 FWD data for this project.

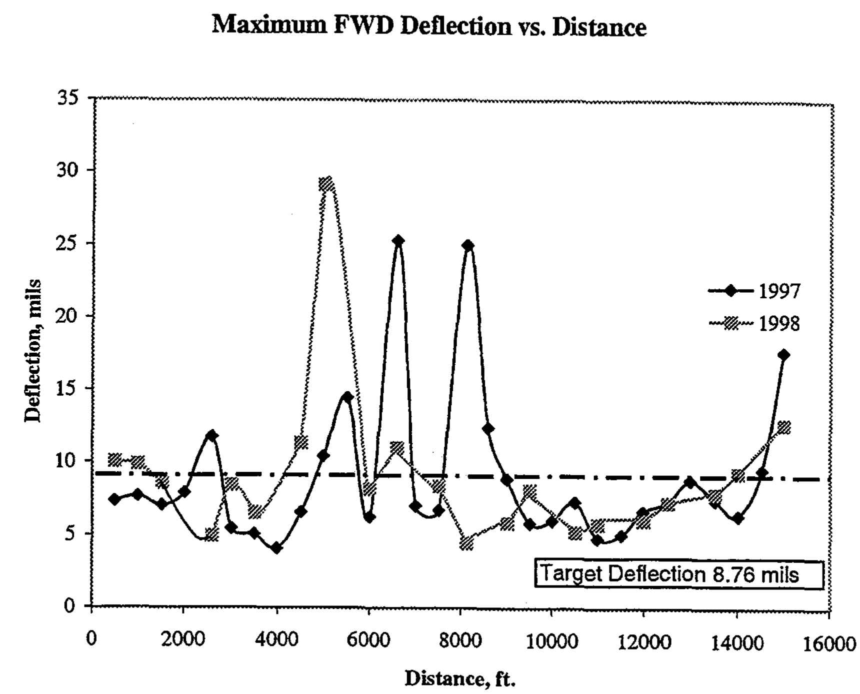


Figure 14. FM 542-2, Target Deflection Analysis.

Table 7. FM 542-2, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 17			MODULI RANGE(psi)		
County: 145		Thickness(in)	Minimum	Maximum	Poisson Ratio Values
Highway/Road: FM542-2	Pavement:	0.70	199,980	200,020	H1: % = 0.35
	Base:	7.00	25,000	600,000	H2: % = 0.35
	Subbase:	10.00	25,000	1,500,000	H3: % = 0.30
	Subgrade:	258.80	10,000		H4: % = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock	
496.000	9,839	7.32	4.31	3.60	2.83	2.17	1.65	1.28	200.	246.2	693.8	20.1	2.38	294.43	
1003.000	10,407	7.67	3.90	3.57	2.85	2.25	1.79	1.45	200.	200.9	1500.0	18.3	6.10	300.00	*
1524.000	10,284	7.05	3.91	3.60	2.91	2.31	1.85	1.16	200.	254.8	1500.0	18.4	4.87	130.85	*
2003.000	10,467	7.89	3.37	3.25	2.60	2.18	1.65	1.44	200.	180.2	1500.0	20.0	11.27	300.00	*
2597.000	9,573	11.76	6.57	5.44	4.27	3.31	2.55	2.05	200.	104.7	712.5	13.1	1.64	300.00	
3011.000	9,982	5.45	2.98	2.55	2.16	1.72	1.42	1.16	200.	437.1	834.1	27.8	6.84	300.00	
3504.000	10,089	5.07	2.77	2.50	2.20	1.87	1.57	1.32	200.	600.0	851.8	28.3	9.50	300.00	*
3998.000	9,426	4.05	3.18	2.77	2.41	2.06	1.70	0.90	200.	600.0	486.4	27.4	14.32	105.83	*
4511.000	9,660	6.55	4.33	3.77	2.79	2.19	1.65	1.29	200.	312.7	726.9	19.9	1.84	300.00	
5005.000	9,811	10.40	7.15	5.40	3.96	2.87	2.17	1.72	200.	364.0	122.6	16.1	1.24	300.00	
5508.000	9,994	14.46	7.06	6.04	4.81	3.72	2.88	2.17	200.	70.2	1155.8	11.5	1.79	251.73	
5998.000	9,899	6.28	4.03	3.49	2.90	2.35	1.90	1.55	200.	337.0	1500.0	17.9	3.16	300.00	*
6603.000	9,128	25.31	12.99	7.44	4.50	3.15	2.56	2.05	200.	67.9	25.0	12.5	2.75	233.55	*
7001.000	9,704	7.02	4.26	3.72	2.98	2.37	1.84	1.52	200.	232.4	1500.0	16.7	2.44	300.00	*
7505.000	9,620	6.72	4.85	4.26	3.43	2.70	2.09	1.65	200.	376.0	882.5	16.1	1.29	300.00	
8125.000	9,104	24.99	11.97	6.67	4.37	3.15	2.42	1.94	200.	57.8	30.7	13.0	2.57	300.00	
8589.000	9,875	12.39	6.91	5.39	4.07	3.11	2.40	1.94	200.	109.9	420.4	14.2	1.96	300.00	
9002.000	9,871	8.86	5.44	4.76	3.87	3.08	1.82	1.55	200.	165.2	1000.5	14.8	4.85	109.06	
9501.000	10,753	5.84	4.18	3.89	3.48	1.93	1.58	1.35	200.	600.0	306.9	25.0	11.32	90.13	*
9996.000	10,109	6.00	4.08	3.53	2.92	2.35	1.90	1.54	200.	398.3	1500.0	18.4	1.96	300.00	*
10503.000	9,918	7.29	4.80	4.35	3.41	2.79	2.04	1.83	200.	253.7	1463.8	15.1	1.77	249.59	
10993.000	9,914	4.80	3.56	3.10	2.55	2.09	1.70	1.43	200.	600.0	417.2	26.9	10.97	300.00	*
11508.000	9,060	5.06	2.99	2.59	2.19	1.80	1.48	0.85	200.	500.8	751.7	25.1	5.81	114.61	
11986.000	9,863	6.66	3.94	3.51	2.92	2.43	2.01	1.70	200.	301.7	1500.0	17.4	5.77	300.00	*
12505.000	10,916	7.28	6.35	2.81	1.99	1.54	1.28	1.17	200.	600.0	59.3	35.6	14.56	143.78	*
12996.000	10,053	8.81	4.79	4.07	3.14	2.40	1.86	1.42	200.	140.8	1500.0	15.7	3.21	267.48	*
13515.000	10,447	7.43	5.83	2.86	2.30	1.86	1.47	1.28	200.	600.0	55.3	31.5	14.94	300.00	*
14019.000	9,640	6.36	3.97	3.37	2.73	2.26	1.81	1.50	200.	396.8	626.1	20.9	3.50	300.00	
14544.000	9,672	9.52	5.50	4.68	3.46	2.77	2.29	2.00	200.	188.4	472.1	15.7	3.97	300.00	
15004.000	9,724	17.65	6.80	5.30	4.04	3.02	2.26	1.86	200.	46.6	495.9	14.9	2.85	300.00	
Mean:		9.06	5.23	4.08	3.17	2.46	1.92	1.54	200.	311.5	819.7	19.6	5.38	276.51	
Std. Dev:		5.27	2.36	1.25	0.77	0.54	0.38	0.34	0.	187.7	530.8	6.1	4.32	157.36	
Var Coeff(%):		58.13	45.20	30.75	24.39	21.76	19.92	22.12	0.	60.3	64.8	31.2	80.26	56.91	

SECTION 6

Road	County	Limits	Date	Structure
FM 3478	Walker	1.8 Mi N of FM 980 to 1.1 Mi N	3-95	LSB (60-40) (6%), 11"FB & 2 CST

RATING Performance (97/98) A/A Structural (97/98) A/A PI<4

FM 3478 seems to be a strong pavement with no deflections significantly above the target value of 282 microns (9.6 mils). This pavement section had a very stiff subbase, and moduli values close to 6900 Mpa (1000 ksi). The subgrade on this section is comprised of low PI sandy soils. This pavement had been in service two years, and no surface cracking was observed in either the 1997 or 1998 surveys. The GPR data indicated that the base and subbase were dry. Figure 15 summarizes the deflection profiles for FM 3478, and Table 8 summarizes analysis of the 1997 FWD data for this project.

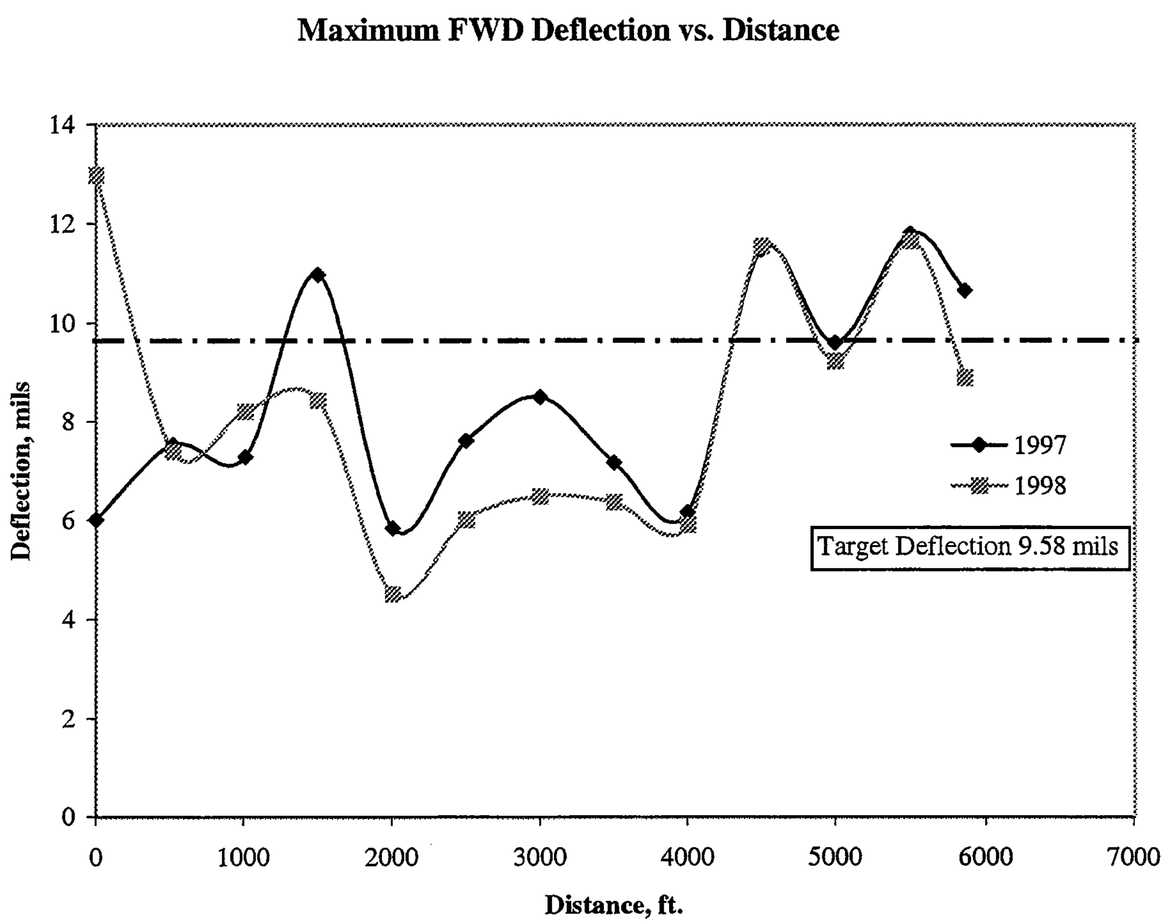


Figure 15. FM 542-2, Target Deflection Analysis.

Table 8. FM 3478, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 0
County: 0
Highway/Road:PxNnnn

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	0.70	199,980	200,020	H1: % = 0.35
Base:	11.00	40,000	500,000	H2: % = 0.35
Subbase:	10.00	40,000	1,500,000	H3: % = 0.30
Subgrade:	278.30	10,000		H4: % = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock	
0.000	11,178	6.02	2.64	2.53	1.96	1.57	1.23	1.03	200.	298.6	1500.0	23.4	6.44	300.00	*
519.000	11,329	7.53	2.87	2.44	2.10	1.69	1.33	1.07	200.	222.6	1500.0	20.4	12.50	300.00	*
1011.000	11,364	7.28	3.26	2.40	2.21	1.80	1.51	1.09	200.	238.3	1500.0	19.9	10.97	300.00	*
1503.000	10,725	10.97	3.96	2.79	2.37	1.87	1.44	1.18	200.	106.9	596.8	23.1	10.81	300.00	
2005.000	10,931	5.85	2.26	1.99	1.59	1.36	0.98	0.88	200.	289.6	1500.0	26.7	11.68	217.10	*
2501.000	10,832	7.61	3.41	2.81	2.39	1.93	1.54	1.28	200.	205.2	1500.0	17.6	7.56	300.00	*
3001.000	10,991	8.50	3.56	3.11	2.68	2.21	1.83	1.56	200.	184.2	1500.0	15.9	9.36	300.00	*
3501.000	10,947	7.16	3.51	2.90	2.37	1.89	1.50	1.23	200.	226.5	1500.0	18.1	4.63	300.00	*
4000.000	11,126	6.16	3.40	3.01	2.52	2.08	1.69	1.42	200.	329.9	1500.0	18.7	2.98	300.00	*
4500.000	10,773	11.47	5.35	3.46	2.98	2.43	1.92	1.59	200.	106.8	1500.0	13.9	7.81	300.00	*
5000.000	10,967	9.59	3.25	2.88	2.47	1.74	1.63	0.83	200.	137.9	694.8	23.2	15.25	284.63	
5500.000	10,959	11.80	4.32	3.44	2.61	2.25	1.73	1.36	200.	109.3	585.2	19.5	11.96	300.00	
5862.000	11,372	10.65	3.59	2.96	2.67	2.16	1.71	1.32	200.	128.2	664.2	22.1	17.12	273.03	
Mean:		8.51	3.49	2.82	2.38	1.92	1.54	1.22	200.	198.8	1233.9	20.2	9.93	300.00	
Std. Dev:		2.15	0.77	0.41	0.36	0.30	0.26	0.24	0.	77.5	416.3	3.5	4.03	46.52	
Var Coeff(%):		25.23	22.08	14.59	14.98	15.58	16.65	19.40	0.	39.0	33.7	17.3	40.56	15.51	

SECTION 7

Road	County	Limits	Date	Structure
FM 1362	Burleson	SH 21 to FM 166	5-95	LSB (80-30) (4%), 7"FB & 2 CST

RATING 0.0 to 1.2 mile Performance (97/98) B/B Structural (97/98) B/B PI<20

RATING 1.2 to 3.0 mile Performance (97/98) A/C Structural (97/98) B/A PI>45

RATING 3.0 to 4.3 mile Performance (97/98) B/C Structural (97/98) B/C PI>25

The average deflection for the pavement section on FM 1362 is 368 microns, which is very close to the target deflection of 361 microns (14.2 mils). Some of the variability in the deflection values was caused by the change in subgrade moduli values. For example, the weak spot at 910 m (2985 ft) from the beginning of the section had a subgrade modulus of 74 Mpa (10.7 ksi) as compared to the average subgrade modulus value of 124 Mpa (17.9 ksi). The USDA soils maps indicated that the soils were very variable along this project. Some of the variability in deflections was caused by variation in base layer thickness, as observed in the GPR data. The base thickness was computed to vary from 127 to 254 mm (5 to 10 in) as compared to the design thickness of 178 mm (7 in).

In 1997, only minor distress was observed in this section, and the modulus value of the base layer looked good. However, deterioration occurred from 1997 to 1998; in 1998, the last 2415 m (1.5 mi) of this project was found to have substantial cracking. This section appeared to be deteriorating rapidly; it should be retested in a few years. Figure 16 summarizes the deflection profiles for FM 1362, and Table 9 summarizes analysis of the 1997 FWD data for this project.

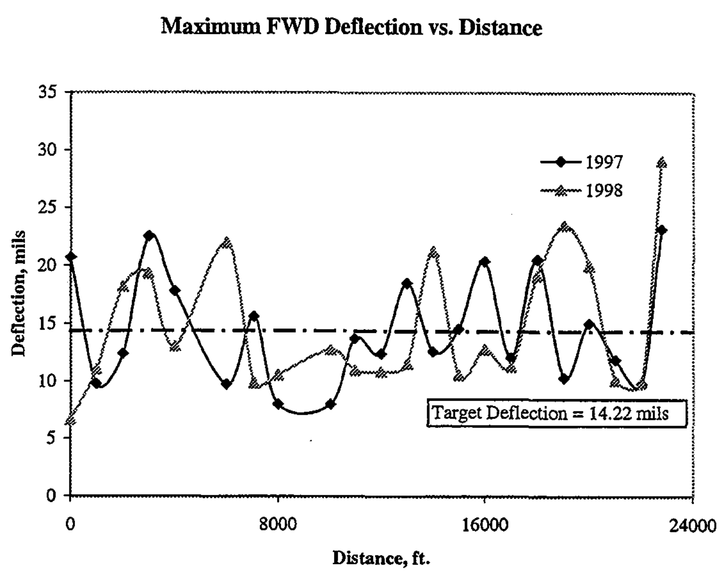


Figure 16. FM 1362, Target Deflection Analysis.

Table 9. FM 1362, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District:	17		Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
County:	26					
Highway/Road:	fm1362	Pavement:	0.70	199,980	200,020	H1: ¥ = 0.35
		Base:	7.00	40,000	250,000	H2: ¥ = 0.35
		Subbase:	10.00	25,000	1,000,000	H3: ¥ = 0.30
		Subgrade:	282.30	17,900		H4: ¥ = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock	
0.000	9,501	20.72	9.76	4.83	3.07	2.24	1.81	1.52	200.	73.2	33.4	18.0	6.20	300.00	
1003.000	10,765	9.76	5.54	4.22	3.34	2.64	2.13	1.79	200.	153.0	867.7	16.7	4.74	300.00	
2010.000	10,061	12.38	7.32	5.36	3.72	2.72	2.04	1.70	200.	164.9	136.1	16.4	0.75	300.00	
2985.000	10,232	22.53	13.87	9.67	6.45	4.37	3.12	2.20	200.	156.5	28.1	10.7	3.12	202.09	
4009.000	10,272	17.79	8.21	5.52	3.92	2.67	1.94	0.98	200.	75.1	102.6	17.1	2.03	230.98	
5987.000	10,471	9.70	5.70	4.76	4.02	3.19	2.51	2.03	200.	167.9	1000.0	13.6	3.18	300.00	*
7070.000	10,590	15.57	8.33	5.40	3.93	2.91	2.22	1.87	200.	107.1	120.9	16.8	2.57	300.00	
7985.000	10,137	7.98	3.93	3.11	2.53	1.97	1.62	1.30	200.	167.0	1000.0	21.1	8.12	300.00	*
10027.000	10,689	7.99	4.36	3.57	2.91	2.26	1.83	1.50	200.	209.3	1000.0	18.9	5.65	300.00	*
10998.000	10,363	13.66	8.31	6.70	5.42	4.24	3.39	2.73	200.	112.2	578.6	10.7	1.82	300.00	
12001.000	10,181	12.39	7.26	5.03	3.54	2.52	1.88	1.52	200.	171.1	116.6	17.7	0.14	300.00	
12999.000	10,586	18.47	8.95	5.77	3.86	2.72	1.92	1.57	200.	87.0	72.9	17.3	1.36	207.88	
14008.000	11,142	12.56	5.89	5.22	4.11	3.09	2.51	1.87	200.	100.8	1000.0	14.9	4.36	300.00	*
15000.000	10,534	14.54	5.57	4.84	3.59	2.75	2.14	1.72	200.	61.9	1000.0	16.2	4.64	300.00	*
16002.000	10,399	20.35	8.09	3.98	2.57	1.84	1.50	1.09	200.	75.0	29.3	25.0	7.66	300.00	
17006.000	10,490	12.07	5.74	4.30	3.34	2.51	1.94	1.57	200.	93.6	577.8	18.7	3.43	300.00	
18028.000	10,633	20.50	7.77	5.16	3.39	2.65	2.00	1.59	200.	50.4	124.5	18.8	2.71	300.00	
19035.000	10,649	10.28	6.07	4.65	3.53	2.59	2.02	1.65	200.	175.0	367.5	17.8	1.97	300.00	
19992.000	10,868	15.01	5.20	4.61	3.29	2.35	1.62	1.32	200.	58.8	1000.0	18.3	6.28	176.70	*
21001.000	10,641	11.86	3.32	2.63	2.17	1.58	1.29	1.04	200.	69.4	1000.0	27.9	10.01	300.00	*
22001.000	11,051	9.77	4.18	3.14	2.50	1.90	1.20	0.99	200.	112.0	1000.0	24.8	3.97	125.00	*
22774.000	9,311	23.14	10.06	4.99	3.25	2.42	1.83	1.51	200.	56.0	31.0	17.1	5.45	300.00	
Mean:		14.50	6.97	4.88	3.57	2.64	2.02	1.59	200.	113.5	508.5	17.9	4.10	300.00	
Std. Dev:		4.75	2.45	1.43	0.95	0.67	0.52	0.41	0.	48.3	433.7	4.1	2.52	118.81	
Var Coeff(%):		32.73	35.06	29.25	26.51	25.30	25.53	25.84	0.	42.5	85.3	22.8	61.38	39.60	

SECTION 8

Road	County	Limits	Date	Structure
FM 246	Freestone	FM 27 to 3.3 Mi E	5-95	LSB (40-60) (6%), 9"FB & 2 CST

RATING Performance (97/98) C/C Structural (97/98) B/Failed, not tested PI>35

The average deflection of 391 microns (15.4 mils) observed on FM 246 is less than the target deflection of 399 microns (15.7 mils). High deflections in the first 305 m (1000 ft) and throughout the section were attributed to low values for the subgrade moduli. The moduli of the subbase is extremely low in the first 305 m (1000 ft); this could be due to "start-up" problems.

Visually, this section is performing very poorly—one of the worst in the study. Between 1997 and 1998, approximately 30 percent of the project was repaired with full depth patches. In the 1998 survey, it was noted that some of the repaired areas were beginning to fail. Because of the level of maintenance, no deflection analysis was performed in 1998. This section is a failure. Figure 17 summarizes the deflection profiles for FM 246, and Table 10 summarizes analysis of the 1997 FWD data for this project.

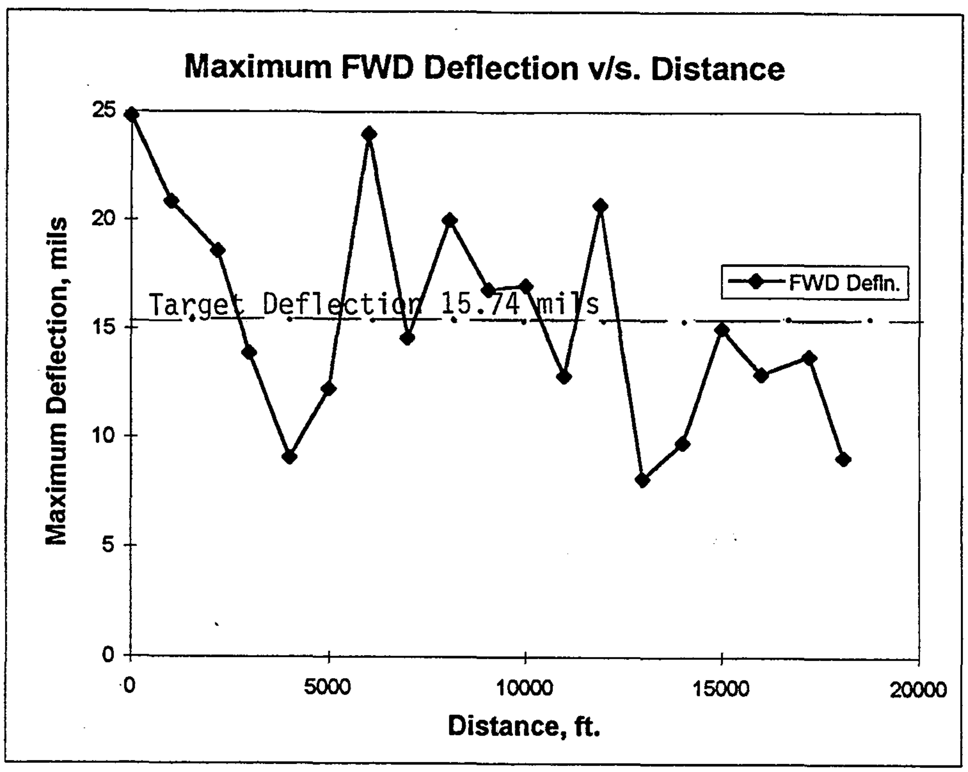


Figure 17. FM 246, Target Deflection Analysis.

Table 10. FM 246, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 0
County: 246
Highway/Road:south

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	0.70	199,980	200,020	H1: ¥ = 0.35
Base:	9.00	25,000	500,000	H2: ¥ = 0.35
Subbase:	10.00	10,000	1,500,000	H3: ¥ = 0.30
Subgrade:	276.80	10,000		H4: ¥ = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock	
0.000	9,589	24.80	15.93	11.10	7.71	5.24	3.53	2.48	200.	132.9	13.6	9.0	2.81	177.72	
1009.000	9,914	20.83	14.45	10.74	7.41	5.14	3.59	2.80	200.	221.4	14.8	9.6	2.26	213.78	
2192.000	9,752	18.57	7.74	5.20	3.68	2.75	2.08	1.71	200.	60.0	108.2	16.2	2.39	300.00	
3003.000	10,427	13.88	7.19	5.21	4.11	3.07	2.53	1.93	200.	102.2	287.7	14.8	3.40	300.00	
4004.000	9,918	9.07	5.96	5.07	4.09	3.21	2.50	2.02	200.	231.2	508.0	13.5	1.02	300.00	
5001.000	10,177	12.24	5.17	4.46	3.60	2.83	2.21	1.78	200.	90.1	1500.0	13.2	3.46	300.00	*
6005.000	9,366	23.97	11.55	8.38	6.03	4.19	2.92	2.27	200.	54.7	61.4	10.5	3.85	200.69	
7010.000	9,986	14.58	6.52	5.63	4.65	3.65	2.81	2.23	200.	72.4	1500.0	10.4	0.81	300.00	*
8068.000	9,386	19.98	11.84	8.08	5.83	4.05	2.93	2.35	200.	110.5	31.8	10.8	2.20	255.72	
9045.000	9,259	16.79	10.74	8.74	6.66	4.73	3.50	2.56	200.	138.8	78.3	9.1	3.44	227.53	
10000.000	10,228	16.99	8.43	6.27	4.98	3.77	2.90	2.32	200.	74.5	302.2	12.4	2.13	300.00	
10999.000	9,700	12.85	6.35	5.13	3.91	2.98	2.24	1.60	200.	91.5	561.0	14.0	1.29	191.94	
11921.000	9,756	20.70	8.61	7.21	5.87	4.52	3.39	2.72	200.	47.0	594.2	10.3	1.45	300.00	
13000.000	10,228	8.12	5.26	4.32	3.43	2.81	2.11	1.71	200.	253.8	603.0	16.0	0.95	300.00	
14010.000	10,018	9.78	5.44	4.86	4.01	3.18	2.46	2.02	200.	148.3	1500.0	11.3	1.20	300.00	*
15001.000	9,799	15.02	5.03	4.28	3.66	2.96	2.26	1.80	200.	60.1	1500.0	13.2	6.84	300.00	*
16020.000	9,700	12.94	5.80	4.95	3.98	2.85	2.13	1.90	200.	81.4	971.3	13.6	1.61	296.91	
17215.000	9,827	13.75	5.28	4.68	3.82	2.92	2.20	1.77	200.	71.1	1500.0	12.7	2.60	300.00	*
18064.000	9,680	9.11	4.20	3.75	3.18	2.62	2.07	1.79	200.	136.3	1500.0	13.6	6.00	300.00	*
Mean:		15.47	7.97	6.21	4.77	3.55	2.65	2.09	200.	114.6	691.3	12.3	2.62	296.51	
Std. Dev:		4.99	3.39	2.20	1.39	0.85	0.54	0.37	0.	61.8	616.8	2.2	1.64	82.29	
Var Coeff(%):		32.22	42.56	35.39	29.25	23.98	20.24	17.52	0.	53.9	89.2	17.9	62.55	27.75	

SECTION 9

Road	County	Limits	Date	Structure
FM 977	Leon	SH 75 to 4.5 Mi E	7-95	LSB (60-40) (3%), 11" FB & 2 CST

RATING 0.0 to 1.4 mile Performance (97/98) A/A Structural (97/98) A/A PI<20

RATING 1.4 to 4.5 mile Performance (97/98) B/C Structural (97/98) A/A PI>35

Overall, this pavement section on FM 977 was computed to be very stiff, with average deflection less than the target values. The average deflections decreased from 1997 to 1998, indicating that the layers are continuing to get stiffer. In 1997, some longitudinal cracks were observed from 3355 to 5490 m (11,000 to 18,000 ft). The subbase moduli in the cracked areas were very high, greater than 6900 Mpa (1000 ksi). In 1998, the condition of the last 4.8 km (3 mi) was found to have deteriorated rapidly; substantial maintenance had already been applied. This section is a failure. Figure 18 summarizes the deflection profiles for FM 977, and Table 11 summarizes analysis of the 1997 FWD data for this project.

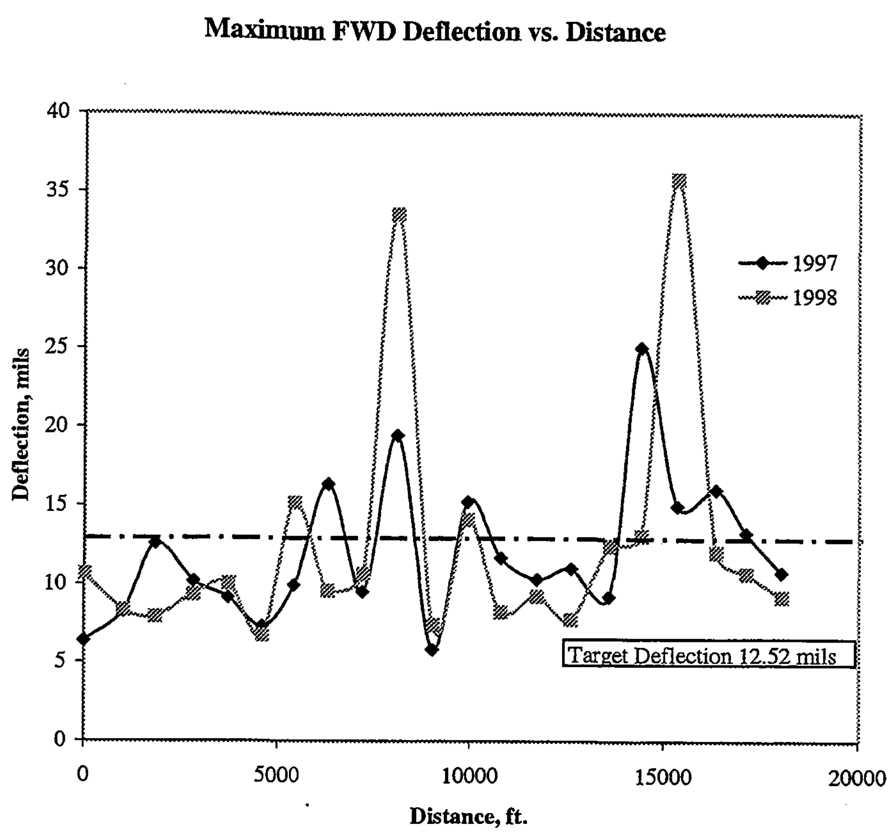


Figure 18. FM 977, Target Deflection Analysis.

Table 11. FM 977, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

			MODULI RANGE(psi)		
District: 0		Thickness(in)	Minimum	Maximum	Poisson Ratio Values
County: 977	Pavement:	0.70	199,980	200,020	H1: % = 0.35
Highway/Road:9nnS	Base:	11.00	25,000	500,000	H2: % = 0.35
	Subbase:	10.00	25,000	1,000,000	H3: % = 0.30
	Subgrade:	268.70	10,000		H4: % = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock	
0.100	9,231	8.18	4.79	3.69	2.96	2.25	1.78	1.41	200.	212.2	284.1	16.7	2.14	300.00	
1207.000	10,053	10.62	3.51	2.78	2.04	1.81	1.31	1.26	200.	99.7	716.0	22.8	12.34	300.00	
2402.000	10,407	5.83	1.98	1.42	1.13	0.90	0.71	0.61	200.	327.2	166.0	53.4	19.79	24.00	
3608.000	9,954	7.65	3.93	2.98	2.28	1.73	1.29	1.03	200.	188.5	573.2	19.5	3.34	284.54	
4806.000	9,779	8.95	4.69	3.22	2.39	1.88	1.35	1.13	200.	169.0	213.0	21.0	3.97	218.67	
6008.000	9,327	11.94	3.68	2.76	2.23	1.70	1.29	1.02	200.	71.0	952.3	20.2	9.05	300.00	
7206.000	9,839	17.67	11.50	8.17	5.87	4.19	2.97	2.24	200.	147.7	25.0	11.2	1.46	220.46	*
8406.000	10,570	14.05	4.13	3.23	2.52	2.05	1.59	1.39	200.	66.9	1000.0	19.5	11.16	300.00	*
9604.000	10,117	8.40	3.40	2.65	2.20	2.04	1.43	1.43	200.	163.5	1000.0	17.9	13.09	300.00	*
10800.000	10,069	12.41	4.00	3.58	2.93	2.59	2.09	1.79	200.	93.0	1000.0	15.2	14.09	300.00	*
11999.000	9,887	10.37	2.74	2.36	2.31	2.02	1.76	1.54	200.	92.6	806.7	24.3	24.31	300.00	
13201.000	10,641	15.61	8.53	6.99	5.92	4.56	3.45	2.58	200.	101.1	339.2	9.8	1.16	236.99	
14398.000	9,819	11.37	3.47	3.17	2.94	2.54	2.18	1.87	200.	104.6	1000.0	15.6	18.29	300.00	*
15616.000	8,635	15.94	9.13	6.67	5.03	3.64	2.61	2.09	200.	98.0	59.9	10.7	1.89	231.57	
16823.000	9,640	12.77	2.47	2.07	1.98	1.74	1.46	1.28	200.	67.2	1000.0	25.4	23.40	300.00	*
18006.000	9,990	14.87	3.49	2.31	2.32	2.04	1.71	1.46	200.	64.3	267.6	26.8	21.75	300.00	
19367.000	9,521	11.61	3.73	3.04	2.57	2.04	1.59	1.37	200.	85.5	1000.0	17.3	11.81	300.00	*
20417.000	10,129	12.56	5.00	3.82	2.98	2.27	1.87	1.63	200.	89.1	731.4	16.2	5.83	300.00	
21606.000	9,334	12.18	4.09	2.67	2.13	1.55	1.22	0.96	200.	79.3	244.8	24.5	7.05	300.00	
22911.000	9,891	13.79	4.02	3.00	2.67	2.43	1.93	1.82	200.	70.2	229.1	22.9	16.06	300.00	
24007.000	9,589	19.72	13.94	9.00	5.94	4.08	2.97	2.37	200.	106.9	25.0	10.1	7.24	280.82	*
Mean:		12.21	5.06	3.79	3.02	2.38	1.84	1.54	200.	118.9	554.0	20.0	10.92	290.47	
Std. Dev:		3.43	3.08	2.06	1.40	0.95	0.68	0.50	0.	63.9	383.9	9.2	7.54	450.23	
Var Coeff(%):		28.07	60.91	54.47	46.50	39.74	36.89	32.38	0.	53.7	69.3	45.9	69.06	155.00	

SECTION 10

Road	County	Limits	Date	Structure
FM 27	Freestone	Curb & Cutter in Wortham to FM 1366	9-95	LSB (60-40) (5%), 12"FB & 2 CST

RATING Performance (97/98) B/B Structural (97/98) A/A PI>35

Most of measured deflections on FM 27 are below the target value of 472 microns (18.6 mils); the average deflection measured was 389 microns (15.3 mils). In 1997, little or no surface distress was observed. Some minor level-up work and some cracking in a short section was observed on this pavement. In 1998, the level of longitudinal cracking had increased, and one short section had failed; however, the overall performance was judged as good. GPR did not indicate any problems. The thickness of the flexible base as measured by the GPR was approximately 305 mm (12 in) with little variability, which is the same as the design thickness. This indicates good construction control. The flexible base was dry with no apparent weak or wet spots. The average flexbase modulus was 518 Mpa (75 ksi), and the moduli for the stabilized subbase was 4575 Mpa (663 ksi). Figure 19 summarizes the deflection profiles for FM 27, and Table 12 summarizes analysis of the 1997 FWD data for this project.

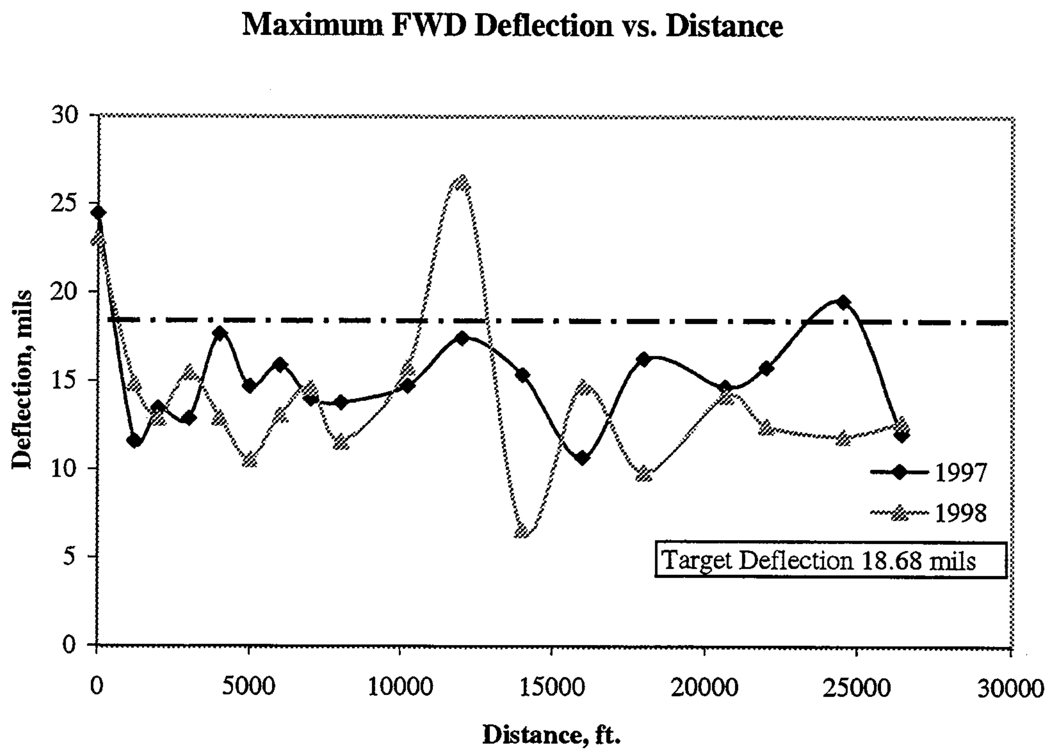


Figure 19. FM 27, Target Deflection Analysis.

Table 12. FM 27, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 0
County: 27
Highway/Road: east b

	Thickness(in)	MODULI RANGE(psi) Minimum	MODULI RANGE(psi) Maximum	Poisson Ratio Values
Pavement:	0.70	199,980	200,020	H1: % = 0.35
Base:	12.00	25,000	250,000	H2: % = 0.35
Subbase:	10.00	25,000	1,000,000	H3: % = 0.30
Subgrade:	277.30	15,000		H4: % = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock	
0.000	9,021	24.47	10.67	4.73	2.94	2.24	1.76	1.56	200.	41.3	25.0	17.3	7.72	153.93	*
1200.000	9,589	11.60	5.28	4.07	3.35	2.65	2.04	1.64	200.	98.7	1000.0	12.1	1.57	300.00	*
1982.000	9,346	13.48	4.97	3.61	3.07	2.43	1.94	1.67	200.	73.6	1000.0	13.5	6.07	300.00	*
3004.000	9,577	12.90	5.01	3.61	3.11	2.48	1.94	1.62	200.	81.7	1000.0	13.5	5.79	300.00	*
4007.000	9,593	17.66	7.46	4.84	4.19	3.39	2.70	2.22	200.	57.0	624.1	11.5	4.91	300.00	
5004.000	9,493	14.72	6.69	4.62	3.84	3.21	2.48	2.09	200.	72.6	756.9	11.4	3.92	300.00	
6001.000	9,104	15.91	6.88	4.87	4.26	3.40	2.70	2.24	200.	62.1	860.1	10.4	2.46	300.00	
7005.000	9,406	14.02	4.62	2.29	2.37	2.07	1.66	1.44	200.	65.9	225.2	22.9	13.24	300.00	
8006.000	10,185	13.78	4.44	2.93	2.75	2.34	1.83	1.52	200.	77.0	223.1	22.3	14.42	300.00	
10215.000	9,577	14.74	5.65	3.42	3.09	2.77	2.21	1.83	200.	66.9	1000.0	13.3	11.03	300.00	*
12014.000	9,414	17.44	6.65	4.80	3.89	3.11	2.47	2.05	200.	54.1	885.0	11.5	3.01	300.00	
14002.000	9,509	15.35	5.56	3.63	3.39	2.81	2.32	1.96	200.	63.2	1000.0	12.9	8.52	300.00	*
16000.000	9,406	10.71	4.73	3.17	2.69	2.15	1.71	1.41	200.	102.3	1000.0	14.2	6.35	300.00	*
18010.000	9,446	16.26	8.21	6.15	5.08	4.00	3.15	2.61	200.	77.3	254.7	10.3	1.31	300.00	
20665.000	9,382	14.66	8.69	6.05	4.51	3.43	2.75	2.23	200.	124.2	52.7	12.1	2.50	300.00	
22024.000	9,259	15.82	6.21	4.11	3.86	3.13	2.58	2.02	200.	61.3	1000.0	11.3	5.58	300.00	*
24519.000	9,303	19.54	9.50	5.31	3.86	2.73	2.10	1.58	200.	67.3	31.0	15.3	3.25	300.00	
26428.000	9,767	12.05	5.90	4.36	3.70	3.02	2.44	2.08	200.	99.8	1000.0	10.9	2.63	300.00	*
Mean:		15.28	6.51	4.25	3.55	2.85	2.27	1.88	200.	74.8	663.2	13.7	5.79	300.00	
Std. Dev:		3.19	1.79	1.03	0.70	0.52	0.42	0.34	0.	20.3	401.3	3.7	3.89	80.86	
Var Coeff(%):		20.88	27.45	24.24	19.83	18.36	18.63	18.04	0.	27.1	60.5	27.0	67.14	26.95	

SECTION 11

Road	County	Limits	Date	Structure
FM 3411	Walker	SH 19 to FM 2929	9-95	LSB (50-50) (6%), 10"FB & 2 CST

RATING Performance (97/98) A/A Structural (97/98) A/A PI<20

FM 3411 was judged as a good pavement section, with no apparent visual or structural problems. The 1997 average measured deflection of 308 microns (12.1 mils) is very close to the target values of 300 microns (11.8 mils). Only a few areas had deflections above the target value. In 1998, the deflections rose, but this was attributed to seasonal variations in deflection rather than a reduction in the base layer modulus. GPR indicated that the base was uniform in thickness and dry. No major structural distresses were observed on this pavement.

However, a concern was the variability of the backcalculated E-values for the stabilized subbase. The average value at 1904 Mpa (276 ksi) was judged as good, but several sections were computed to be less than 621 Mpa (90 ksi). This section should be retested in several years to determine if the reductions in layer moduli continue and if they correlate to changes in pavement performance. Figure 20 summarizes the deflection profiles for FM 3411, and Table 13 summarizes analysis of the 1997 FWD data for this project.

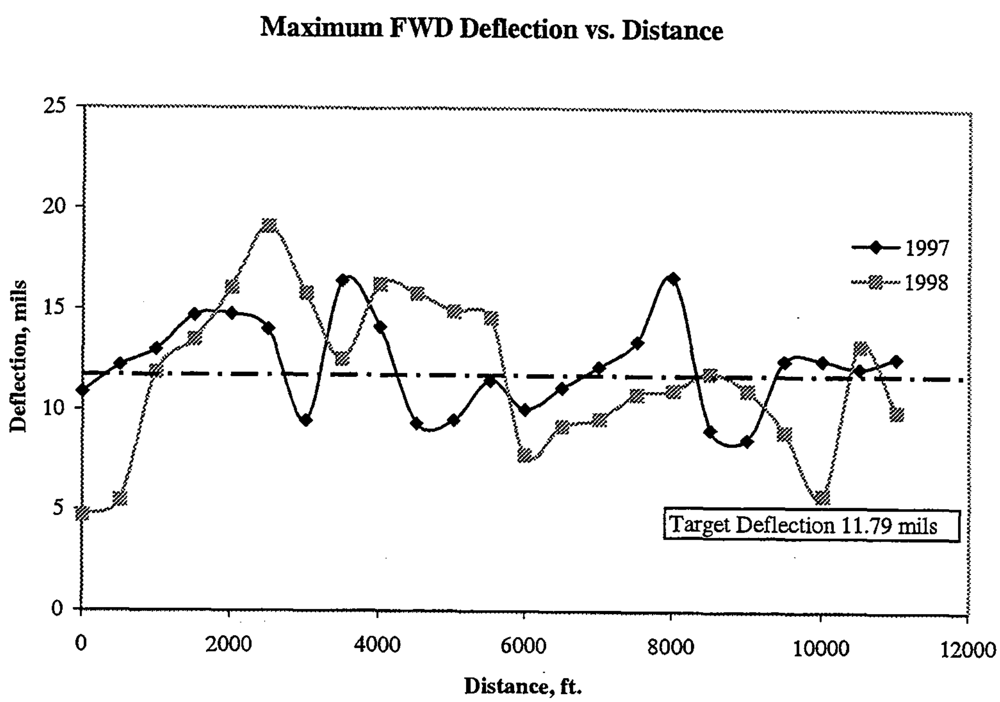


Figure 20. FM 3411, Target Deflection Analysis.

Table 13. FM 3411, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 0		Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
County: 0					
Highway/Road:PxNnnn	Pavement:	0.70	199,980	200,020	H1: ¥ = 0.35
	Base:	10.00	25,000	500,000	H2: ¥ = 0.35
	Subbase:	10.00	25,000	1,500,000	H3: ¥ = 0.30
	Subgrade:	241.10	10,000		H4: ¥ = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
0.000	10,574	10.85	4.76	3.13	2.58	1.75	1.30	1.23	200.	116.1	333.6	23.9	3.29	209.34
504.000	10,224	12.19	6.32	4.72	3.84	2.91	2.34	1.54	200.	106.7	653.3	13.8	3.28	145.53
1000.000	10,308	12.98	6.37	4.13	2.70	1.95	1.43	1.11	200.	123.6	76.8	21.9	1.28	263.48
1507.000	9,970	14.69	7.85	5.47	4.07	3.06	2.23	1.76	200.	107.1	112.6	14.5	1.75	251.56
2011.000	9,795	14.77	8.36	5.76	4.24	3.13	2.33	1.93	200.	120.7	75.8	14.0	1.19	300.00
2502.000	10,010	14.00	6.15	4.37	3.43	2.67	2.14	1.72	200.	80.3	417.9	16.0	4.65	300.00
3011.000	9,624	9.44	5.39	2.97	2.18	1.62	1.21	0.92	200.	194.8	73.0	25.5	5.55	300.00
3506.000	10,042	16.41	9.47	6.84	5.09	3.82	2.90	2.32	200.	111.2	86.1	11.7	1.00	300.00
4013.000	9,970	14.11	8.90	6.22	4.59	3.45	2.66	2.20	200.	166.0	63.6	13.3	1.21	300.00
4507.000	10,169	9.31	5.48	3.07	2.09	1.52	1.16	0.91	200.	235.8	55.7	28.3	4.84	300.00
5018.000	9,744	9.47	5.72	4.51	3.39	2.53	1.92	1.52	200.	206.2	197.5	16.5	0.89	300.00
5531.000	9,740	11.45	7.38	5.02	3.59	2.61	1.98	1.58	200.	224.2	55.0	16.8	1.17	300.00
5998.000	9,760	10.01	5.57	4.13	3.10	2.23	1.60	1.17	200.	165.6	178.2	17.9	0.48	211.02
6501.000	9,358	11.11	3.50	2.41	1.86	1.38	1.03	0.78	200.	82.2	282.3	28.2	7.72	248.01
7006.000	10,014	12.15	5.55	3.66	2.50	1.67	1.18	0.89	200.	116.3	96.5	23.9	2.25	193.66
7521.000	10,343	13.44	6.63	4.87	3.83	2.89	2.21	1.75	200.	99.2	340.3	15.1	2.40	300.00
8004.000	9,839	16.61	7.18	4.71	3.44	2.52	1.91	1.53	200.	71.9	110.1	17.3	2.40	300.00
8501.000	10,479	9.04	5.70	4.05	2.87	1.98	1.39	1.03	200.	333.8	65.7	23.4	1.30	193.66
9001.000	10,113	8.54	5.14	3.87	3.00	2.22	1.63	1.23	200.	231.9	226.0	19.3	0.71	247.45
9501.000	9,779	12.48	6.51	4.87	4.16	3.13	1.51	1.87	200.	106.3	321.7	14.6	9.16	66.23
10005.000	10,153	12.51	5.65	4.02	3.23	2.37	1.81	1.34	200.	94.9	439.7	17.4	2.95	220.91
10515.000	10,081	12.08	4.38	3.35	2.66	2.01	1.51	1.15	200.	84.1	691.8	20.3	6.14	258.66
11010.000	10,085	12.59	5.40	4.08	3.33	2.51	1.90	1.45	200.	84.1	1413.5	13.8	3.24	263.22
Mean:		12.18	6.23	4.36	3.29	2.43	1.79	1.43	200.	141.9	276.8	18.6	2.99	261.82
Std. Dev:		2.28	1.43	1.07	0.82	0.65	0.50	0.42	0.	66.9	310.0	4.9	2.35	152.85
Var Coeff(%):		18.73	23.01	24.51	24.92	26.64	28.08	29.48	0.	47.1	100.0	26.4	78.48	58.38

SECTION 12

Road	County	Limits	Date	Structure
FM 111	Burleson	FM 60 to 3.4 Mi E	12-95	LSB (50-50) (3%), & 2 CST

RATING 0.0 to 2.8 mile Performance (97/98) A/B Structural (97/98) C/C PI>35

RATING 2.8 to 3.4 mile Performance (97/98) A/A Structural (97/98) C/C PI>35

The FWD analysis on FM 111 indicates highly variable results. The first half of the section had reasonable deflections and good backcalculated moduli values. The second half was found to be considerably weaker. At location 3965 m (12,999 ft), the measured deflection of 1092 microns (43 mils) was more than twice the target value of 483 microns (19 mils). This section was stabilized with only 3 percent lime. The stabilizer content may have been insufficient. The section from 3660 to 4880 m (12,000 to 16,000 ft) had a deflection and modulus profile similar to that anticipated for a flexible base. The high deflection values indicate little benefit from the lime. It is anticipated that this section will deteriorate rapidly under heavy load.

This is an interesting section as it is built on poor subgrade comprised of highly plastic clays; however, there is currently little or no distress. The 1998 visual survey noted the beginning of load associated distress. If this section performs well for several years, it could have a major impact on the criteria used to design these recycled bases to be used on high PI soils. This low strength project is performing better than all of the other projects where high moduli layers were placed over high PI soils. Figure 21 summarizes the deflection profiles for FM 111, and Table 14 summarizes analysis of the 1997 FWD data for this project.

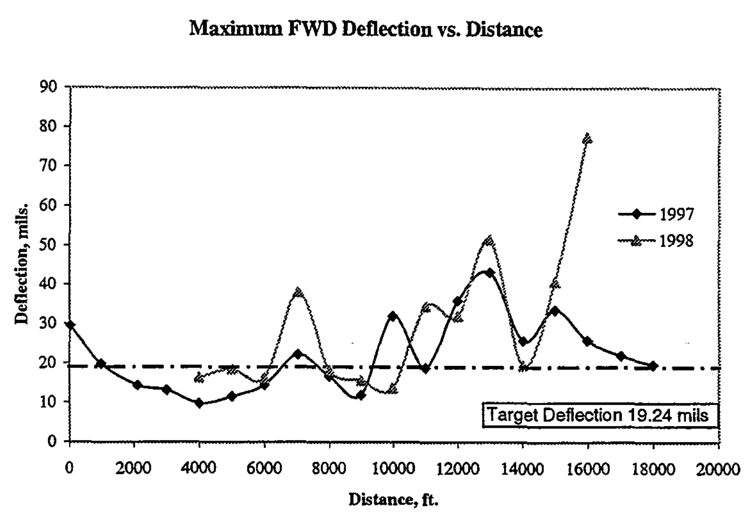


Figure 21. FM 111, Target Deflection Analysis.

Table 14. FM 111, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

			MODULI RANGE(psi)		
District: 17		Thickness(in)	Minimum	Maximum	Poisson Ratio Values
County: 26	Pavement:	0.70	199,980	200,020	H1: ¥ = 0.35
Highway/Road: fm111	Base:	10.00	25,000	1,000,000	H2: ¥ = 0.30
	Subbase:	0.00	0	0	H3: ¥ = 0.30
	Subgrade:	204.10	10,000		H4: ¥ = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
31.000	9,708	29.42	12.32	5.28	3.21	2.64	2.18	1.87	200.	42.8	0.0	16.6	13.49	116.88
1003.000	10,780	19.79	13.14	8.19	5.51	3.92	3.01	2.50	200.	182.6	0.0	13.0	3.60	300.00
2120.000	10,566	14.30	9.98	5.96	3.90	2.74	2.13	1.83	200.	251.6	0.0	17.5	4.55	300.00
2998.000	10,204	13.16	9.86	6.87	4.71	3.22	2.45	1.98	200.	384.8	0.0	14.7	1.60	247.80
3990.000	10,828	9.81	7.50	5.32	3.85	2.89	2.15	1.80	200.	733.1	0.0	18.1	3.34	294.79
5000.000	10,792	11.48	8.04	5.35	3.61	2.72	2.13	1.79	200.	441.0	0.0	19.3	5.50	300.00
6000.000	9,763	14.40	9.76	6.21	4.20	3.07	2.38	2.02	200.	259.0	0.0	15.4	4.66	300.00
7033.000	10,526	22.09	11.14	7.02	4.79	3.54	2.76	2.29	200.	119.8	0.0	14.6	9.85	300.00
7997.000	10,753	16.50	12.78	9.49	6.56	4.65	3.40	2.71	200.	382.4	0.0	10.9	0.84	285.57
9000.000	10,832	11.85	9.63	6.73	4.94	3.48	2.70	2.09	200.	604.7	0.0	14.4	2.38	300.00
9992.000	10,133	31.88	11.43	6.65	4.31	3.11	2.44	2.00	200.	44.2	0.0	15.2	11.90	300.00
11001.000	10,359	18.74	10.91	6.49	3.88	2.70	2.10	1.81	200.	135.0	0.0	16.3	3.38	199.20
12002.000	9,243	35.85	20.21	10.33	5.56	3.80	2.83	2.34	200.	42.6	0.0	9.7	6.36	110.59
12999.000	9,823	43.17	17.91	9.07	5.11	3.04	1.53	1.41	200.	27.5	0.0	12.2	10.19	96.54
14016.000	10,073	25.64	14.52	7.76	4.65	3.12	2.34	1.91	200.	76.8	0.0	13.4	4.02	210.51
15000.000	8,941	33.50	22.33	13.67	7.85	4.98	3.14	2.37	200.	59.6	0.0	7.5	9.51	172.53
15997.000	9,962	25.76	16.75	6.92	3.17	1.85	1.51	1.32	200.	58.5	0.0	15.6	18.21	69.47
17030.000	10,030	21.90	14.63	8.37	4.87	3.03	2.15	1.72	200.	106.5	0.0	13.1	6.74	142.14
18005.000	10,562	19.57	12.91	7.90	5.11	3.48	2.49	2.00	200.	159.5	0.0	13.7	1.40	239.77
Mean:		22.04	12.93	7.56	4.73	3.26	2.41	1.99	200.	216.4	0.0	14.3	6.40	214.79
Std. Dev:		9.31	4.00	2.05	1.14	0.72	0.49	0.35	0.	204.7	0.0	2.9	4.66	134.25
Var Coeff(%):		42.25	30.96	27.11	24.12	22.10	20.26	17.43	0.	94.6	0.0	20.3	72.79	62.50

SECTION 13

Road	County	Limits	Date	Structure
FM 1124	Freestone	FM 488 to 1.8 Mi E	12-95	LSB (50-50) (4%), 5"FB & 2 CST

RATING 0.0 to 1.5 mile Performance (97/98) A/A Structural (97/98) B/A PI<25

RATING 1.5 to 1.8 mile Performance (97/98) B/C Structural (97/98) C/C PI>35

The problem with the pavement section on FM 1124 is in the last 0.5 km (0.3 mi). The ending deflections are substantially higher than the target values. The higher deflections were primarily attributed to a weaker subgrade. For example, from 2.25 to 2.90 km (1.4 to 1.8 mi), the subgrade was 30-40 percent weaker than the rest of the section. The remainder of this section appeared to be good, with deflections below the target value of 272 microns (10.7 mils).

In the 1997 condition survey, only a minor amount of longitudinal cracking was noted in the last 0.5 km (0.3 mil); substantially more cracking of a more severe nature was recorded in 1998. Figure 22 summarizes the deflection profiles for FM 1124, and Table 15 summarizes analysis of the 1997 FWD data for this project.

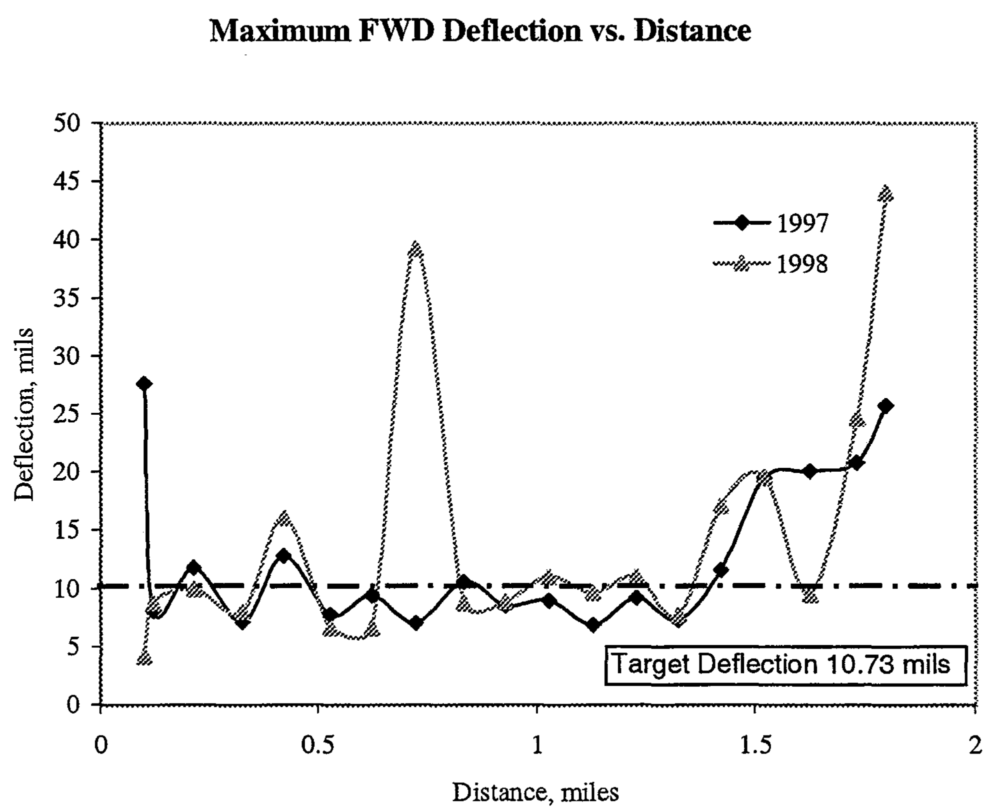


Figure 22. FM 1124, Target Deflection Analysis.

Table 15. FM 1124, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

		Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
District: 0					
County: 0					
Highway/Road:PxNnnn	Pavement:	0.70	199,980	200,020	H1: % = 0.35
	Base:	5.00	40,000	600,000	H2: % = 0.35
	Subbase:	10.00	25,000	1,000,000	H3: % = 0.30
	Subgrade:	238.40	10,000		H4: % = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock	
0.100	8,369	27.61	14.47	7.06	4.11	2.76	2.04	1.65	200.	55.7	25.0	12.1	5.85	166.76	*
0.121	10,721	8.11	5.13	4.07	3.17	2.35	1.89	1.48	200.	241.2	748.8	20.3	3.47	300.00	
0.213	10,864	11.83	6.72	5.25	3.90	2.80	2.28	1.74	200.	106.7	599.2	17.1	2.17	300.00	
0.326	10,927	7.16	5.27	4.71	3.87	3.11	2.41	1.89	200.	536.9	1000.0	17.0	2.02	300.00	*
0.422	11,027	12.83	8.94	5.72	3.52	2.28	1.65	1.22	200.	600.0	54.0	20.9	3.76	164.92	*
0.529	11,245	7.77	4.96	4.46	3.39	2.75	2.06	1.76	200.	308.5	1000.0	19.5	2.74	300.00	*
0.624	10,697	9.42	5.96	5.00	3.61	2.86	2.19	1.75	200.	189.6	729.7	17.0	2.84	300.00	
0.723	10,792	7.03	5.40	4.55	3.49	2.74	2.13	1.70	200.	600.0	656.1	19.0	1.66	300.00	*
0.832	10,216	10.59	8.09	6.57	4.96	3.83	2.89	2.23	200.	600.0	233.2	13.0	1.40	300.00	*
0.928	10,606	8.52	5.95	5.07	3.91	3.11	2.31	2.04	200.	255.9	881.8	16.1	0.77	284.78	
1.029	10,546	8.99	6.46	5.51	4.66	3.52	2.86	2.26	200.	266.6	1000.0	13.7	1.56	300.00	*
1.130	11,102	6.89	4.23	3.21	2.29	1.59	1.26	1.03	200.	299.7	531.2	30.0	2.23	259.46	
1.229	11,023	9.20	5.47	4.20	3.09	2.22	1.60	1.26	200.	171.9	566.7	22.0	0.54	226.32	
1.325	11,090	7.31	4.39	3.65	2.73	2.07	1.53	1.24	200.	252.6	941.6	23.6	2.93	268.42	
1.423	10,332	11.56	7.61	6.16	4.65	3.39	2.47	1.85	200.	154.7	441.0	14.1	1.53	246.77	
1.523	10,387	19.46	12.93	9.07	6.03	4.05	3.02	2.35	200.	352.9	46.6	11.7	2.35	224.27	
1.627	10,765	20.06	13.02	8.63	5.72	3.83	2.59	1.80	200.	360.7	38.5	13.0	2.99	173.55	
1.733	10,904	20.80	11.46	8.53	6.06	4.28	2.98	2.19	200.	79.7	128.8	12.3	2.76	196.92	
1.798	9,684	25.72	18.51	5.80	3.20	2.16	1.71	1.41	200.	84.4	25.0	15.3	16.02	53.90	*
Mean:		12.68	8.16	5.64	4.02	2.93	2.20	1.73	200.	290.4	507.7	17.2	3.14	254.14	
Std. Dev:		6.62	4.03	1.68	1.08	0.74	0.52	0.38	0.	179.7	374.5	4.7	3.34	185.69	
Var Coeff(%):		52.23	49.39	29.83	26.78	25.21	23.58	22.20	0.	61.9	73.8	27.4	106.46	73.07	

SECTION 14

Road	County	Limits	Date	Structure
FM 244	Grimes	FM 3090 to SH 30	5-95	LSB (60-40) (4%), 12"FB & 2 CST

RATING 0.0 to 0.6 mile Performance (97/98) A/A Structural (97/98) B/A PI<10

RATING 1.2 to 2.3 mile Performance (97/98) A/A Structural (97/98) A/A PI<15

In general, the pavement section on FM 244 was found to be stiff, with high average base and subbase moduli values. A section of high deflection was detected from 1495 to 2288 m (4900 to 7500 ft). This middle section was a different structure altogether and was not part of this study. Outside of this section, the deflection data were generally below the target value. There was one location around 2160 m (7200 ft) where the 1998 deflections were substantially lower than the 1997 data; however, this is probably a referencing problem. In the last 1.6 km (mi) of the project, the stabilized layer modulus was computed to be high in both 1997 and 1998.

This section is performing very well; no cracks were noted in either the 1997 or 1998 surveys. Table 23 summarizes the deflection profiles for FM 244, and Table 16 summarizes analysis of the 1997 FWD data for this project.

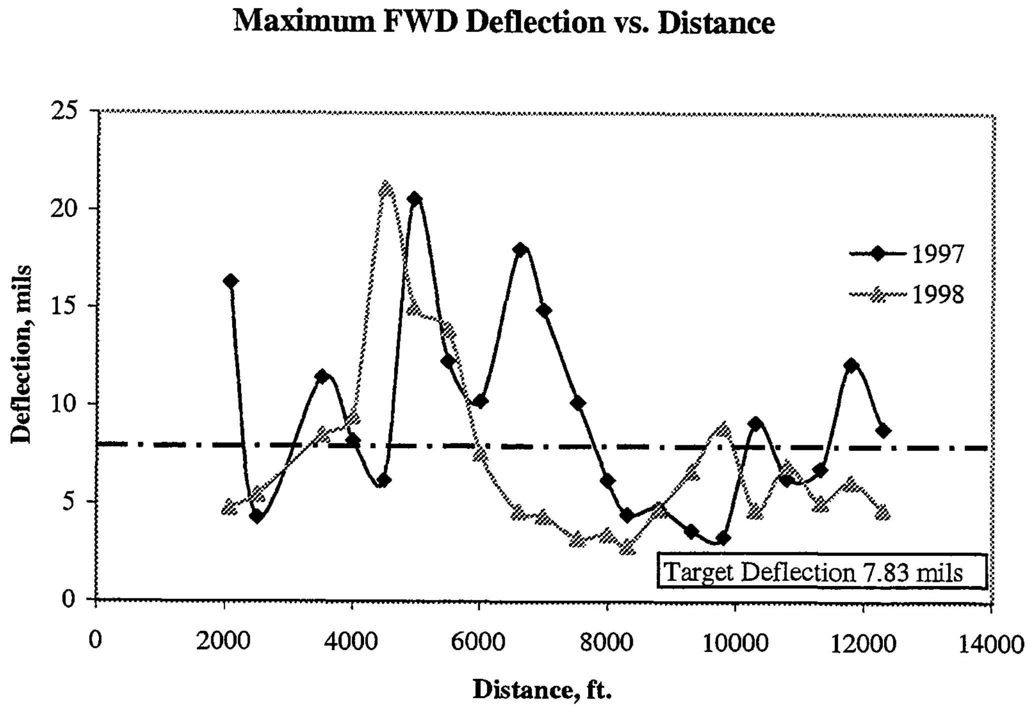


Figure 23. FM 244, Target Deflection Analysis.

Table 16. FM 244, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 17
County: 94
Highway/Road: FM0244

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	0.70	199,980	200,020	H1: ¥ = 0.35
Base:	12.00	25,000	900,000	H2: ¥ = 0.35
Subbase:	10.00	10,000	1,250,000	H3: ¥ = 0.30
Subgrade:	277.30	10,000		H4: ¥ = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
2075.000	9,295	16.34	11.46	7.90	5.69	4.21	3.11	2.34	200.	173.5	13.3	11.2	1.49	271.66
2503.000	10,455	4.31	3.11	2.55	1.83	1.33	0.92	0.71	200.	900.0	62.6	39.5	4.92	175.06 *
3517.000	9,636	11.45	6.93	4.20	2.89	1.94	1.37	1.01	200.	185.7	22.6	21.7	1.64	203.08
3997.000	9,843	8.19	4.97	4.30	3.41	2.71	2.11	1.72	200.	231.9	490.9	13.6	1.20	300.00
4500.000	9,553	6.19	3.77	2.89	2.29	1.80	1.41	1.07	200.	343.7	241.9	22.4	1.44	251.92
4960.000	8,878	20.60	8.48	3.44	1.48	0.93	0.70	0.62	200.	44.2	27.4	27.4	18.27	36.00 *
5502.000	9,354	12.28	5.68	3.30	2.24	1.67	1.29	1.09	200.	103.9	66.5	23.0	4.36	300.00
6004.000	8,953	10.22	5.72	3.49	2.55	1.94	1.52	1.25	200.	150.9	69.9	19.5	5.01	300.00
6613.000	9,918	18.02	8.00	4.06	2.85	2.13	1.63	1.36	200.	68.5	37.7	20.3	5.82	300.00
6999.000	10,169	14.91	4.74	3.03	2.14	1.67	1.34	1.18	200.	70.1	131.1	25.9	7.51	300.00
7528.000	9,632	10.15	5.50	3.82	2.96	2.37	1.95	1.66	200.	141.1	278.3	15.5	5.14	300.00
7997.000	9,644	6.17	4.31	3.62	3.01	2.50	2.03	1.72	200.	417.0	505.0	14.7	0.46	300.00
8315.000	9,847	4.44	3.59	3.22	2.59	2.28	1.80	1.59	200.	900.0	451.0	18.0	2.62	300.00 *
8801.000	10,046	4.75	3.69	3.39	2.78	2.30	1.92	1.59	200.	900.0	290.9	19.0	3.15	300.00 *
9306.000	10,443	3.63	2.63	2.39	2.03	1.71	1.45	1.32	200.	900.0	781.2	24.7	3.76	300.00 *
9816.000	10,419	3.32	3.05	2.67	2.36	2.04	1.76	1.56	200.	900.0	1250.0	18.9	7.28	300.00 *
10310.000	9,593	9.12	2.97	2.60	2.41	2.06	1.76	1.50	200.	141.2	1250.0	15.2	16.50	300.00 *
10799.000	9,851	6.27	3.04	2.64	2.04	1.67	1.33	1.17	200.	242.5	1250.0	18.1	4.36	300.00 *
11319.000	9,732	6.78	4.69	3.90	3.21	2.50	1.95	1.54	200.	394.1	273.4	16.2	1.31	300.00
11800.000	9,815	12.18	4.70	3.52	2.76	2.16	1.72	1.38	200.	93.4	506.2	16.9	7.64	300.00
12297.000	9,986	8.84	4.33	3.78	3.37	2.81	2.39	1.97	200.	179.9	1209.9	11.2	7.11	300.00
Mean:		9.44	5.02	3.56	2.71	2.13	1.69	1.40	200.	356.3	438.5	19.7	5.29	300.00
Std. Dev:		4.91	2.18	1.14	0.85	0.66	0.51	0.40	0.	326.7	447.6	6.4	4.62	351.41
Var Coeff(%):		51.99	43.47	32.17	31.37	30.78	30.39	28.54	0.	91.7	100.0	32.3	87.39	117.14

SECTION 15

Road	County	Limits	Date	Structure
FM 2223	Brazos	OSR to FM 974	9-96	LSB (50-50) (10%), 12"FB & 2 CST

RATING Performance (97/98) A/A Structural (97/98) A/not tested

The pavement section on FM 2223 had two distinct pavement cross-sections. The first has 330 mm (13 in) of flexbase, and the second has 230 mm (9 in) over a heavily stabilized 254 mm (10 in) subbase. The subbase, in both cases, was backcalculated to have a moduli value close to 3450 Mpa (500 ksi). The moduli values for the base were high in both sections. The one slightly weaker spot at 458 m (1500 ft) can be attributed to a poorer subgrade value.

No problems were detected in the GPR. The base and subbase were dry, and no distress was observed in this pavement in either survey. This pavement is performing well. Figure 24 summarizes the deflection profiles for FM 2223, and Tables 17 and 18 summarize the analysis of the 1997 FWD data for this project.

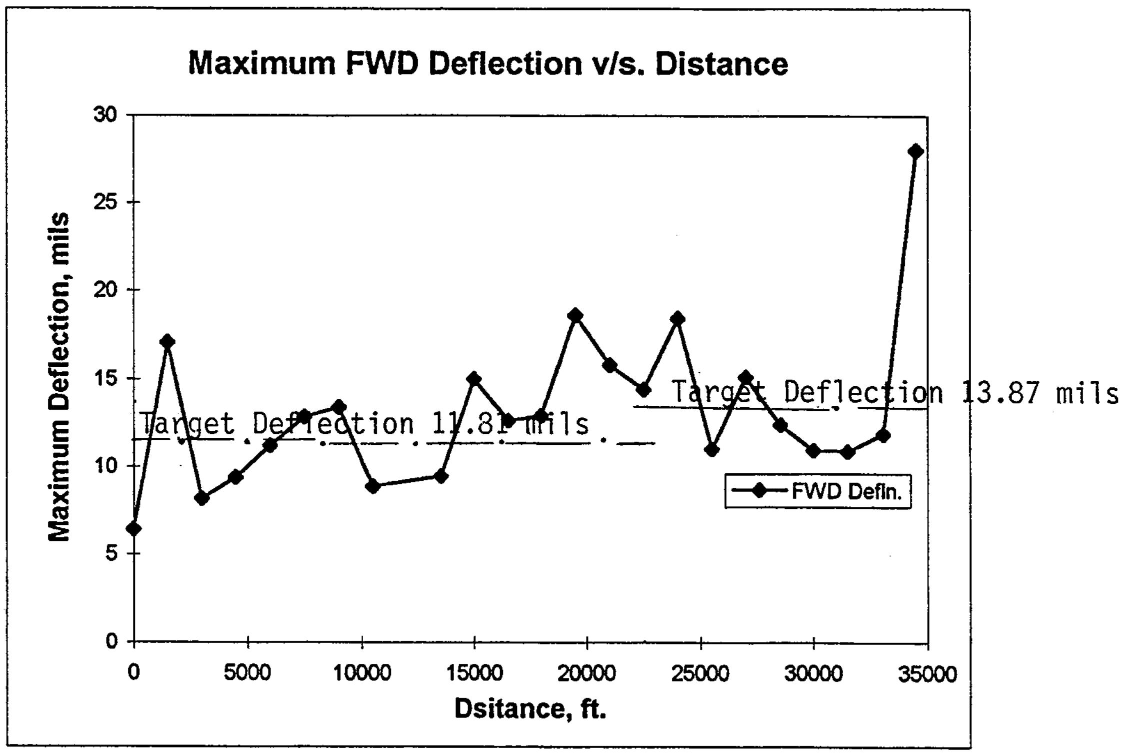


Figure 24. FM 2223, Target Deflection Analysis.

Table 17. FM 2223, FWD Analysis (325 mm (13 in) thk. Flexbase).

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 0
County: 222
Highway/Road: Braz

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	0.70	199,980	200,020	H1: ϒ = 0.35
Base:	13.00	25,000	500,000	H2: ϒ = 0.35
Subbase:	10.00	25,000	1,200,000	H3: ϒ = 0.30
Subgrade:	276.30	10,000		H4: ϒ = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock	
0.000	10,375	6.43	3.74	3.00	2.48	2.11	1.76	1.53	200.	414.7	242.2	21.5	6.25	300.00	
1500.000	10,006	17.06	6.33	4.45	3.58	2.76	2.18	1.79	200.	61.8	458.2	13.9	4.29	300.00	
3000.000	10,411	8.15	5.30	4.00	3.07	2.45	1.98	1.63	200.	315.6	117.0	18.7	1.43	300.00	
4501.000	11,233	9.38	4.63	3.81	3.26	2.69	2.22	1.90	200.	184.3	1170.1	11.7	5.78	300.00	
6004.000	10,506	11.19	5.28	3.52	2.94	2.27	1.86	1.52	200.	121.3	467.6	15.6	5.81	300.00	
7501.000	10,892	12.83	5.70	3.96	3.26	2.56	2.07	1.74	200.	106.7	451.2	15.0	5.47	300.00	
9001.000	10,554	13.39	4.29	3.46	3.01	2.39	1.87	1.50	200.	89.1	1200.0	13.9	10.46	300.00	*
10500.000	10,534	8.87	3.87	2.44	1.86	1.30	1.02	0.94	200.	147.8	242.2	28.4	5.41	269.38	
12004.000	8,834	6.06	3.70	2.49	1.96	1.59	1.30	1.12	200.	313.3	151.8	23.4	4.91	300.00	
13503.000	11,106	9.43	5.12	3.81	3.22	2.63	2.18	1.88	200.	179.7	945.7	12.5	4.37	300.00	
15001.000	11,023	14.99	4.16	3.30	2.70	2.47	1.92	2.05	200.	76.9	228.3	22.8	15.34	300.00	
16534.000	11,329	12.61	4.31	3.33	2.71	2.09	1.81	1.07	200.	106.5	546.5	18.2	12.67	300.00	
17999.000	11,134	12.92	4.60	3.79	2.85	2.31	1.87	1.56	200.	104.8	507.0	16.9	10.19	300.00	
19504.000	10,693	18.58	5.53	3.93	3.42	2.73	2.20	1.82	200.	53.2	611.2	15.9	10.30	300.00	
21000.000	10,804	15.80	5.83	4.31	3.48	2.72	2.12	1.77	200.	76.6	464.7	15.0	6.41	300.00	
22500.000	10,995	14.41	3.77	2.49	2.31	2.02	1.61	1.49	200.	77.1	259.8	26.0	16.80	300.00	
Mean:		12.01	4.76	3.51	2.88	2.32	1.87	1.58	200.	151.8	504.0	18.1	7.87	300.00	
Std. Dev:		3.71	0.84	0.63	0.52	0.42	0.33	0.31	0.	106.4	335.7	5.0	4.30	31.26	
Var Coeff(%):		30.94	17.61	17.99	18.00	18.03	17.82	19.87	0.	70.1	66.6	27.5	54.61	10.42	

Table 18. FM 2223, FWD Analysis (250 mm (10 in) thk. Flexbase).

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 0
County: 222
Highway/Road: Braz

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	0.70	199,980	200,020	H1: ν = 0.35
Base:	9.00	25,000	500,000	H2: ν = 0.35
Subbase:	10.00	10,000	1,200,000	H3: ν = 0.30
Subgrade:	273.00	10,000		H4: ν = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
24008.000	11,384	18.42	3.58	2.94	2.43	1.98	1.52	1.26	200.	46.3	1055.5	26.6	15.09	300.00
25505.000	11,035	11.02	4.85	3.76	2.96	2.33	1.86	1.48	200.	120.3	598.2	19.9	7.29	300.00
27011.000	10,975	15.09	4.04	3.22	2.85	2.31	1.84	1.55	200.	61.6	870.2	22.4	12.68	300.00
28538.000	10,641	12.38	5.30	3.86	2.94	2.30	1.68	1.40	200.	92.8	584.9	19.5	3.91	246.28
30000.000	11,047	10.92	6.24	4.59	3.40	2.52	1.87	1.43	200.	180.4	197.1	18.5	0.91	273.59
31501.000	10,916	10.87	4.77	3.23	2.49	1.86	1.36	1.04	200.	113.9	442.8	24.6	3.98	244.90
33073.000	10,900	11.83	5.24	3.91	2.95	2.19	1.63	1.58	200.	104.2	537.0	20.3	2.53	284.51
34503.000	9,350	28.03	12.03	4.89	2.87	2.06	1.66	1.43	200.	40.3	20.3	17.8	12.54	85.60
Mean:		14.82	5.76	3.80	2.86	2.19	1.68	1.40	200.	95.0	538.3	21.2	7.37	292.71
Std. Dev:		5.95	2.66	0.68	0.30	0.22	0.18	0.17	0.	46.1	333.2	3.1	5.39	182.57
Var Coeff(%):		40.14	46.22	17.88	10.55	9.82	10.69	12.47	0.	48.5	61.9	14.5	73.15	62.37

SECTION 16

Road	County	Limits	Date	Structure
FM 1687	Brazos	FM 50 to OSR	5-96	LSB (60-40) (XX), XX"FB(1%L) & 2 CST

RATING 0.0 to 3.1 mile Performance (97/98) A/B Structural (97/98) A/A PI<35

RATING 3.1 to 3.9 mile Performance (97/98) A/B Structural (97/98) B/C PI>50

In 1997, FM 1687 was rated as a good pavement section, with an average deflection of 279 microns (11 mils)—very close to the target value of 234 microns (9.2 mils). The only slightly weaker section was from 2745 to 3355 m (9000 ft to 11,000 ft). This increase in deflection was caused by a weaker subgrade condition. The GPR indicated that the base was dry, with a thickness between 203 and 254 mm (8 and 10 in). No apparent subsurface defects were found, and no surface distress was evident.

This section was only one year old at the time of the 1997 survey. In 1998, more distress was noted, primarily in the last .8 km (0.5 mi) of the project. Some severe longitudinal cracking was found. For this last subsection, the 1998 deflections were substantially higher than the 1997 data; it is interesting to note that this section is built on very high PI clay. It will be important to follow this change in stiffness with time. Figure 25 summarizes the deflection profiles for FM 1687, and Table 19 summarizes analysis of the 1997 FWD data for this project.

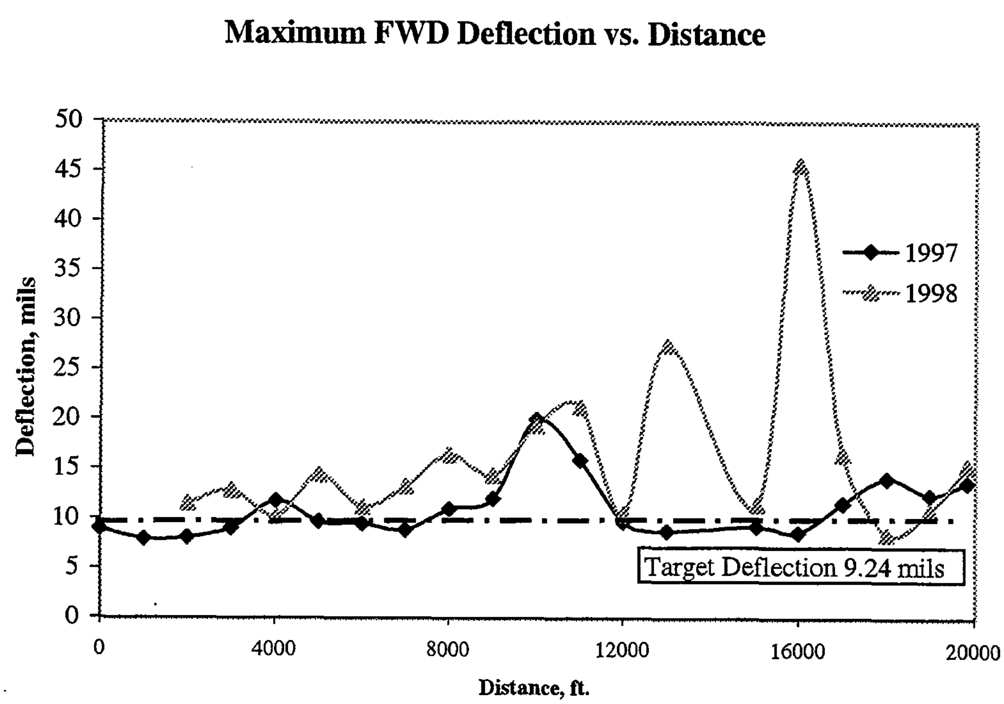


Figure 25. FM 1687, Target Deflection Analysis.

Table 19. FM 1687, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 0
County: 0
Highway/Road:PxNnnn

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	0.70	199,980	200,020	H1: % = 0.35
Base:	7.00	25,000	750,000	H2: % = 0.35
Subbase:	10.00	25,000	700,000	H3: % = 0.30
Subgrade:	282.30	10,000		H4: % = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
0.000	11,158	8.95	6.16	4.42	3.35	2.66	2.15	1.78	200.	364.2	261.0	20.1	2.97	300.00
1001.000	10,411	7.93	4.99	3.85	3.13	2.57	2.11	1.78	200.	293.7	562.3	19.1	3.82	300.00
2002.000	11,138	8.06	6.19	4.52	3.54	2.82	2.35	1.90	200.	625.6	257.3	19.2	2.97	300.00
3000.000	10,097	9.05	6.91	5.30	4.18	3.30	2.60	2.13	200.	577.5	193.0	15.0	0.78	300.00
4017.000	10,459	11.78	8.00	5.74	4.34	3.22	2.59	2.09	200.	269.3	146.6	15.2	1.97	300.00
5003.000	10,280	9.72	7.04	5.33	4.13	3.27	2.57	2.13	200.	396.0	207.8	15.4	0.97	300.00
6012.000	10,320	9.52	6.99	5.09	3.93	3.07	2.44	1.98	200.	425.8	188.0	16.3	1.98	300.00
7000.000	9,974	8.87	6.07	5.00	3.73	2.98	2.29	1.93	200.	309.7	320.8	16.2	1.09	300.00
8013.000	10,463	10.97	7.21	5.34	4.11	3.24	2.60	2.15	200.	230.0	246.6	15.4	2.10	300.00
9003.000	10,637	12.02	7.00	4.52	3.41	2.61	2.20	1.62	200.	157.1	180.2	18.3	5.48	300.00
10004.000	10,355	19.96	12.41	7.78	5.32	3.68	2.73	1.83	200.	163.1	34.5	12.9	0.55	282.20
11004.000	10,093	15.91	11.01	7.06	4.80	3.57	2.56	2.06	200.	287.3	38.9	13.7	1.94	236.99
12008.000	10,208	9.72	6.55	4.63	3.41	2.66	2.11	1.78	200.	289.4	193.1	18.1	2.73	300.00
13003.000	10,296	8.80	5.84	4.34	3.26	2.56	2.05	1.73	200.	304.2	272.7	19.1	2.17	300.00
15049.000	9,950	9.21	6.76	5.01	3.57	2.70	2.01	1.72	200.	631.2	100.1	18.0	1.47	298.67
16002.000	10,101	8.64	6.05	4.65	3.54	2.72	2.06	1.64	200.	418.2	212.9	18.1	0.51	300.00
17000.000	10,308	11.56	8.43	6.18	4.61	3.45	2.60	2.07	200.	443.8	100.0	14.5	0.62	300.00
18012.000	10,073	14.11	7.42	4.69	3.51	2.75	2.16	1.86	200.	100.6	162.6	17.4	5.29	300.00
19006.000	9,187	12.39	6.78	4.39	3.31	2.54	1.98	1.65	200.	114.0	162.4	16.9	4.32	300.00
19831.000	8,949	13.63	7.91	4.72	3.21	2.37	1.88	1.59	200.	145.7	61.6	16.8	3.74	300.00
Mean:		11.04	7.29	5.13	3.82	2.94	2.30	1.87	200.	327.3	195.1	16.8	2.37	300.00
Std. Dev:		3.03	1.73	0.95	0.60	0.39	0.26	0.19	0.	159.8	116.7	2.0	1.53	51.71
Var Coeff(%):		27.45	23.70	18.59	15.58	13.11	11.40	10.00	0.	48.8	59.8	11.8	64.28	17.24

SECTION 17

Road	County	Limits	Date	Structure
FM 977	Leon	3.2 Mi E of FM 1119 to FM 1119	2-96	LSB (30-60) (4%), & 2 CST

RATING Performance (97/98) A/A Structural (97/98) A/A PI<25

Despite having two locations where the measured deflections were higher than the target, this project was given a "A" structural rating. In both of these cases, the increase was attributed to substantially weaker subgrade conditions—almost 50 percent less than the average section subgrade moduli. In all cases, the stiffness of the stabilized layer appeared reasonable.

No distresses were found on the pavement in either year, and the GPR indicated that the base was dry (low dielectric constant). The average base thickness was 254 mm (10 in), but these thicknesses were observed to vary by $\pm$ 51 mm (2 in). Figure 26 summarizes the deflection profiles for FM 977, and Table 20 summarizes analysis of the 1997 FWD data for this project.

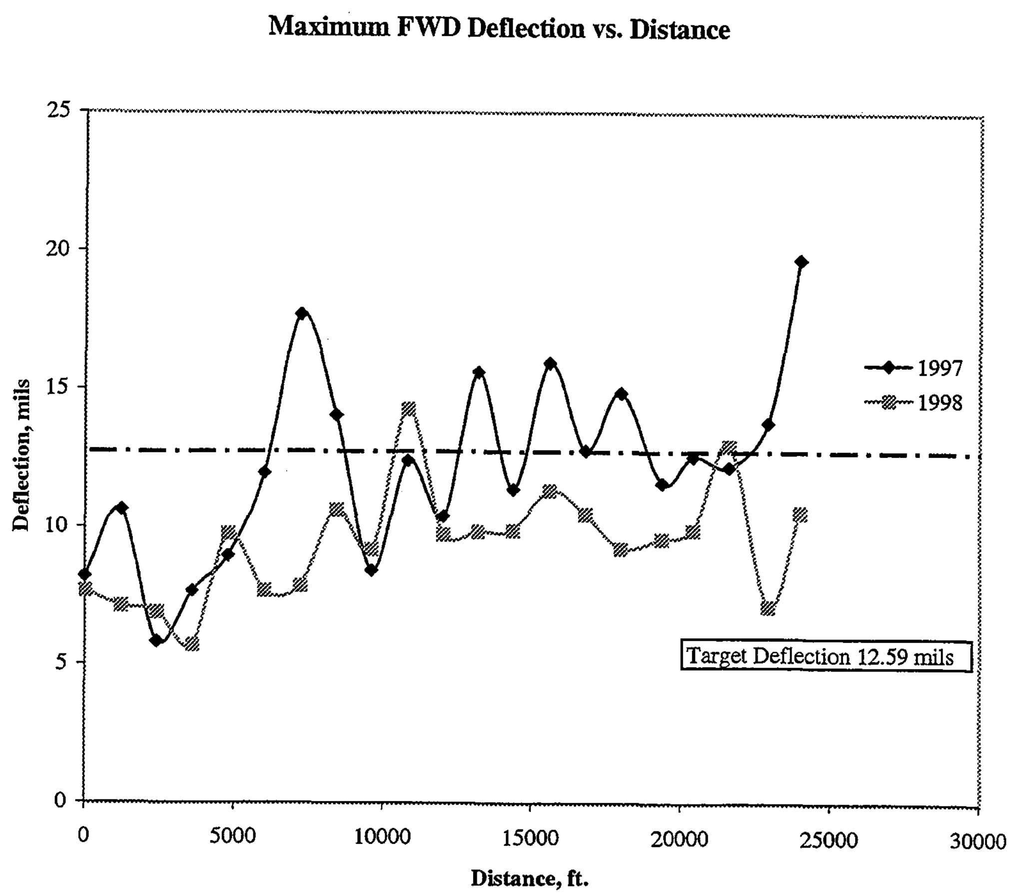


Figure 26. FM 977, Target Deflection Analysis.

Table 20. FM 977, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 0
County: 977
Highway/Road: 10NB

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	0.70	199,980	200,020	H1: % = 0.35
Base:	10.00	25,000	1,500,000	H2: % = 0.30
Subbase:	0.00	0	0	H3: % = 0.40
Subgrade:	183.40	10,000		H4: % = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock	
0.100	10,435	6.37	5.08	3.75	2.64	1.92	1.39	1.18	200.	1228.2	0.0	24.2	1.91	234.27	
1015.000	10,085	8.26	6.50	4.73	3.33	2.22	1.54	1.16	200.	749.0	0.0	19.9	1.79	184.06	
1831.000	9,970	12.58	8.72	5.89	3.78	2.47	1.63	1.28	200.	293.7	0.0	17.7	2.12	152.54	
2822.000	10,769	10.19	8.15	6.47	4.93	3.81	2.67	2.11	200.	1101.6	0.0	12.5	2.22	194.99	
3712.000	10,264	9.11	8.06	6.76	5.38	4.06	3.07	2.25	200.	1500.0	0.0	10.2	1.13	217.19	*
4602.000	10,224	7.28	6.02	4.70	3.56	2.52	1.87	1.27	200.	1377.4	0.0	17.0	0.55	159.78	
5429.000	10,276	9.88	7.67	5.37	3.91	2.51	1.89	1.26	200.	605.4	0.0	17.6	1.65	152.70	
6307.000	10,308	16.34	12.55	8.59	5.75	3.68	2.57	1.96	200.	275.2	0.0	12.2	3.67	159.30	
7206.000	10,395	9.48	7.81	5.98	4.38	3.12	2.21	1.61	200.	913.1	0.0	14.6	1.49	209.05	
8103.000	9,434	19.42	14.57	8.80	5.81	3.91	2.81	2.27	200.	167.4	0.0	10.7	3.13	231.70	
9029.000	9,668	5.80	5.12	4.11	3.11	2.17	1.70	1.24	200.	1500.0	0.0	18.7	2.90	275.47	*
9942.000	9,624	15.21	11.85	7.73	5.00	3.22	2.21	1.54	200.	248.2	0.0	12.9	4.78	165.77	
10809.000	9,938	11.64	10.08	7.61	5.54	3.86	2.71	1.97	200.	675.0	0.0	11.2	2.70	204.51	
11745.000	10,224	10.29	8.56	6.69	4.98	3.68	2.68	1.84	200.	951.5	0.0	12.0	0.89	169.55	
12607.000	9,851	10.98	8.85	6.61	4.65	3.19	2.11	1.63	200.	574.8	0.0	13.9	2.95	153.03	
13595.000	9,636	9.19	7.38	5.45	3.78	3.81	0.85	0.59	200.	487.6	0.0	19.0	26.55	69.49	
14423.000	9,787	25.07	22.23	15.27	9.24	4.74	2.94	2.28	200.	135.4	0.0	7.6	18.87	90.99	
15345.000	10,260	14.97	10.62	5.37	3.57	2.61	1.81	1.58	200.	189.5	0.0	17.6	6.89	187.35	
16334.000	9,895	16.02	12.98	8.97	5.87	3.87	2.53	1.96	200.	281.6	0.0	11.3	5.37	153.95	
17116.000	9,903	13.24	2.88	2.41	2.20	2.07	1.76	1.57	200.	126.9	0.0	40.1	41.53	300.00	
18021.000	9,859	10.75	1.85	1.35	1.59	1.44	1.28	1.15	200.	97.1	0.0	66.7	47.90	300.00	
18073.000	9,704	9.32	2.07	1.65	1.74	1.60	1.38	1.24	200.	213.9	0.0	53.5	43.56	300.00	
Mean:		11.88	8.62	6.10	4.31	3.02	2.07	1.59	200.	622.4	0.0	20.0	10.21	194.19	
Std. Dev:		4.53	4.54	2.95	1.70	0.91	0.61	0.45	0.	475.3	0.0	14.7	15.21	82.43	
Var Coeff(%):		38.14	52.69	48.34	39.46	30.15	29.49	28.26	0.	76.4	0.0	73.4	148.98	42.45	

SECTION 18

Road	County	Limits	Date	Structure
FM 978	Madison	FM 39 to FM 2289	4-96	LSB (40-60) (5%), & 2 CST

RATING 0.0 to 4.0 mile Performance (97/98) A/B Structural (97/98) B/B PI>35

RATING 4.0 to 7.8 mile Performance (97/98) B/B Structural (97/98) A/B PI>25

In 1997, this pavement section had high deflections from 4575 to 5338 m (15,000 to 17,500 ft). The high deflections were caused by lower subgrade stiffness and not attributed to problems with the stabilized base. In 1997, there was no obvious distress in the section; however, the GPR did indicate a potential future problem. The base dielectric constant (moisture content) was substantially higher on this section than on the other pavements that were tested. Dielectric constants exceeding 12 indicate trouble for stabilized materials. Normally, the dielectric constants of the stabilized base run from 7 to 10, indicating a dry condition.

In 1997, the section was only one year old. In 1998, several locations of severe longitudinal cracks were found. In general, the pavement condition was good, with good ride. However, this section should be monitored to evaluate long-term performance. Figure 27 summarizes the deflection profiles for FM 978, and Table 21 summarizes analysis of the 1997 FWD data for this project.

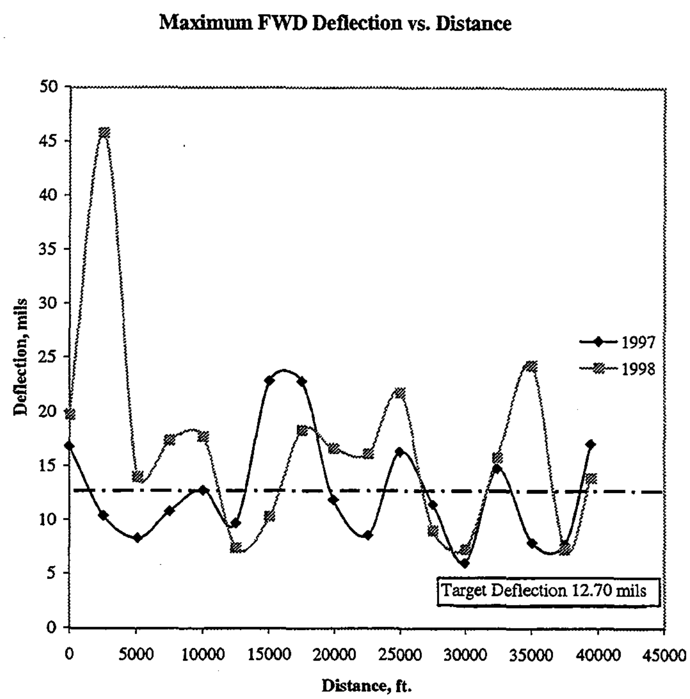


Figure 27. FM 978, Target Deflection Analysis.

Table 21. FM 978, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 0			MODULI RANGE(psi)		
County: 978		Thickness(in)	Minimum	Maximum	Poisson Ratio Values
Highway/Road:Madiso	Pavement:	0.70	199,980	200,020	H1: ¥ = 0.35
	Base:	10.00	25,000	1,500,000	H2: ¥ = 0.30
	Subbase:	0.00	0	0	H3: ¥ = 0.40
	Subgrade:	209.10	10,000		H4: ¥ = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
0.000	10,554	16.81	14.14	10.96	8.22	5.99	4.27	3.19	200.	524.9	0.0	8.3	1.50	232.85
2501.000	10,916	10.43	7.83	5.85	4.27	3.11	2.31	1.80	200.	732.5	0.0	16.9	2.40	293.17
5089.000	10,538	8.32	7.10	6.00	4.62	3.82	2.54	2.32	200.	1500.0	0.0	13.2	3.51	151.60 *
7525.000	10,451	10.80	8.77	6.53	4.72	3.38	2.44	1.92	200.	694.3	0.0	14.6	1.38	240.15
10010.000	9,914	12.80	9.76	6.59	4.38	3.06	2.23	1.77	200.	354.0	0.0	15.2	1.76	267.32
12503.000	10,745	9.72	7.78	5.68	4.06	2.71	1.93	1.39	200.	683.4	0.0	18.2	2.25	196.34
15048.000	10,073	22.87	18.27	12.41	8.08	5.16	3.22	2.02	200.	169.2	0.0	8.9	8.14	139.01
17506.000	9,839	22.86	16.32	9.93	5.87	3.56	2.27	1.56	200.	108.7	0.0	11.2	9.91	130.43
19931.000	10,641	11.90	8.60	5.59	3.76	2.58	1.88	1.50	200.	369.5	0.0	19.1	2.02	249.04
22553.000	10,804	8.58	7.61	6.06	4.57	3.43	2.53	1.99	200.	1398.2	0.0	13.9	2.14	275.59
25000.000	10,630	16.35	12.03	8.32	5.72	3.88	3.00	2.12	200.	302.5	0.0	12.8	1.75	236.49
27513.000	10,645	11.45	9.61	7.35	5.31	3.74	2.58	1.87	200.	692.0	0.0	13.3	2.87	183.81
30015.000	10,157	6.04	6.74	5.54	4.29	3.13	2.32	1.78	200.	1500.0	0.0	15.6	10.52	*** *
32508.000	10,530	14.82	11.30	7.96	5.39	3.64	2.65	2.13	200.	343.7	0.0	13.4	1.82	227.60
35059.000	11,098	7.94	5.87	4.52	3.36	2.59	1.97	1.61	200.	1207.5	0.0	20.7	4.66	300.00
37510.000	11,142	7.74	6.95	5.53	4.54	3.24	2.55	1.78	200.	1500.0	0.0	15.6	3.82	300.00 *
39494.000	10,423	17.10	12.76	8.98	5.93	3.81	2.59	1.91	200.	252.3	0.0	12.4	4.39	164.19
Mean:		12.74	10.08	7.28	5.12	3.58	2.55	1.92	200.	725.5	0.0	14.3	3.81	219.90
Std. Dev:		5.05	3.56	2.19	1.35	0.87	0.56	0.41	0.	502.0	0.0	3.3	2.93	65.82
Var Coeff(%):		39.66	35.34	30.06	26.34	24.35	22.17	21.12	0.	69.2	0.0	23.0	76.73	29.93

SECTION 19

Road	County	Limits	Date	Structure
FM 1373	Robertson	Falls County Line to 6 Mi E	---	LSB (50-50) (XX), & 2 CST

RATING Performance (97/98) A/B Structural (97/98) A/A PI<25

In 1997, it was concluded that this pavement section has an extremely stiff, uniform, stabilized base. It was surprising that no stabilization cracks were present. Visual condition was very good, and the GPR did not find any defects. The average deflection was almost 50 percent lower than the target deflection. If this section does not crack, forensic studies need to be carried out to find out what "works." Evaluation of the material used, construction techniques, and time of the year when constructed should be carried out to document what "works."

Again, this section was only one year old at the time of the 1997 inspection. In 1998, the section was still performing well; only one short 30 m (100 ft) long crack was found in the entire 9.5 km (6 mi) project. Figure 28 summarizes the deflection profiles for FM 1373, and Table 22 summarizes analysis of the 1997 FWD data for this project.

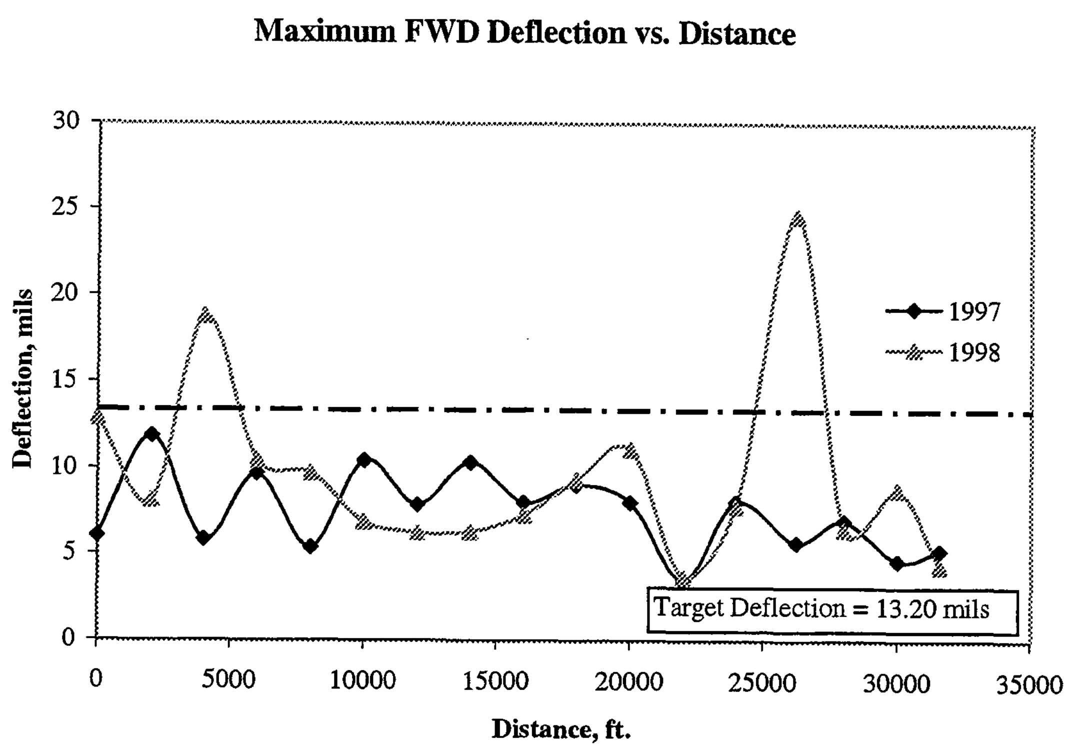


Figure 28. FM 1373, Target Deflection Analysis.

Table 22. FM 1373, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 0
County: 137
Highway/Road: Robe

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	0.70	199,980	200,020	H1: ¥ = 0.35
Base:	10.00	100,000	2,100,000	H2: ¥ = 0.30
Subbase:	0.00	0	0	H3: ¥ = 0.40
Subgrade:	289.30	10,000		H4: ¥ = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
0.000	10,610	6.01	5.66	4.48	3.52	2.74	2.13	1.76	200.	2100.0	0.0	19.4	2.64	300.00 *
2020.000	10,498	11.87	9.59	7.55	5.62	4.19	3.17	2.46	200.	739.5	0.0	13.0	0.69	300.00
4001.000	10,729	5.85	5.06	4.06	3.23	2.55	2.08	1.75	200.	2100.0	0.0	21.6	3.15	300.00 *
5998.000	10,820	9.69	7.98	6.52	5.04	3.90	2.98	2.39	200.	1227.7	0.0	14.2	0.97	300.00
8011.000	10,486	5.39	5.06	4.15	3.37	2.69	2.26	1.82	200.	2100.0	0.0	20.2	6.11	300.00 *
10002.000	10,665	10.44	9.17	7.57	6.01	4.56	3.54	2.74	200.	1271.9	0.0	11.5	1.30	300.00
12027.000	10,443	7.87	7.02	5.85	4.66	3.66	2.83	2.26	200.	1899.8	0.0	13.8	1.35	300.00
14001.000	10,459	10.31	8.56	6.94	5.37	4.07	3.04	2.41	200.	1039.8	0.0	13.2	1.01	300.00
16019.000	10,653	8.01	7.47	6.17	4.84	3.78	2.97	2.39	200.	1895.7	0.0	13.5	2.30	300.00
18000.000	10,912	9.03	7.43	5.86	4.51	3.52	2.69	2.23	200.	1245.7	0.0	16.1	1.38	300.00
20000.000	10,677	8.01	6.56	5.08	3.83	2.85	2.15	1.72	200.	1170.0	0.0	19.3	1.18	300.00
21999.000	10,200	3.48	3.43	2.82	2.27	1.79	1.41	1.14	200.	2100.0	0.0	34.7	11.72	300.00 *
24001.000	10,633	8.06	5.27	3.94	3.15	2.44	1.92	1.49	200.	1010.4	0.0	24.0	7.97	294.19
26230.000	10,391	5.70	5.11	4.21	3.29	2.55	1.97	1.56	200.	2100.0	0.0	20.7	2.16	300.00 *
28002.000	10,367	6.95	5.96	4.72	3.65	2.78	2.11	1.64	200.	1611.3	0.0	18.9	1.51	300.00
30005.000	10,852	4.63	3.97	3.24	2.55	2.00	1.48	1.21	200.	2100.0	0.0	30.6	4.64	266.88 *
31566.000	10,570	5.25	4.30	3.33	2.62	1.84	1.59	1.31	200.	1996.5	0.0	28.0	2.17	283.78
Mean:		7.44	6.33	5.09	3.97	3.05	2.37	1.90	200.	1629.9	0.0	19.6	3.07	300.00
Std. Dev:		2.32	1.84	1.50	1.13	0.86	0.64	0.49	0.	485.9	0.0	6.6	2.98	34.50
Var Coeff(%):		31.16	29.01	29.58	28.57	28.16	26.79	25.77	0.	29.8	0.0	33.9	97.05	11.50

SECTION 20

Road	County	Limits	Date	Structure
FM 3178	Leon	FM 1511 to FM 542	---	CSB (50-50) (4%), & 2 CST

RATING Performance (97/98) A/B Structural (97/98) C/C PI<25

The pavement section on FM 3178 looked considerably weaker than other sections in this study. The target deflection was 396 microns (15.6 mils), and the average measured in 1997 was 663 microns (26.1 mils). Several areas have deflections over 889 microns (35 mils). Normally, for newly constructed, unstabilized granular base, the ratio of base to subgrade modulus is around 3.0. In several areas, the cement-treated base has less than a 3:1 ratio; see 5187 m (17,005 ft). The GPR indicated that the base was not wet, but in some instances, it appeared less than the planned thickness of 254 mm (10 in). The thickness appeared closer to 203 mm (8 in).

This highway could experience load-related damage, particularly the section from 4877 to 5486 m (16,000 to 18,000 ft). In 1998, the overall performance was still rated as very good, but in one short location, some apparent load-associated rutting was found. Monitoring should be continued. Figure 29 summarizes the deflection profiles for FM 3178, and Table 23 summarizes analysis of the 1997 FWD data for this project.

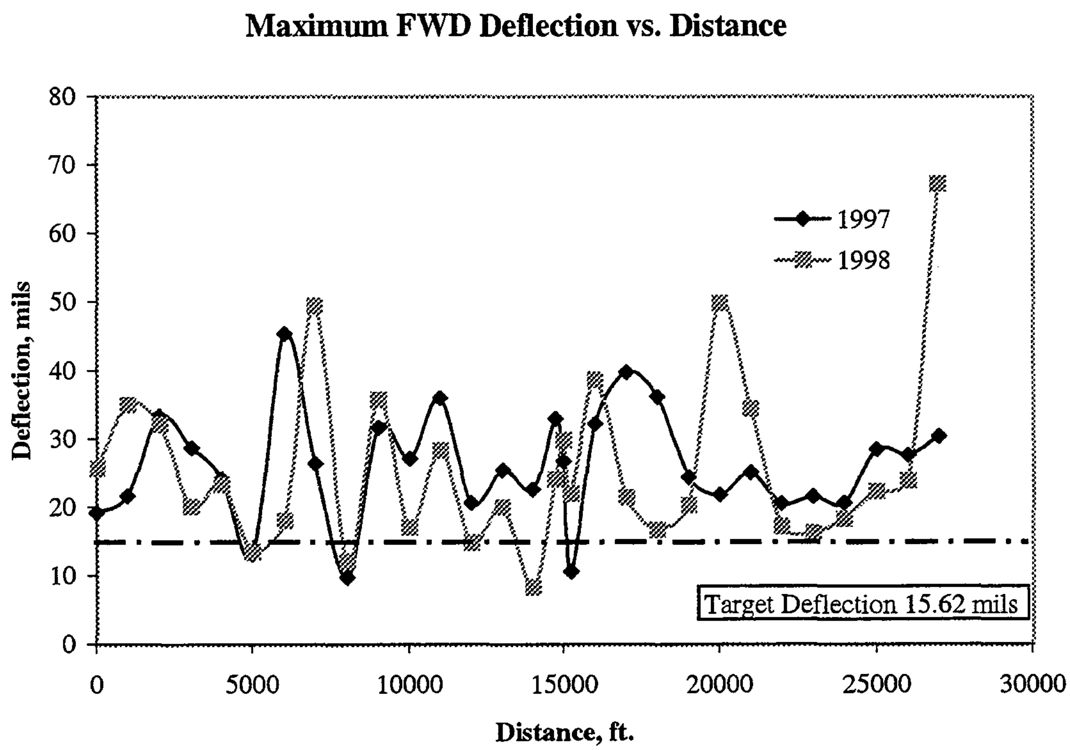


Figure 29. FM 3178, Target Deflection Analysis.

Table 23. FM 3178, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 17			MODULI RANGE(psi)			
County: 145		Thickness(in)	Minimum	Maximum	Poisson Ratio Values	
Highway/Road: FM3178	Pavement:	0.70	199,980	200,020	H1: ¥ = 0.35	
	Base:	10.00	10,000	500,000	H2: ¥ = 0.30	
	Subbase:	0.00	0	0	H3: ¥ = 0.40	
	Subgrade:	107.60	10,000		H4: ¥ = 0.40	

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock	
0.000	11,535	19.11	4.60	4.15	3.11	2.48	1.87	1.24	200.	104.5	0.0	29.6	36.72	148.46	
1000.000	10,065	21.57	12.48	7.93	5.33	3.86	2.97	2.35	200.	168.6	0.0	9.8	12.59	300.00	
2032.000	9,732	33.28	19.16	10.26	6.21	4.12	2.83	2.00	200.	65.9	0.0	7.8	5.04	203.15	
3031.000	10,173	28.69	17.04	8.06	3.89	2.15	1.42	1.09	200.	60.1	0.0	12.0	12.53	76.75	
4000.000	10,546	24.11	13.59	8.01	4.65	2.72	1.82	2.05	200.	100.6	0.0	12.0	1.25	111.40	
5000.000	9,716	13.13	10.22	7.42	5.31	3.87	2.83	2.12	200.	500.0	0.0	8.7	6.55	259.98	*
6021.000	9,394	45.37	25.95	13.63	8.27	5.24	3.23	2.02	200.	43.1	0.0	5.6	3.16	142.07	
7000.000	9,668	26.33	16.65	9.17	5.04	2.95	1.81	1.30	200.	83.7	0.0	9.5	5.63	115.59	
8026.000	9,744	9.73	6.90	4.23	2.59	1.57	0.92	0.75	200.	385.8	0.0	19.5	2.54	110.07	
9010.000	9,450	31.55	18.37	11.07	6.85	4.32	2.82	2.23	200.	78.2	0.0	7.1	4.16	158.65	
10004.000	9,748	27.07	16.49	9.69	5.48	3.62	2.63	2.04	200.	91.0	0.0	8.6	4.32	140.24	
11006.000	9,497	35.89	14.48	5.39	2.87	2.06	1.73	1.50	200.	30.9	0.0	14.2	13.14	64.12	
11999.000	10,749	20.55	8.52	4.13	2.21	1.41	1.18	1.02	200.	75.5	0.0	23.4	7.72	97.69	
13011.000	9,640	25.34	14.19	5.65	2.57	1.45	1.09	1.02	200.	53.3	0.0	15.5	14.99	68.00	
14000.000	10,129	22.52	11.96	6.16	3.08	1.84	1.33	1.02	200.	77.9	0.0	16.0	5.85	81.32	
14731.000	9,418	32.88	15.31	6.09	3.00	1.78	1.30	1.07	200.	36.0	0.0	13.2	11.19	78.85	
14976.000	9,930	26.69	10.89	6.81	3.57	2.18	1.56	1.13	200.	60.1	0.0	14.8	6.39	94.25	
15225.000	10,324	10.60	6.14	3.21	1.87	1.21	0.90	0.70	200.	218.3	0.0	30.0	3.00	151.59	
15999.000	9,581	32.10	16.29	6.62	3.08	1.92	1.42	1.20	200.	40.7	0.0	12.5	11.71	71.10	
17005.000	9,136	39.65	14.60	4.96	2.31	1.50	1.12	0.96	200.	23.3	0.0	15.5	13.56	55.97	
17996.000	9,708	36.06	17.53	8.91	5.04	3.37	2.52	2.02	200.	46.4	0.0	9.3	6.45	138.60	
19001.000	10,336	24.27	16.54	10.15	6.41	4.11	2.75	1.94	200.	153.6	0.0	8.2	3.75	167.60	
20007.000	10,050	21.77	15.10	8.98	5.31	3.27	2.14	1.64	200.	147.7	0.0	9.5	2.85	136.56	
20999.000	9,926	25.00	16.29	8.44	4.82	2.87	1.95	1.56	200.	92.2	0.0	10.1	4.60	119.78	
22003.000	10,248	20.43	15.73	10.93	7.00	4.69	3.25	2.47	200.	277.0	0.0	7.0	2.86	209.73	
23003.000	9,982	21.50	14.08	8.37	4.61	2.63	1.76	1.35	200.	126.2	0.0	10.8	4.53	105.68	
24000.000	10,093	20.51	14.63	9.86	6.62	4.49	3.16	2.39	200.	260.1	0.0	7.4	5.00	222.31	
24999.000	9,664	28.36	17.85	11.23	6.97	4.70	3.30	2.58	200.	113.6	0.0	6.9	6.28	236.58	
26000.000	9,887	27.53	17.58	9.88	5.43	3.37	2.34	1.91	200.	87.6	0.0	8.7	2.75	119.69	
26977.000	9,585	30.21	15.97	8.60	4.56	3.30	2.56	2.15	200.	62.8	0.0	9.8	7.55	100.94	
Mean:		26.06	14.50	7.93	4.60	2.97	2.08	1.63	200.	122.2	0.0	12.4	7.62	118.35	
Std. Dev:		7.98	4.28	2.51	1.70	1.16	0.77	0.56	0.	108.8	0.0	6.2	6.71	50.12	
Var Coeff(%):		30.62	29.54	31.64	36.94	38.92	37.01	34.50	0.	89.1	0.0	49.6	88.01	42.35	

SECTION 21

Road	County	Limits	Date	Structure
FM 977	Leon	FM 3 to 2 Mi E	5-96	CSB (50-50) (4%), & 2 CST

RATING Performance (97/98) A/A Structural (97/98) A/B PI<15

The 1997 average deflection of 305 microns (12 mils) was less than the target deflection of 390 microns (15.4 mils). No distress was found in either visual survey, and no apparent defects were observed from the GPR survey. The high deflection at 0.8 km (0.5 mi) was attributed to a weak subgrade modulus of 41 Mpa (6 ksi) against the measured average subgrade modulus of 111 Mpa (16.1 ksi). The 1998 deflections at these locations were higher than the 1997 readings. The high deflections at the end of the project are from the next pavement section and not part of this study. Figure 30 summarizes the deflection profiles for FM 977, and Table 24 summarizes analysis of the 1997 FWD data for this project.

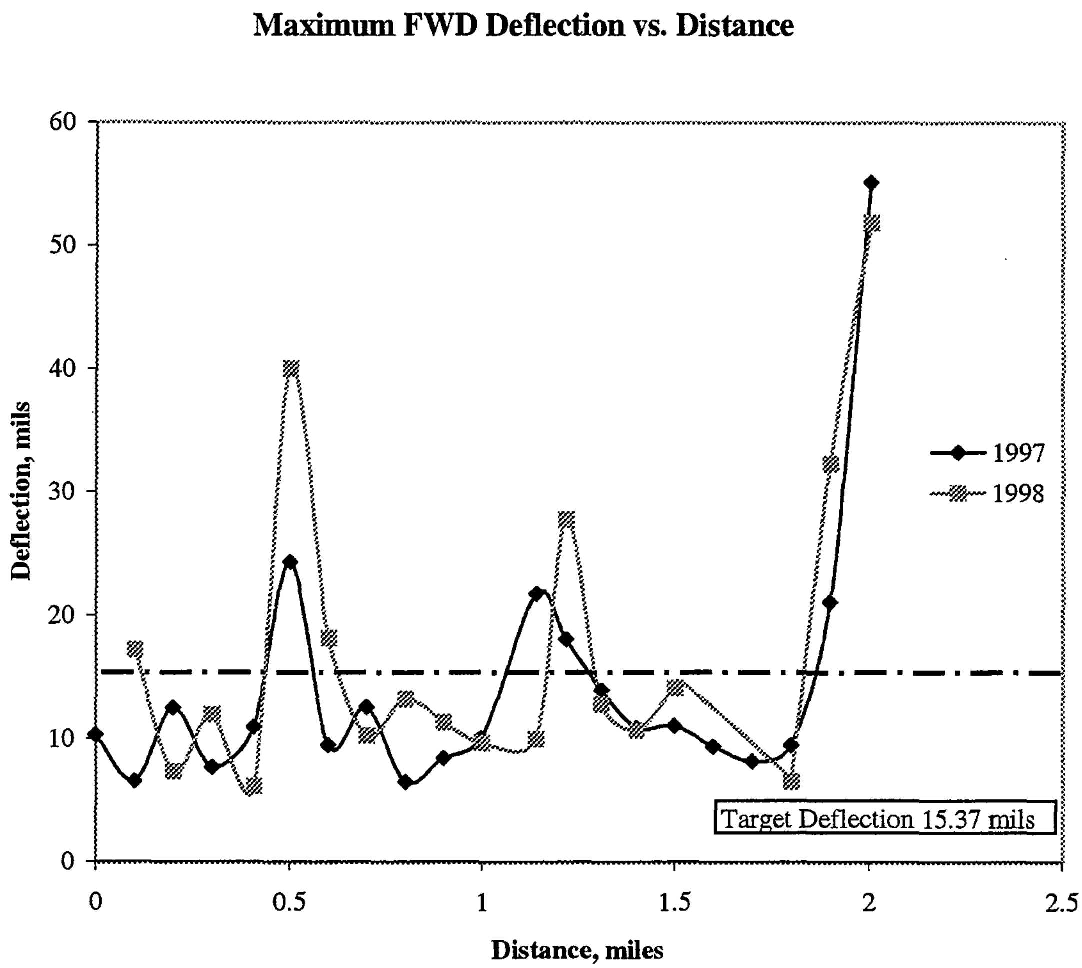


Figure 30. FM 977, Target Deflection Analysis.

Table 24. FM 977, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 17
County: 123
Highway/Road: PxNnnn

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	0.70	199,980	200,020	H1: ¥ = 0.35
Base:	10.00	25,000	1,000,000	H2: ¥ = 0.30
Subbase:	0.00	0	0	H3: ¥ = 0.40
Subgrade:	206.60	10,000		H4: ¥ = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
0.001	11,015	10.33	9.11	6.98	5.09	3.65	2.72	2.02	200.	936.6	0.0	13.4	2.53	238.92
0.100	11,190	6.52	5.28	4.24	3.28	2.36	1.70	1.21	200.	1000.0	0.0	24.8	8.82	185.18 *
0.200	10,343	12.50	10.00	7.61	5.85	4.45	3.43	2.51	200.	766.5	0.0	10.8	2.64	220.33
0.300	10,387	7.70	6.91	5.78	4.56	3.62	2.81	2.24	200.	1000.0	0.0	15.7	11.42	300.00 *
0.407	10,379	10.92	8.40	6.12	4.38	3.06	2.15	1.57	200.	571.6	0.0	16.1	0.90	200.70
0.500	9,918	24.30	19.15	14.30	10.39	7.59	5.67	4.35	200.	266.3	0.0	6.2	0.81	300.00
0.600	9,847	9.50	7.50	5.54	3.75	2.56	1.78	1.33	200.	591.4	0.0	17.8	2.69	190.68
0.700	10,224	12.59	11.16	8.99	6.82	4.99	3.63	2.62	200.	839.7	0.0	9.1	2.43	211.22
0.800	10,073	6.47	5.58	4.46	3.51	2.63	1.65	1.54	200.	1000.0	0.0	20.4	7.49	124.43 *
0.900	10,761	8.44	6.71	5.32	3.61	2.69	1.83	1.31	200.	941.4	0.0	18.9	3.02	168.98
1.000	10,558	10.09	8.51	6.84	5.30	4.00	3.01	2.28	200.	1000.0	0.0	12.3	2.48	264.66 *
1.139	10,022	21.75	8.89	4.89	3.27	2.41	1.89	1.61	200.	75.5	0.0	19.5	13.05	300.00
1.216	9,541	18.07	14.91	10.29	7.06	5.17	3.80	3.09	200.	291.3	0.0	8.8	2.29	300.00
1.308	9,791	13.93	8.75	4.69	2.60	1.65	1.23	0.88	200.	155.3	0.0	21.8	5.37	115.61
1.400	10,232	10.89	8.09	5.64	3.66	2.51	1.72	1.27	200.	421.2	0.0	18.9	2.34	177.91
1.500	10,038	11.06	7.60	4.70	2.86	1.92	1.33	1.21	200.	287.3	0.0	22.4	2.56	187.02
1.601	10,240	9.35	8.09	5.86	4.15	2.86	2.02	1.50	200.	740.6	0.0	16.3	3.47	206.98
Mean:		12.02	9.10	6.60	4.71	3.42	2.49	1.91	200.	640.3	0.0	16.1	4.37	217.40
Std. Dev:		5.01	3.40	2.57	1.95	1.48	1.14	0.87	0.	331.4	0.0	5.3	3.63	74.57
Var Coeff(%):		41.70	37.40	38.85	41.47	43.32	45.89	45.48	0.	51.8	0.0	32.9	83.12	34.30

SECTION 22

Road	County	Limits	Date	Structure
FM 1935	Washington	FM 390 to End	---	-----

RATING Performance (97/98) A/A Structural (97/98) A/C PI<15

No plan thicknesses or construction details were available for the pavement section on FM 1935. In 1997, no apparent problems were found with GPR and FWD, and no distresses were observed. The section was still performing very well in 1998, but the deflections had increased significantly. In 1997, the average maximum deflection was 465 microns (11.8 mils); this increased to 784 microns (19.9 mils) in 1998. The backcalculated moduli decreased from 3036 to 2139 Mpa (440 to 310 ksi), and in several locations, the benefits of the stabilization appeared to be disappearing. This will be an interesting section to continue to monitor. Figure 31 summarizes the deflection profiles for FM 1935, and Table 25 summarizes analysis of the 1997 FWD data for this project.

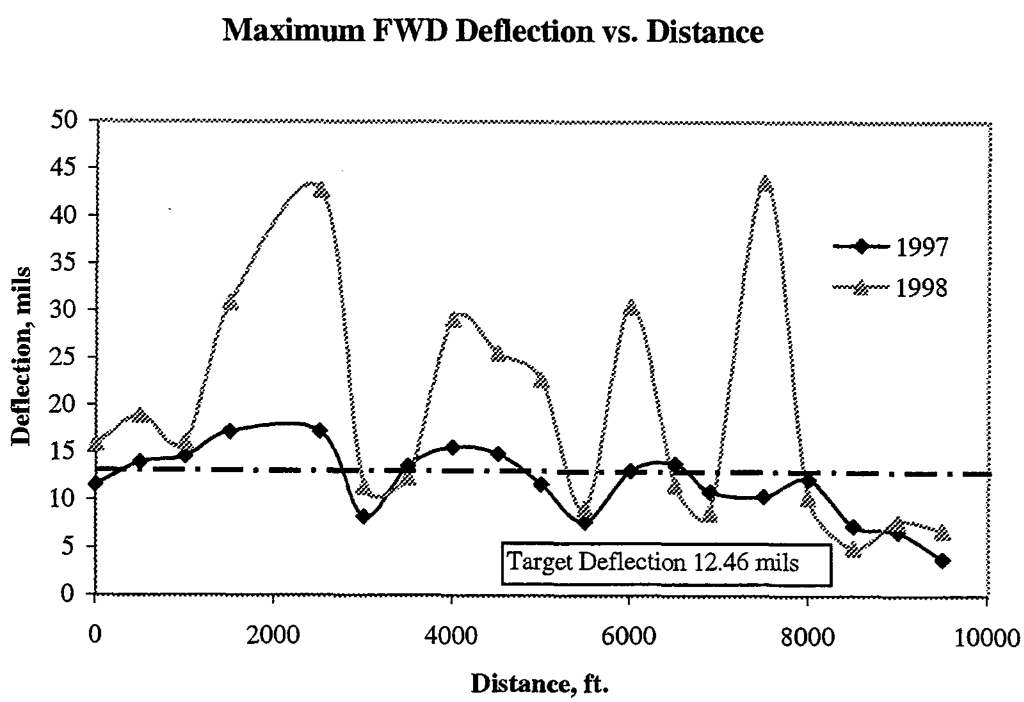


Figure 31. FM 1935, Target Deflection Analysis.

Table 25. FM 1935, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 0
County: 193
Highway/Road: Washi

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	0.70	199,980	200,020	H1: ¥ = 0.35
Base:	10.00	25,000	1,000,000	H2: ¥ = 0.30
Subbase:	12.00	4,000	50,000	H3: ¥ = 0.40
Subgrade:	185.50	10,000		H4: ¥ = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock	
0.100	9,779	11.61	8.39	5.72	3.98	2.93	2.13	1.77	200.	340.7	27.9	15.5	1.75	253.68	
500.000	10,169	13.97	9.20	6.11	4.12	2.89	2.16	1.68	200.	222.6	27.4	15.6	1.07	300.00	
1002.000	9,843	14.62	10.89	7.82	5.41	3.52	2.65	2.00	200.	361.9	9.7	13.0	1.25	173.27	
1502.000	9,887	17.22	12.43	8.43	5.66	3.85	2.56	2.03	200.	263.2	8.1	12.8	0.92	162.02	
2014.000	9,303	28.27	17.19	8.63	4.77	2.94	1.93	1.45	200.	77.4	5.2	13.9	4.10	121.83	
2506.000	9,585	17.23	13.81	8.24	5.67	3.91	2.75	2.04	200.	258.7	6.7	12.4	4.12	214.24	
3009.000	9,998	8.22	6.30	4.75	3.54	2.65	2.10	1.61	200.	696.7	50.0	16.6	1.77	281.27	*
3503.000	9,954	13.58	9.34	6.40	4.44	3.09	2.29	1.74	200.	266.3	24.4	14.6	0.68	289.77	
4002.000	9,712	15.55	11.14	6.91	4.13	2.70	1.94	1.50	200.	241.9	7.9	17.0	3.28	182.50	
4511.000	9,656	14.86	10.12	6.22	3.87	2.70	2.04	1.66	200.	205.5	15.8	15.9	3.22	300.00	
5003.000	9,887	11.67	7.86	5.41	3.72	2.53	1.92	1.49	200.	292.1	30.8	17.1	1.23	230.36	
5503.000	10,014	7.68	5.79	4.23	3.16	2.28	1.74	1.36	200.	686.1	49.5	19.4	1.36	300.00	
6005.000	10,212	13.12	9.78	6.07	3.97	2.74	2.12	1.56	200.	310.0	15.7	16.9	4.10	269.33	
6502.000	9,724	13.86	8.15	4.87	2.94	1.92	1.44	1.07	200.	167.6	22.8	20.2	2.25	173.38	
6901.000	9,644	10.94	5.29	3.72	2.32	1.54	1.14	0.92	200.	172.8	50.0	25.4	3.29	188.11	*
7504.000	10,252	10.45	7.09	4.72	3.15	2.11	1.46	0.94	200.	367.0	24.6	21.8	0.61	142.41	
8003.000	9,807	12.19	6.41	4.21	2.63	1.69	1.25	1.08	200.	161.9	41.0	22.8	2.18	154.82	
8507.000	9,970	7.30	5.61	4.02	2.88	2.11	1.56	1.18	200.	778.2	36.9	21.4	1.71	256.26	
9005.000	10,117	6.67	4.59	3.05	1.92	1.33	1.02	0.83	200.	617.5	35.7	33.6	3.18	259.59	
9502.000	10,359	3.81	3.06	2.24	1.48	0.94	0.62	0.38	200.	1000.0	50.0	52.5	9.64	36.00	*
Mean:		12.64	8.62	5.59	3.69	2.52	1.84	1.41	200.	374.4	27.0	19.9	2.59	208.20	
Std. Dev:		5.15	3.40	1.81	1.17	0.79	0.56	0.44	0.	246.0	15.7	9.2	2.03	181.77	
Var Coeff(%):		40.75	39.43	32.33	31.65	31.49	30.51	31.28	0.	65.7	58.0	46.1	78.66	87.31	

SECTION 23

Road	County	Limits	Date	Structure
FM 975	Burleson	SPRR to 5.5 Mi S	6-96	LSB (60-40) (3%), & 2 CST

RATING 0.0 to 1.1 mile Performance (97/98) B/C Structural (97/98) A/C PI>55

RATING 1.1 to 2.5 mile Performance (97/98) A/B Structural (97/98) A/B PI>25

RATING 2.5 to 5.3 mile Performance (97/98) A/C Structural (97/98) C/C PI<15

In 1997, the overall deflections for this pavement section on FM 975 looked very good. The average deflection of 391 microns (15.4 mils) was below the target deflection of 439 microns (17.3 mils). The only area of high deflection was between 4575 and 4880 m (15,000 and 16,000 ft). This increase was largely related to the weaker subgrade in the area. However, the GPR did detect localized areas of wetter base which could be problematic. It is apparent that the subgrade material varies substantially along this project.

In 1998, the deflections for this section had increased substantially, and the moduli values had decreased. Little distress was found in 1997, but several areas were showing moderate cracking in 1998. In 1998, the section was only two years old; therefore, continued monitoring is recommended. Figure 32 summarizes the deflection profiles for FM 975, and Table 26 summarizes analysis of the 1997 FWD data for this project.

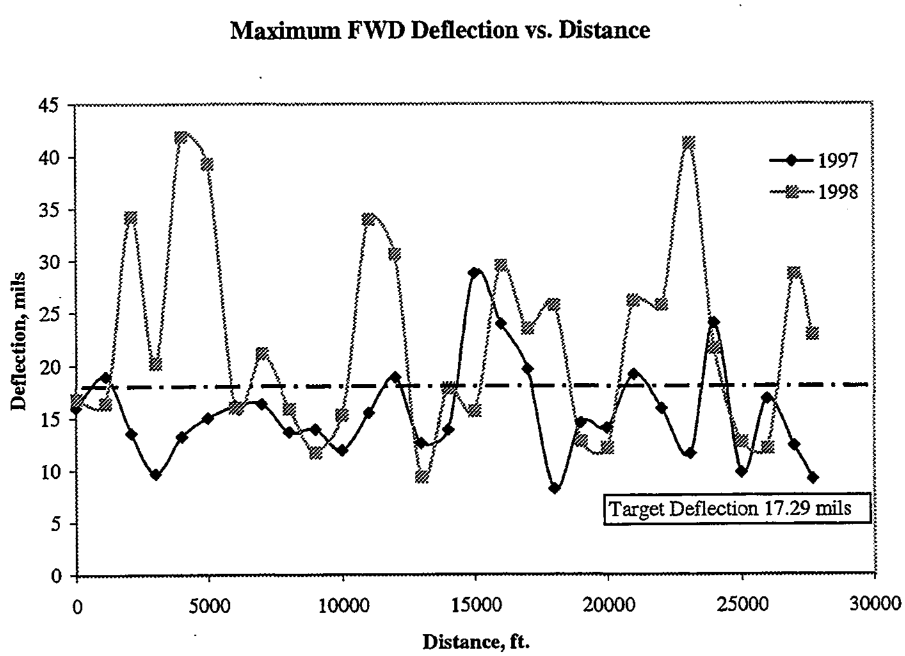


Figure 32. FM 975, Target Deflection Analysis.

Table 26. FM 975, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

			MODULI RANGE(psi)		
District: 17		Thickness(in)	Minimum	Maximum	Poisson Ratio Values
County: 26	Pavement:	0.70	199,980	200,020	H1: ¥ = 0.35
Highway/Road: fm975	Base:	10.00	40,000	600,000	H2: ¥ = 0.30
	Subbase:	0.00	0	0	H3: ¥ = 0.40
	Subgrade:	190.50	15,000		H4: ¥ = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock	
0.000	10,641	15.98	11.74	8.14	5.56	4.02	2.99	2.44	200.	330.0	0.0	12.5	2.37	300.00	
1125.000	10,737	18.93	13.04	8.73	6.05	4.44	3.37	2.71	200.	245.6	0.0	11.8	4.34	300.00	
2102.000	10,705	13.57	8.80	5.73	4.09	2.96	2.15	1.63	200.	322.2	0.0	17.7	5.51	247.69	
2989.000	11,225	9.64	6.72	4.56	2.93	2.11	1.64	1.41	200.	526.1	0.0	24.3	4.07	300.00	
4002.000	10,745	13.24	9.03	6.49	4.66	3.37	2.52	1.96	200.	438.8	0.0	15.4	4.59	300.00	
4999.000	10,383	14.98	9.68	5.96	3.96	2.84	2.13	1.74	200.	229.8	0.0	17.0	4.50	300.00	
6009.000	10,780	16.11	12.30	9.19	6.51	4.72	3.31	2.67	200.	418.9	0.0	10.8	0.97	207.97	
7002.000	10,598	16.31	8.95	5.77	3.47	2.12	1.30	0.96	200.	151.9	0.0	20.1	3.84	122.79	
8024.000	11,019	13.67	9.72	5.98	3.76	2.36	1.62	1.22	200.	257.7	0.0	19.3	3.85	139.64	
8995.000	10,828	13.88	10.01	6.59	4.19	2.71	1.92	1.79	200.	288.7	0.0	17.2	2.90	165.19	
10000.000	11,519	11.88	7.83	5.34	3.85	2.59	2.01	1.52	200.	445.4	0.0	20.5	4.57	205.65	
10998.000	11,063	15.48	11.77	8.20	5.59	4.21	3.30	1.76	200.	404.0	0.0	12.4	3.98	113.24	
12002.000	10,510	18.83	14.39	10.24	6.16	4.39	3.20	2.80	200.	241.0	0.0	10.8	3.79	230.22	
12995.000	10,836	12.58	10.01	7.60	5.48	3.94	2.94	2.32	200.	600.0	0.0	12.7	1.44	300.00	*
14000.000	10,796	13.83	10.60	7.61	4.45	3.44	2.71	2.21	200.	383.4	0.0	14.5	5.35	167.95	
14999.000	8,623	28.78	20.47	6.50	3.91	2.97	2.58	2.23	200.	47.2	0.0	11.3	17.80	54.23	
15995.000	10,189	23.92	15.53	9.26	5.63	3.73	2.78	2.26	200.	108.8	0.0	11.4	2.63	208.87	
17001.000	10,415	19.59	12.52	7.55	4.26	3.04	1.76	1.56	200.	128.8	0.0	15.0	6.01	132.19	
17999.000	11,313	8.16	4.66	2.96	2.03	1.51	1.19	0.98	200.	457.2	0.0	36.9	9.12	300.00	
19012.000	10,808	14.51	10.01	6.53	4.20	2.79	2.03	1.65	200.	267.4	0.0	17.0	1.47	196.53	
19999.000	10,784	14.02	9.25	5.09	3.03	2.00	1.44	1.07	200.	196.9	0.0	21.3	3.77	185.36	
20987.000	10,502	19.11	13.87	9.39	6.63	4.78	3.70	2.94	200.	265.9	0.0	10.5	3.31	300.00	
22026.000	10,812	15.87	12.82	9.97	7.39	5.35	3.90	2.95	200.	555.7	0.0	9.1	0.32	271.18	
23101.000	10,765	11.57	7,87	5.24	3.04	2.02	1.30	1.03	200.	302.6	0.0	22.5	4.52	151.95	
23996.000	10,411	24.00	14.21	8.06	4.49	2.69	1.78	1.24	200.	84.7	0.0	14.5	7.53	121.49	
25006.000	11,380	9.77	7.61	5.59	3.91	2.71	1.86	1.41	200.	600.0	0.0	20.0	2.72	175.22	*
25997.000	10,590	16.78	12.15	7.94	5.12	3.38	2.58	1.78	200.	237.0	0.0	13.7	2.78	193.20	
27008.000	10,415	12.32	9.62	6.67	4.46	3.02	2.10	1.52	200.	414.0	0.0	15.6	2.61	193.95	
27700.000	10,820	9.15	6.76	4.70	3.08	2.17	1.52	1.26	200.	581.7	0.0	22.8	1.69	195.20	
Mean:		15.40	10.76	6.95	4.55	3.19	2.33	1.83	200.	328.7	0.0	16.5	4.22	201.25	
Std. Dev:		4.65	3.19	1.78	1.27	0.97	0.77	0.61	0.	157.2	0.0	5.7	3.22	112.86	
Var Coeff(%):		30.18	29.64	25.64	27.96	30.48	33.00	33.45	0.	47.8	0.0	34.8	76.22	56.08	

SECTION 24

Road	County	Limits	Date	Structure
FM 2446	Robertson	Intersection of FM 46 to 1.0 Mi E	---	-----

RATING Performance (97/98) A/A Structural (97/98) A/B PI<15

No plan thicknesses or construction details are available for this pavement section on FM 2446. However, it is known that on this project, new base material was placed on the surface prior to recycling the pavement. In 1997, no apparent problems were found with GPR and FWD, and no distresses were observed. In 1998, the FWD data increased in the middle of the project, but the performance remained very good, with no major surface distresses. The average base moduli for the section remained constant at around 2000 Mpa (300 ksi). This section should be monitored for long-term changes. Figure 33 summarizes the target deflection profile for FM 2446, and Table 27 summarizes analysis of the FWD data for this project.

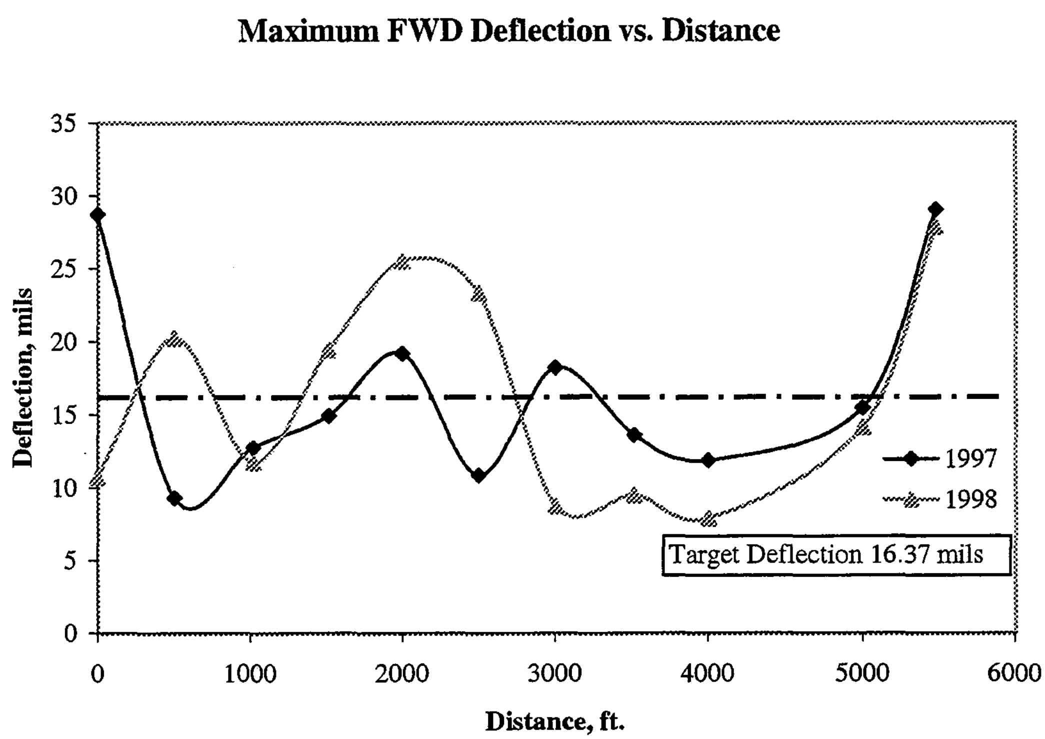


Figure 33. FM 2446, Target Deflection Analysis.

Table 27. FM 2446, FWD Analysis.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 0		Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
County: 244	Pavement:	0.70	199,980	200,020	H1: ¥ = 0.35
Highway/Road: Robe	Base:	10.00	25,000	1,000,000	H2: ¥ = 0.30
	Subbase:	0.00	0	0	H3: ¥ = 0.40
	Subgrade:	155.40	10,000		H4: ¥ = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
0.000	9,060	28.70	20.45	9.56	4.96	3.48	2.75	2.06	200.	63.3	0.0	9.4	9.90	95.47
501.000	10,093	9.30	5.85	4.06	2.93	2.22	1.78	1.51	200.	605.8	0.0	20.7	10.67	300.00
1018.000	9,672	12.74	8.68	5.94	4.09	2.74	0.72	1.37	200.	237.4	0.0	18.2	26.55	55.88
1516.000	9,545	14.92	9.22	5.23	3.26	2.24	1.63	1.31	200.	166.5	0.0	17.6	3.67	258.51
2001.000	9,172	19.14	12.34	6.19	3.92	2.69	2.07	1.62	200.	111.5	0.0	13.7	6.02	263.29
2501.000	9,847	10.85	7.54	4.95	3.22	2.20	1.59	1.25	200.	376.1	0.0	18.8	2.84	240.38
3006.000	9,791	18.17	14.28	10.08	6.74	4.53	3.11	2.28	200.	275.8	0.0	8.9	2.31	194.17
3520.000	9,684	13.58	10.70	7.67	5.26	3.66	2.54	1.86	200.	423.2	0.0	11.0	1.05	195.76
4003.000	9,561	11.85	10.09	7.40	5.10	3.57	2.36	1.81	200.	548.7	0.0	10.9	3.17	155.50
4503.000	9,668	30.13	19.02	11.17	7.60	4.47	2.90	1.94	200.	79.2	0.0	8.4	5.20	115.17
5002.000	9,597	15.44	11.31	7.37	4.79	3.30	2.31	1.87	200.	255.2	0.0	12.3	2.08	208.17
5480.000	9,211	29.02	18.27	8.98	4.68	2.89	2.29	1.90	200.	57.5	0.0	10.6	8.98	95.89
Mean:		17.82	12.31	7.38	4.71	3.17	2.17	1.73	200.	266.7	0.0	13.4	6.87	166.07
Std. Dev:		7.46	4.73	2.22	1.40	0.81	0.66	0.32	0.	187.4	0.0	4.3	6.98	96.48
Var Coeff(%):		41.85	38.44	30.03	29.66	25.65	30.60	18.46	0.	70.3	0.0	32.2	101.66	58.10

SECTION 25

Road	County	Limits	Date	Structure
FM 2780	Washington	FM 1697 to FM 1697	---	-----

RATING Performance (97/98) A/C Structural (97/98) C/C PI>35

FM 2780 is a recently constructed project which was added to the evaluation because it was experiencing rapid deterioration. The first 1.61 km (1 mi) of the pavement section looked bad. It currently has extensive cracking and little structural strength. It was noted that the subgrade contains excessive organics (peat). It appears that the subgrade and base blend did not react with the lime stabilizer. The subgrade and base are weak in this area. In 1998, severe structural deterioration was noted in this area.

After the first 1.61 km (mi), the remainder of section 11.26 (7 mi) looked fairly good as far as the 1997 FWD data were concerned. However, some severe cracking was observed, and the condition appears to be deteriorating rapidly. This project is a failure. Figure 34 summarizes the deflection profiles for FM 2780, and Tables 28 and 29 summarize analysis of the 1997 FWD data for this project.

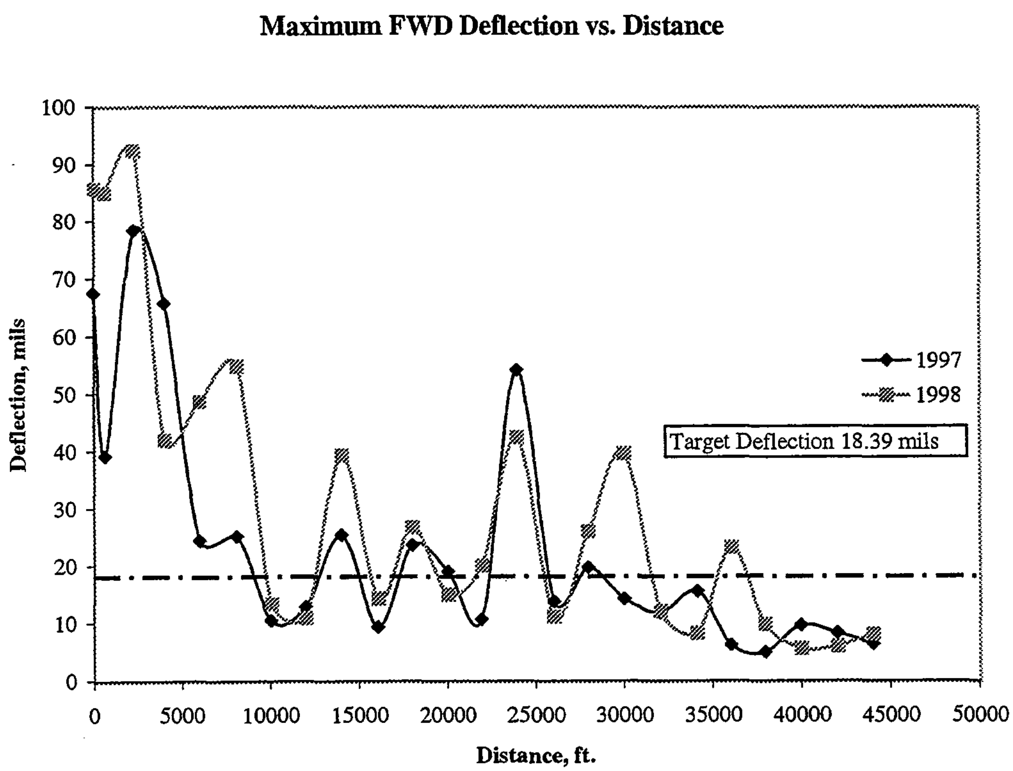


Figure 34. FM 2780, Target Deflection Analysis.

Table 28. FM 2780, FWD Analysis 9 - 11.58 km (0 - 7.2 miles).

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 0			MODULI RANGE(psi)	
County: 278		Thickness(in)	Minimum / Maximum	Poisson Ratio Values
Highway/Road: Washi	Pavement:	0.70	199,980 / 200,020	H1: ¥ = 0.35
	Base:	10.00	5,000 / 2,500,000	H2: ¥ = 0.30
	Subbase:	0.00	0 / 0	H3: ¥ = 0.30
	Subgrade:	107.50	10,000	H4: ¥ = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
5.000	8,027	67.54	32.26	11.40	6.33	5.65	3.26	3.49	200.	15.0	0.0	4.9	16.81	62.05
163.000	7,706	90.29	42.18	13.09	6.94	5.48	4.14	3.74	200.	9.7	0.0	4.1	18.50	58.67
607.000	8,178	39.30	21.98	7.43	3.04	1.95	1.55	1.44	200.	25.5	0.0	9.1	19.42	58.00
2274.000	8,127	78.48	35.41	11.65	5.50	3.90	2.79	2.15	200.	11.5	0.0	5.3	15.10	59.04
4002.000	8,758	65.85	22.00	4.73	2.04	1.22	1.03	0.66	200.	10.8	0.0	13.3	28.11	92.58
6003.000	8,846	24.49	17.99	10.65	6.01	3.86	2.80	2.11	200.	117.8	0.0	7.0	5.79	143.56
8113.000	8,858	25.17	18.03	10.09	5.65	3.24	1.96	1.38	200.	95.5	0.0	7.7	7.18	111.37
10053.000	9,847	10.49	8.53	6.31	4.51	3.13	2.22	1.59	200.	801.2	0.0	10.2	2.28	200.43
12023.000	9,720	12.93	10.15	7.02	4.72	3.14	2.20	1.59	200.	471.0	0.0	10.0	3.25	201.04
14018.000	8,905	25.41	20.11	13.04	7.65	4.50	2.73	1.79	200.	132.2	0.0	5.8	6.40	124.87
16075.000	9,446	9.37	7.10	5.17	3.63	2.40	1.72	1.30	200.	725.9	0.0	12.8	2.97	184.73
18002.000	9,819	23.56	18.22	11.02	6.31	4.02	2.68	1.93	200.	150.7	0.0	7.4	4.84	155.81
20024.000	9,176	19.01	17.51	14.07	10.67	7.60	5.31	3.80	200.	559.2	0.0	3.6	2.52	218.85
22002.000	9,601	10.72	7.71	5.80	4.01	2.83	2.02	1.51	200.	674.9	0.0	11.5	4.66	224.02
24003.000	8,611	54.15	21.52	5.89	3.03	2.28	2.11	1.37	200.	15.9	0.0	10.4	20.11	58.45
26028.000	9,958	13.73	9.79	5.85	3.25	1.75	1.10	0.94	200.	228.9	0.0	15.7	7.36	90.14
27981.000	9,354	19.69	15.85	11.40	7.59	5.30	3.73	2.72	200.	324.4	0.0	5.7	2.95	235.09
30001.000	9,569	14.26	11.88	8.39	5.52	3.89	2.87	2.26	200.	481.3	0.0	7.9	4.55	300.00
32037.000	9,549	11.88	10.52	7.79	5.28	3.56	2.51	2.15	200.	653.6	0.0	8.3	3.87	216.89
34119.000	9,183	15.54	14.19	11.04	8.01	5.46	3.48	2.24	200.	550.8	0.0	5.2	3.86	142.24
36047.000	10,375	6.21	5.66	4.91	3.87	3.23	2.60	2.11	200.	2500.0	0.0	9.8	6.91	300.00 *
38004.000	10,308	5.01	4.74	4.26	3.21	2.57	1.96	1.52	200.	2500.0	0.0	13.2	8.50	299.69 *
Mean:		29.23	16.97	8.68	5.31	3.68	2.58	1.99	200.	502.5	0.0	8.6	8.91	118.24
Std. Dev:		25.25	9.76	3.12	2.06	1.52	0.99	0.83	0.	699.9	0.0	3.3	7.29	70.78
Var Coeff(%):		86.38	57.52	35.95	38.86	41.31	38.44	41.83	0.	100.0	0.0	38.6	81.81	59.86

Table 29. FM 2780, FWD Analysis 11.58 - 12.87 km (7.2 - 8.0 miles).

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1

District: 0
County: 0
Highway/Road:PxNnnn

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	0.70	199,980	200,020	H1: ¥ = 0.35
Base:	10.00	5,000	2,500,000	H2: ¥ = 0.30
Subbase:	0.00	0	0	H3: ¥ = 0.30
Subgrade:	289.30	10,000		H4: ¥ = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
0.000	9,986	9.68	8.18	6.46	4.90	3.67	2.76	2.10	200.	966.6	0.0	14.0	1.38	274.74
2023.000	10,073	8.33	6.88	5.43	4.20	3.22	2.50	1.91	200.	1246.2	0.0	16.0	1.11	274.98
4032.000	10,546	6.37	5.76	4.98	4.18	3.46	2.89	2.40	200.	2500.0	0.0	15.2	4.96	300.00 *
Mean:		8.13	6.94	5.62	4.43	3.45	2.72	2.14	200.	1570.9	0.0	15.1	2.48	300.00
Std. Dev:		1.66	1.21	0.76	0.41	0.23	0.20	0.25	0.	816.6	0.0	1.0	2.15	39.03
Var Coeff(%):		20.48	17.45	13.49	9.26	6.53	7.31	11.56	0.	52.0	0.0	6.9	86.53	13.01

CHAPTER 4
FAILURE INVESTIGATION

GENERAL

Of the 25 projects evaluated in this study, pavement structural distress was observed in a few projects, including FM 542-2 in Leon County, from 8 km (5 mi) south of Oakwood to Shiloh; FM 2780 in Washington County, from FM 1697 to FM 1697; FM 977 in Leon County, from SH 75 to FM 1119; and FM 246 in Freestone County, from FM 27 to 5.3 km (3.3 mi) east. Unusually high deflections in the pavement structure were also observed in FM 111 in Burleson County, from FM 60 to 5.4 km (3.4 mi) east. In view of the anomalies encountered in the above mentioned projects, it was decided to conduct additional investigations to identify factors influencing the cause of severe longitudinal cracking.

FIELD CORING

The following additional field investigations were conducted for the three sites: FM 111, FM 246, and FM 542-2:

- Two boreholes, each 6 m (20 ft) deep, were drilled at each of the three sites. One borehole was drilled through the pavement structure as close as possible to the crack. The second borehole was drilled through the ditch (500 mm (20 in) below the pavement surface), 4.3-6 m (15-20 ft) away from the center line of the pavement.
- Continuous shelby tube samples were obtained from the top 3 m (10 ft) of the borehole. In the bottom 3 m (10 ft) of the borehole, samples were obtained at .6 m (2 ft) intervals.
- In addition to sampling in boreholes, 200 mm (8 in) diameter samples were taken from the in-situ pavement structure. Two samples each were taken from FM 542-2 and FM 246, and one sample was taken from FM 111. Photographs of the cores are shown in Figures 35, 36, and 37.
- Exhaustive laboratory testing was conducted to determine Atterberg Limits, particle size distribution, moisture content, density, and strength. Additional bore holes were drilled on FM 2780 and FM 977.

Figure 35. Core from FM 111.

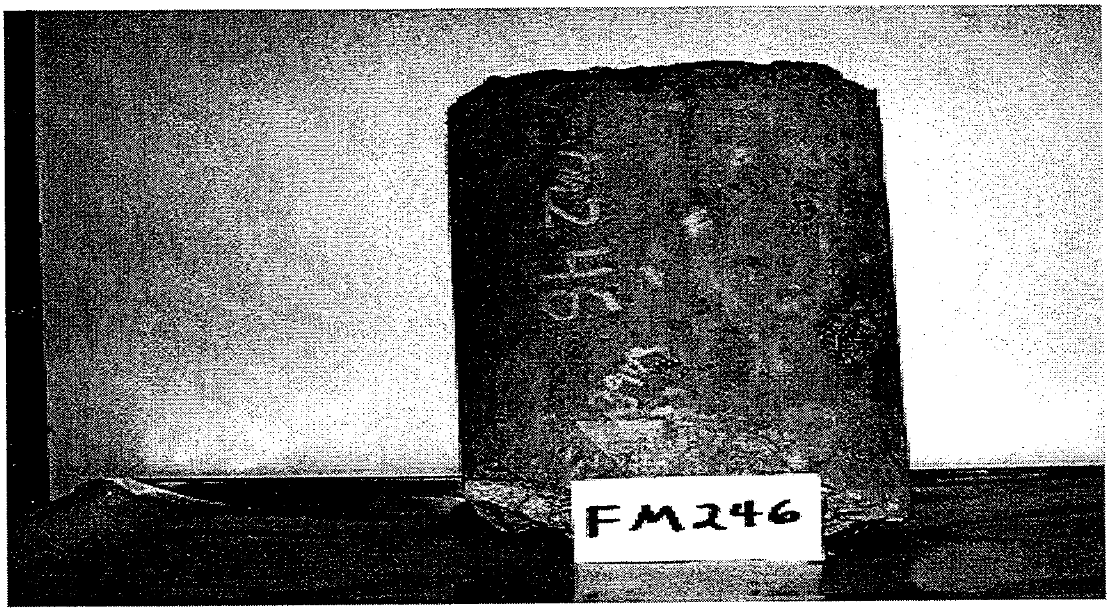


Figure 36. Core from FM 246.

Figure 37. Core from FM 542-2.

SITE DESCRIPTION

FM 542-2

The area of distress is located in a fill zone. Figures 38, 39, and 40 give some indication of the site location. The groundwater table was measured in the borehole that was made through the ditch. The depth of the groundwater table was about 4.5 m (15 ft) from the existing ground surface of the ditch.

The subgrade was comprised of hard, light brown clay with traces of sand and calcareous nodules. Field estimates of the compressive strength in both the boreholes indicate a strength of 335 to 383 kN/m^2 (3.5 to 4.0 tsf) and above. Laboratory compression tests yielded values between 298 to 503 kN/m^2 (3.11 to 5.25 tsf), with predominant values around 480 kN/m^2 (5 tsf). Particle size distribution indicates 90 to 98 percent passing the 75 micron (# 200) sieve, with predominant values of 95 percent. Atterberg limits are as follows:

Liquid Limit, %:	51 ± 12
Plastic Limit, %:	21 ± 3
Plasticity Index, %:	34 ± 11
Moisture Content, %:	19 ± 6
Dry Density, kN/m^3:	14 ± 1 (108 ± 9 pcf)

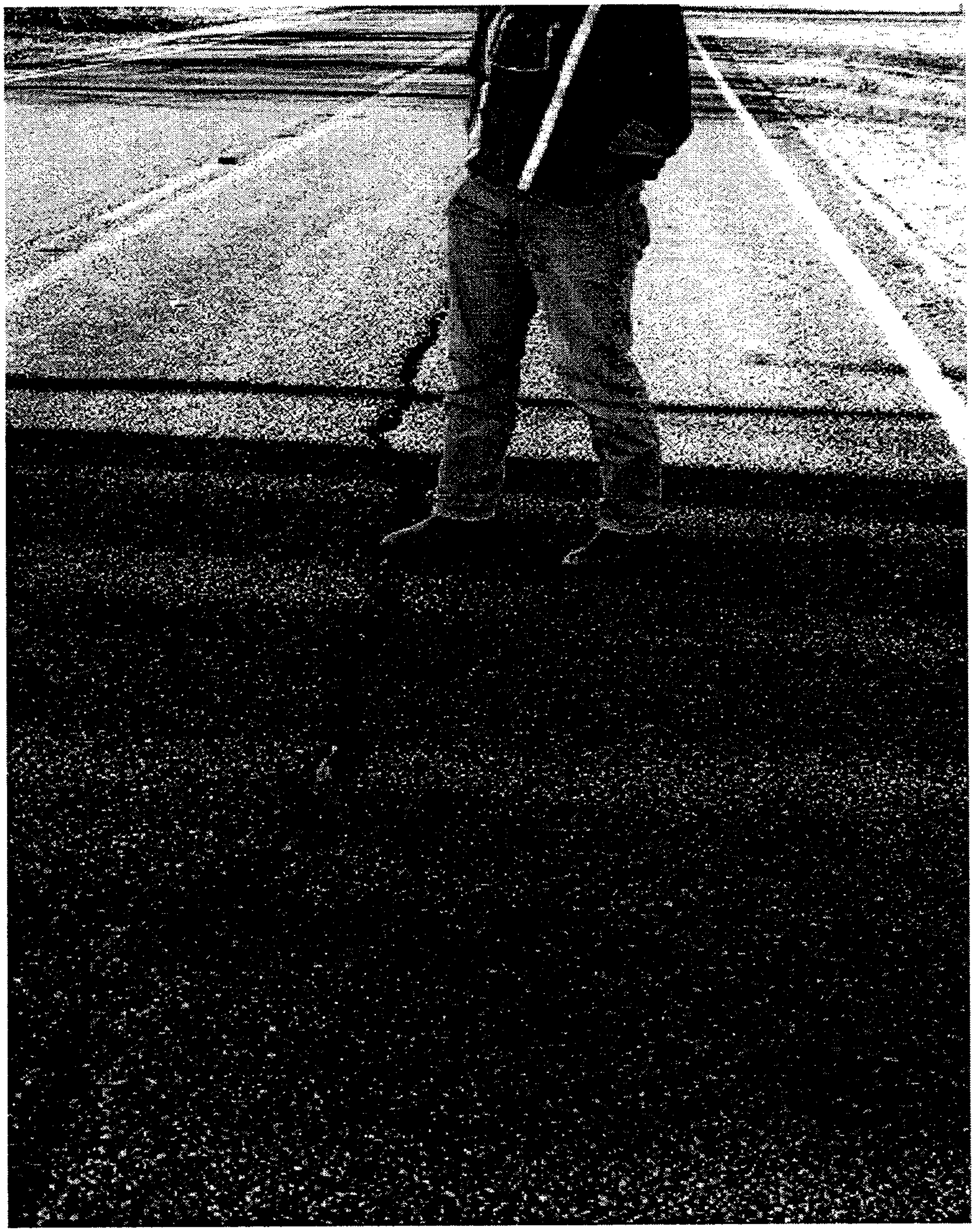


Figure 38. Longitudinal Crack in the Middle of the Lane, FM 542-2.

Figure 39. Longitudinal Crack Along the Edge of the Pavement Structure, FM 542-2.

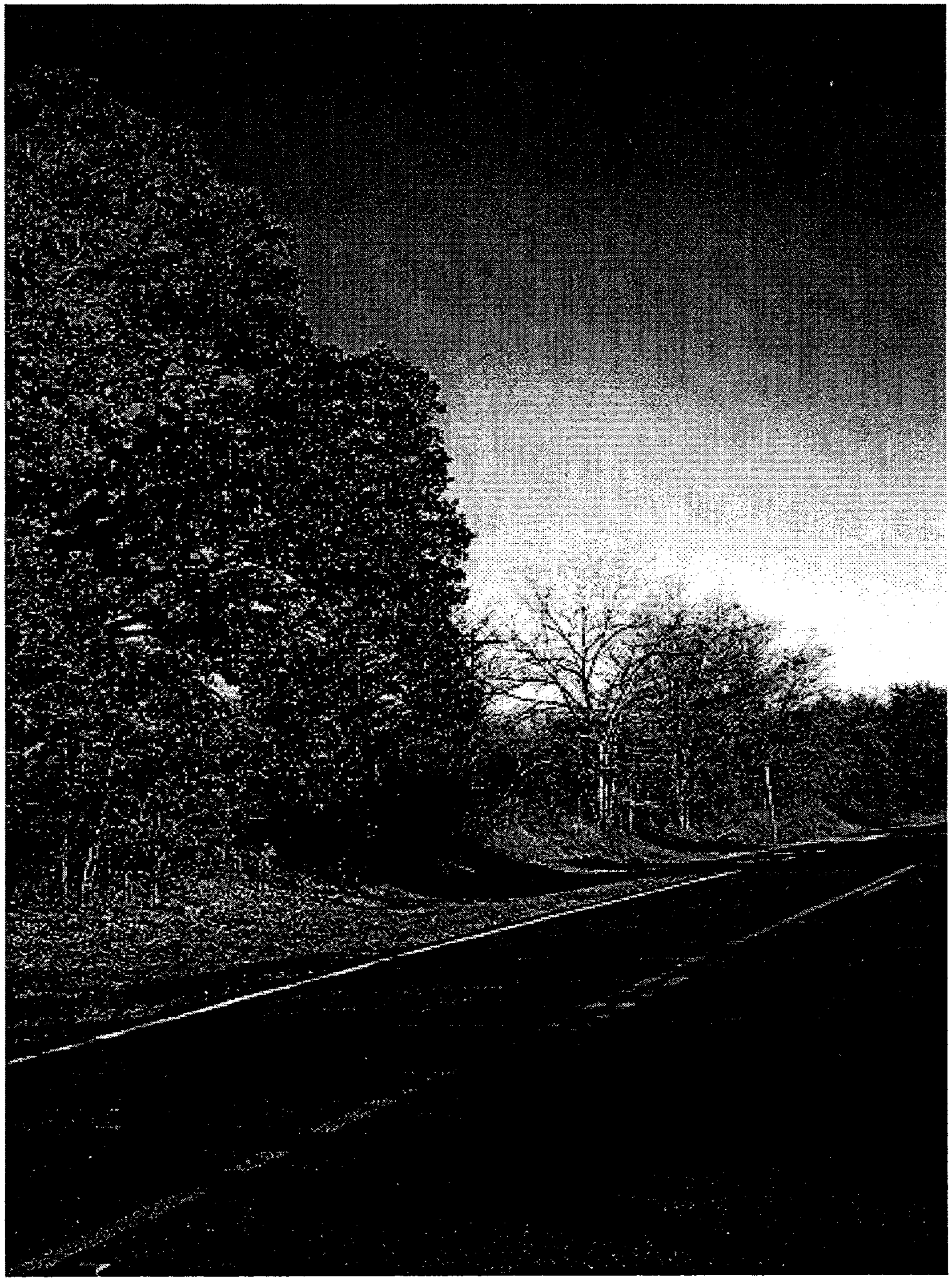


Figure 40. Large Trees Adjacent to the Cracked Section.

FM 111

Unusually high deflections were observed in isolated spots along this road. The groundwater table was measured in the borehole that was made through the ditch. The depth of the groundwater table was 4.57 m (15 ft) from the existing ground surface of the ditch.

The subgrade was comprised of hard, light brown clay with traces of sand and calcareous nodules. Field estimates of the compressive strength (ASTM D 2573-72) indicate a strength of 335 to 383 kN/m^2 (3.5 to 4.0 tsf) and above up to 3.05 m (10 ft) below the existing pavement level. At lower depths, i.e., 3.05 m to 6.10 m (20 ft), the field compressive strength drops to 335 kN/m^2 (3.5 tsf). The borehole that was made through the ditch indicated the field compressive strength between 192 to 383 kN/m^2 (2.0 to 4.0 tsf) and above up to 4.57 m (15 ft) below the existing ground level. At lower depths, i.e., 4.57 m to 6.10 m (20 ft), the field compressive strength drops to 287 kN/m^2 (3.0 tsf).

Laboratory compression tests yielded values between 230 to 421 kN/m^2 (2.4 to 4.4 tsf), with predominant values around 383 kN/m^2 (4 tsf). Particle size distribution indicates 65 to 98 percent passing the 75 micron (# 200) sieve, with predominant values of 95 percent. The lower limit of particle size passing the 75 microns was observed only in the surface layers. Atterberg limits are as follows:

Liquid Limit, %:	25 to 92 percent with predominant values around 83
Plastic Limit, %:	14 to 31 percent with predominant values around 26
Plasticity Index, %:	11 to 61 percent with predominant values around 56
Moisture Content, %:	10 to 30 percent with predominant values around 28
Dry Density, kN/m^3:	11.35 to 15.13 (90 to 120 lb/ft^3) with predominant values around 12 (95 lb/ft^3)

FM 246

Some longitudinal cracks were also observed on this road. The groundwater table was measured in the borehole that was made through the ditch. The depth of the groundwater table was about 2.24 m (7 ft 4 in) from the existing ground surface of the ditch.

The subgrade was comprised of hard, light brown clay with traces of sand and calcareous nodules. Field estimates of the compressive strength indicate a strength of 239 to 383 kN/m^2 (2.5 to 4.0 tsf) and above. Laboratory compression tests (ASTM D 2166-91) yielded values between 145 to 753 kN/m^2 (1.51 to 7.86 tsf), with average values around 436 kN/m^2 (4.55 tsf). Particle size distribution indicates 93 to 98 percent passing the 75 micron (# 200) sieve, with predominant values of 96 percent. Atterberg limits are as follows:

Liquid Limit, %:	49 ± 6
Plastic Limit, %:	18 ± 2
Plasticity Index, %:	32 ± 5
Moisture Content, %:	18 ± 4
Dry Density, kN/m^3:	14 (107 ± 4 lb/ft^3)

FM 2780

The project limits for this road start at FM 1697 and loop back to FM 1697. The performance of this road is extremely variable. In the first 800 m (0.5 mi), longitudinal cracks and bearing capacity failure were observed. The GPR and FWD data also corroborate with the visual information. After about 1600 m (1 mi), the performance of this road starts improving, and in the last 3200 m (12 mi) the stabilized base is extremely stiff and dry. This is reflected from the FWD and GPR data.

Four boreholes were drilled on this road (chainages 152 m (500 ft), 4300 m (14,000 ft), 9150 m (30,000 ft), and 12,200 m (40,000 ft)). The groundwater table was very shallow in the first borehole, at a chainage of 152 m (500 ft). The sub-surface material encountered in this borehole had a very high moisture content. The groundwater table was not encountered in the remaining three boreholes drilled on this road. The subsurface materials encountered in the remaining three boreholes at chainages 4300 m (14,000 ft), 9150 m (30,000 ft), and 12,200 m (40,000 ft) were very dry.

The subsurface was comprised of a weak and nearly disintegrated recycled- and lime-stabilized base. The subgrade was comprised of a stiff tan and white to gray sandy clay with calcareous nodules. Field estimates of the compressive strength indicate a strength of 192 to 383 kN/m^2 (2.0 to 4.0 tsf). The sections at chainages 4300 m (41,000 ft), 9150 m (30,000 ft) and 12,200 m (40,000 ft) had fill material varying in depth from 0.6 m (2 ft) to 3 m (10 ft).

FM 977

The additional bore holes drilled on this road actually comprised two project sections. One was between the limits of SH 75 and 7250 m (4½ mi) east, whereas, the other was from 7250 m (4½ mi) east of SH 75 to FM 1119. Two bore holes were drilled in the section between SH 75 and 7250 m east, while one bore hole was drilled in the section between 7250 m (4½ mi) east of SH 75 to FM 1119.

DISCUSSION OF NON-DESTRUCTIVE TESTING DATA

FM 542-2

Severe longitudinal cracking was found between two bridge structures (approximately 9.5 km (6 mils) south of US 79, 1659 m (5508 ft) from the start of the section). The subgrade modulus is very low in this area, approximately 50 percent less than the average subgrade moduli. The DCP reading indicated very poor subgrade in the cracked areas, noted as heavy clays. Additional site investigations have been conducted in this area, with drilling and sampling down to 6.1 m (20 ft).

The subbase is very strong in this location, with subbase moduli of 7970 MPa (1155 ksi). The average moduli for the stabilized layer is stiff for this section, which could be related to severity of cracking over the heavy clays. Outside the cracked areas, deflections are low, and no distresses are present.

FM 246

The average deflection of 391 microns (15.4 mils) is less than the target deflection of 399 microns (15.7 mils). The high deflections in the first 300 m (1000 ft) and throughout the section can be attributed to low values of the subgrade moduli. The values of the subbase moduli are low in the first 300 m (1000 ft); this could be due to "start-up" problems.

FM 111

The FWD analysis on this pavement section indicates highly variable results. The first half of the section has reasonable deflections and good back-calculated moduli values. The second half is considerably weaker. At location 3902 m (12999 ft), the measured deflection of 1092 microns (43 mils) was more than twice the target value of 483 micron (19 mils). This section was stabilized with only 3 percent lime. This may have been insufficient. Several locations indicated poorer subgrade strength in the weak areas. The section from 3600 m to 4800 m (12,000 ft to 16,000 ft) appears similar to an unstabilized flexible base section with high deflection values; there is little apparent benefit from lime. It is anticipated that this section may deteriorate under load.

This is an interesting section since it is built on a poor subgrade of highly plastic clays, but currently there is little or no distress.

FM 2780

The FWD analysis on this pavement section indicates highly variable results. The first 1.61 km (1 mi) of the pavement section was experiencing rapid deterioration, and bearing capacity failure of the supporting materials was observed. Unsuccessful attempts were made to extract cores of the stabilized base material. The cores completely disintegrated. From visual observations, it looked like the blend of subgrade and existing pavement material did not react with the lime stabilizer. Moisture content of the base material was very high in the first 1.61 km (1 mi) of the pavement section. This observation is supported by the GPR and FWD data along with the bore hole information.

For the remainder of the section, bearing capacity failure was not observed, but some severe cracking was observed, and the condition appears to be deteriorating rapidly. The magnitude of the problems encountered on this project is highly variable. This highway section will be the subject of ongoing study to explain the different failure mechanisms observed in this project.

INITIAL CONCLUSION

A Triaxial Compression Test (Test Method Tex-117-E) was conducted on the cores from the three sites. Table 30 contains the results:

Table 30. Preliminary Test Results.

Road	Triaxial Compr. Strength	Moduli of stabilized layer	Distress observed
FM 111	1647.6 kPa	1492 MPa	High Deflections, no cracking
FM 246	5864.4 kPa	4767 MPa	Longitudinal cracking in few areas
FM 542-2	6288.2 kPa	5652 MPa	One big longitudinal crack

From the above-tabulated results, it appears that the severity of cracking in the pavement layers is correlated to the high compressive strength and the high moduli values of the stabilized pavement layers. Although FM 111 has a few areas with higher deflections, little or no distress was observed on this pavement structure in the first year of study. However, the visual survey carried out in the second year noted the beginning of load-associated distress. FM 542 and FM 246 have achieved a higher compressive strength and a very high moduli for the stabilized layer, but severe localized cracks were observed along the pavement surface. One of the cracks observed on FM 542-2 was approximately 30-50 mm (1.2-2.0 in) wide, 900 mm (36 in) deep, and extend longitudinally for about 300 m (1000 ft). These clearly are subgrade cracks associated with edge drying.

The deflections on FM 542-2 are also very low compared to the target deflection, thereby indicating that the layers are very stiff. The deflections on FM 246 are also on the lower side, as compared to the target deflection value. However, the deflection on FM 111 is much higher, compared to the target deflection value.

It seems that the cracking of these pavement layers is due to both the plasticity of the subgrade soils and to the extremely high compressive strength and moduli values of the stabilized subbases. As discussed with Craig Hogan, P.E. of TxDOT, other issues that are impacting the cracking are the steepness and height of the front slope, the presence of trees, and the summer drought of 1996.

CHAPTER 5

CORRELATION OF DEFLECTION PARAMETERS

GENERAL

Correlations between layer moduli, dynamic cone penetrometer, stiffness coefficients calculated from Dynaflect, and stabilizer content were prepared. The general trend of the correlations, the regression analysis performed, and the suggested design line depend on a number of variable factors. The design charts presented in this chapter may not be the solution to all the problems, but they are a first attempt to develop design guidance for recycled and stabilized pavement layers.

With more research and performance monitoring, these design charts can be modified to provide an adequate design methodology for recycled and stabilized pavement layers.

The object of any design procedure is to produce a quality product which will perform according to the expectations of the designer. Reliability is defined as the probability that a product or system will perform a specified function for a specified time without failure, or reliability is 1.0 minus the risk of failure. Whether we design a new pavement or we design a pavement rehabilitation scheme with recycled and stabilized pavement layers, one has to bear in mind that what is being designed does not yet exist. Design of pavements is a process in which the variability and uncertainty in the factors that will control the performance are taken into account in providing an acceptable level of risk that the product will meet its performance expectations (*7*).

There is a lot of scatter in the data in the correlations. The design line was selected by subtracting one standard deviation from the best fit lines. It can be observed that this suggested design line generally falls near or below the lower bound envelope of the data points.

GROUND PENETRATING RADAR

It has been shown in the past, and it has been proved yet again in this project, how useful a tool GPR can be in performance evaluation of pavement layers in terms of checks on consistency, moisture trapping, and confirmation of layer thickness. Chapter 3 describes the projects where GPR was used successfully to interpret the problems in the pavement layers.

CORRELATION OF DCP INDEX WITH SUBGRADE MODULI

Procedures presented in the US Army Corps of Engineers report on *"Description and Application of Dual Mass Dynamic Cone Penetrometer" Report # GL-92-3*, May 1992, were used for measuring soil strength and correlating the DCP index with subgrade moduli values. On selected sites, the DCP was used to estimate a modulus strength with depth to compare the backcalculated values with those computed from the FWD.

Figure 41a illustrates the correlation of the DCP index with the subgrade moduli for CH (highly plastic clay) soils; whereas, Figure 41b shows the correlation of the DCP index with subgrade moduli for CL (clay of low plasticity) soils. The solid line indicates the regression line, or the "best fit" line, for the particular distribution. The dotted line indicates the recommended design line, which was obtained by subtracting the standard deviation from the best fit line.

CORRELATION BETWEEN DYNAFLECT AND FWD

Figures 42a and 42b present a correlation between the stiffness coefficients obtained from the Dynaflect and layer moduli values obtained from the FWD.

For the purpose of developing the base correlation, only three-layer pavement systems—subgrade, stabilized base, and surface layers—were considered. From Figure 42a, all of the average base stiffness coefficients were above the value (0.7) typically used within FPS 11. Figure 42b presents the correlation between the subgrade modulus and subgrade stiffness coefficient. It is apparent that the line is very flat, ranging from 0.22 for low strength materials to 0.27 for good subgrades. When reviewing all of these data, it is important to remember that the correlations are only appropriate for pavements with stabilized bases. These correlations are not appropriate for pavements with unstabilized granular base materials.

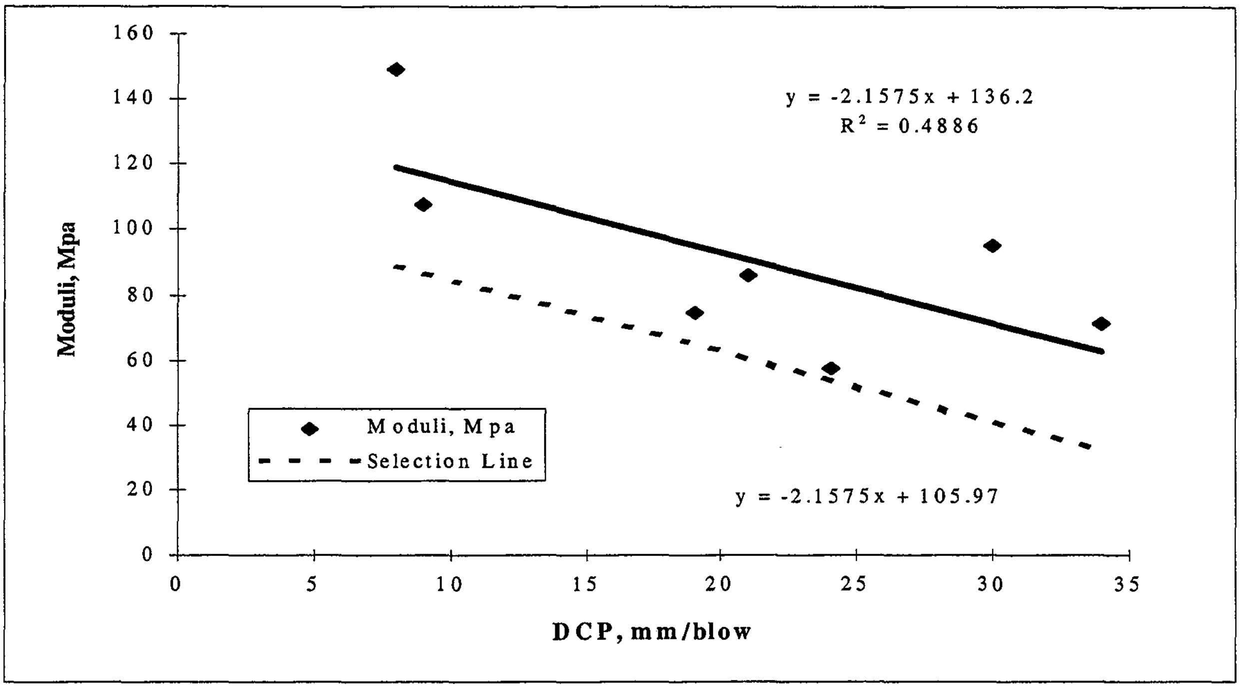


Figure 41a. Correlation of DCP Index v/s. Subgrade Moduli (CH Soils).

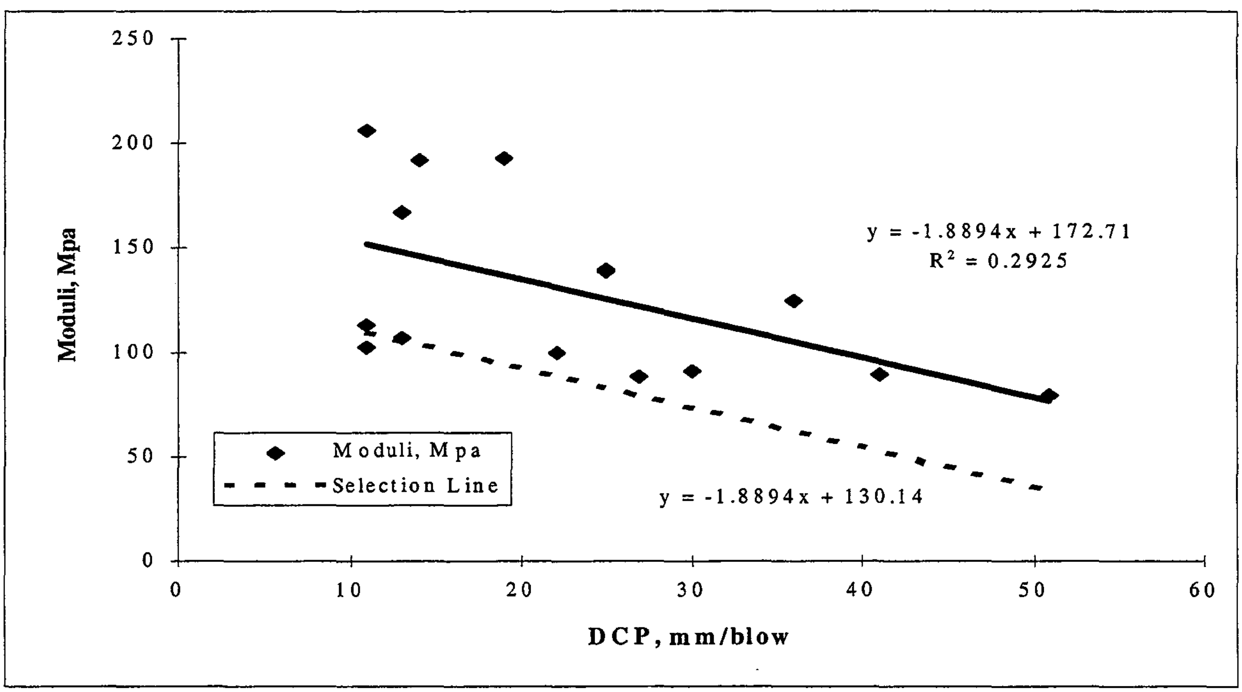


Figure 41b. Correlation of DCP Index v/s. Subgrade Moduli (CL Soils).

***Note: valid only for pavements with stabilized base layers**

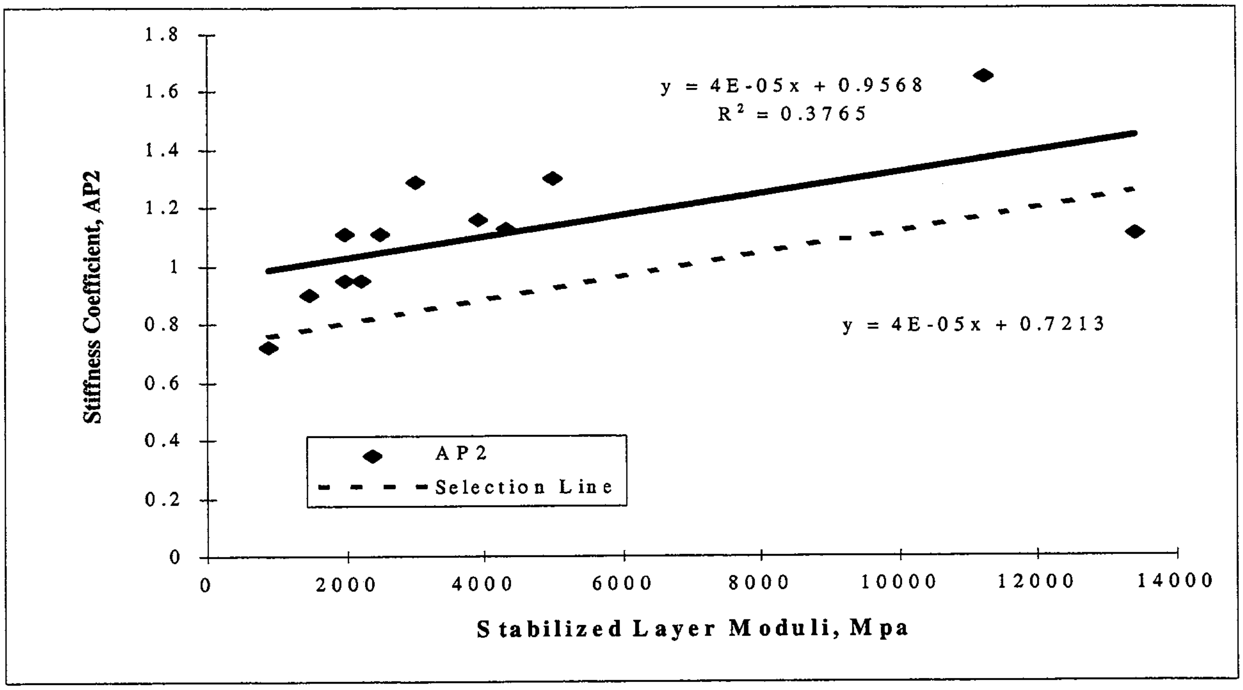


Figure 42a. Correlation between Dynaflect and FWD.

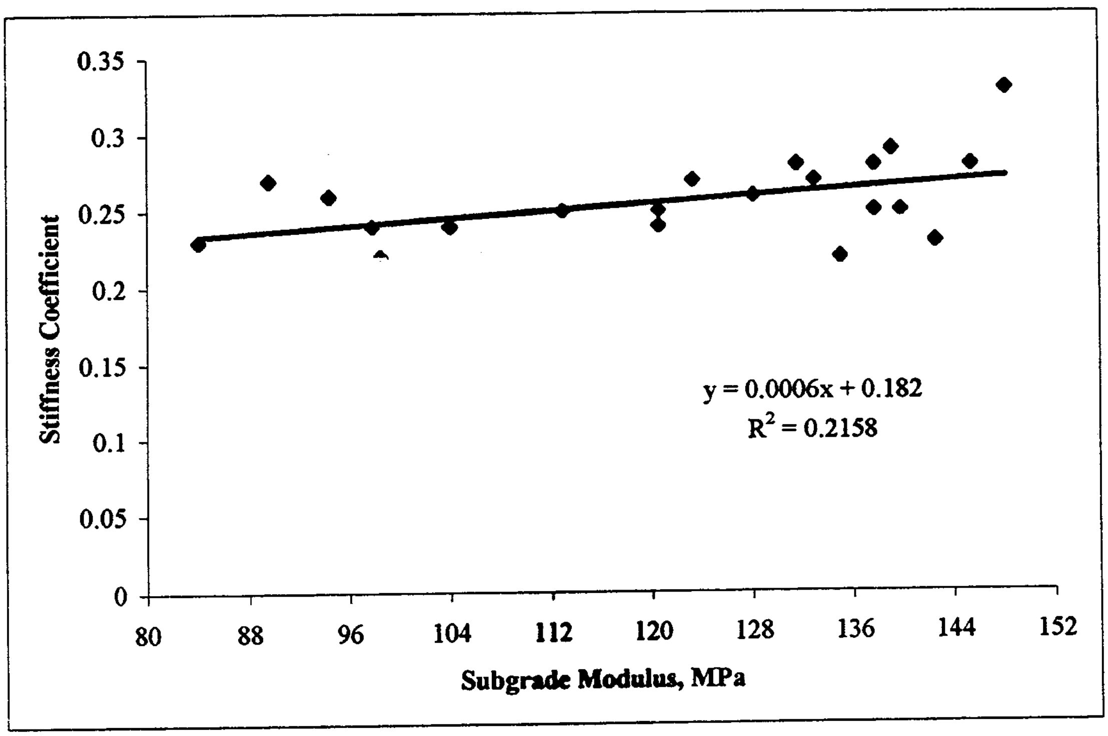


Figure 42b. Correlation of Subgrade Stiffness Coefficient with Subgrade Modulus.

CORRELATION BETWEEN STABILIZER CONTENT AND DYNAFLECT

Figure 43 illustrates the correlation between stabilizer content and Dynaflect stiffness values based on the field-measured stiffness of the stabilized pavement layers.

The solid line in Figure 43 indicates the best fit line based on the field data. The dotted line is the suggested "design line." The design line was obtained by deducting one standard deviation from the average (best fit) line. For the purpose of developing this correlation, only three-layer pavement systems—subgrade, subbase, and surface layers—were considered.

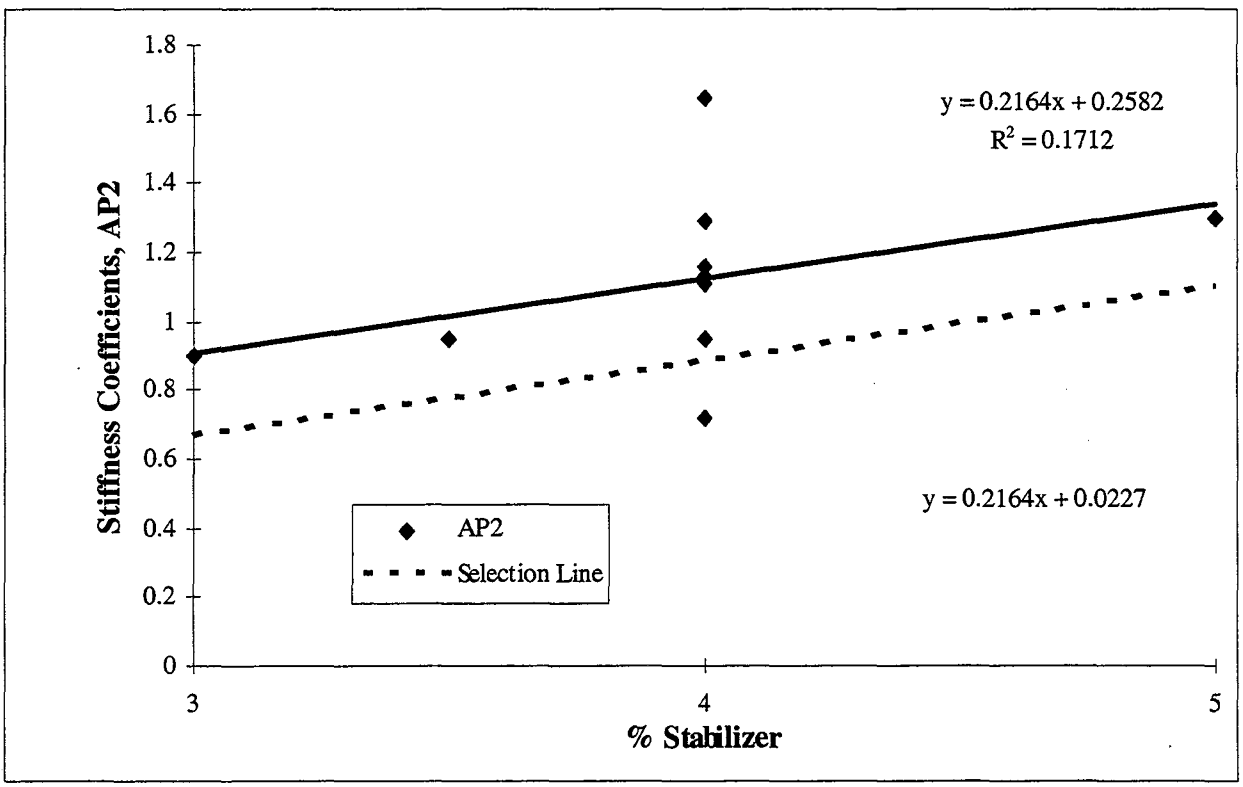


Figure 43. Correlation between Stabilizer Content and Dynaflect.

CORRELATION BETWEEN STABILIZER CONTENT AND FWD

Figures 44 and 45 present the correlations between the stabilizer content and the layer moduli values, as calculated from FWD and MODULUS 5.1, based on the field-measured stiffness of the stabilized pavement layers. Figure 44 indicates the correlation for a three-layer pavement system, while Figure 45 indicates the correlation for a four-layer pavement system. The solid line in the figures indicate the average line based on the field performance data. The dotted line is the suggested "design line." The design line was obtained by deducting one standard deviation from the average (best fit) line.

In Figure 45, the correlation line and the design line show no increase in the moduli of the stabilized pavement layer beyond 6 percent of stabilizer. This is because it has been found in study 1287 (5) that stabilizer in excess of 6 percent does not serve any useful purpose in terms of strength gain.

For the purpose of developing this correlation, both three-layer and four-layer pavement systems—subgrade, subbase, base, and surface layers—were considered.

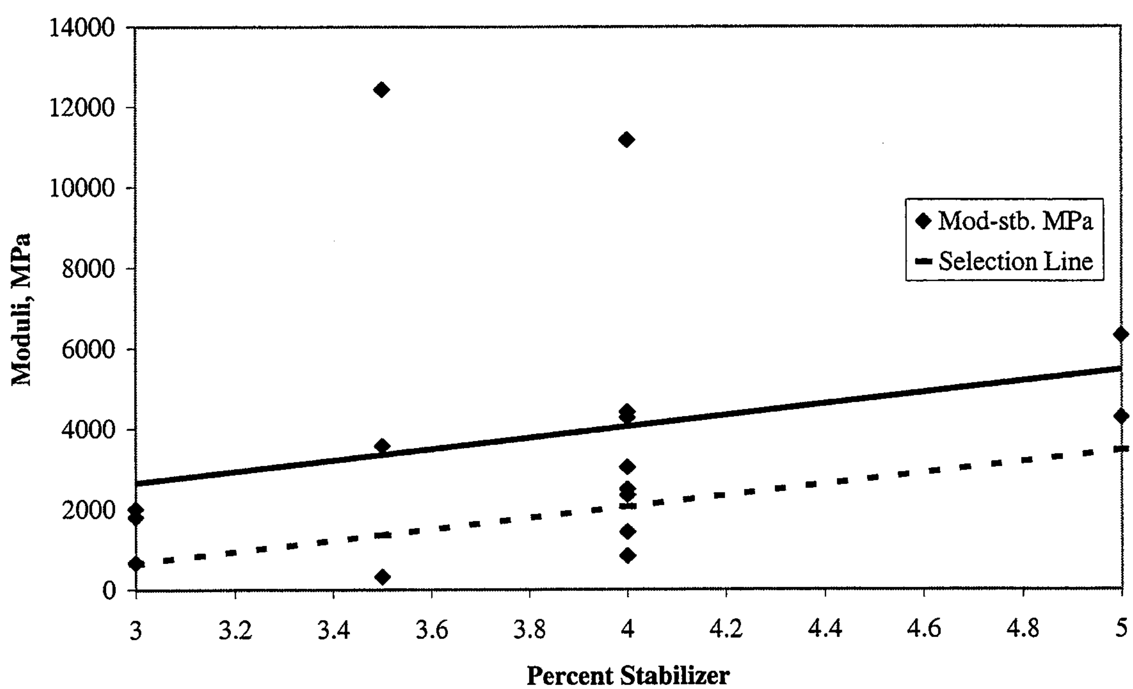


Figure 44. Correlation between Stabilizer Content and FWD (Three-Layer).

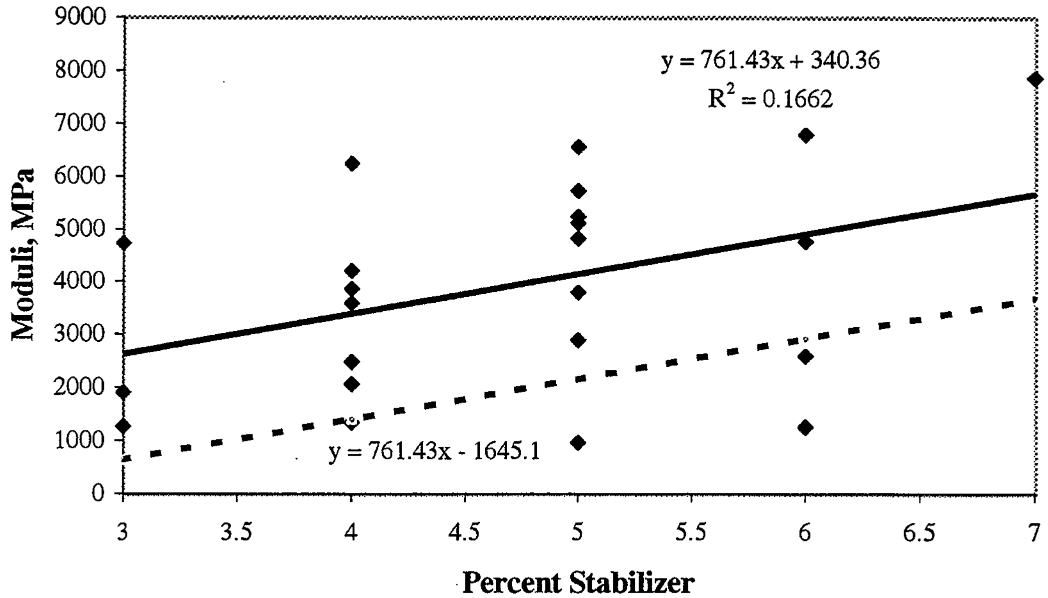


Figure 45. Correlation between Stabilizer Content and FWD (Four-Layer).

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS TO IMPROVE RECLAIMED PAVEMENT PERFORMANCE

CURRENT DESIGN APPROACH

The Bryan District's current design approach for reclaimed pavements is summarized below. Their goal is to chemically stabilize the top 250 mm (10 in) of the existing structure and to use it as either a base or subbase layer. In the case of a subbase layer, a granular overlay is placed over the treated layer followed by a two-course surface treatment. The current design procedure includes the following steps:

1. At approximately 1.6 km (1 mi) intervals, core each project to a depth of 2 m (7 ft); log the pavement structure, and return samples to the laboratory for testing.
2. For each location, complete TxDOT Form 476A, noting the basic soil properties including PI and soil gradation. Of particular interest are the values for the existing base and the top of subgrade.
3. Use the PI and soil binder (percent passing #40 sieve) as inputs to TxDOT method 121-E to determine the amount of lime stabilization required. The goal is to stabilize the top 250 mm (10 in), so for thin structures, a weighted average is often used to arrive at the final stabilizer content. For low PI materials (PI < 10), cement stabilization would be recommended.
4. If a granular base is to be used, its thickness is designed using one of the Department's approved design procedures, either Texas Triaxial, PVR, or the FPS programs.

Several issues arise which make the design process complex. The major issue is the variability of the existing pavement structure. These are low volume roadways which have received substantial maintenance over their life; it is not uncommon to find from 75 to 100 mm (3 to 4 in) of base at one location and 250 to 300 mm (10 to 11 in) at the next. The Bryan District also has a large variability in subgrade soils; in several counties, the soil type can change from sand to expansive clay in the same construction section.

PAVEMENT PERFORMANCE SUMMARY

Chapter 3 presented a discussion of the FWD data and observed pavement performance for each section. In order to summarize these data and demonstrate the impact of soil type on pavement performance summary, Table 31 is presented. It is noted that several of the sections are broken into subsections; this is where the available soils information indicated that there was a significant break in soil type. This information was from either the TxDOT drill logs or the USDA county soil series maps. This is an important table which presents most of the data collected in this study; the key to reading the table is:

Blend % — Percentage of base to subgrade in the top 250 mm (10 in) of existing structure.

Eb — Modulus of the granular base layer in ksi, backcalculated from the FWD data using MODULUS 5.1.

Esb — Modulus of the stabilized layer in ksi, backcalculated from FWD data using MODULUS 5.1. In the case where a granular base is placed over this layer, this layer is the subbase; with no granular base, this is the base layer.

Visual — Pavement performance indicator based primarily on the amount of cracking found in the sections:

- A No Distress,
- B Minor or Localized Cracking, and
- C Major cracks in more than 25 percent of section.

Str. — Structural strength indicator based on FWD deflection data; its variability along the section and its comparison to the target deflection value were discussed in Chapter 3:

- A Deflection profile close to target deflection,
- B Localized weak sections, >10 percent of readings with deflections greater than 50 percent above target deflection, and
- C Several weak locations, >25 percent of readings with deflections greater than 50 percent above target deflection.

Table 31. Summary Results.

	Constr.				1997	1997	1998	1998	1997	1997	1998	1998	
									Performance		Performance		
	Date	Soil Type	% Stabl.	Blend, %	Eb	Esb	Eb	Esb	Visual	Str.	Visual	Str.	
FM3090	May-94	PI<20	3 Lime	60-40	157	276	144	210	A	B	A	B	
FM149	Jun-94	PI>35	7 Lime	60-40	197	1140	176	1220	B	A	C	A	
FM1486	Jul-94	CL, PI<15	5 Lime	60-40	165	700			A	A	A		
		CH, PI>35	5 Lime	60-40	152	140			A	C	C		
FM542-1	Jun-94	CH, PI>25	3 Lime	60-40	107	185	88	122	A	C	A	C	
		SM, PI<10	3 Lime	60-40	149	685	165	780	A	A	A	A	
FM542-2	Apr-95	CH, PI>25	5 Lime	60-40	160	830	175	950	B	B	B	A	
		CL, PI<25	5 Lime	60-40	162	742	141	884	B	B	B	A	
FM3478	May-95	SM, PI<4	6 Lime	80-20	165	985	164	1240	A	A	A	A	
FM1362	May-95	CL, PI<20	4 Lime	60-40	130	360	172	222	B	B	B	B	organics
		CH, PI>45	4 Lime	60-40	127	610	135	650	A	B	C	A	
		CH, PI>25	4 Lime	60-40	85	520	103	460	B	B	C	C	organics
FM246	May-95	CH, PI>35	6 Lime	40-60	115	690	Failed	Failed	C	B	C	Failed	
FM977	May-96	CL, PI<20	3 Lime	60-40	173	420	196	370	A	A	A	A	
		CL, PI>35	3 Lime	60-40	84	951	125	1075	B	A	C	A	
FM 27	Sep-95	CH, PI>45	5 Lime	60-40	68	760	85	670	B	A	B	A	
		CH, PI>35	5 Lime	60-40	89	550	130	450	B	A	B	A	
FM3411	Sep-95	CL, PI<20	6 Lime	50-50	150	183	174	143	A	A	A	A	
		SM, PI<4	6 Lime	50-50	133	375	185	420	A	A	A	A	
FM 111	Dec-95	CH, PI>35	3 Lime	50-50		260		180	A	C	B*	C	
		CL, PI>35	3 Lime	50-50		96		93	A	C	A	C	
FM1124	Dec-95	CL, PI<25	4 Lime	50-50	150	560	170	540	A	B	A	A	
		CH, PI>35	4 Lime	50-50	152	195	138	70	B	C	C	C	
FM244	May-95	SM, PI<10	4 Lime	60-40	145	300	170	716	A	B	A	A	
		CL, PI<25	4 Lime	60-40	160	905	182	950	A	A	A	A	
FM2223	Sep-96		10 Lime	50-50	150	505			A	A	A		
FM1687	May-96	CL, PI<25	8 Lime	60-40	300**	343	237	375	A	A	B	A	
		CH, PI>50	8 Lime	60-40	235	200	210	120	A	A	B	C	
FM977	Feb-96	CL, PI<25	4 Lime	40-60		620		600	A	A	A	A	
FM978	Apr-96	CH, PI>35	5 Lime	40-60		620		400	A	B	B	B	
		CH, PI>25	5 Lime	40-60		915		715	B	A	B	B	
FM1373		CL, PI<25	4 Lime	50-50		1620		1610	A	A	B	A	
FM3178	Aug-95	SC, PI<15	4 Cement	50-50		120		125	A	C	B*	C	
FM977	Aug-95	CL, PI<15	4 Cement	50-50		640		509	A	A	A	B	
FM1935		CL, PI<15	4 Lime	???		440		310	A	A	A	C	
FM975	Sep-96	CH, PI>55	3 Lime	60-40		360		155	B	A	C	C	
		CH, PI>25				340		200	A	A	B	B	
		CL, PI<15				206		280	A	C	B	C	
FM2446	Aug-94	CL, PI<15	3 Lime***	???		290		300	A	A	A	B	
FM2780		CH, PI>35	3.5 Lime	???		45		20	A	C	C*	C	organic
		CH, PI>35				517.4		436					alkaline
		CH, PI>35				1803.2		1586.5					alkaline

* - mainly load associated distress

** - lime added to base layer

*** - new base added before recycling

Many comparisons can be made based on the results presented in Table 31, but the most important one is shown below in Table 32. This relates the observed performance to the PI of the subgrade soil. The higher the PI, the worse the sections are performing.

Table 32. Number of Sections in Each Performance Group (1998 Data).

Soil PI	A Good	B Fair	C Poor
> 35	1	5	8
15-35	6	7	1
<15	9	2	0

Based on Table 32, it is concluded that the current pavement reclamation process is working well for sections built on low to moderate PI soils, but the process is not performing well on sections constructed on high PI material. In the Bryan District, the majority of the high PI soils are also expansive in nature. The addition of a stabilized (stiff) base layer on top of a high shrink/swell soil does not appear to be working. Other contributing factors to the severity of the cracking are the steepness of side slopes, the presence of trees, long dry summers, and the stiffness of the stabilized layer. Later in this chapter, design alternatives will be presented for these high PI sections.

One interesting observation from Table 32 is that in only one location was good performance observed where the subgrade soils had a PI of greater than 35. This was on the last part of FM 111. It must be emphasized that this was also the section that by far had the lowest backcalculated layer moduli (93 ksi) from the FWD data. This section was stabilized with only 3 percent lime. In the first subsection, only a small amount of longitudinal cracking was found; the main concern here was the overall structure capacity. In a few locations, it appeared that some load-associated damage was initiating. This section clearly demonstrates the balancing act that must be undertaken to achieve good, long-term performance on reclaimed sections: first, make the sections strong enough to withstand the traffic loads and minimize subgrade shearing forces; second, make the layers durable so they do not lose strength over time; and third, avoid making the layer too rigid so that it does not crack under environmentally induced forces.

CHALLENGE—BALANCING STRENGTH AND PERFORMANCE

The moduli values backcalculated for the stabilized layers are summarized below in Tables 33 and 34.

Table 33. Backcalculated Moduli Values for All Sections.

% Stabilizer	n	Moduli (ksi)			
		High	Low	Avg.	S.D.
3	12	1075	20	270	198
4	12	1610	70	508	244
5	6	950	400	680	178
6	3	1240	143	-	-
7	1	1279	-	-	-

Table 34. Backcalculated Moduli Values for Stabilized Base Sections Only (No Subbases).

% Stabilizer	n	Moduli (ksi)		
		High	Low	Avg.
3	7	300	20	180
4	4	1610	125	450
5	2	715	400	557

For those sections with granular bases over stabilized subbases, the range of backcalculated moduli for the granular bases is 587 to 1352 Mpa (85 to 196 ksi), with an average value of approximately 1035 Mpa (150 ksi). It must be emphasized that all of these average values are high. For example, the moduli value for a top quality granular base is often found to be three to four times higher than the subgrade value. For typical soils in the Bryan District, this would give a base moduli in the 207 to 276 Mpa (30 to 40 ksi) range. The 3 percent stabilizer gave an average base modulus of approximately 1242 Mpa (180 ksi), five to six times the modulus of a good unstabilized material. The higher modulus will do a better job of distributing the loads to the subgrade and should greatly reduce vehicle load damage. To demonstrate the impact of base modulus on pavement life predictions, several runs of the Department's FPS 19 program were made. These are shown below in Table 35.

Table 35. FPS 19 Life Predictions for Stabilized vs. Unstabilized Bases. N.F. not Feasible (Confidence Level C, Base Thickness 250 mm (10 in), 2 CST, Subgrade Modulus 70 Mpa (10 ksi)).

Design Loads (millions)	3% Stabilizer	Flexible Base
0.5	20+	7
1.0	20+	4
2.0	13	N.F.
3.0	9	N.F.

Clearly, the stabilized layer shows great potential to minimize load-associated damage, but this will only be achieved if the following also occur:

1) The stabilization is uniform along the section, and the resulting layer moduli is relatively uniform. Given the variability of the pavements, this is difficult to achieve. Achieving an average value of 1242 Mpa (180 ksi) means little if the standard deviation of strength is high.
2) Provide permanent stabilization. Several of the layers in Table 31 show a loss in modulus from the 1997 to 1998 data; for example, a section of FM 975 changed from 2484 Mpa (360 ksi) in 1997 to 1070 Mpa (155 ksi) in 1998.

3) Ensure that the sections are not too stiff to prevent cracking caused by shrinkage of either the base or subgrade.

The main conclusion from Tables 33 through 35 is that the moduli values are very high, and that even at the lowest level of stabilization, the average moduli values appear more than adequate to carry the loads for these low volume roads, all of which have 20-year design traffic levels of less than 500,000 equivalent axle loads. The balancing act involves reducing the base strength to still provide adequate load-carrying capability but improving resistance to environmental shrinkage cracking while maintaining a durable layer over the 20-year design life.

ALTERNATIVES FOR HIGH PI SECTIONS

Field Testing Improvements

The data shown above clearly demonstrate that the performance of the reclaimed sections is strongly dependent upon the subgrade soils. If the section is placed over a high PI soil, then the current procedure gives unsatisfactory results. For the moderate and low PI materials, the performance has been good. For these sections, the current procedures are working well.

One major problem within the Bryan District is the variability of both the existing pavement structure and subgrade soils. A single project may have both clay and sand subgrades, and areas with thick and thin base. The presence of fill materials in sections will also impact performance. What is required for future designs is a rapid, nondestructive testing method of identifying major base and subgrade soil breaks along a project. If clear breaks exist and potential problem areas can be identified before design, then localized special treatments can be applied. For the soils problem, the existing USDA soil series maps are a major step in the right direction. However, they are not available for all areas, and they are sometimes not specific enough for this application. GPR has tremendous potential in this area. Research work is underway at the Texas Transportation Institute to evaluate if a combined Ground Coupled and Air Launched survey can be used to map both existing pavement structure (primarily at a depth of 0 - 250 mm (0 - 10 in) and upper subgrade soil type (250 - 750 mm (10 - 30 in).

Design Alternatives

Lime is the primary stabilizer used to date in the Bryan District; a limited number of sections have been built with cement. Mostly, the existing top 250 mm (10 in) is stabilized. This means that when the existing structure has only 100 mm (4 in) of base, then 150 mm (6 in) from the top of the subgrade will become part of the new base. On heavy clay sections, this would lead to a higher optimal stabilizer content, which often results in a stiff brittle slab being placed on the poor clay subgrade. Alternatives to this approach are discussed below.

1. Adding New Material

In one section, FM 2446, a new thin flexible base was placed on top of the existing structure prior to the milling operation. This section is performing well, but the subgrade in this area had a relatively low PI (< 15). Other sections (not included in this study) have been constructed where RAP materials have been placed over the existing structure. The benefit of adding better material before milling is that the final stabilizer content will be reduced; indeed, results from the Houston District (*21*) have shown that high strengths can be obtained with very low stabilizer contents. Compressive strength results from several stabilized base materials are shown in Figure 46, these being for materials which pass TxDOT's base specifications. For these materials, high compressive strengths can be obtained at relatively low cement contents. At 3 percent cement, the minimum seven day compressive strength was 2.1 Mpa (300 psi), with the RAP material having a strength of 3.80 Mpa (550 psi). However, it must be emphasized that the use of compressive strength criteria alone is not sufficient to guarantee a good performing pavement. Other criteria such as durability, shrinkage, strain at break, etc., have an impact on long-term performance.

This technique of adding new base and reducing stabilizer content has not been applied to sections with high PI clay subgrades sections in the Bryan District. It may provide part of the solution to the longitudinal cracking problem.

Action Item 1 *On problem sections (high PI, steep slopes, trees close to roadway), consider adding new base prior to stabilization. This will permit the use of lower levels of stabilizer.*

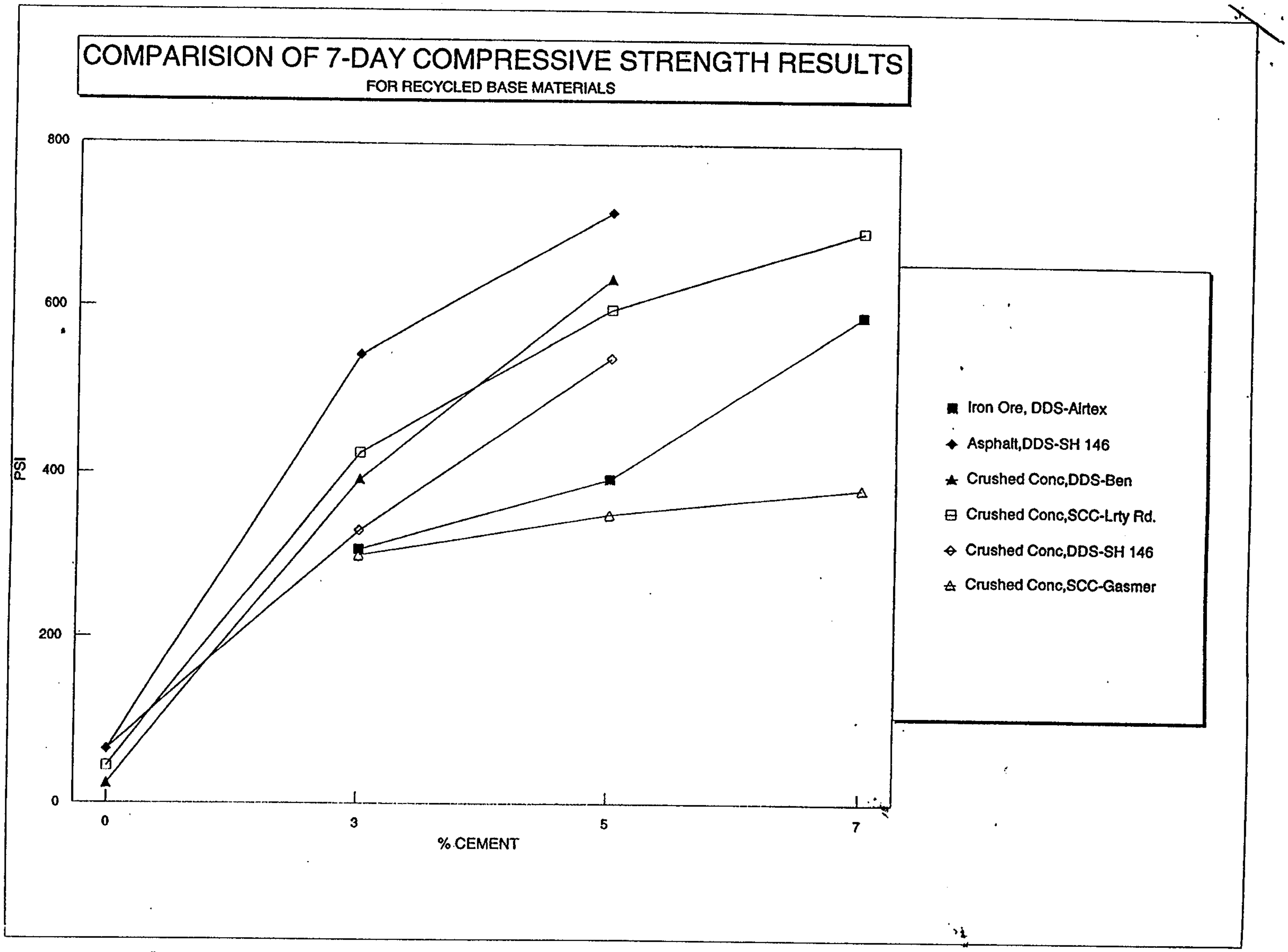


Figure 46. Compressive Strengths for Houston Materials (Yin, 1997).

2. Reducing Stabilizer Content

The only section built on the High PI soils which is performing adequately is FM111. This section has some minor cracking in one location; otherwise, no other cracking was found. The section was reported to have PIs > 35, and the existing base was only 125 mm (5 in) thick. This means that 50 percent subgrade was blended into the new base. Only 3 percent lime was used as the stabilizer, and the average base modulus was low at only 642 Mpa (93 ksi). In several instances, the in-place modulus for several sections seems to be approaching that of unstabilized granular material, which is three to four times the subgrade modulus values. Also on this section, the stabilized layer was used as the base. Although cracking was not found to be a problem on this section, it was apparent that in one short area deep rutting was found. This appeared to be a load associated subgrade problem. Clearly, monitoring should continue on this section.

This section is a very good example of the dilemma facing engineers in trying to select the optimum stabilizer content. Reducing the stabilizer content will reduce the amount and severity of cracking; however, it may also lead to durability problems. New tests to identify durability problems will be discussed later in this chapter.

3. Adding Fibers

Synthetic fibers have been available for many years, but they have not found widespread usage in the highway industry. The benefits of adding fibers to any stabilized material are shown in Figure 47 (22). The stress strain curve for the non fibers sample has a clear maximum after which the strength greatly decreases. The 5 percent cement with 0.5 percent Fibers has a completely different stress strain curve. Even at high strain levels of 4 - 5 percent, the stabilized material continues to have high strength. The potential benefit in reclaimed highways would be: a) reduced crack width, and b) improved post cracking strength. It is recommended that fibers be used on an experimental basis in potential problem locations on an upcoming project. Namely, high PI soils, trees close to road, steep side slopes, etc.

Action Item 2

Evaluate the use of fibers and low stabilizer contents in problem areas.

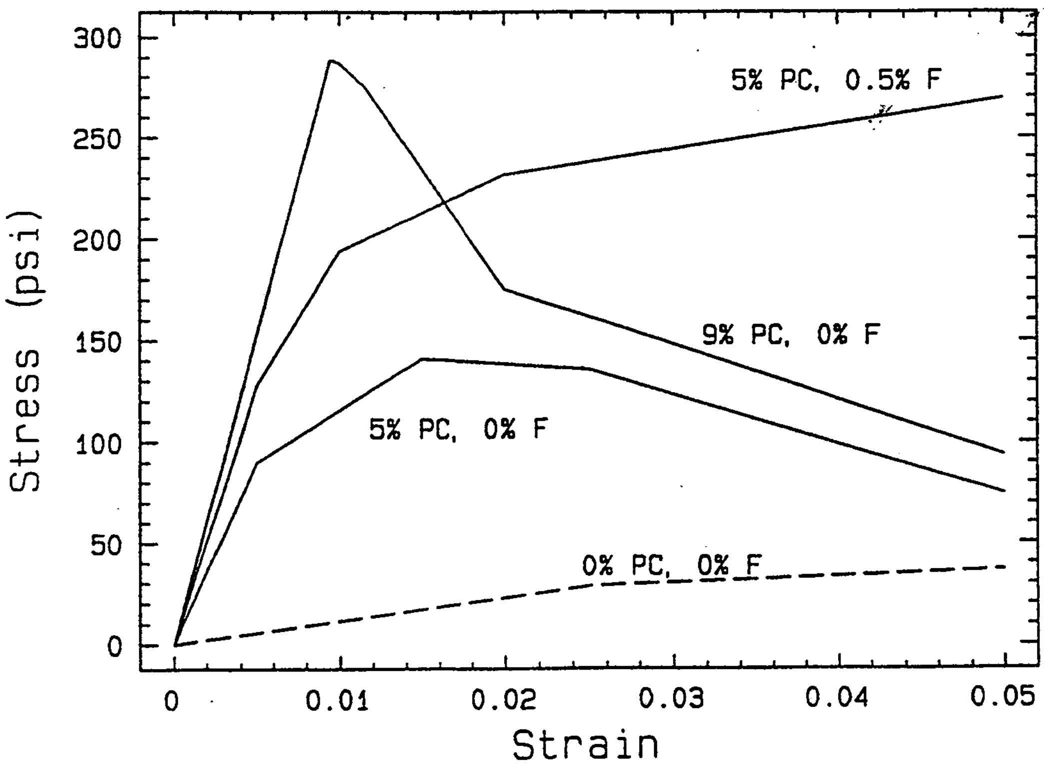


Figure 47. Stress-Strain Curve for Sand (200 psi ≈ 1.4 Mpa).

(Texas Triaxial Test with 5 psi Confining).

4. Asphalt Stabilization

Asphalt stabilization should be considered on an experimental basis in the localized problem areas. It has the potential to provide a more flexible, durable subbase layer to resist the movements of the clay subgrade. Substantial improvements have been made in custom designing emulsified asphalt materials for a range of applications. Their use is not recommended if the existing base has an excessive amount of clay; however, this may not be a problem if new material is added to the surface prior to stabilization. Alternatively, a pre-treatment with lime may be required. Critical issues that need to be resolved with asphalt stabilization relate to construct ability, how to compact the material, and when to apply the surface seal. TxDOT has experienced problems with meeting density requirements on cold recycled asphalt materials which then exhibit hot weather rutting problems. Other problems with emulsified asphalts have been reported, with sealing the sections too early leading to moisture becoming trapped in the base layer.

With careful design and construction control, these problems can be overcome. Asphalt stabilization should be considered as an alternative to chemical stabilization.

Action Item 3 *Where feasible, construct an experimental test section with asphalt stabilization.*

5. Fabrics

Two types of fabrics (web type and impermeable) are available, which provide different potential capabilities. The impermeable fabric may assist with durability (waterproofing), and the web type may add tensile strength to the base and minimize cracking severity.

The sections with major durability problems, such as FM 2780, all have areas of wet subgrades. If moisture is available in the subgrade, it can, through capillary action, move into the stabilized base. With wetting and drying action, some stabilized bases have been observed to lose strength. In situations with wet subgrade conditions, two alternatives should be considered. Firstly, select the stabilizer type and content so the material is not water susceptible. Recommendations on how to use the tube suction test to achieve this will be given later in this section. Second, to waterproof the base, it may be possible to place an impermeable fabric over the subgrade under the stabilized base. Again, the problem with this operation is constructability. However, in addition to waterproofing, the fabric may provide a shear plane to reduce the crack propagation from subgrade drying. If problem areas are detected in upcoming projects, the use of an impermeable fabric on top of the subgrade should be tried on an experimental basis.

Web or grid fabrics are usually placed in the unstabilized flexible base on top of the stabilized layer. Again, these should be tried on an experimental basis.

6. Encapsulation

The major cracking problems are primarily caused by edge drying of the subgrade soils. These soils crack, and the crack propagates rapidly—sometimes with dramatic consequences—through the subbase, base, and surface layers. In some instances, the cracks are 25 to 52 mm (1 to 2 in) wide, and faulting often occurs 25 to 52 mm (1 to 2 in) deep, similar to a slope failure. The only guaranteed method of fixing this subgrade problem would be soil replacement. The PVR test could be run to compute how much soil needs to be replaced or imported. For these low volume roads, this is often cost prohibitive. A less costly alternative could be encapsulation. In this process, an impermeable fabric is wrapped around the subgrade. It is placed horizontally across the layer and vertically in trenches cut at the edge of the pavement. A depth would probably have to be calculated for every location, but a depth of 1.2 m (4 ft) could be used for planning purposes. The aim is to prevent the subgrade material from drying out. The downside of this option is constructability and drying caused by tree roots. This option would probably only be feasible if relatively short potential problem areas were detected.

7. Non-traditional Stabilizers

In recent years, numerous non-traditional stabilizers have become available for soil stabilization. The most widely used one is probably fly ash, which is used alone or more often blended with lime. Fly ash is used extensively in the Lubbock District with good results (*23*). It produces a low strength base, which has been reported to have autogenous healing characteristics. No evidence exists to suggest that fly ash will help with the cracking problem in the Bryan District; durability of the material is a question. In the next section of this report, recommendations on laboratory evaluations of these materials will be given. All non-standard materials should be tested in the laboratory with the project specific materials before including in upcoming jobs.

Other more exotic materials (sulphonated oils, enzymes, silicon products, etc.) are now available. Some have shown potential in reducing the moisture susceptibility of base and subgrade materials. Again, these materials should be evaluated in the laboratory before being considered for inclusion in upcoming jobs.

New Testing Procedures

One cause for concern in Table 31 is the very large variation of in-place moduli values for the same stabilizer content. From Table 34, at the 4 percent stabilizer level, the base modulus varied from 862.5 to 11,109 Mpa (125 to 1610 ksi). The modulus values backcalculated are from in-service pavements three to four years; these results are dependent upon many factors including: a) percentage stabilizer used, b) the variability of existing pavement layers, c) construction quality control, and d) subgrade conditions—particularly the availability of moisture. However, the in-place moduli are so variable, it is proposed that the Bryan District evaluate if alternative design procedures would result in more uniform layer moduli. In this section, a series of laboratory testing procedures are proposed for district evaluation.

1. Strength Criteria

In the Bryan District, stabilizer contents are currently estimated from Texas Method 121-E, and no initial compressive strength measurements are made. Other districts do not use 121-E; most use a seven day compressive strength requirement to estimate stabilizer content. The strength criteria are frequently in the range 1.04 to 1.73 Mpa (150 to 250 psi) or a strength gain in relation to the raw material strength (a factor of three is common). Insufficient evidence exists to conclude that more uniform moduli would be obtained if 121-E were replaced by a strength-based method; however, it is proposed that the district investigate this in upcoming jobs. Two action items are proposed.

Action Item 4 *On future jobs, use 121-E to compute the design stabilizer content. Mold Texas triaxial samples at both the recommended stabilizer level and with the raw material; conduct a Triaxial Test at 15 psi confining on both samples. These data will not be used initially to change the stabilizer content but used simply to evaluate the variability in strengths obtained with 121-E. This initial strength will be used to compare with field moduli and to allow the district to determine if 121-E should be replaced with a strength-based procedure.*

Action Item 5 *With a select number of base materials, investigate the difference in design stabilizer contents using 121-E—a strength-based procedure—and the pH test discussed below. This will be a research type investigation.*

Another issue that has been raised by the Lubbock District (*23*) is that the type of failure in the compression test should also be considered. Stabilizer contents resulting in high strength samples that are rigid and fail suddenly at high loads with very little deformation should be avoided. Samples that fail at vertical strain levels less than 1 percent should be avoided as they are too brittle for flexible pavements.

2. Locating Organic Material

The major problem identified with organic material in the subgrade soil is demonstrated in the first section of FM 2780. This 1.3 km (0.8 mi) section is a failure; 3.5 percent lime did not stabilize the existing base. After only a few years in service, this section is showing severe load-associated distress in the form of subgrade shear failures. The base modulus is very low, in the order of 138 Mpa (20 ksi). Sampling of the section concluded that the subgrade soil is very wet and is high in organic content. Clearly, a simple laboratory test is needed to identify this problem in future projects.

Organic matter is heterogeneous, containing a diverse population of live organisms as well as plant and animal residues in different stages of decomposition. A discussion of the impact of organic material on the stabilization process is given in Appendix B. Two simple tests are proposed to identify if organic matter is in sufficient quantities to influence the stabilization process. These are the hydrogen peroxide test and the pH test. Both should be used, but the pH test is preferred as it involves an assessment of the soil/stabilizer mix.

3. Locating Sulfate Soils

One additional problem with stabilizing clay soils is the presence of excessive sulfates, which can cause dramatic early failure of the stabilized soils. Sulfate-rich soils are not widely found in the Bryan District; however, the major problems experienced on SH 6 near Benchley were attributed to adding lime to a soil containing excessive amounts of selenite (a form of gypsum, high in sulfates). With the addition of large amounts of rain during construction, the soil/sulfate blend can lead to the formation of the highly expansive ettringite mineral.

If a simple test were available, it could provide a good indicator of potential future problems. A simple procedure was proposed by Bredenkamp (*24*) involving measuring the conductivity of a soil/water mix. The test procedure and equipment needed are shown in Appendix C. It is noted that the total test set-up costs less than $600, and the conductivity/pH probe costs less than $300. This single probe can be used to conduct the pH test for organics and the conductivity test for sulfates. While the pH test is not conclusive, it is a good indicator test. If a soil passes the test, it will probably not experience problems; however, if a soil fails the test, it may or may not be a sulfate problem. If a sample fails, more tests should be run before selecting stabilizer types.

Action Item 6 *Purchase and test in the laboratory a pH/conductivity probe.*

4. Tube Suction Test for Durability

One major concern with base and subbase stabilization is durability or permanency of stabilization. A loss of stabilization with time is a complex phenomena which is not well understood. Several researchers attribute the loss of stabilization to carbonation where the long term stabilization processes are reversed in the presence of carbon dioxide; others have detected leaching problems where moisture movement in the stabilized layer removes the stabilizing agents.

In the sections monitored in 1997 and 1998, as shown in Table 31, eight sections showed a significant increase in deflection and a corresponding drop of backcalculated moduli values (> 40 percent drop). These are FMs 542-1, 1362, 111, 1124, 1687, 978, 975, and 2780. While some of this can be attributed to seasonal variations in deflections, an underlying trend is clear. It would be very interesting to retest these sections in about three years from now to see what layer strengths remain. In research conducted at TTI, it is clear that the major factors influencing the permanency of stabilization is both the availability of moisture in the subgrade soils and the ability of this moisture to "wick" in and out of the stabilized layer. Wetting and drying cycles have a major impact on stabilized layer performance.

To test the moisture susceptibility of stabilized bases and subbases, the Bryan District should conduct tube suction tests on stabilized materials that will be placed on clay subgrades. This test was developed by TTI initially to detect moisture-susceptible granular materials but has subsequently been recommended for stabilized materials. It can be used as a durability test to support the traditional compressive strength test. This new test can also check the suitability of both stabilizer type and stabilizer content. In the test, standard triaxial samples are molded and cured according to standard TxDOT procedures. The samples are then placed in a 40 °C (104 °F) room for four days, after which they are subjected to capillary rise for 10 days. During the capillary rise, the surface dielectric of the sample is measured each day. The proposed criteria is a follows:

a) For base material, the surface dielectric should be less than 10 after 10 days. (Values higher than this indicate significant amounts of free moisture in the material.)

b) For subbase material, two criteria are proposed: 1) the surface dielectric should be less than 16, and 2) the percentage retained strength should be greater than 75 percent after 10 days soak. The second criteria will require strength testing of both samples which have and have not been subjected to the capillary rise.

<u>Action Item 7</u> *Purchase a dielectric probe, and evaluate the durability of layers to be placed over clay subgrades.*

REFERENCES

1. Hoffman, M. S., and Thompson, M. R., *Nondestructive Testing of Pavements Field Testing Program Summary,* Transportation Engineering Series No. 31, Illinois Cooperative Highway and Transportation Research Program, Series No. 190, University of Illinois, Urbana-Champaign, Illinois, 1981.
2. Hoffman, M. S., and Thompson, M. R., *Mechanistic Interpretation of Nondestructive Pavement Testing Deflections,* Transportation Engineering Series No. 32, Illinois Cooperative Highway and Transportation Research Program, Series No. 190, University of Illinois, Urbana-Champaign, Illinois, 1981.
3. Letto, A. R., *A Computer Program for Function Optimization Using Pattern Search and Gradient Summation Techniques,* Master of Engineering Thesis in Industrial Engineering, Texas A&M University, College Station, Texas, 1980.
4. Little, D. N., *Handbook for Stabilization of Pavement Subgrades and Base Courses with Lime*, Kendall/Hunt Publishing Company, Dubuque, Iowa 52002.
5. Little, D. N., Scullion, T., Kota, P. B. V. S., and Bhuiyan, J., *Identification of the Structural Benefits of Base and Subgrade Stabilization,* Research Report 1287-2F, Texas Transportation Institute, Highway Materials Division, College Station, Texas, 1994.
6. Lytton, R. L., Germann, F., and Chou, Y. J., *Determination of Asphaltic Concrete Pavement Structural Properties by Nondestructive Testing,* National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington D.C., 1989.
7. Lytton, R. L., and Zollinger, D. G., *Modeling Reliability in Pavement*, Transportation Research Board, Washington, D.C., 1993.
8. Maser, K. S., and Scullion, T., *Automated Detection of Pavement Layer Thickness and Subsurface Moisture Using Ground Penetrating Radar,* Transportation Research Board, Washington D.C., 1991.

9. Maser, K. S., Scullion, T., and Briggs, R., *Use of Radar Technology for Pavement Layer Evaluation,* Research Report 930-5F, Texas Transportation Institute, Highway Materials Division, College Station, Texas, 1991.

10. Michalak, C. H., Lu, D. Y., and Turman, G. W., *Determining Stiffness Coefficients and Elastic Moduli of Pavement Materials from Dynamic Deflections*, Research Report 201-1, Texas Transportation Institute, College Station, Texas, 1976.

11. Ralston, A., and Rabanowitz, P., *A First Course in Numerical Analysis,* McGraw Hill, New York, 1978.

12. Rohde, G. T., and Scullion, T., *Modulus 4.0: Expansion and Validation of the MODULUS Backcalculation System*, Research Report 1123-3, Texas Transportation Institute, Highway Materials Division, College Station, Texas, 1990.

13. Rohde, G. T., and Smith, R. E., *Determining Depth to Apparent Stiff Layer from FWD Data*, Research Report 1159-1, Texas Transportation Institute, Highway Materials Division, College Station, Texas, 1991.

14. Scrivner, F. H., and Moore, W. M., *An Empirical Equation for Predicting Pavement Deflections*, Research Report 32-12, Texas Transportation Institute, College Station, Texas, 1968.

15. Scullion, T., Uzan, J., Yazdani, J. I., and Chan, P., *Field Evaluation of the Multi-depth Deflectometers*, Research Report 1123-2, Texas Transportation Institute, Highway Materials Division, College Station, Texas, 1988.

16. Scullion, T., Chen, Y., and Lau, C. L., *Colormap - User's Manual with Case Studies*, Research Report 1341-1, Texas Transportation Institute, Highway Materials Division, College Station, Texas, 1995.

17. Syed, I. M., and Scullion, T., *Determining In-place Engineering Properties of Recycled Pavement Layers,* Research Report 3903-1, Texas Transportation Institute, Highway Materials Division, College Station, Texas, 1997.

18. Uzan, J., Scullion, T., Michalak, C. H., Paredes, M., and Lytton, R. L., *A Microcomputer Based Procedure for Back-calculating Layer Moduli from FWD Data*, Research Report 1123-1, Texas Transportation Institute, Highway Materials Division, College Station, Texas, 1988.

19. Webster, S. L., Grau, R. H., and Williams, T. P., *Description and Application of Dual Mass Dynamic Cone Penetrometer*, Research Report GL-92-3, USAE Waterways Experiment Station, Geotechnical Laboratory, Vicksburg, Mississippi, 1992.

20. Huang, Y. H., *Pavement Analysis and Design*, Prentice Hall, Englewood Cliffs, New Jersey, 1993.

21. Yin, S., Laboratory Engineer of Houston District of TxDOT, private communication, 1998.

22. Crockford, W. W., Grogan, P. G., and Chill, D. S., "Strength and Life of Stabilized Layers Containing Fibrillated Polypropylene," TRB paper 930888, Texas Transportation Institute, College Station, Texas, January 1993.

23. Baker, R., Construction Engineer Lubbock District of TXDOT, private communication, 1998.

24. Bredenkamp, S., and Lytton, R. L., "Reduction of Sulfate Swell in Expansive Clay Subgrades in District 18," TTI Report 1994-5, November 1994.

APPENDIX A

VISUAL ASSESSMENT OF PAVEMENT CONDITION

(June/July 1998)

- **soils data obtained from USDA county soils maps (PI = Plasticity Index)**

CH = Clays of High plasticity

CL = Clays of Low to Medium Plasticity

SM = Silty Sands

ML = Silty of Low to Medium Plasticity

SC = Clayey Sands

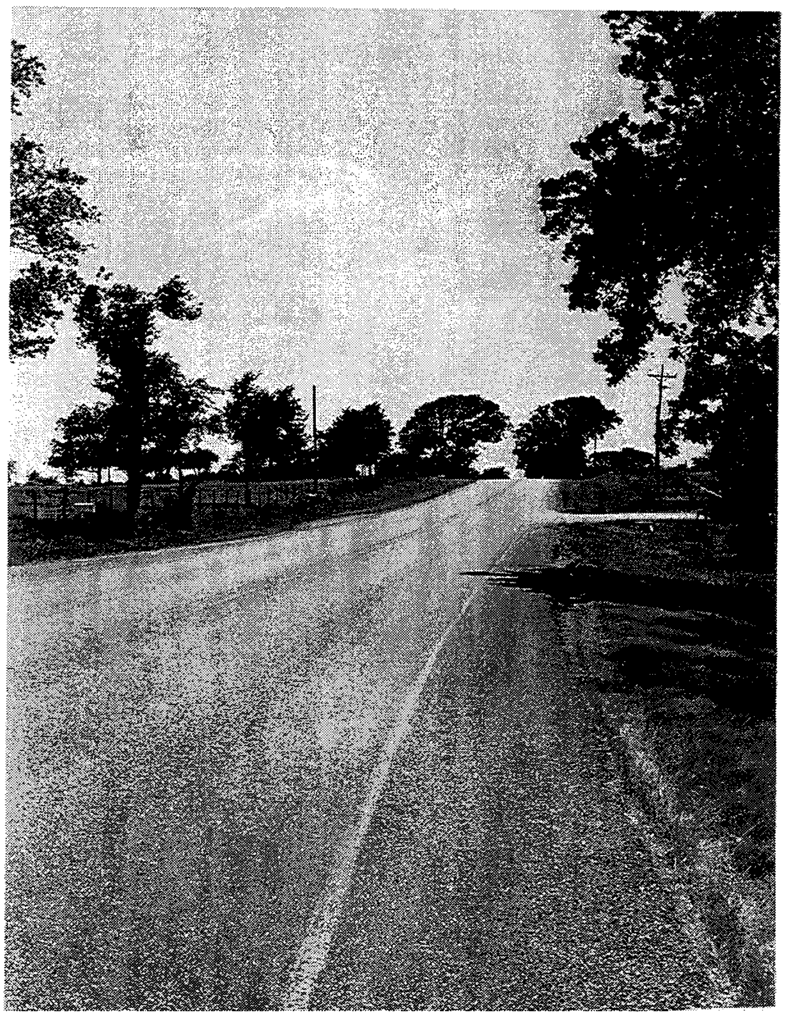
SECTION 1

Road	County	Limits	Date	Structure
FM 3090	Grimes	From SH 6 to FM 3455	5-94	LSB (60-40) (3%), 8"FB & 2 CST

Mile

0 - 0.1 A short section of minor rutting at very start of project, inside wheel path.

0.1 - 1.9 No cracks or ruts - good performance. Overall condition is very good.



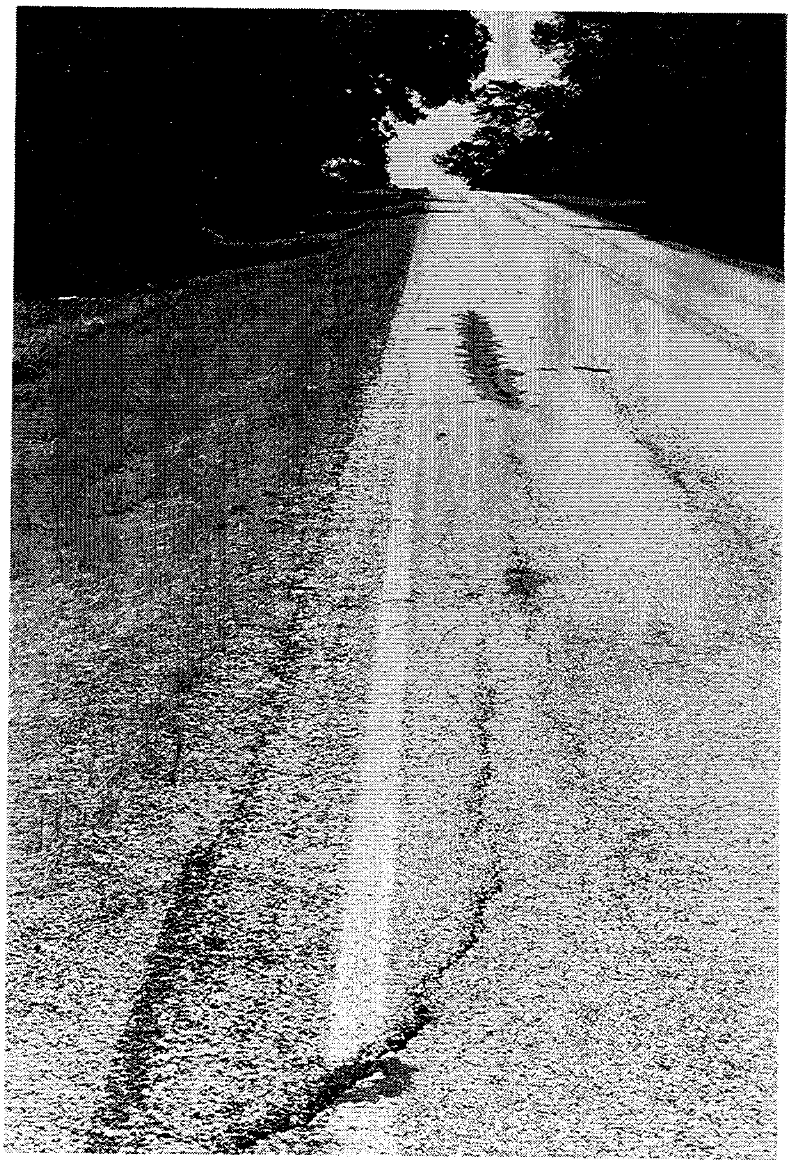
Conclusion: Good Performer.

SECTION 2

Road	County	Limits	Date	Structure
FM 149	Grimes	Montgomery C/L to 2.2 Miles West	6-94	LSB (60-40) (7%), 8"FB & 2 CST

<u>Mile</u>

0.1 - 0.15 Edge cracking along tree line. (CL, PI = 17).



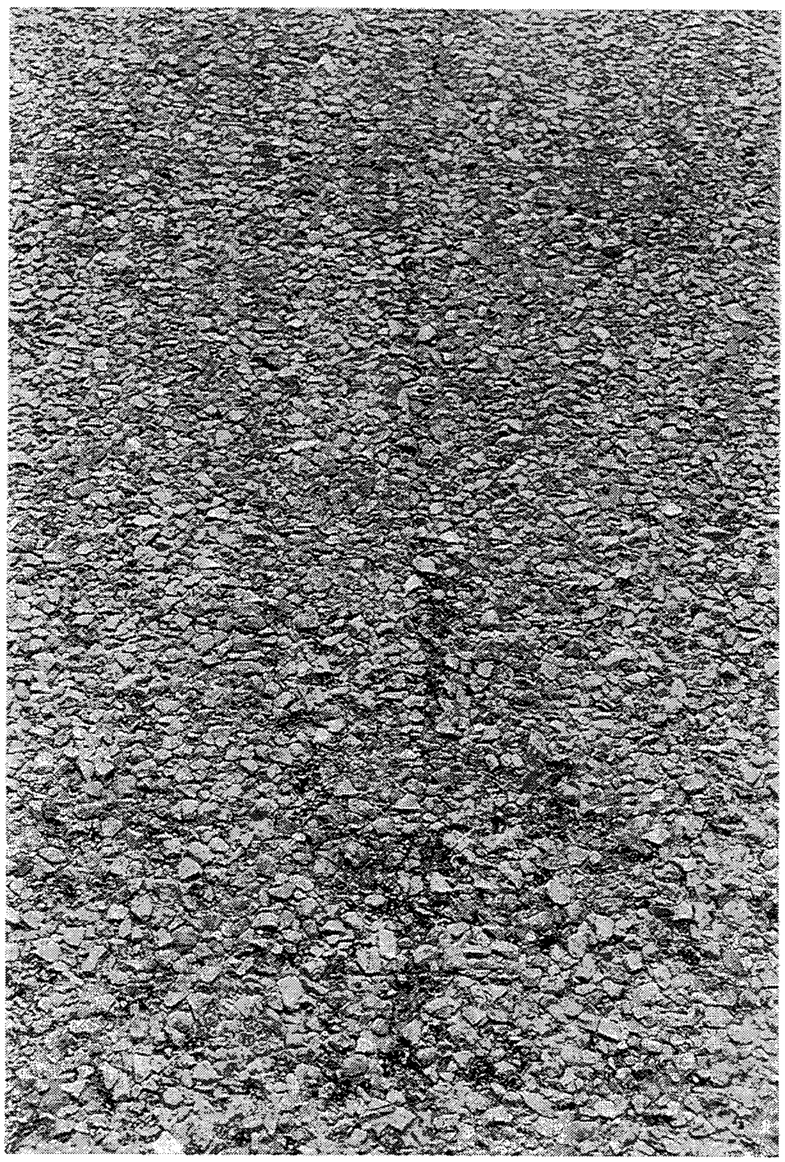
SECTION 2

Road	County	Limits	Date	Structure
FM 149	Grimes	Montgomery C/L to 2.2 Miles West	6-94	LSB (60-10) (7%), 8"FB & 2 CST

<u>Mile</u>

0.3 - 0.4 Steep slopes, close trees no cracks, bridges at 0.8 to 1.0 km (0.5 to 0.6 mi). (CL, PI = 17).

0.7 - 1.3 Slight transverse cracks. Road in good condition.



SECTION 2

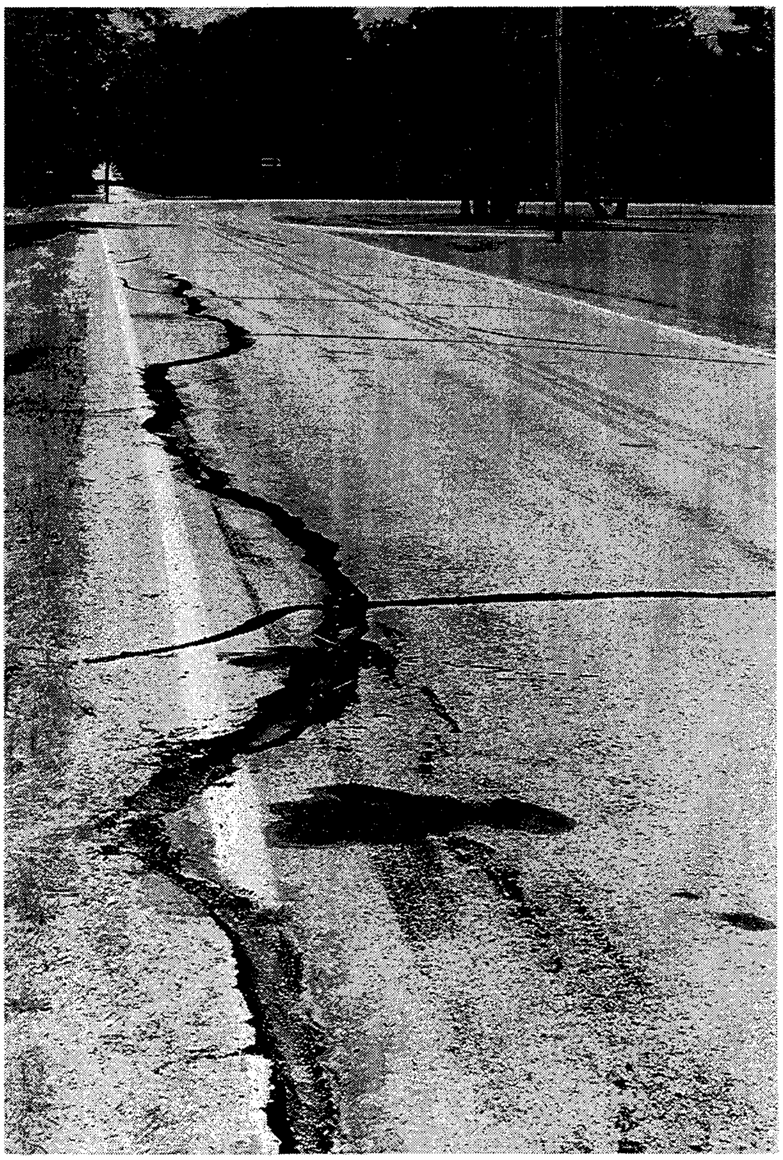
Road	County	Limits	Date	Structure
FM 149	Grimes	Montgomery C/L to 2.2 Miles West	6-94	LSB (60-10) (7%), 8"FB & 2 CST

<u>Mile</u>

1.3 - 2.3 Several crack sealing operations, extensive longitudinal and transverse.

Some long-edge cracking; severe problems close to trees.

Looks like traditional stabilization block cracking. (CH, PI > 35).



<u>Conclusion</u>:

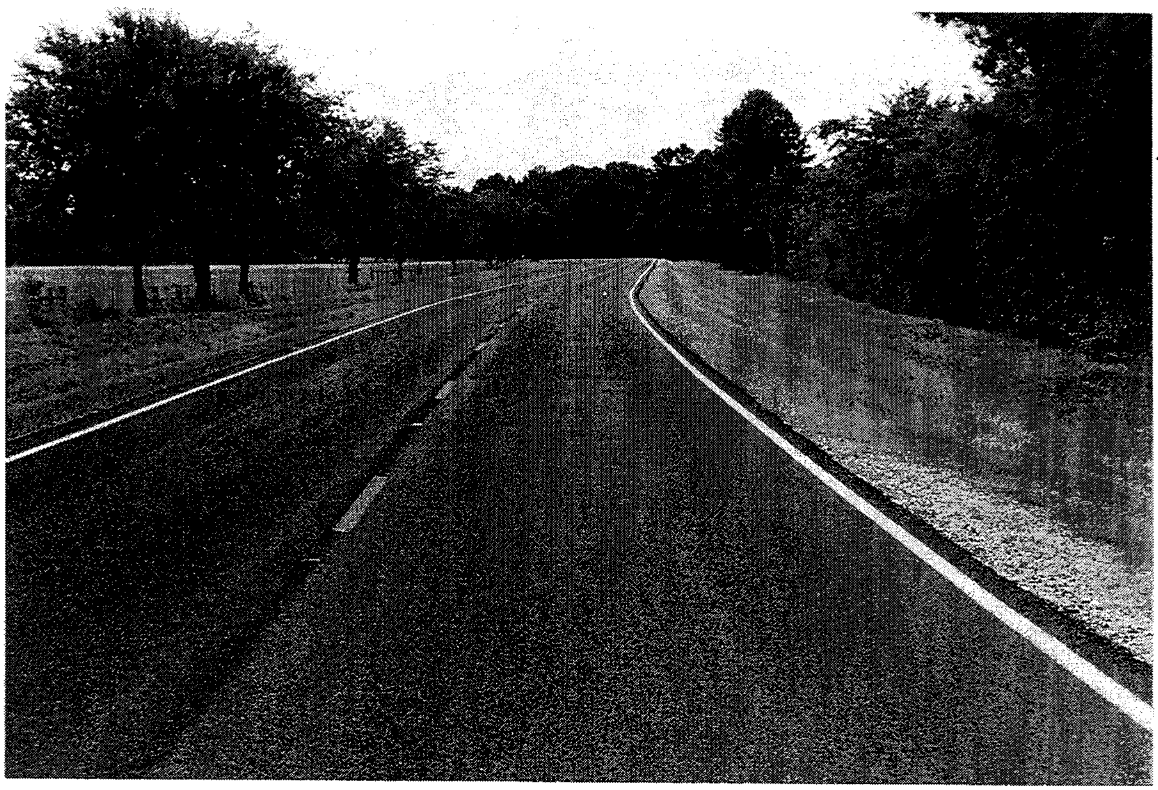
Variable performance. Last 1.6 km (mile) looks like traditional stabilization cracks. Too much stabilizer. Some severe problems in this section; very wide cracks.

SECTION 3

Road	County	Limits	Date	Structure
FM 1486	Grimes	2.3 Mi S of FM 149 to 2.3 Mi N	7-94	LSB (60-40) (8%), 10"FB & 2 CST

Mile

0 - 1.5 Road in very good shape. Lots of trees. No major cracks. Minor cracking around 1.13 km (0.7 mi) from start. (CL, PI < 15).



SECTION 3

Road	County	Limits	Date	Structure
FM 1486	Grimes	2.3 Mi S of FM 149 to 2.3 Mi N	7-94	LSB (60-40) (5%), 8"FB & 2 CST

<u>Mile</u>

1.5 - 1.9 Short section of crack seal operation. Transverse cracks every 9-12 m (30-40 ft). Some moderate longitudinal cracking. This is over crest of hill. Localized problem. (CH, PI > 35).

1.9 - 2.3 Road in good shape.

<u>Conclusion</u>:

Overall, road in good condition; one short 0.6 km (0.4 mi) section with longitudinal/transverse cracking.

SECTION 4

Road	County	Limits	Date	Structure
FM 542-1	Leon	US 79 to 5 Miles South	6-94	LSB (60-40) (3%), 8"FB & 2 CST

<u>Mile</u>

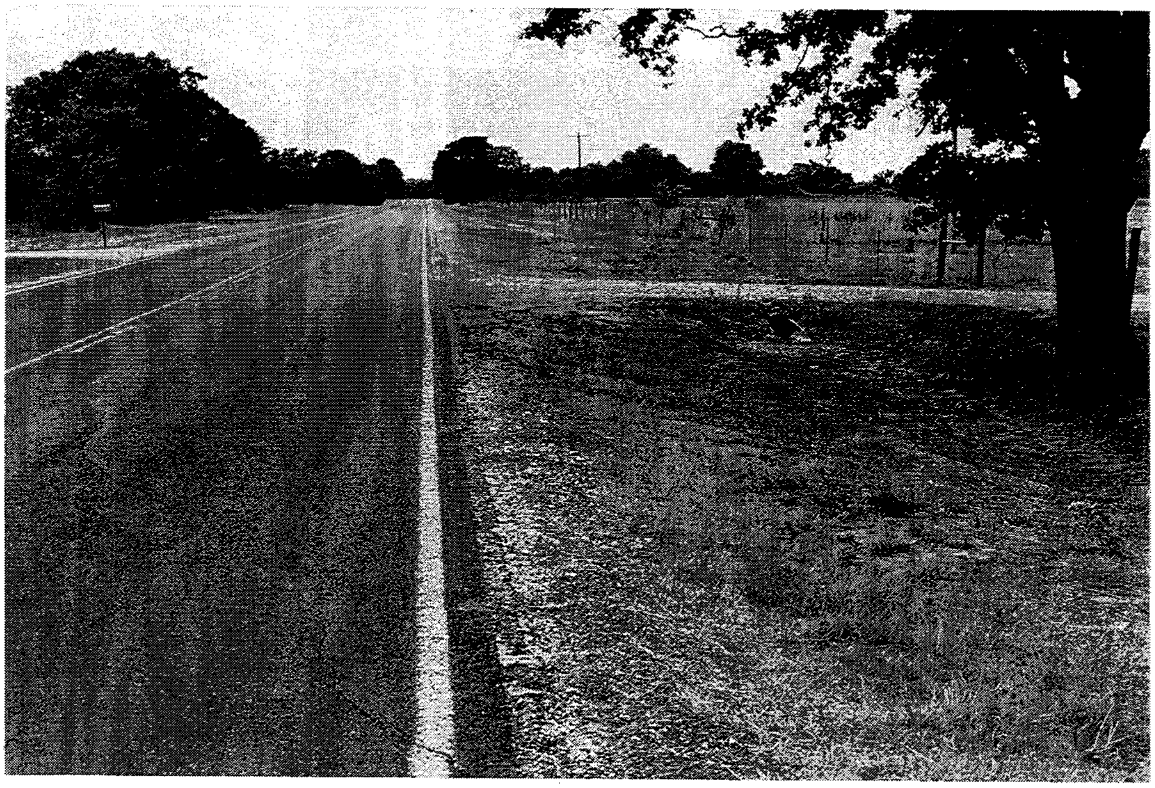
0 - 5.0	Very good - mostly sandy subgrade - no distress even close to trees. Only one longitudinal crack in entire 8 km (5 mi).	
	Soils Info	
0 - 1.2	CL - CH	PI 26-34
1.2 - 5	SM, ML	PI 3 - 11

SECTION 5

Road	County	Limits	Date	Structure
FM 542-2	Leon	5 Mi S of Oakwood to Shiloh	4-95	LSB (60-40) (5%), 7"FB & 2 CST

Mile

0.5 mile Large crack around tree. CL, CH PI 28-38.



SECTION 5

Road	County	Limits	Date	Structure
FM 542-2	Leon	5 Mi S of Oakwood to Shiloh	4-95	LSB (60-40) (5%), 7"FB & 2 CST

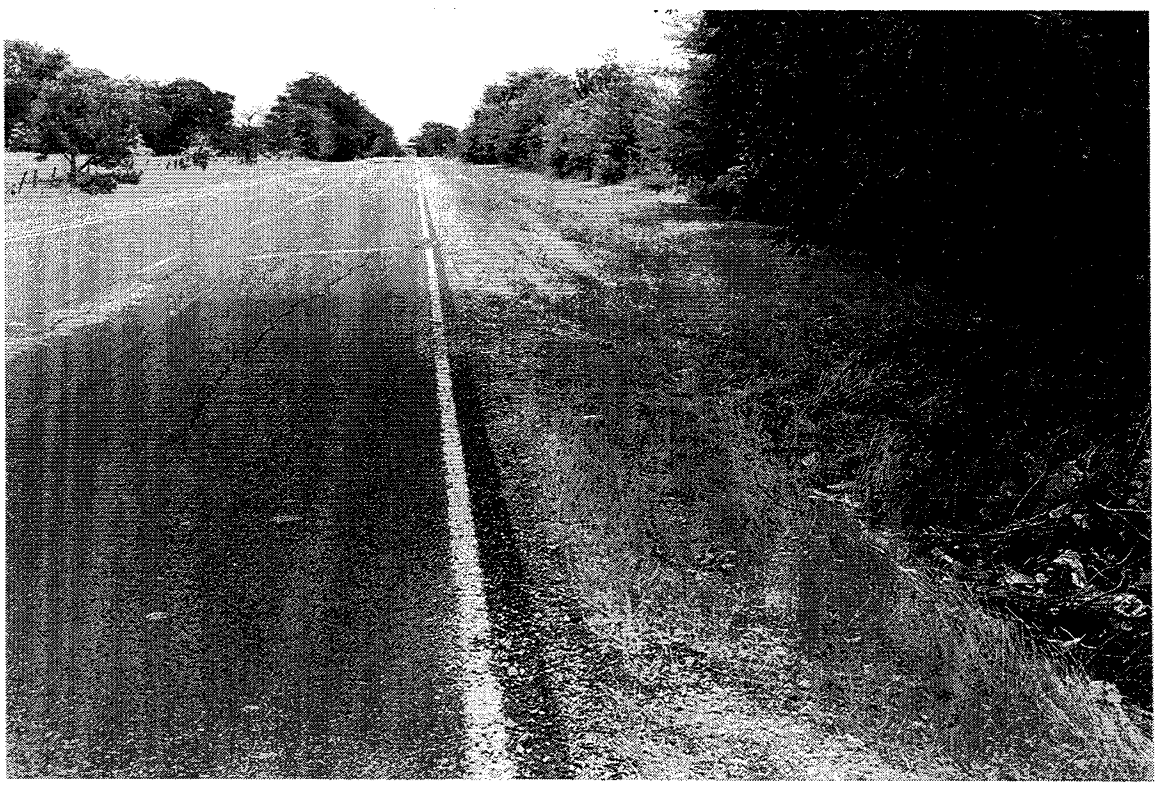
<u>Mile</u>

1.3 Bridge.

1.45 Large crack filled with HMAC. A few other large unsealed cracks. CH - CL (PI 28-35).

1.5 Bridge.

2.1 Large crack around tree NB. Cl PI < 25.



<u>Conclusion</u>:

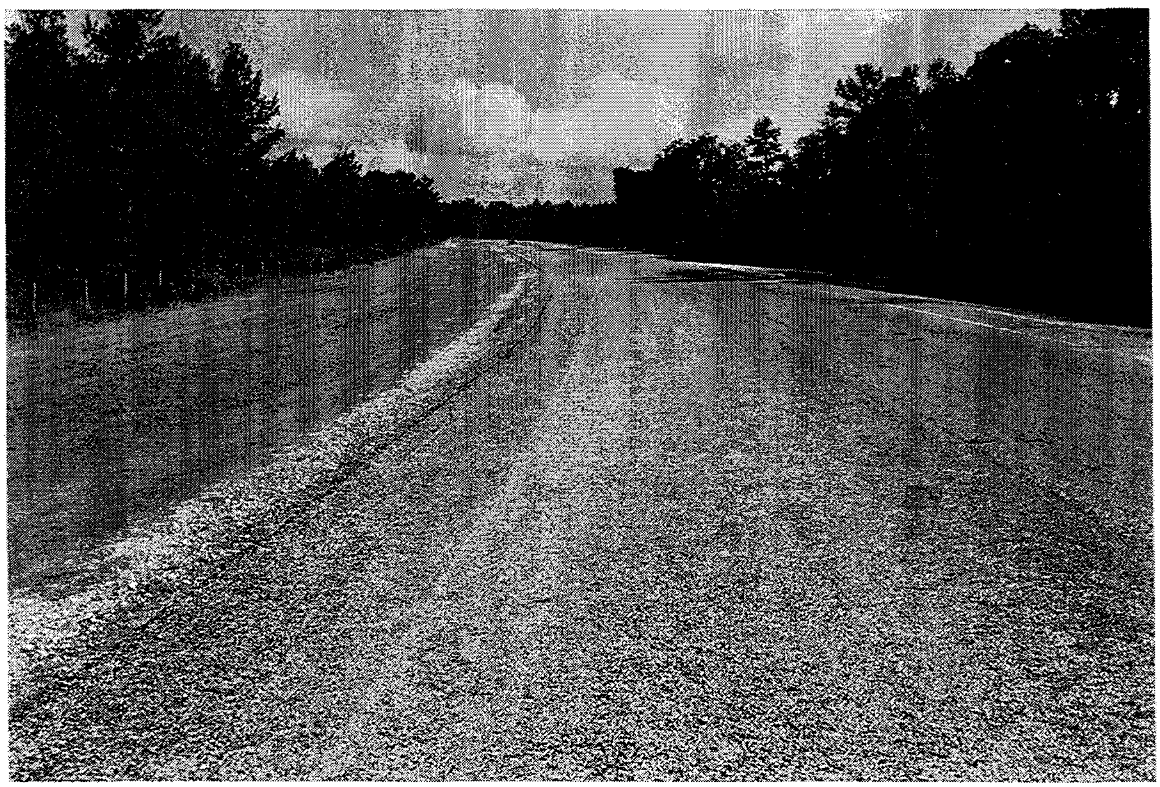
Overall section in good shape. A few localized cracks - no further major deterioration - maintenance seems to fix problem.

SECTION 6

Road	County	Limits	Date	Structure
FM 3478	Walker	1.8 Mi N of FM 980 to 1.1 Mi N	3-95	LSB (60-20) (6%), 11"FB & 2 CST

<u>Mile</u>

0 - 1.1 Highway in very good condition - tree line a long way from pavement edge ±40 ft.



<u>Conclusion</u>: Very good performance.

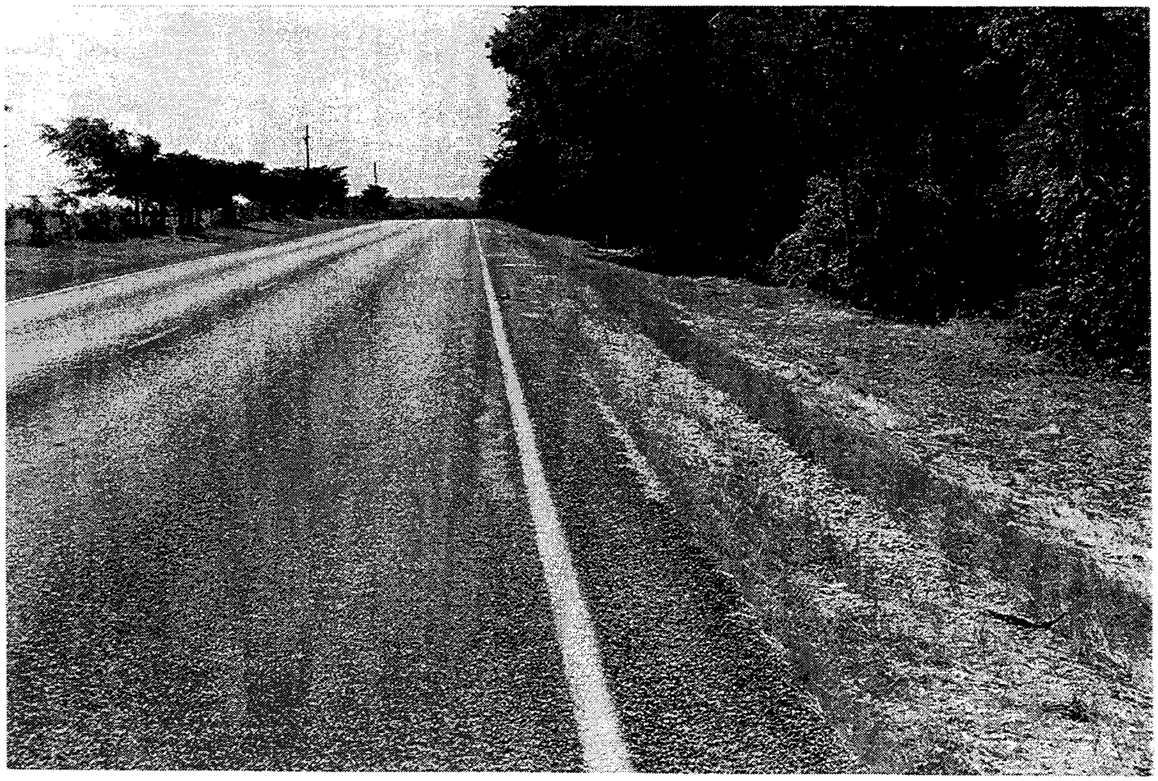
SECTION 7

Road	County	Limits	Date	Structure
FM 1362	Burleson	SH 21 to FM 166	5-95	LSB (80-20) (4%), 7"FB & 2 CST

Mile

0.25 - 1.2 Minor longitudinal cracking along pavement edge. Trees present most places - cracks are sealed and not deteriorating. (CL, PI < 20, some organics).

1.2 - 2.0 Good Condition.



SECTION 7

Road	County	Limits	Date	Structure
FM 1362	Burleson	SH 21 to FM 166	5-95	LSB (80-40) (4%), 7"FB & 2 CST

2.0 - 2.2 Some severe edge deterioration after bridge next to large oak trees. (CH, PI > 45).

SECTION 7

Road	County	Limits	Date	Structure
FM 1362	Burleson	SH 21 to FM 166	5-95	LSB (80-20) (4%), 7"FB & 2 CST

<u>Mile</u>

3.0 - 3.6 Moderate cracking whenever tree line comes close to road.

3.4 Major crack on curve, steep slopes, and many trees. (CL, CH PI 20-38).

<u>Conclusion</u>:

Very variable performance. Last 2.4 km (1.5 mi) fair to poor.

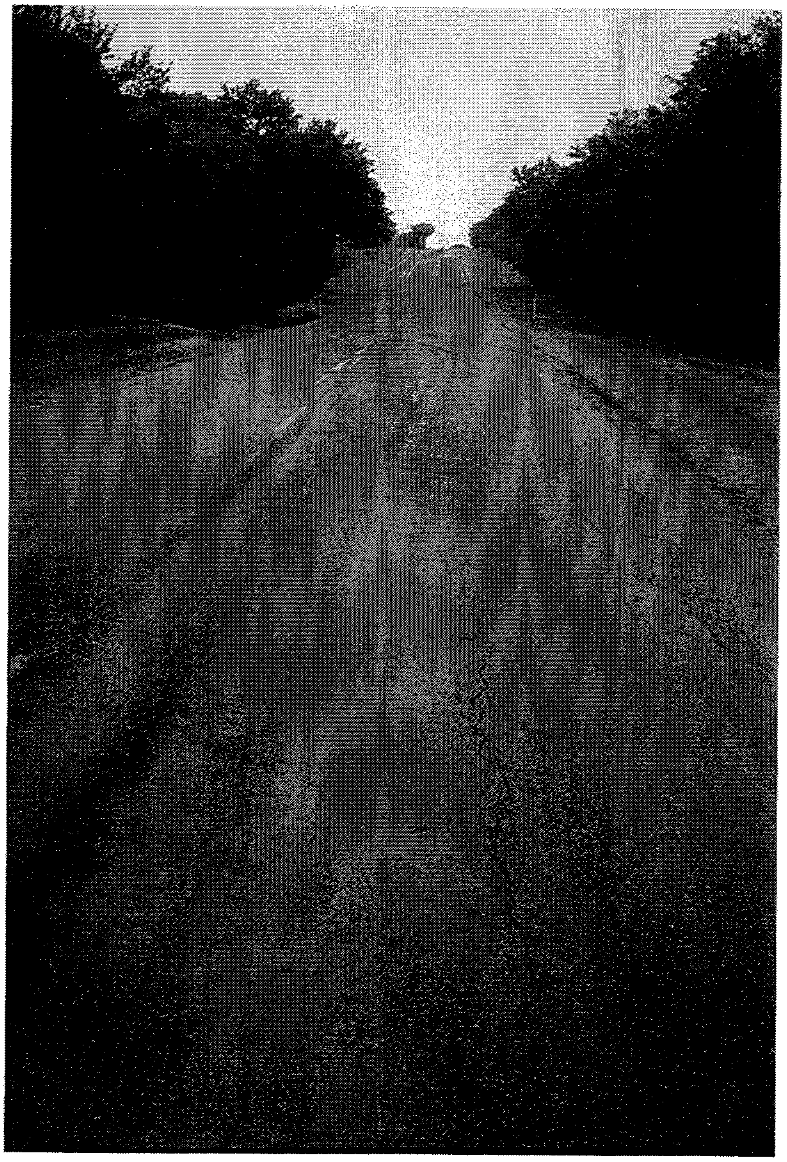
SECTION 8

Road	County	Limits	Date	Structure
FM 246	Freestone	FM 27 to 3.3 Mi E	5-95	LSB (40-60) (6%), 9"FB & 2 CST

<u>Mile</u>

1.8 - 2.1 No trees-road in good shape.

2.1 - 2.9 Distress starts at tree line, multiple repairs. (PI > 35).



<u>Conclusion</u>:

Between 1.3/1.8 2.1/2.9 at least 50 percent patching. The repaired section is again failing. In non-tree areas, some minor longitudinal + transverse-all sealed. Poorly performing section. One of worst performers in study.

SECTION 8

Road	County	Limits	Date	Structure
FM 246	Freestone	FM 27 to 3.3 Mi E	5-95	LSB (40-60) (6%), 9"FB & 2 CST

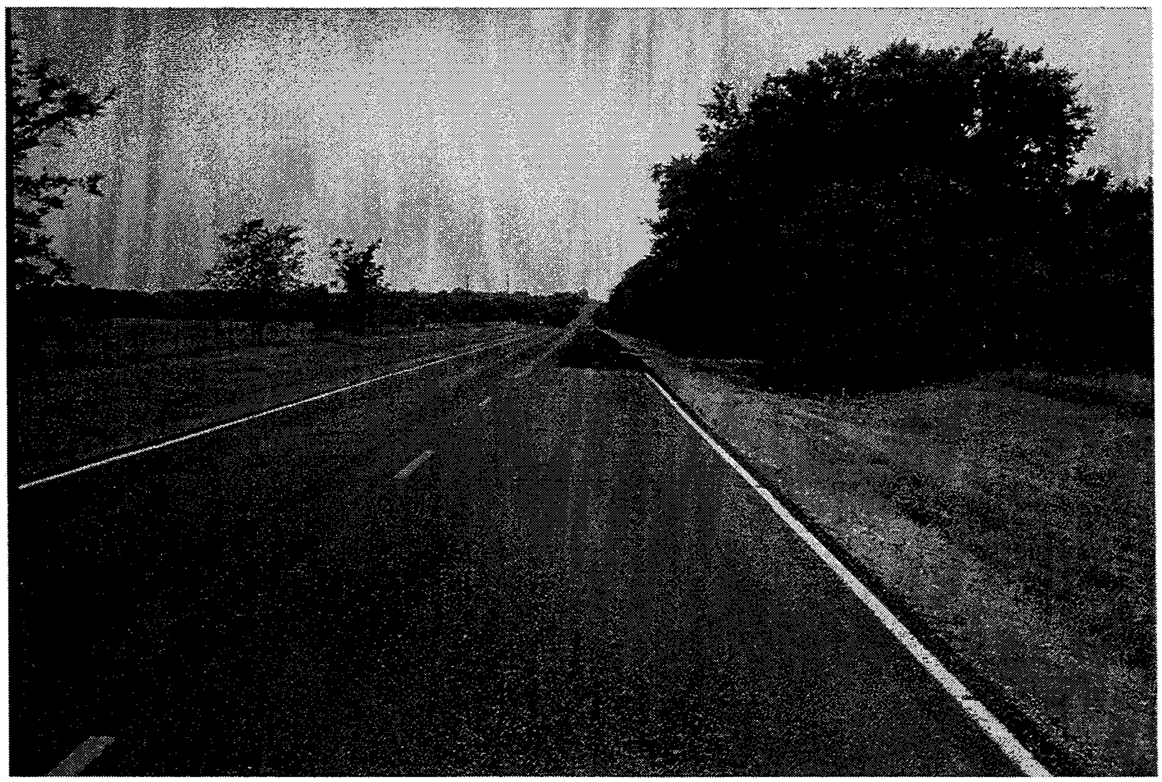
<u>Mile</u>

Start 3.3 East of 27 - road condition poor - multiple patches (CH, PI > 35).

0 - 1.3 No trees at side of road. Minor trans/longitudinal cracking.

1.3 - 1.5 Repairs start at tree line. Multiple patches.

Cracking in repaired area; also some edge failures (trees + slopes).



SECTION 9

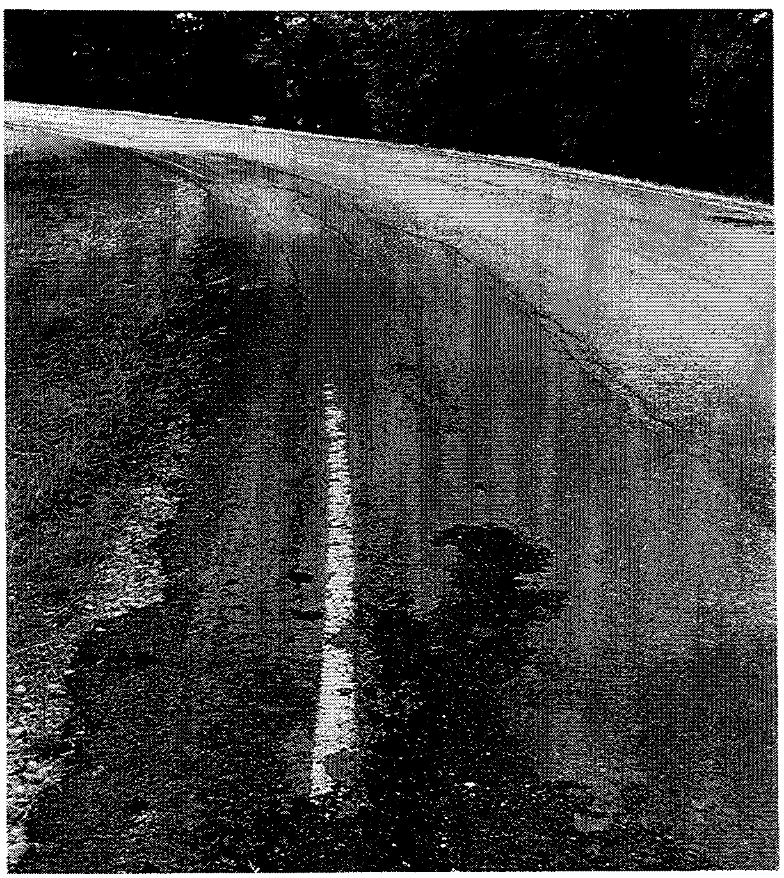
Road	County	Limits	Date	Structure
FM 977	Leon	Sh 75 to 4.5 MI E	7-95	LSB (60-40) (3%), 11" FB & 2 CST

Mile

0 - 1.4 Trees next to highway - no distress-suspect sandy soil in area (red soil). (CL, ML PI 8-20).

1.4 - 3.0 Longitudinal cracks, some severe/edge drying patched with HMAC. Trees near edge severe condition-lots of maintenance on section. Poorly performing section. (CH, PI > 35).

3.0 -4.5 Distress not as severe, but edge-long cracking intermittent throughout project.



Conclusion:

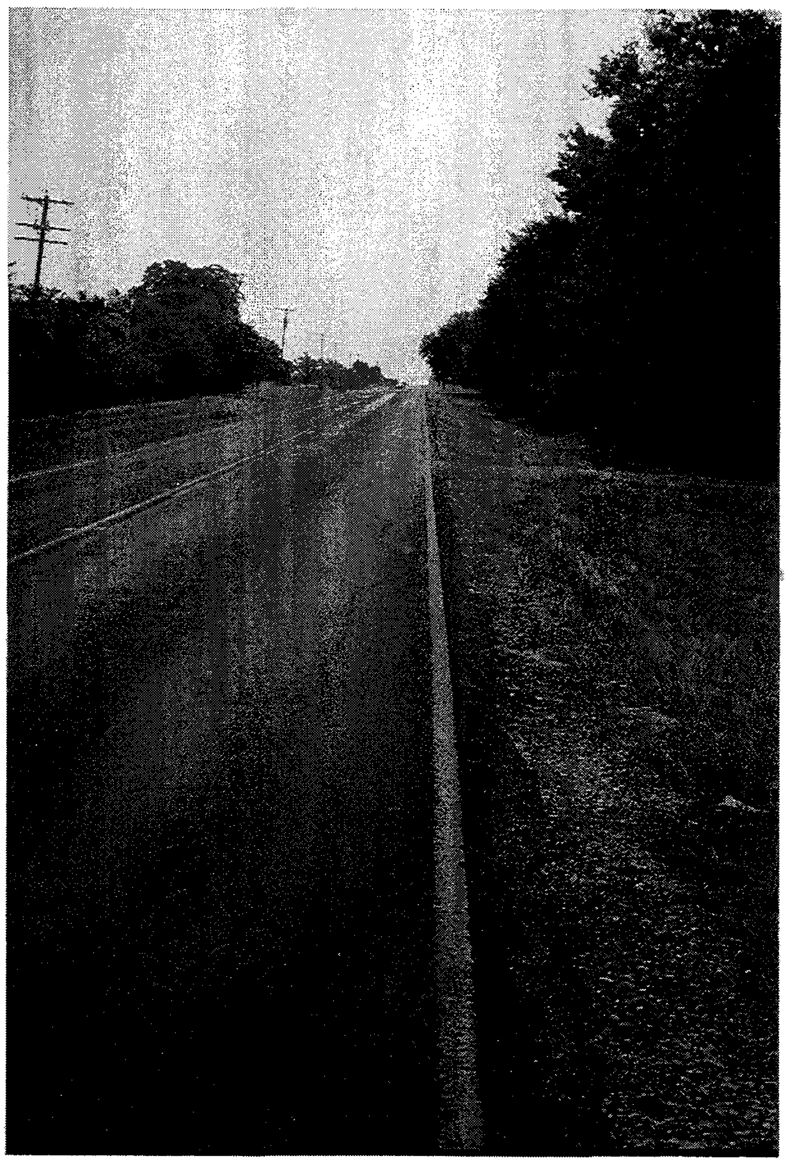
Last 4.8 km (3 mi) poorly performing. Investigate why 0-2.2 km (0-1.4 mi) is good.

SECTION 10

Road	County	Limits	Date	Structure
FM 27	Freestone	Curb & Cutter in Wortham to FM 1366	9-95	LSB (60-40) (5%), 12"FB & 2 CST

<u>Mile</u>

0.1 - 0.3 Continuous moderate longitudinal cracking with tree line close to edge and steep slopes. Most of section has tree line set back at least 6 m (20 ft) from pavement edge - No major crack - flushed seal.



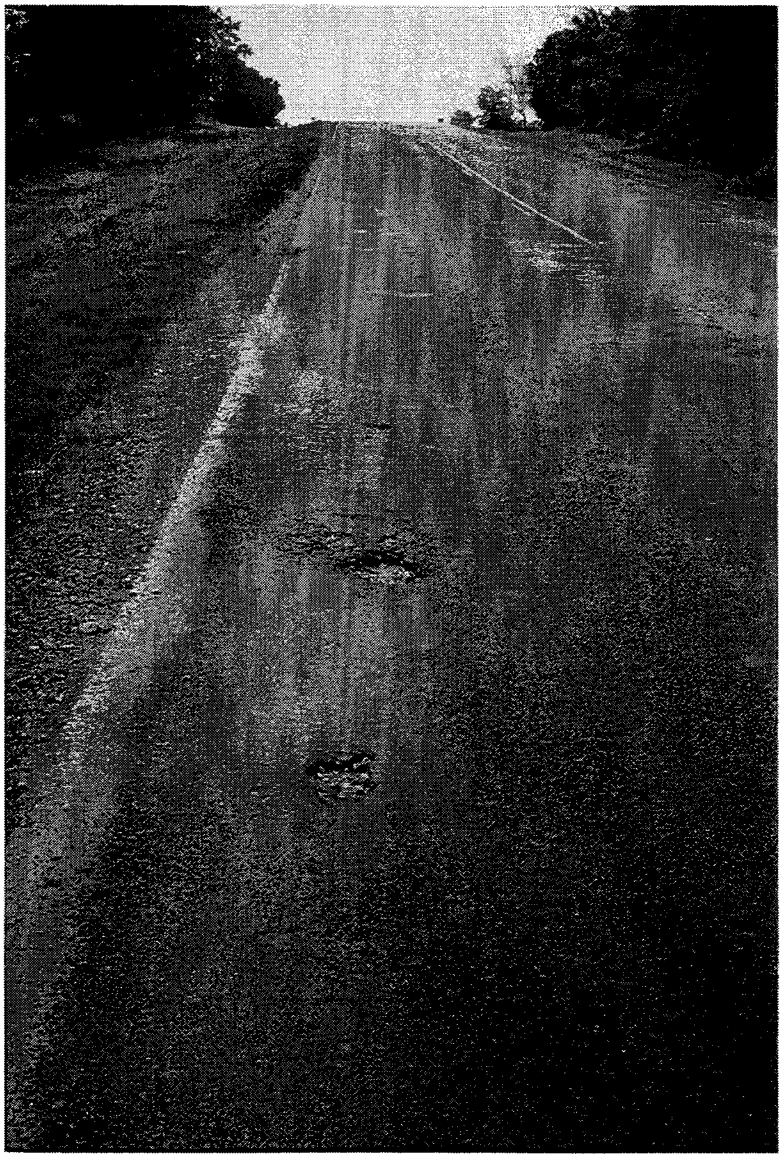
SECTION 10

Road	County	Limits	Date	Structure
FM 27	Freestone	Curb & Cutter in Wortham to FM 1366	9-95	LSB (60-40) (5%), 12"FB & 2 CST

Mile

0.3 - 2.4 Fair/good condition flush.

2.4 - 2.55 Short failed section with rutting alligator - just after bridge - both directions failed. Not typical.



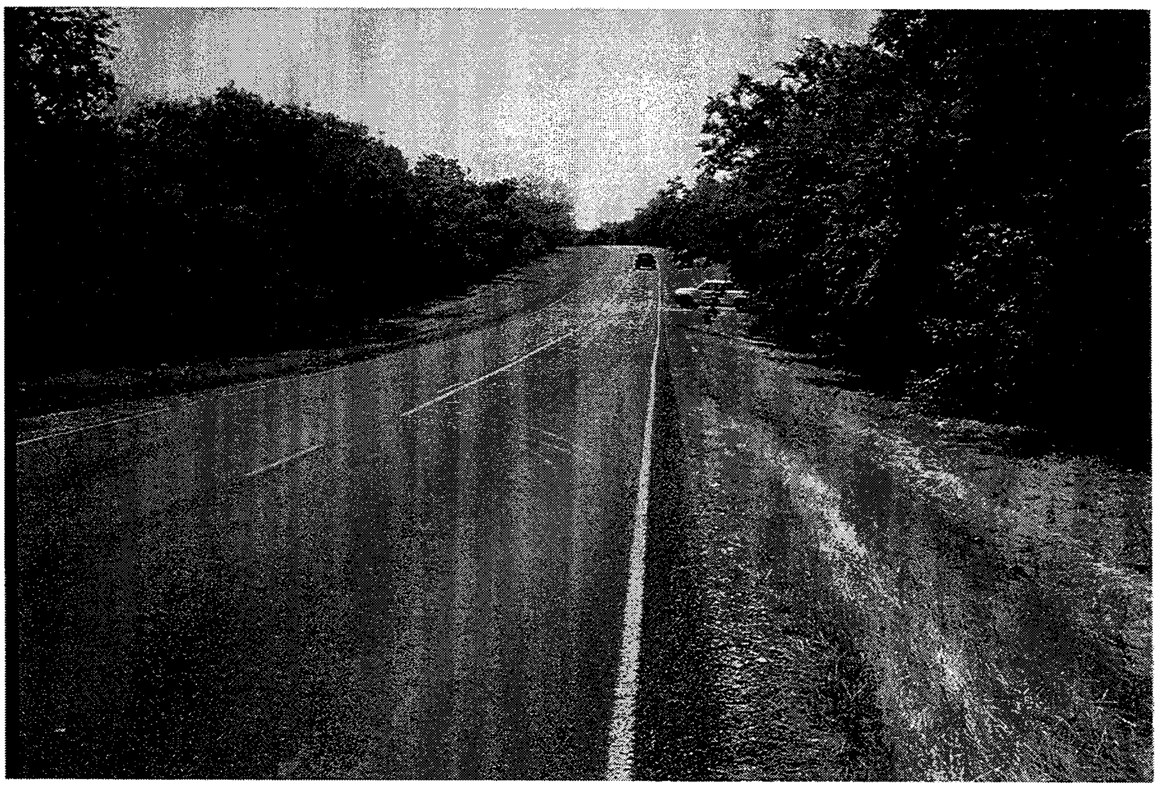
SECTION 10

Road	County	Limits	Date	Structure
FM 27	Freestone	Curb & Cutter in Wortham to FM 1366	9-95	LSB (60-40) (5%), 12"FB & 2 CST

Mile

2.55 - 4.2 Fair/good condition.

4.2 - 5.0 Minor longitudinal crack + small area of seal failure, trees close to road. (CH, PI 28-35).



Conclusion:

Overall, no maintenance yet - one short failed sections + one seal failure, some minor longitudinal cracking.

SECTION 11

Road	County	Limits	Date	Structure
FM 3411	Walker	SH 19 to FM 2929	9-95	LSB (50-50) (6%), 10"FB & 2 CST

<u>Mile</u>

0 - 2.2 Road in excellent condition. No distress. Trees very close to highway, both sides of road. Looks like sandy subgrade (red/tan color soils). (CL, ML, SMI PI < 20).

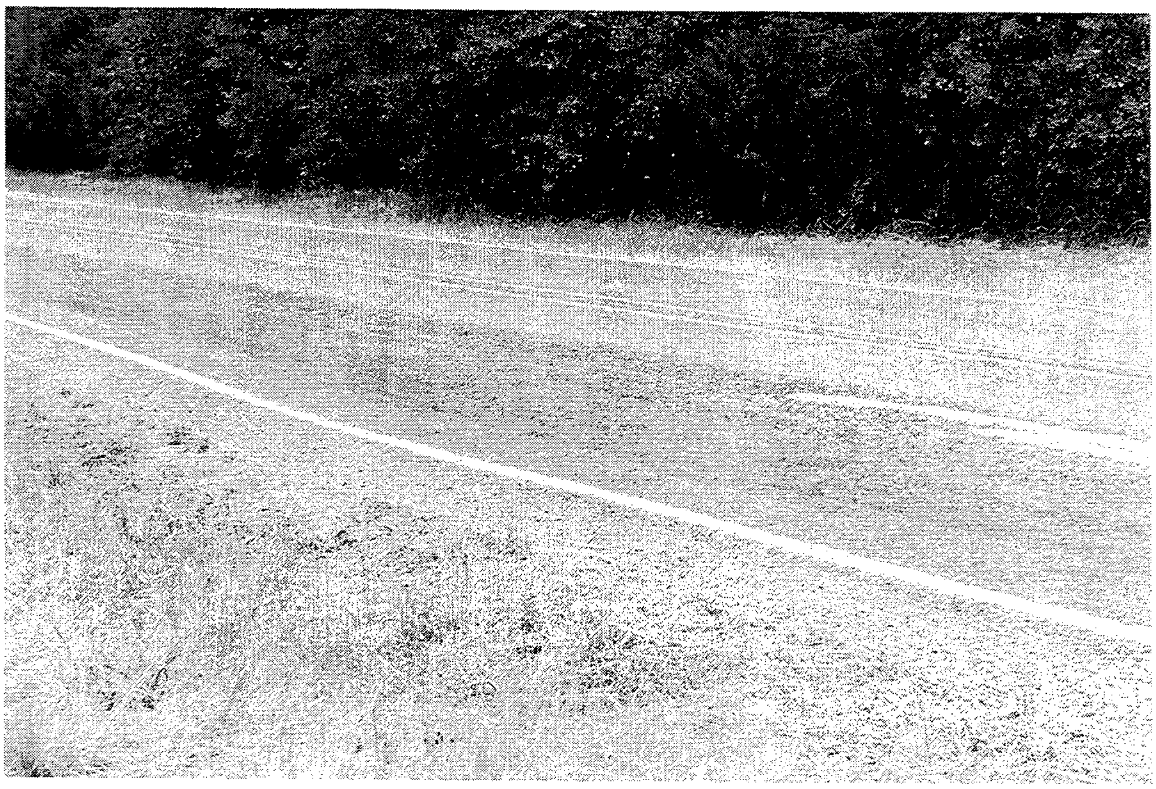
<u>Conclusion</u>: Good Performer.

SECTION 12

Road	County	Limits	Date	Structure
FM 111	Burleson	FM 60 to 3.4 Mi E	12-95	LSB (50-50) (3%), & 2 CST

Mile

0 - 0.6	No problem, no trees.
0.6 - 0.65	Minor cracking in curve, no major trees.
0.65 - 2.65	No cracks - few trees near road - road very good condition. (CH, PI > 35).
2.65 - 2.85	Minor cracks - trees. Some evidence of initial edge rutting localized, possible subgrade shear problem.
2.85 - 3.4	Many trees/no cracks. (CL, PI > 25).



Conclusion:

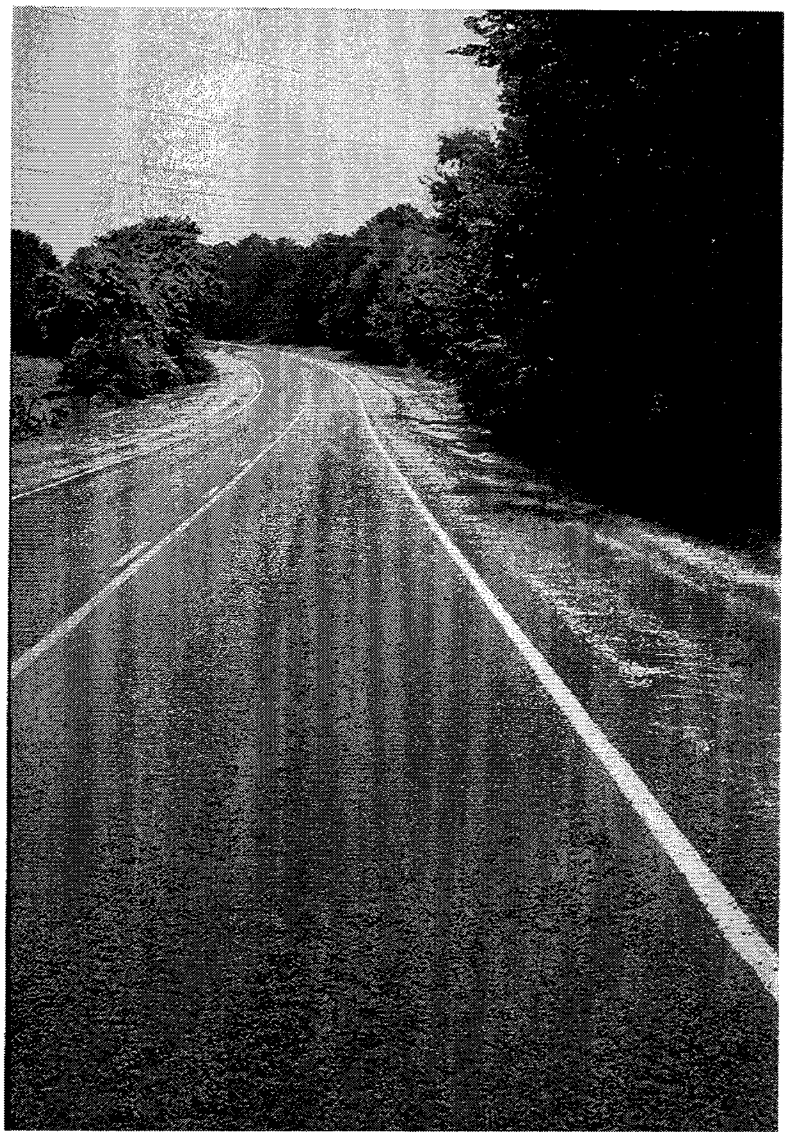
Road generally in very good condition. No major problems with cracking. This road could have structural problem; edge depressions evident in some locations. Structural check FWD/GPR data.

SECTION 13

Road	County	Limits	Date	Structure
FM 1124	Freestone	FM 488 to 1.8 Mi E	12-95	LSB (50-50) (4%), 5"FB & 2 CST

<u>Mile</u>

0 - 1.5 No problem first 2.4 km (1.5 mi). For most part, trees set back from highway ± 6 m (± 20 ft). Even when trees close to highway, no cracking found. (CL, PI 14-30).

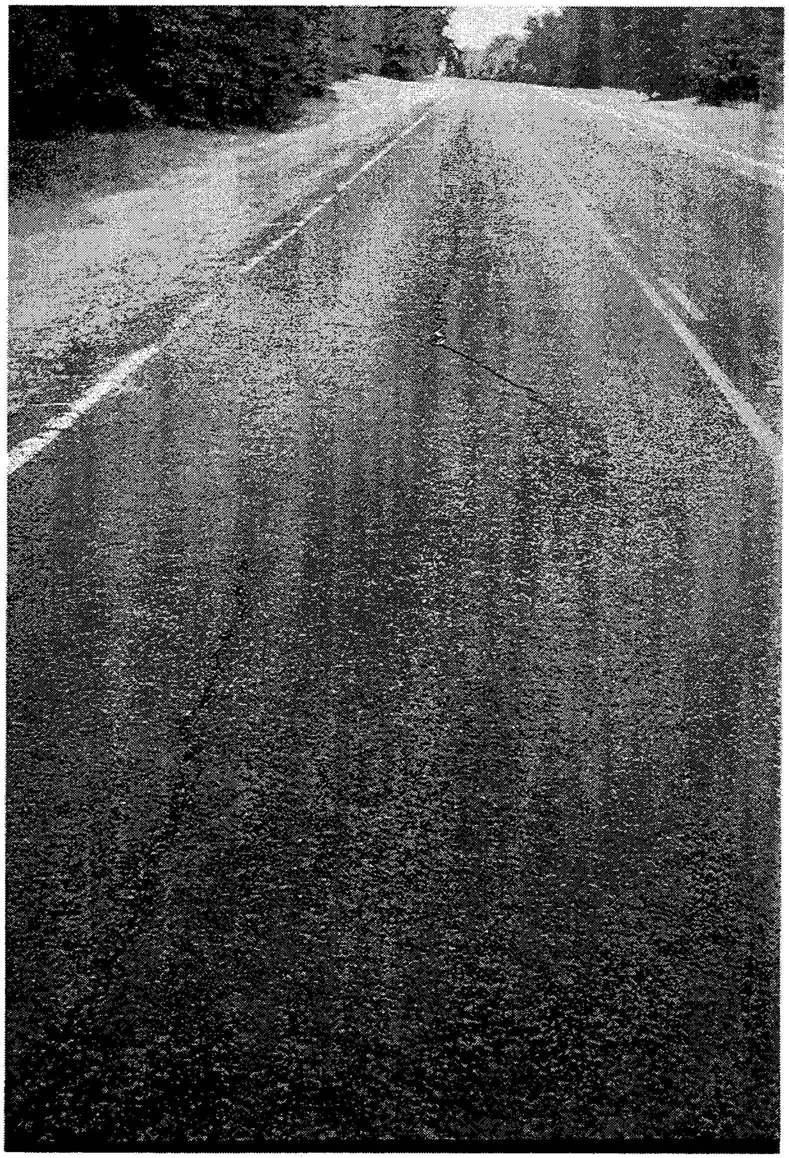


SECTION 13

Road	County	Limits	Date	Structure
FM 1124	Freestone	FM 488 to 1.8 Mi E	12-95	LSB (50-50) (4%), 5"FB & 2 CST

Mile

1.5 - 1.8 Substantial severe longitudinal cracking in last 0.48 km (0.3 mi). (CH, PI 40-60).



Conclusion:

Check for change of soil type around 2.4 km (1.5 mi).

Overall, very good condition except for last 0.48 km (0.3 mi), major longitudinal cracking.

SECTION 14

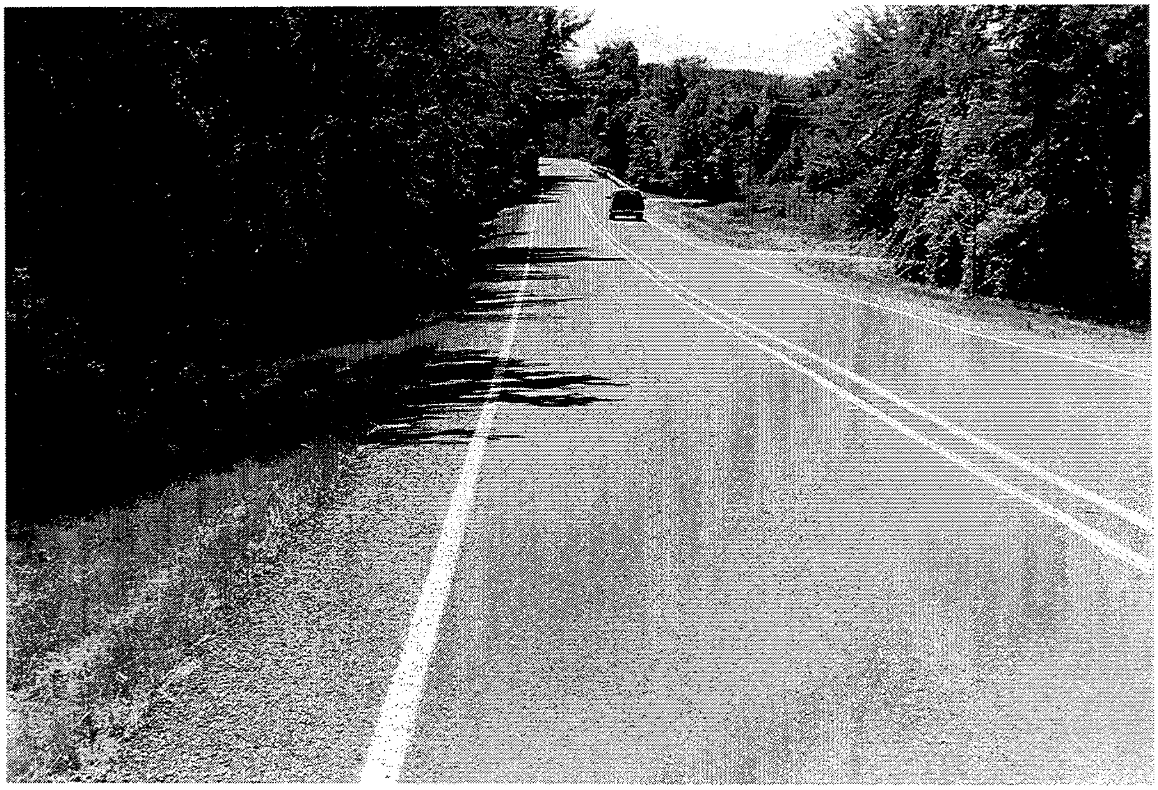
Road	County	Limits	Date	Structure
FM 244	Grimes	FM 3090 to SH 30	5-95	LSB (60-40) (4%), 12"FB & 2 CST

<u>Mile</u>

0 - 0.6 Very good condition. Some washboarding on curves.

0.6 - 1.2 Different structure associated with bridge (not in study).

1.2 - 2.3 Lots of trees, no cracking - road in excellent shape. (CL, PI < 15).
Very well performing highway. Heavy truck traffic.



<u>Conclusion</u>: Good Performer.

SECTION 16

Road	County	Limits	Date	Structure
FM 1687	Brazos	FM 50 to OSR	5-96	LSB (60-40) (XX), XX", & 2 CST

Mile

0 - .5 Bad flushed seal → faint long edge crack 1.5 m (5 ft) from edge.

0.2 Surface seal (flush problem).

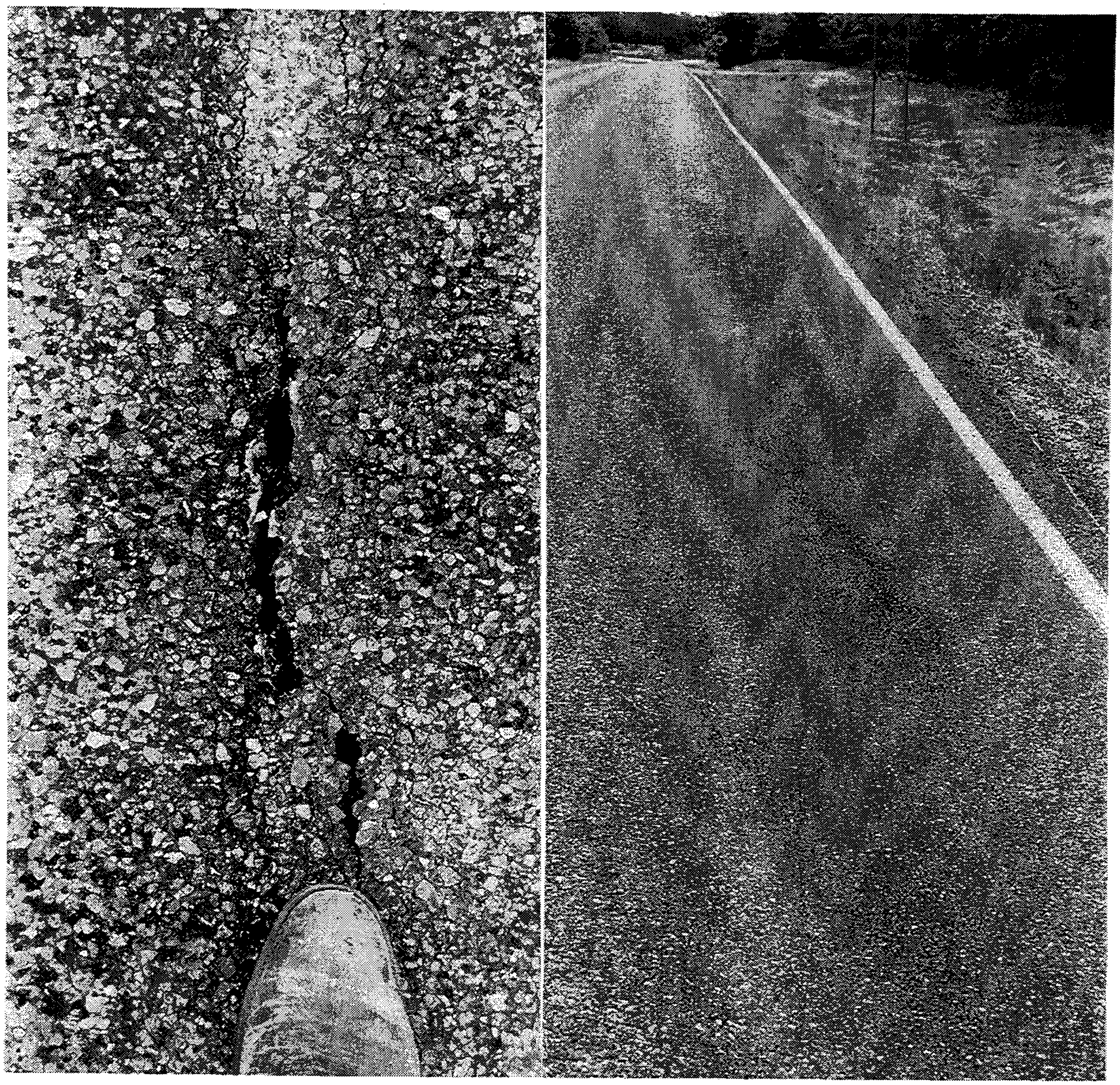
1.2 Surface seal. Fine edge crack (CL, PI 9-28).

SECTION 16

Road	County	Limits	Date	Structure
FM 1687	Brazos	FM 50 to OSR	5-96	LSB (60-40) (XX), XX", & 2 CST

Mile

3.2 - 3.6 Longitudinal cracks on uphill section. Some severe cracking initiating, situation deteriorating. (CH, PI > 50).



Conclusion

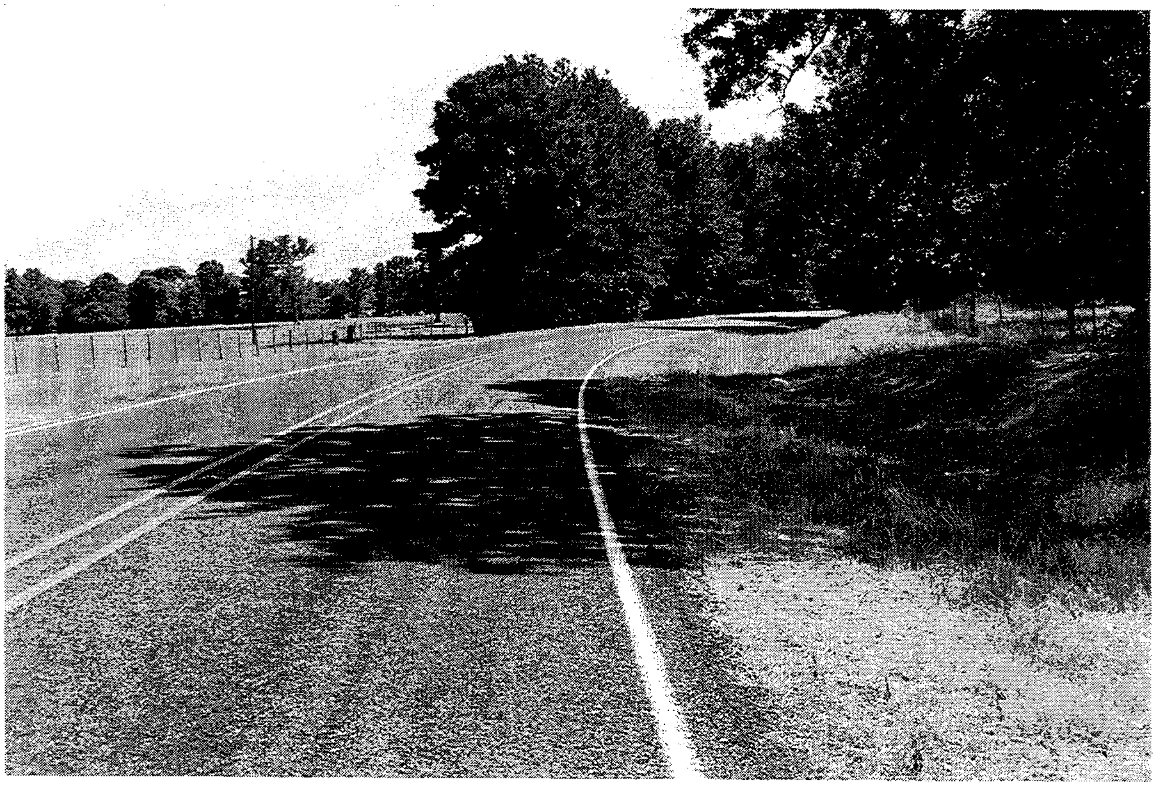
Overall condition good - concern about deteriorating longitudinal cracks in short section at end of project. Problem section after bridge, check for change in soil type.

SECTION 17

Road	County	Limits	Date	Structure
FM 977	Leon	3.2 Mi W of FM 1119 to FM 1119	2-96	LSB (30-60) (4%), & 2 CST

Mile

0.3 - 0.4 Slight longitudinal cracking at pavement edge WB nearest trees. Remainder looks good. (CL, PI < 25).



Conclusion

In general, a good performing section. Longitudinal cracking found in only one location. Many trees, but little or no cracking. Large tree, no crack.

SECTION 18

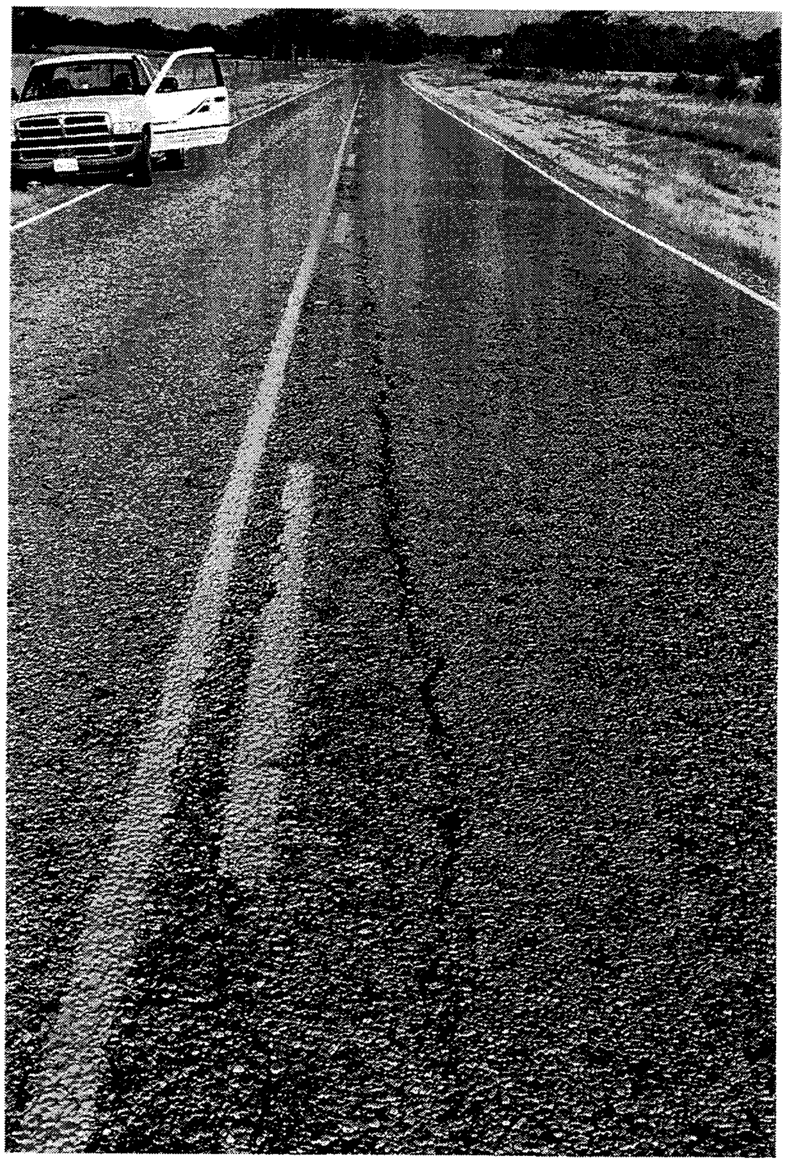
Road	County	Limits	Date	Structure
FM 978	Madison	FM 39 to FM 2289	4-96	LSB (40-60) (5%), & 2 CST

<u>Mile</u>

0 - 1.5 Very good/ No cracks, lots of trees.

1.5 - 2.0 Severe longitudinal cracks - no trees.

Some transverse. Mostly westbound lane. (CH, PI > 35).

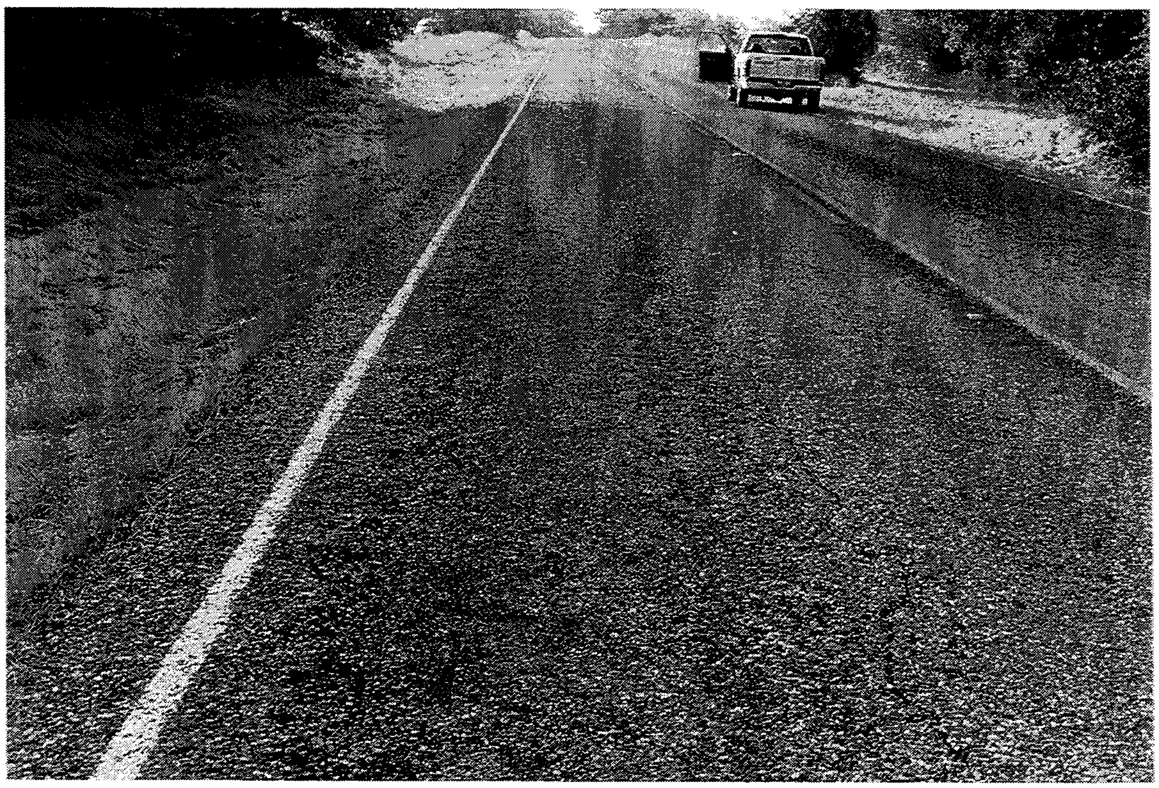


SECTION 18

Road	County	Limits	Date	Structure
FM 978	Madison	FM 39 to FM 2289	4-96	LSB (40-60) (5%), & 2 CST

Mile

1.7	Cracks around trees at edge.
2.0 - 2.5	No cracks - good condition.
2.8 - 3.2	Large cracks around trees. Several locations. (CH, PI > 35).
4.6 - 4.8	Localized moderate cracking around trees. (CL, PI > 25).
6.6 - 7.8	Localized moderate cracking along tree line.



Conclusion

Good condition, "good ride" - Severe long cracking in localized places. Variable soils along project.

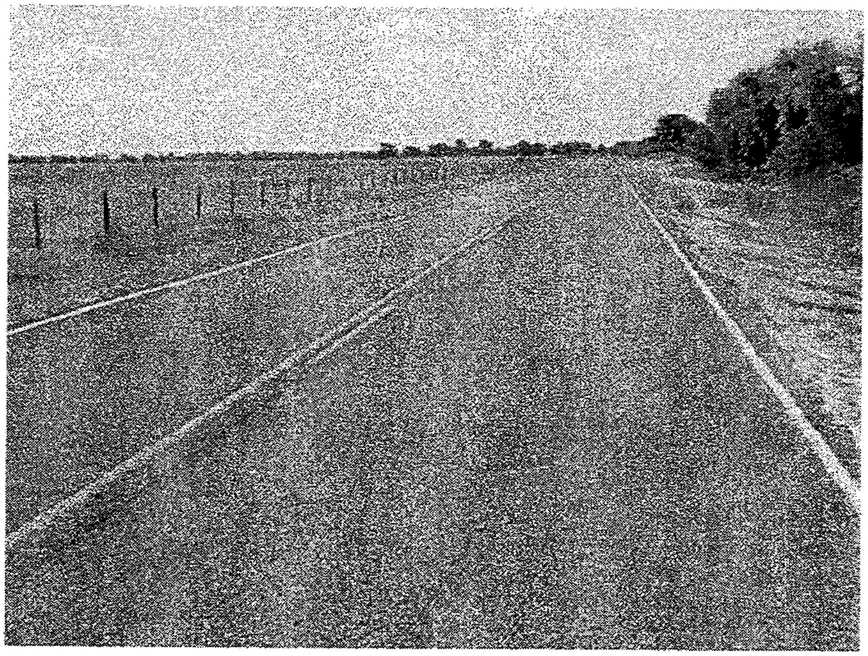
SECTION 19

Road	County	Limits	Date	Structure
FM 1373	Robertson	Falls County Line to 6 Mi E	---	LSB (50-50) (XX), & 2 CST

Mile

0 - 0.8 Excellent condition - no trees.

0.8 - 1.1 Trees 9 m (30 ft) from edge - no cracking. (CL, ML PI < 25).



SECTION 19

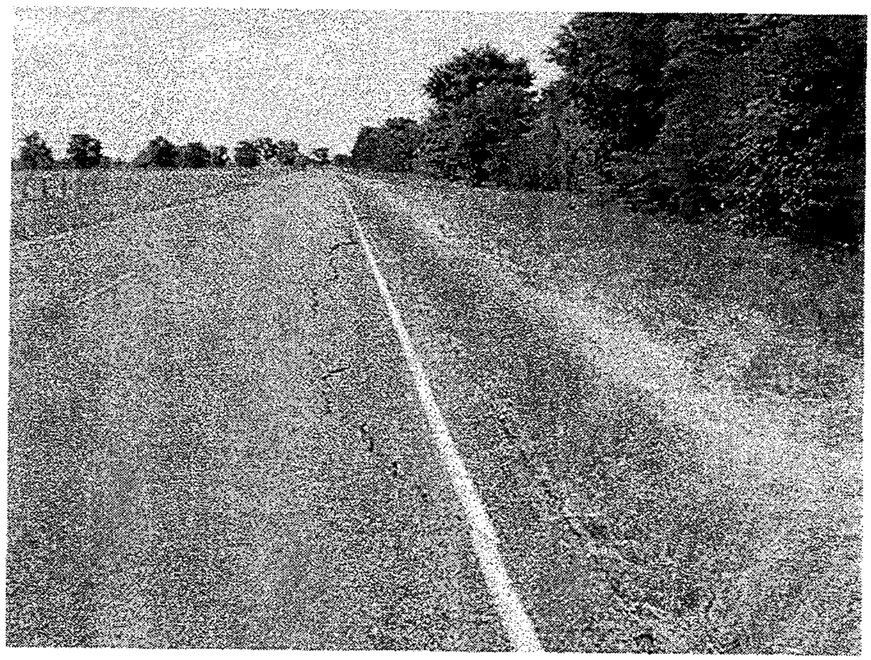
Road	County	Limits	Date	Structure
FM 1373	Robertson	Falls County Line to 6 Mi E	---	LSB (50-50) (XX), & 2 CST

Mile

1.1 - 1.7 Trees close to edge of road. Two areas with minor edge cracking.
30 m (100 ft) of severe edge cracking/faulting at 2.17 km (1.35 mi).
(CH, Pi > 35).

1.7 - 6.2 Road in excellent condition - few trees. Even when big tree close to edge - no problems, (CL, PI < 25).

5.7 Minor long cracking next to trees.

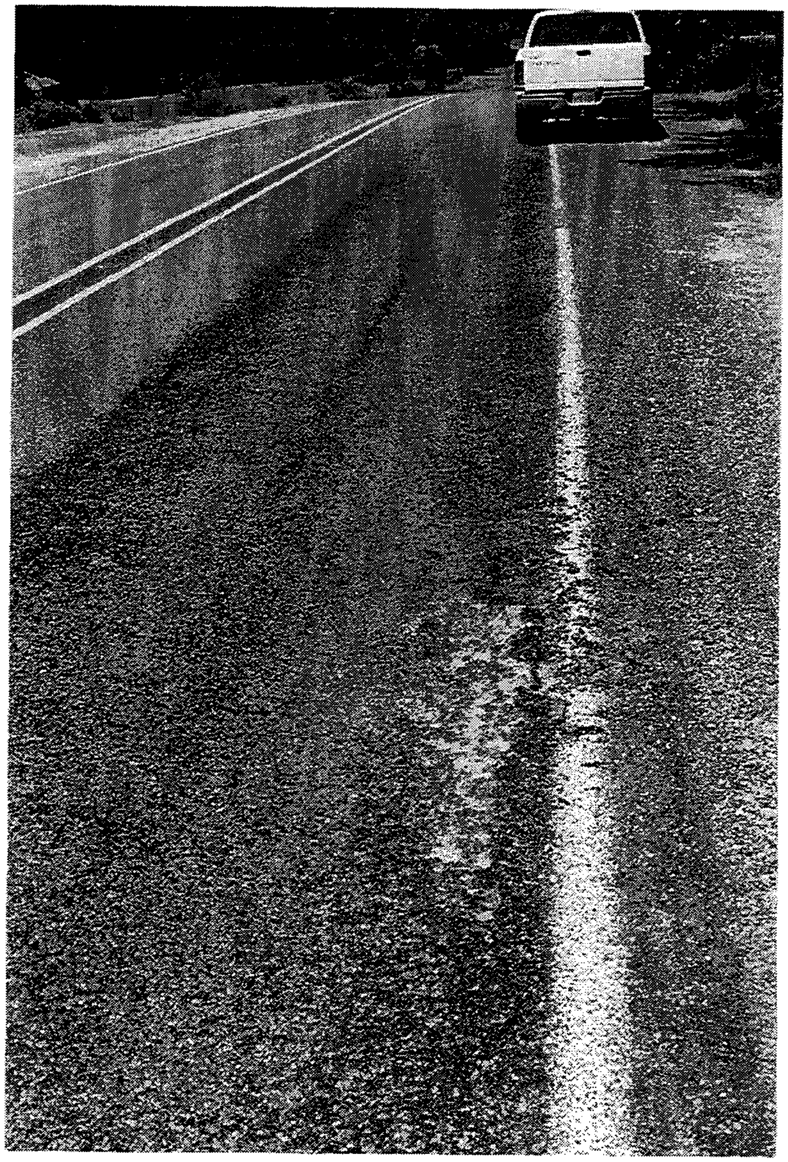


Conclusion:

Road in excellent condition. First few km (mile) red/tan silty/sandy soils, only 30 m (100 ft) of edge cracking in entire 9.5 km (6 mi) project.

SECTION 20

Road	County	Limits	Date	Structure
FM 3178	Leon	FM 1511 to FM 542	---	CSB (50-50) (4%), & 2 CST

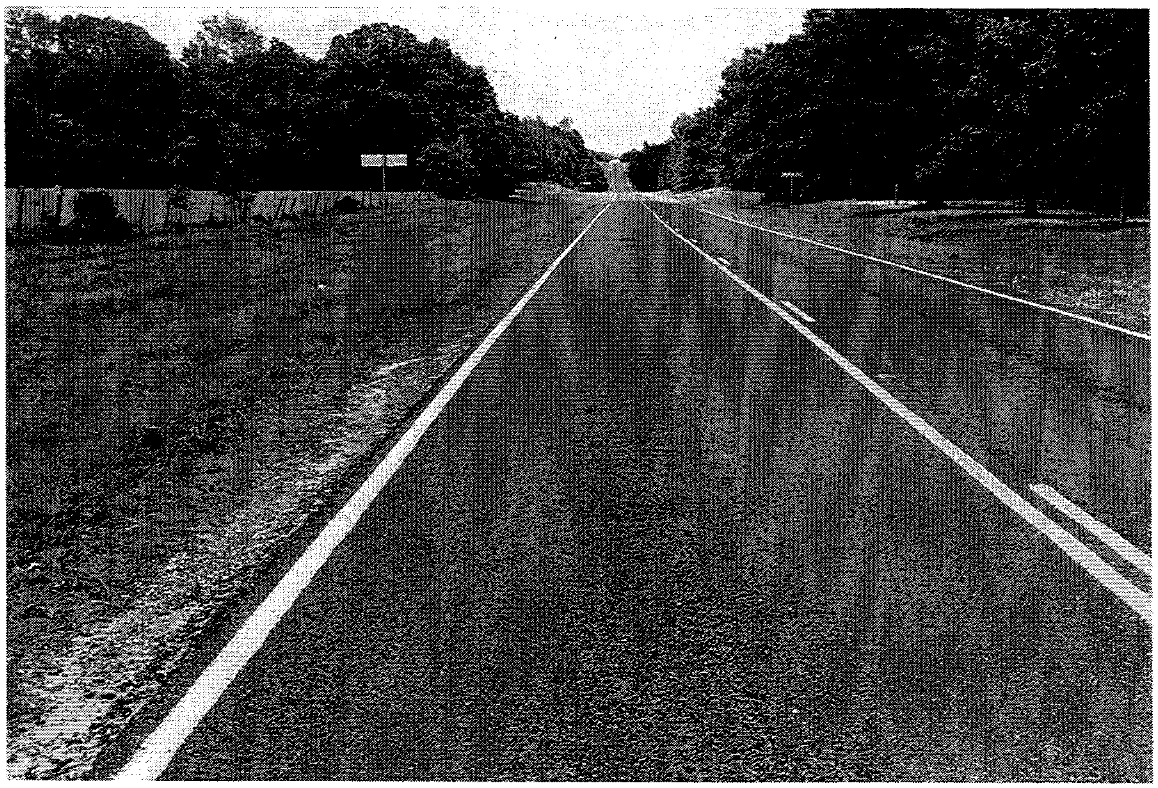


Conclusion

Sandy Subgrade. Highway in excellent condition. No cracking in entire section - one small failure at 2.9 km (1.8 mi). (Photo of failure + rutted section ± 30 m (± 100 ft)). Check FWD for overall structural strength. (SM, SC, PI < 10).

SECTION 20

Road	County	Limits	Date	Structure
FM 3178	Leon	FM 1511 to FM 542	---	CSB (50-50) (4%), & 2 CST



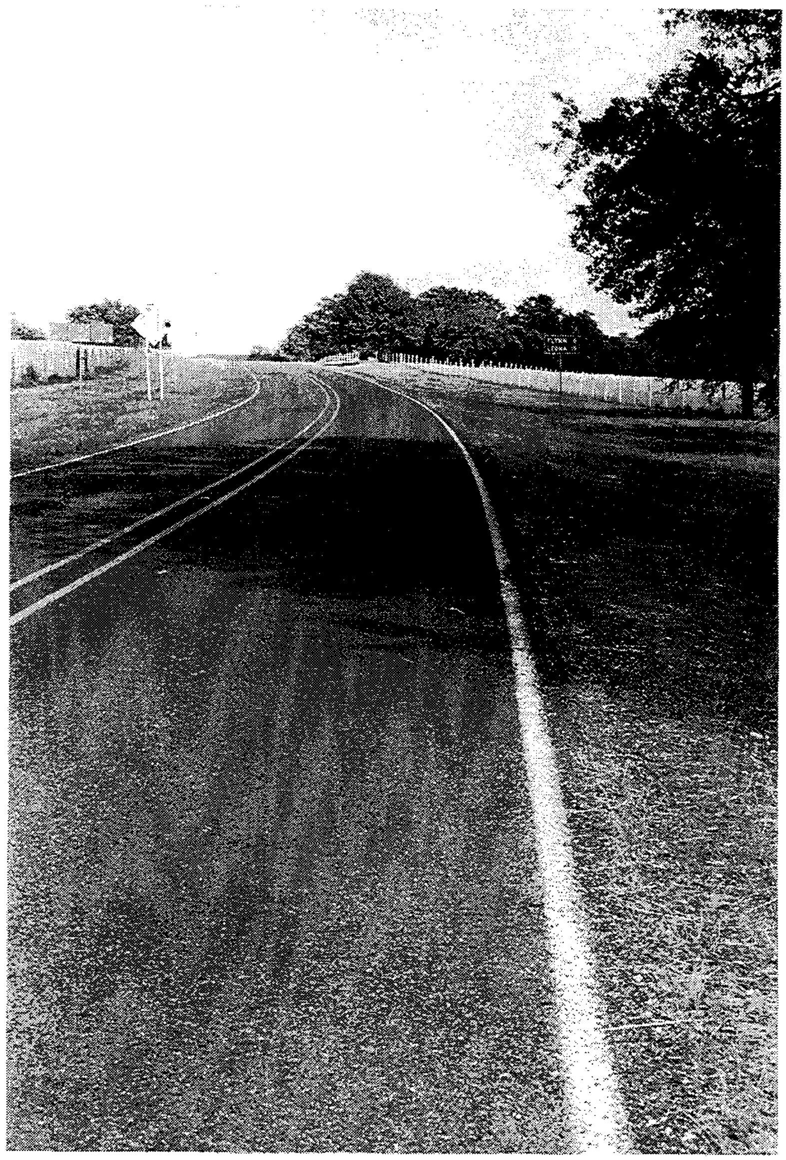
Conclusion

Sandy Subgrade. Highway in excellent condition. No cracking in entire section - one small failure at 2. 9 km (1.8 mi). (Photo of failure + rutted section ± 30 m (± 100 ft)). Check FWD for overall structural strength. (SM, SC PI < 10).

SECTION 21

Road	County	Limits	Date	Structure
FM 977	Leon	FM 3 to 2 Mi E	5-96	CSB (50-50) (4%), & 2 CST

Excellent performance no cracks - even though trees very close to highway in several locations. (CL, PI < 15).



Conclusion: Good Performer.

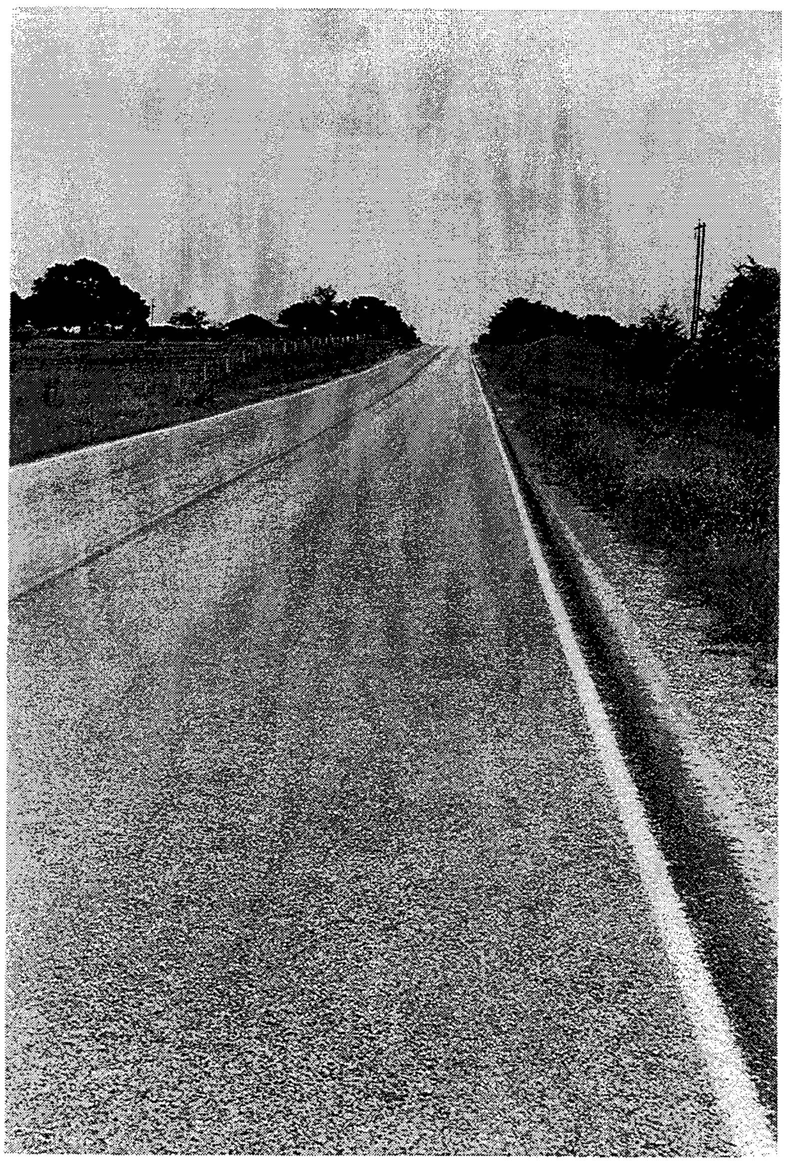
SECTION 22

Road	County	Limits	Date	Structure
FM 1935	Washington	FM 390 to End	---	-----

<u>Mile</u>

0 - 2.8 Excellent, no obvious defect, smooth ride.

Short lengths of slight long cracking in two short locations. Not a problem. One of the best sections in study.



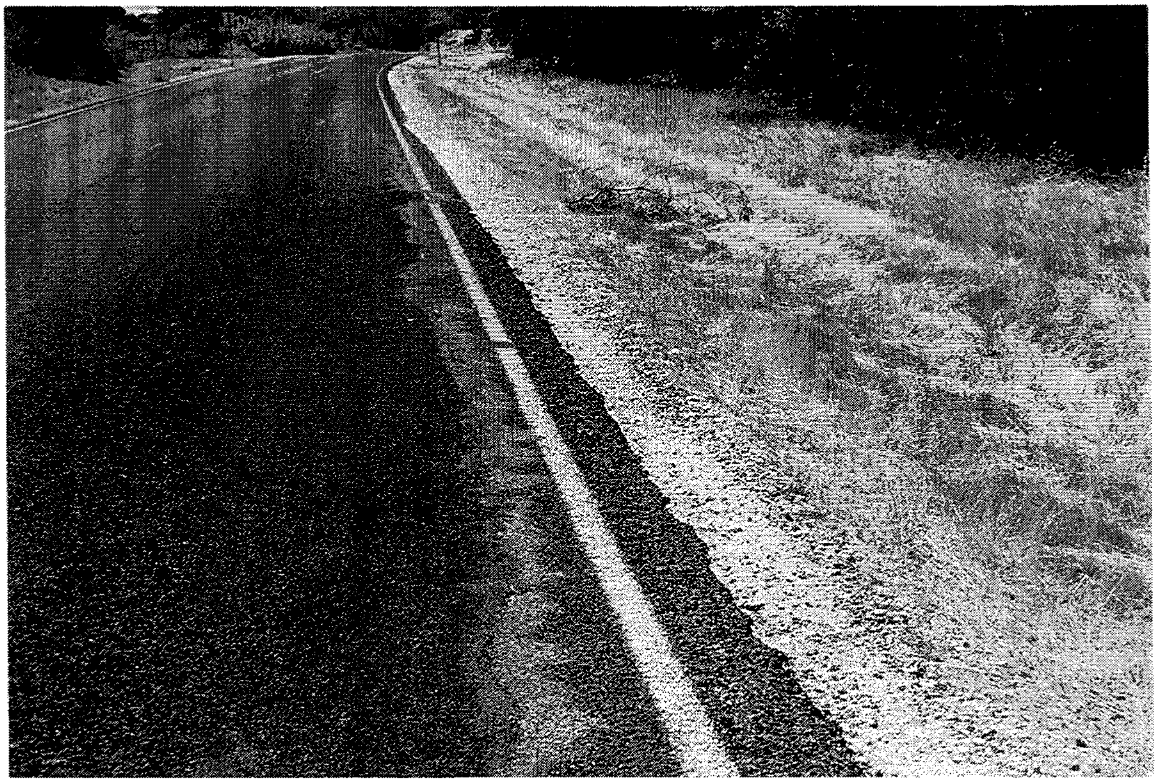
<u>Conclusion</u>: Good Performer.

SECTION 23

Road	County	Limits	Date	Structure
FM 975	Burleson	SPRR to 5.5 Mi S	6-96	LSB (60-40) (3%), & 2 CST

Mile

0.2 - 1.1 Longitudinal cracking along tree line. Cracks filled HMAC - Some severe, some loss in ride around 0.8 - 0.9 problem area. Some edge failures. (CH, PI > 55).



SECTION 23

Road	County	Limits	Date	Structure
FM 975	Burleson	SPRR to 5.5 Mi S	6-96	LSB (60-40) (3%), & 2 CST

Mile

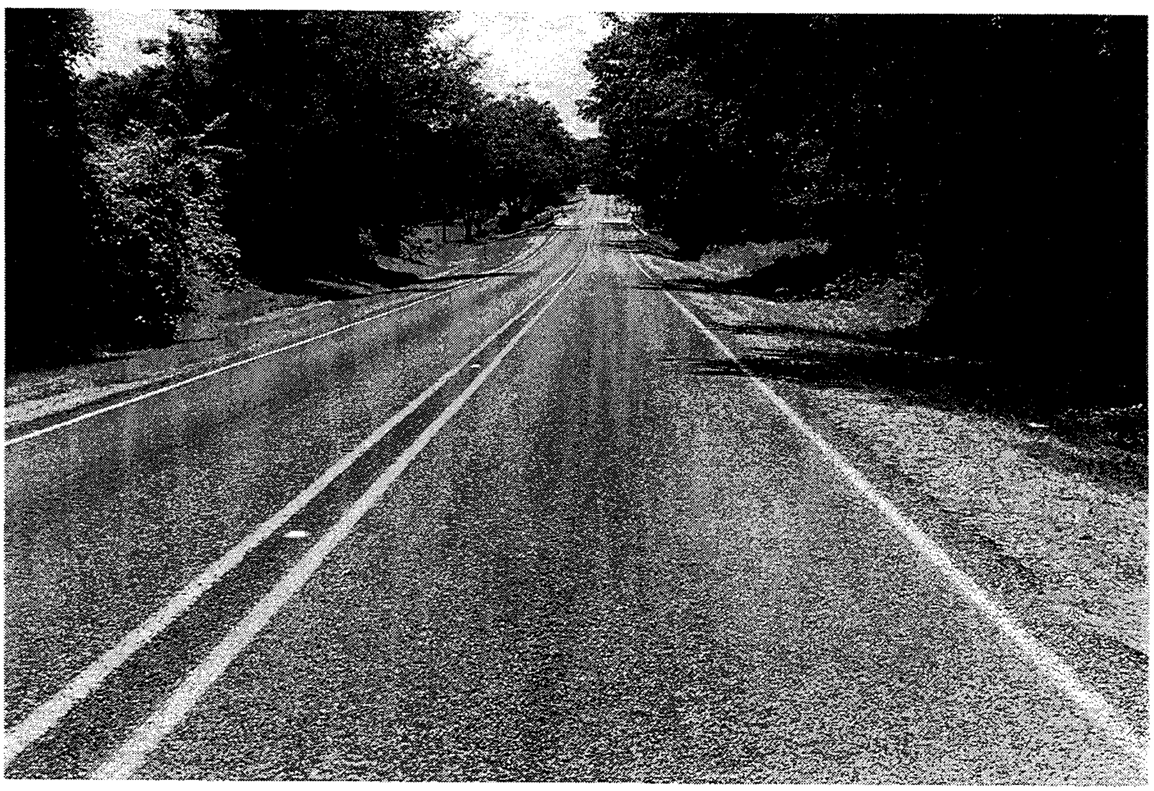
1.1 - 1.9 No cracks, several large trees close to road - check soil type. (CH, PI > 25).

1.9 - 2.5 Lots of trees - some cracking in places.

2.5 - 3.5 Many major trees - no cracks. (CL, PI < 15).

3.9 - 4.5 Moderate longitudinal. Throughout this length. (CH, PI > 15).

4.6 - 5.3 No problems.



Conclusion

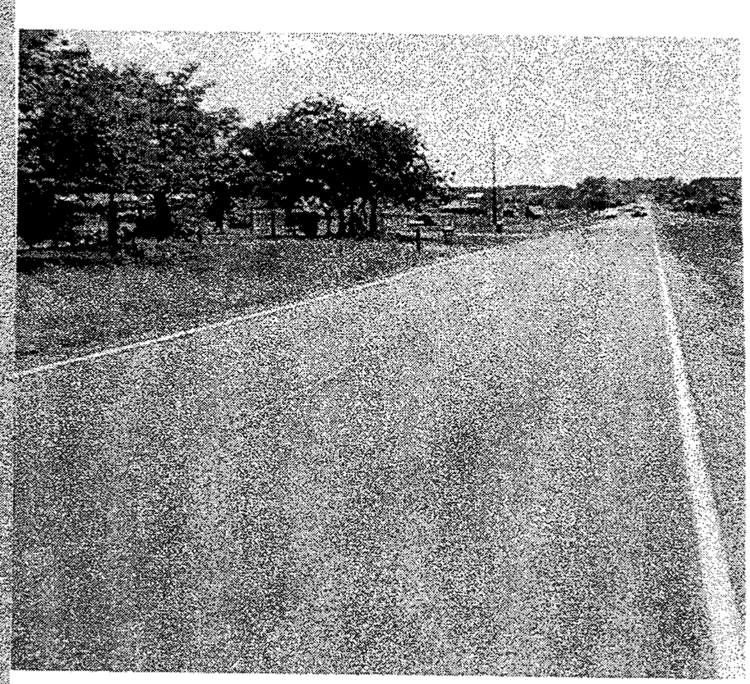
Very variable section - overall condition good. Some localized problem areas. Check for varying soil types.

SECTION 24

Road	County	Limits	Date	Structure
FM 2446	Robertson	Intersection of FM 46 to 1.0 Mi E	---	-----

<u>Mile</u>

0 - 1.0 In general, section is in very good condition. No cracking or rutting. Many trees close to edge. One edge failure about 95 m (300 ft) from start of section.



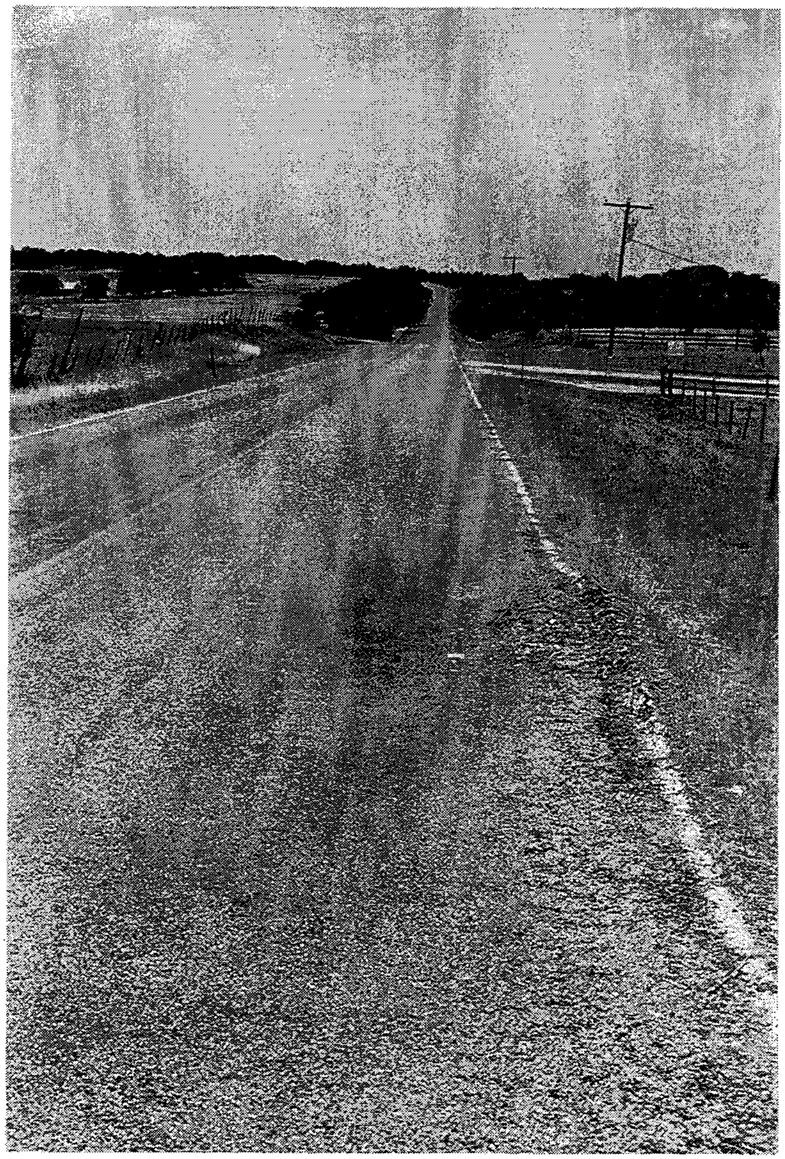
<u>Conclusion</u>: Good Performer.

SECTION 25

Road	County	Limits	Date	Structure
FM 2780	Washington	FM 1697 to FM 1697	---	-----

Mile

0 - 0.6 Several edge failures in first .8 km (.5 mi) structural problem. (ch, pi > 35, with organics).



SECTION 25

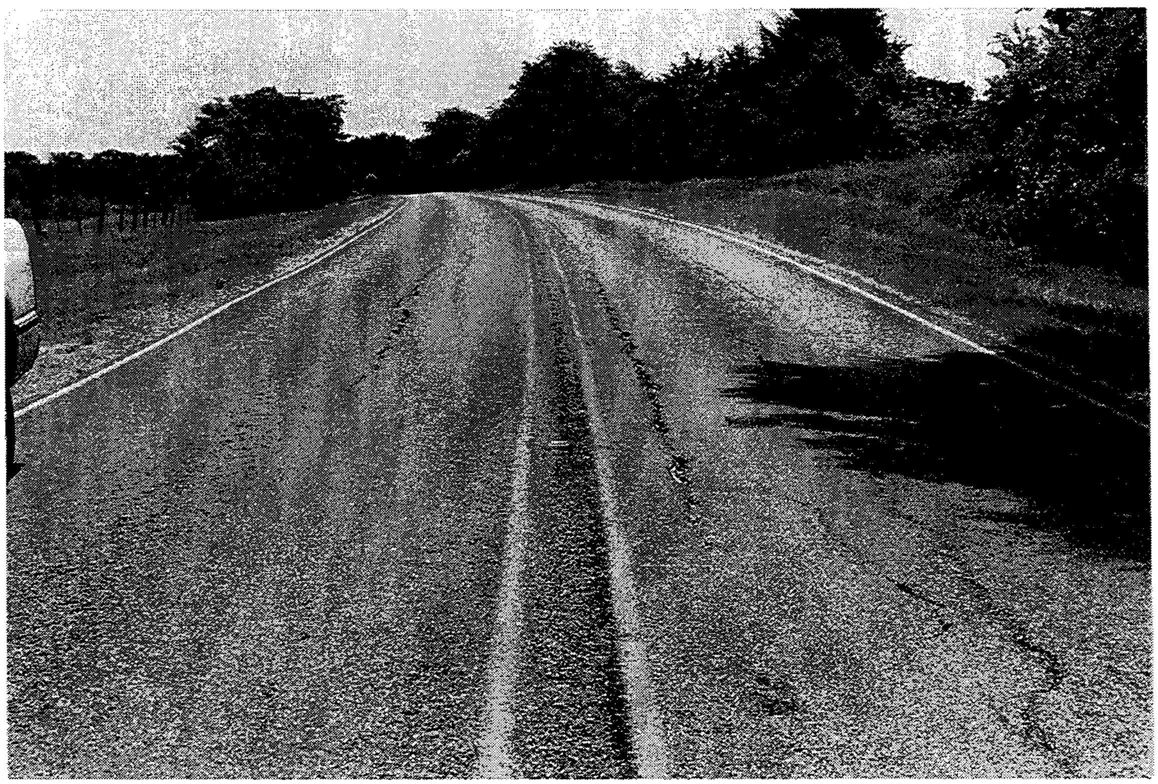
Road	County	Limits	Date	Structure
FM 2780	Washington	FM 1697 to FM 1697	---	-----

<u>Mile</u>

0.6 - 1.0 Ok, no problem.

1.0 - 1.25 Edge cracking on tree line, some severe. (CH PI > 35).

1.3 - 1.5 Major cracks close to tree - stiff base. Very bad.



SECTION 25

Road	County	Limits	Date	Structure
FM 2780	Washington	FM 1697 to FM 1697	---	-----

Mile

1.5 - 2.3	Many trees - section OK.
2.4 - 2.8	Moderate longitudinal cracking along tree line.
2.8 - 4.0	Moderate longitudinal cracking intermittent.
4.0 - 6.2	Road performing well, no cracking.
6.2 - 7.3	Regular moderate longitudinal cracks.
7.3 - 8.0	Good, no cracks.
8.0 - 8.8	Longitudinal cracks along tree line.

Conclusion

Most variable section in study. Some of everything. Structural failure, severe edge cracking, good sections. Check soil on this project. Very poor in several sections.

APPENDIX B

A SIMPLE TEST TO DETECT ORGANIC MATTER IN SOILS

ORGANIC MATTER

Soil organic matter is very heterogeneous, containing a diverse population of live, active organisms as well as plant and animal residues in different stages of decomposition. Organic matter in soil may be responsible for high plasticity, high shrinkage, high compressibility, low hydraulic conductivity, and low strength. Soil organic matter is complex, both chemically and physically, and a variety of reactions and interactions between the soil and the organic matter are possible. In residual soils, organic matter is most abundant in the surface horizons. Organic particles may range down to 0. 1 mm (3.90 mils) in size. The specific properties of the colloidal particles vary greatly depending upon parent material, climate, and stage of decomposition.

Organic particles may be strongly adsorbed on mineral surfaces, and this adsorption modifies both the properties of the minerals and the organic matter itself. Soils containing significant amounts of decomposed organic matter are usually characterized by a dark gray to black color and an odor of decomposition. At high moisture contents, decomposed organic matter may behave as a reversible swelling system. The maximum compacted densities and compressive strength decrease significantly with increased organic content. Increased organic content also causes an increase in the optimum water content for compaction.

The effect of organic matter on the strength and stiffness of soils depends largely on whether the organic matter is decomposed or consists of fibers which can act as reinforcement. In the former case, both the undrained strength and the stiffness, or modulus, are usually reduced as a result of the higher water content and plasticity contributed by the organic matter. In the latter, the fibers can act as reinforcements, thereby increasing the strength.

Organic carbon can inhibit the reaction between calcium and clay mineral surface. This occurs generally because the organic molecule is quite complex and can adsorb calcium cations or interact with soil exchange sites and hence prevent them from reacting with the soil as they normally would to produce cation exchange and pozzolanic reaction. It is debatable to exactly what level of organic material is enough to substantially interfere with soil-lime reactivity. The type of soil being stabilized and the nature of the organic material influence this.

Organic matter may affect the reactivity of lime/soil systems by a "coating effect." This is similar to the coating effect of Fe_2O_3 in well-drained soils of temperate regions. Organic matter is often deleterious in the A horizon. The fact that lime reactivity indications vary with soil environment and geographical location is evidenced by Hardy's work with tropical and subtropical soils. No single property proved to be an accurate predictor of lime reactivity for these soils. Two or more soil properties or characteristics are required. Unlike the midwestern soils studied by Thompson, soil profile drainage is not a valuable index of lime reactivity for tropically and subtropically weathered soils, nor is the calcium-magnesium ratio a useful weathering index relating to probable lime reactivity in these soils. Furthermore, high extractable iron contents do not significantly inhibit the lime reactivity of tropical and subtropically weathered soils. Hardy's work revealed that organic carbon in excess of 1 percent hindered stabilization.

As a general rule, an organic content in excess of 1 percent is cause for concern that the organic material will interfere with the pozzolanic reaction. The solution to this problem will range from removing the soil because it cannot be effectively stabilized with a calcium-based stabilizer to simply adding additional lime.

DETERMINATION OF ORGANIC MATTER

Organic matter can be readily detected by treatment of the soil with a 15 percent hydrogen peroxide solution. The H_2O_2 and organic matter react to give vigorous effervescence. This is more of a qualitative determination. Quantitative analysis methods for soil organic matter are given by the American Society for Testing and Materials (1970).

pH tests on stabilized soil mixtures can be used to determine the organic content.

Procedure:

1. Standardize the pH meter with a buffer solution having a pH of 12.00.
2. Weigh to the nearest 0.01-gram representative samples of air-dried soil passing the # 40 sieve and equal to 25.0 grams of oven-dried soil.
3. Pour the soil samples into 150-ml plastic bottles with screw-top lids.
4. Add cement/lime.
5. Thoroughly mix the soil and the stabilizer.

6. Add sufficient distilled water to make a thick paste. (CAUTION: Too much water will reduce the pH and lead to an erroneous result.)
7. Stir the soil-cement and water until thorough blending is achieved.
7. After 15 minutes, transfer parts of the paste to a plastic beaker and measure the pH.
9. If the pH is 12.0 or greater, the soil organic matter content should not interfere with the cement-stabilizing mechanism.

APPENDIX C

A SIMPLE PROCEDURE TO DETECT THE PRESENCE OF SULFATES IN SOILS

(Reference Bredenkamp, S., and Lytton, R. L., "Reduction of Sulfate Swell in Expensive Clay Subgrades in District 18," TTI Report 1994-5, Nov. 1994)

The equipment needed to perform a field evaluation of the sulfate content in soils, includes the following:

a. Wide mouth plastic containers with water-proof lids,
b. Distilled water,
c. Battery driven digital scale that can measure up to 500 g (Figure C1),
d. Hand held conductivity meter (Figure C2), and
e. Calibration solutions for the conductivity meter.

The entire package costs less than $600.

The procedure is as follows:

Step 1: Find the location where the sulfate test is to be performed, and use an auger to obtain two small soil samples at approximately 10 and 20 cm below the soil surface. Only 5 grams of soil is needed to perform the test.

Step 2: Weigh approximately 5 g of each soil sample into two separate plastic containers. If the soil is wet, break lumps apart and leave the soil to air-dry for one to two hours. Record the exact dry weight of the samples.

Step 3: Now add distilled water with a mass of <u>exactly</u> 20 times the dry weight of the soil sample to the dry sample. Tightly close the lid of the plastic container and shake vigorously until the soil dissolves and forms a homogeneous solution.

Step 4: Calibrate the conductivity meter as described in the instruction manual accompanying the device.

Step 5: Take conductivity measurements on each soil water mixture, and record the data in milli Siemens (mS).

Note: 1 uS = 0.001 mS

From our limited experience, mixtures with a conductivity of more than 8 mS have a potential to cause problems.

Step 6: Use the following equation to determine an estimated amount of expansion that would occur upon lime stabilization:

% Expansion = 0.43Ln (EC) = 0.488(EC) - 0.784

Where EC = Electrical Conductivity measurement in mS, and % Expansion = Anticipated swell after curing in moist environment for seven days.

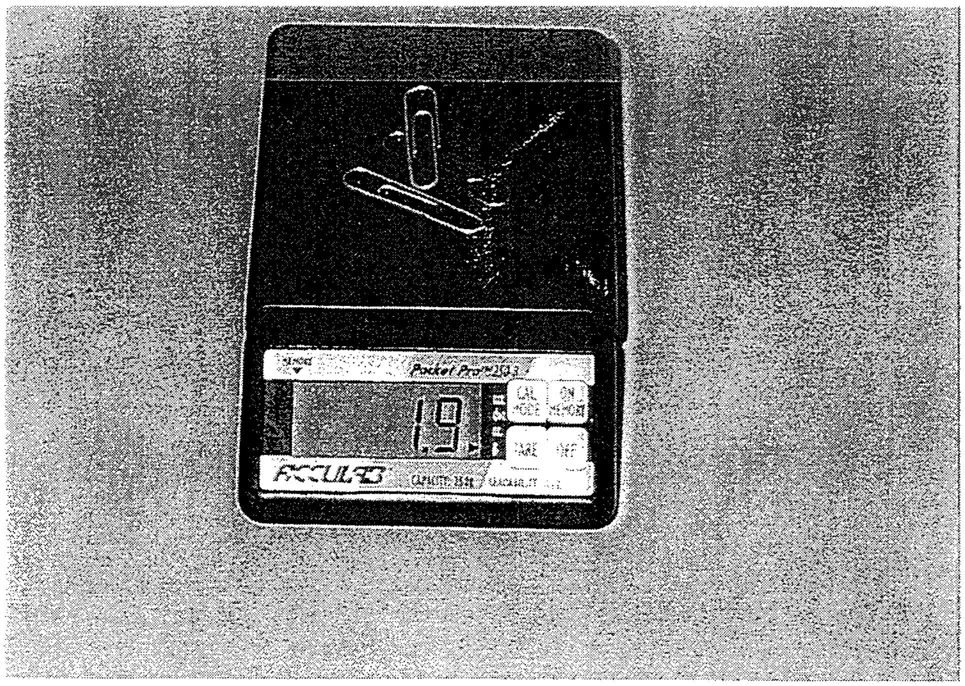


Figure C1. Battery Driven Digital Scale.

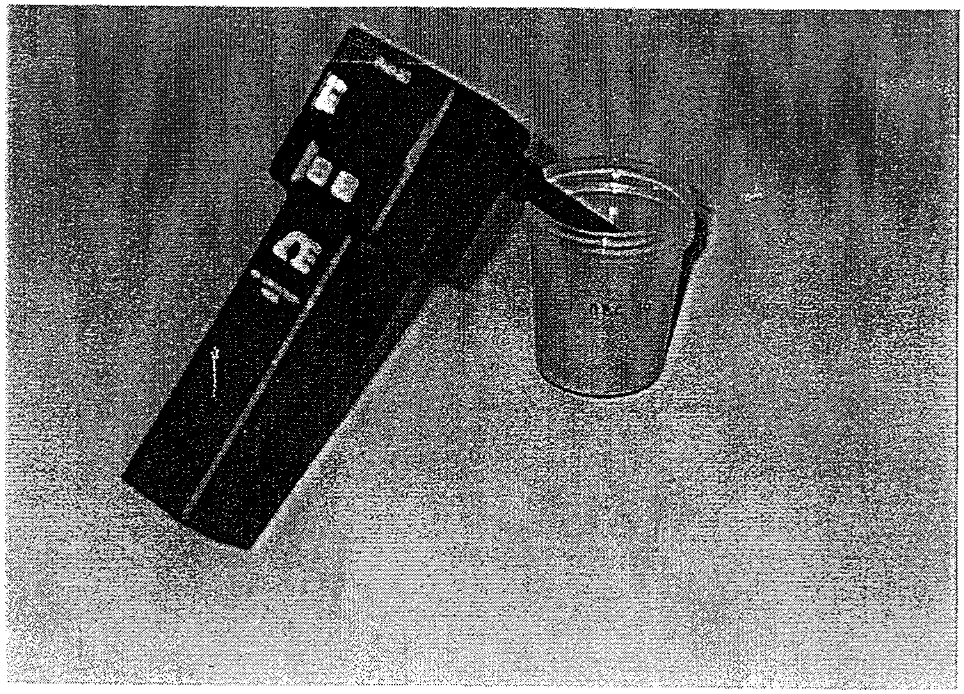


Figure C2. Handheld Electrical Conductivity Meter.