Technical Report Documentation Page

1. Report No. FHWA/TX-01/1712-4	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle SELECTING REHABILITATION OPTIONS FOR FLEXIBLE PAVEMENTS: GUIDELINES FOR FIELD INVESTIGATIONS		5. Report Date January 2001 6. Performing Organization Code
7. Author(s) Tom Scullion		8. Performing Organization Report No. Report 1712-4
9. Performing Organization Name and Address Texas Transportation Institute The Texas A&M University System College Station, Texas 77843-3135		10. Work Unit No. (TRAIS) 11. Contract or Grant No. Project No. 0-1712
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P. O. Box 5080 Austin, Texas 78763-5080		13. Type of Report and Period Covered Research: September 1999-August 2000 14. Sponsoring Agency Code

15. Supplementary Notes

Research performed in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.
Research Project Title: Development of Strategy Selection Procedure for Rehabilitation of Flexible Pavements

16. Abstract

The first step in selecting the optimum rehabilitation strategy for a flexible pavement is identifying the cause of the existing pavement distress. The rehabilitation selection process is often straightforward once the cause has been identified. This report presents an updated summary of the techniques and interpretation guidelines that have been developed by the Texas Transportation Institute over the past two decades. A summary is presented of the different distress types frequently found on Texas flexible pavements. The possible causes for each are described together with guidelines on how to conduct a failure investigation. For each distress type a series of rehabilitation options are also presented.

In recent years nondestructive testing procedures have developed to a stage where they can provide critical inputs to this selection process. In particular the application of both Falling Weight Deflectometer (FWD) and Ground Penetrating Radar (GPR) technology has advanced to where they are now used by Texas Department of Transportation (TxDOT) Engineers as routine tools. In this report guidelines are presented on how TxDOT personnel can incorporate both GPR and FWD information into the evaluation process.

In addition, several districts are currently using full depth pavement reclamation techniques to strengthen low volume roadways. As part of this study, recommendations were also developed to provide the design engineer with new laboratory procedures for selecting the optimum stabilizer type and stabilizer content for reclamation projects.

The integrated nondestructive testing procedures and laboratory tools described in this report are beginning to be used on a regular basis by several TxDOT districts. They have also been incorporated into a pavement rehabilitation school which was developed as part of this project.

17. Key Words Flexible Pavements, Rehabilitation, Distress, NDT, GPR, FWD		18. Distribution Statement No restrictions. This document is available to the public through NTIS: National Technical Information Service 5285 Port Royal Road Springfield, Virginia 22161	
19. Security Classif.(of this report) Unclassified	20. Security Classif.(of this page) Unclassified	21. No. of Pages 86	22. Price

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SELECTING REHABILITATION OPTIONS FOR FLEXIBLE PAVEMENTS: GUIDELINES FOR FIELD INVESTIGATIONS

by

Tom Scullion
Research Engineer
Texas Transportation Institute

Report 1712-4
Project Number 0-1712
Research Project Title: Development of Strategy Selection Procedure for Rehabilitation of Flexible Pavements

Sponsored by the
Texas Department of Transportation
In Cooperation with the
U.S. Department of Transportation
Federal Highway Administration

January 2001

TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, TX 77843-3135

DISCLAIMER

ACKNOWLEDGMENTS

This project was funded by TxDOT's Research Management Committee (RMC 1) on pavements and materials. The project director was Dr. Andrew Wimsatt, P.E., the pavement engineer of the Fort Worth District. Andrew's support and willingness to field test new ideas are acknowledged and greatly appreciated. Mr. Spencer Guthrie of TTI developed the Tube Suction Test protocol given in the Appendix.

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CHAPTER 1
INTRODUCTION

This report attempts to pull together in one document guidelines on how to conduct field and laboratory investigations to identify the cause of distresses in flexible pavements. The optimum rehabilitation strategy for any flexible pavement can not be determined unless the cause of the distress is known. Experience has shown that this is often a complex issue with many factors contributing to pavement failures. This report presents an updated version of TTI's recommended pavement evaluation process, as described in the following three chapters:

Chapter 2	Provides updated guidelines on how to interpret the visual condition data commonly found on Texas pavements. It also suggests possible causes and rehabilitation options.
Chapter 3	Provides a summary of the advances that have been made in nondestructive testing capabilities and how they can be used to identify subsurface problems.
Chapter 4	Addresses new laboratory test methods that have been recently developed to assist in selecting the best stabilizer to use in pavement reclamation projects. Full depth reclamation is rapidly growing within TxDOT, and the guidelines presented are based on recent district experiences.

The level of analysis required to select the appropriate rehabilitation treatment depends on the importance of the route and also the past pavement performance. If the pavement has performed well and if only minor distresses are found after several years in service then a full investigation is not warranted. However in many instances this is not the case. Pavements which fail rapidly or develop unanticipated distress types or pavements which have deteriorated too far for standard maintenance treatments are good candidates for application of the techniques described in this report.

Based on the experience gained in this research study and other forensic investigations the recommended approach to conducting a full field investigation of flexible pavements is as follows:

Step 1 Assemble all background information (typical sections, traffic information, construction dates, maintenance activities, etc).

Step 2 Conduct a Ground Penetrating Radar (GPR) and visual distress survey of the project. TxDOT's current GPR systems have integrated video which can be used to identify and document the existing surface distress.

Step 3 Review the GPR and distress data and develop a plan for additional field testing. This review should involve strength testing with the Falling Weight Deflectometer (FWD) or Dynamic Cone Penetrometer (DCP). It should also involve collecting field samples to verify a certain subsurface condition or be used to run laboratory tests.

Experience to date has indicated that a thorough visual distress survey with a combined GPR and FWD survey is adequate for the majority of flexible pavement investigations. The integrated FWD and GPR approach is essential; both devices provide useful information but when combined they give a comprehensive survey of the subsurface conditions that influence pavement performance. This approach maximizes the use of nondestructive testing. However limited field verification with coring or bag samples is critical.

The pavement investigation approach recommended in this report has been integrated into a Pavement Rehabilitation School for TxDOT. In study 1712 this school was taught twice. The approach to establishing the school in a host district is as follows. First the host district nominates a number of upcoming pavement rehabilitation projects. These nominations ideally are difficult projects where the district staff are uncertain about which strategy to use. The projects are then tested with the techniques recommended in this report, and a full field and laboratory investigation is completed. The results of this investigation are included as case studies in the school. Students are first instructed on how to process data from both the FWD and GPR. The students are then shown the video of the existing pavement condition and are given the raw FWD and GPR data to perform their own analysis.

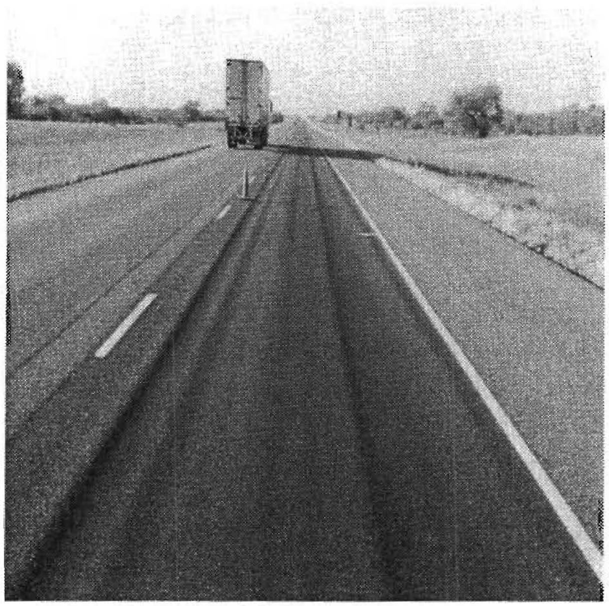
The selection of the appropriate rehabilitation treatments for its aging highway network is one of the top priorities of TxDOT. The guidelines developed in this study can hopefully help in this regard.

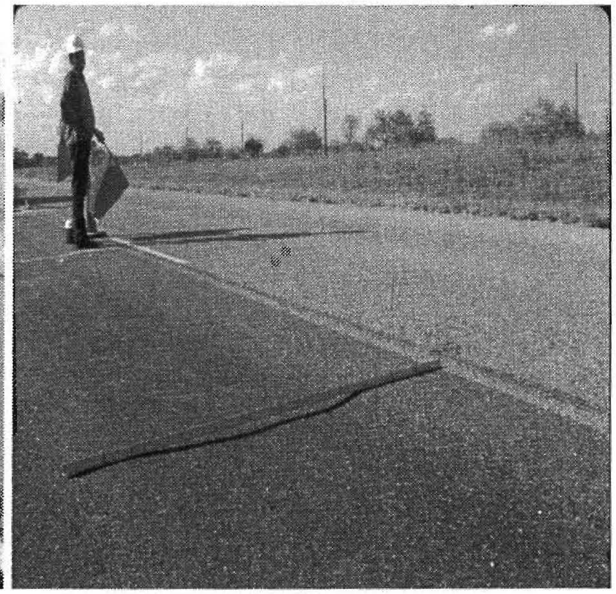
CHAPTER 2
INTERPRETING VISUAL DISTRESS DATA

The first step in the process of selecting a rehabilitation strategy of any roadway is interpretation of the existing pavement condition data. TxDOT has an existing Pavement Management Information System (PMIS) which provides an annual visual condition assessment of both surface distress and riding quality. The Texas Transportation Institute (TTI) has assisted with the development of this system and with providing interpretation guidelines for over the past two decades. This chapter gives a description of the various distress types that are recorded in the PMIS system. This is an updated version of the guidelines which were first developed at TTI by Finn and Epps in Study 214 in 1980 (*1*).

This information will provide the field engineers with guidelines on 1) how to identify the various types of distress, 2) mechanisms which may have caused this distress and 3) the possible maintenance and rehabilitation options. If the decision is made to conduct a full field investigation then guidelines are also presented on what type of laboratory and nondestructive field tests to conduct. These guidelines will help define the scope of the testing required to attempt to conclusively define the cause of the pavement problems. Experience has shown that the cause of premature pavement failures is often complex. Assistance in conducting full field investigations is provided to districts by TxDOT Construction Division's Materials and Pavements Section in Austin. A forensic support study is also ongoing to provide districts with specialized help from the research agencies in Texas.

RUTTING





DESCRIPTION

Rutting is the channeling of the pavement surface in longitudinal depressions, which develop in the wheel path area. Rutting is a traffic load-related distress.

POSSIBLE CAUSES

Hot Mix Asphalt (HMA) mix design	(too much asphalt, rounded aggregates, etc.)
Asphalt cement properties	(poor hot weather properties)
Defects in HMA layer	(stripping)
Structural deficiency	(weak base or subgrade, thin layers)
Compaction (density) - all layers	

FAILURE INVESTIGATION

1. Traffic (per lane)
   a. Accumulated equivalent 18 kip axle loads to date and future 20 year estimates
   b. Average of the10 heaviest wheel loads daily (ATHWLD)
2. Construction Records
   a. Mix design - all stabilized materials
   b. Material properties - base, subbase, subgrade
   c. Pavement layer thickness
   d. Date of construction and last overlay
3. Maintenance Activities
   a. Type
   b. Amount
   c. Effectiveness
4. Field Evaluation
   a. Condition survey, extent and severity of problem
   b. Drainage
   c. GPR survey (moisture in base, stripping in HMA, layer thickness)
   d. FWD survey (layer moduli)
   e. DCP survey (strength profile in base and subgrade)
   f. Samples (from rutted and non-rutted areas, cores and/or bag samples)
   g. Trenching (if all else fails: trench to identify problem layer)
5. Laboratory Testing of Field Samples
   a. HMA properties
      Hveem stability, water susceptibility
      Asphalt content, asphalt penetration, air void content
      Aggregate properties (gradation, absorption, shape, surface texture)
      Wheel tracker performance (Asphalt Pavement Analyzer (APA), Hamburg)
      Repeated load test
   b. Base, Subbase, Subgrade
      Gradation, field moisture content
      Triaxial classification
      Tube suction test (moisture susceptibility)
   c. Stabilized bases
      Asphalt treated (see item 5a above)
      Cement or lime - compressive strength before and after Tube Suction Test

REHABILITATION ALTERNATIVES

If rutting in upper HMA layer (with no defects in lower layers)
- Cold milling including profile requirements, with or without overlay
- Heater scarification with surface treatment or thin overlay
- Heater planning with surface treatment or thin overlay

If rutting in lower HMA layer
- Removal and replace
- Structural overlay (FPS 19W design)

If rutting in base or subgrade
- Full depth reclamation with stabilization (thin HMA pavements)
- Surface removal (base stabilization)
- Structural overlay (FPS 19W design)

If rutting is shallow (<0.5 inches) and no surface cracking
Micro-surfacing or other rut filling material

DISCUSSION

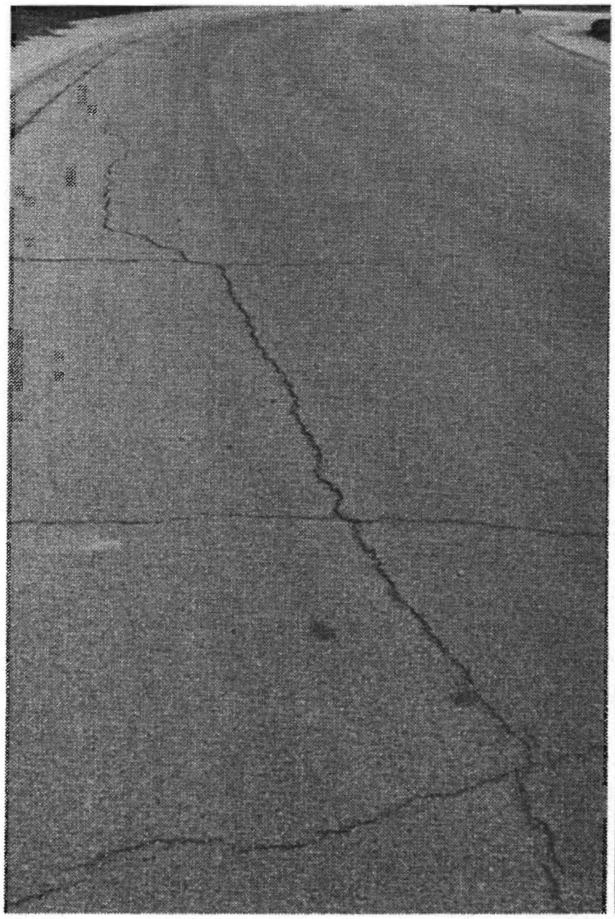
Rutting and alligator cracking are two of the most difficult types of distress to rehabilitate and require careful planning of alternative treatments. The layer causing the rutting must be determined, and a full structural analysis is often warranted. In general, wide ruts indicate that the problem may be deep within the structure. Narrow ruts are usually associated with problems in the upper HMA layer.

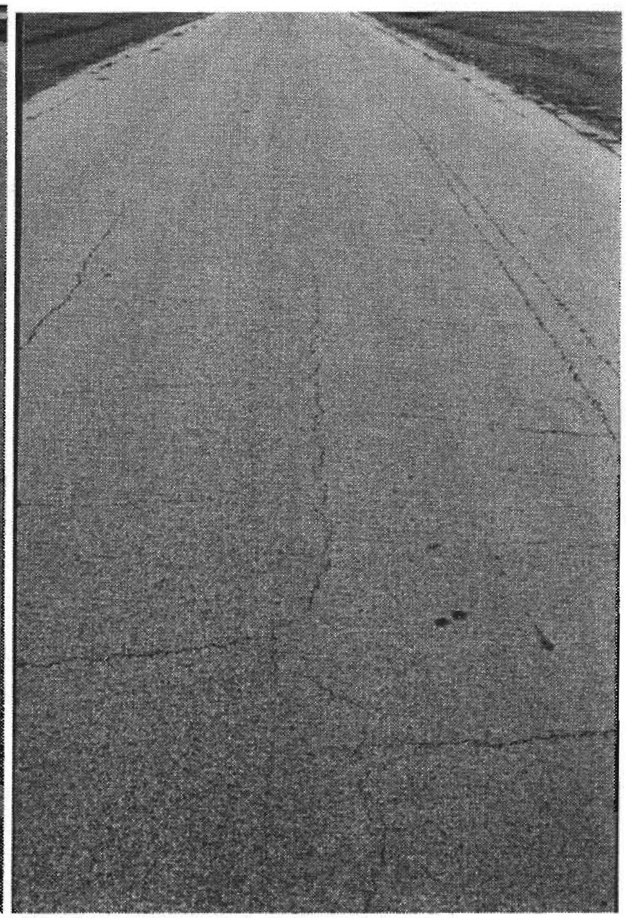
Rutting is most frequently associated with a lack of stability in the upper HMA layer. However if the rutting occurs shortly after construction it could be caused by inadequate layer thicknesses or inadequate shear strength in the base, subbase or foundation layers. Research has shown that rutting can also be a function of deflection. The greater the deflection the greater is the chance for permanent or non-recoverable deformation. Recent studies have also found that moisture susceptibility of treated and untreated base layers can contribute to rutting. Moisture susceptibility is measured in the tube suction test.

Stripping of the asphalt from the aggregate can effectively reduce the tensile strength (cohesion) within the HMA and contribute to rutting and should be considered a possible cause of rutting. This condition can be detected by GPR surveys.

Investigations of rutting will normally concentrate on HMA material properties (stability and asphalt content) and in-place condition (density). However, the investigation should not overlook the possibility of structural deficiencies and excessive deflections.

BLOCK CRACKING





DESCRIPTION

The cracks on the surface form definite blocks typically ranging in size from 1 foot by 1 foot to 10 foot by 10 foot. Cracks cover the entire surface; this is not a load-associated problem.

POSSIBLE CAUSES

Shrinkage of stabilized layers	(Too much stabilizer)
Asphalt cement properties	(Burnt binder)

FAILURE INVESTIGATION

1. Construction Records
   a. Mix design - all stabilized materials
   b. Material properties - base, subbase, subgrade; particularly design criteria and stabilizer content used
   c. Pavement layer thicknesses
   d. Date of construction and last overlay (quality control data such as lay-down temperatures)
2. Maintenance Activities
   a. Type
   b. Amount
   c. Effectiveness
3. Field Evaluation
   a. Condition survey, extent and severity of problem, when distress first observed
   b. GPR survey (high sampling rate GPR survey to detect extent of cracking in base)
   c. FWD survey (layer moduli, load transfer efficiency of cracks)
   d. Samples (core samples both over cracks and in solid areas)
4. Laboratory Testing of Field Samples
   a. HMA properties
      Asphalt content, asphalt penetration, viscosity or Dynamic Shear Rheometer (DSR)
   b. Stabilized Bases
      Asphalt treated (see item 4a above)
      Cement or lime - compressive strength

REHABILITATION ALTERNATIVES

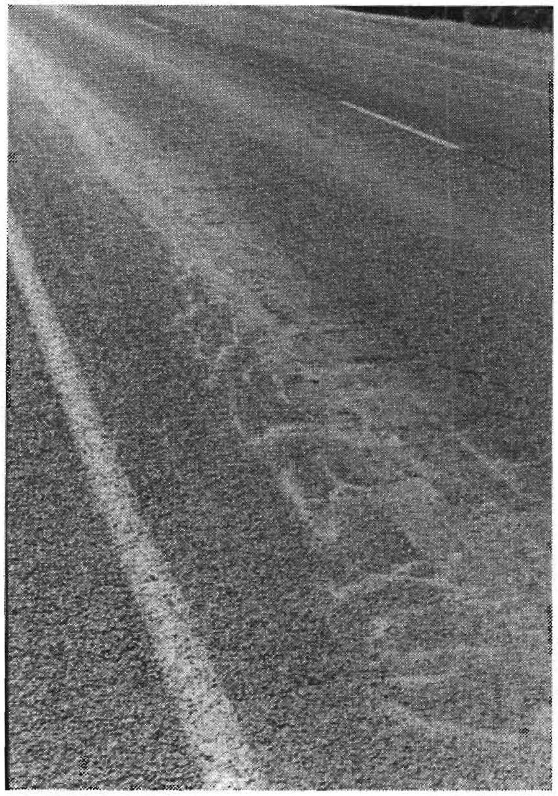
Low severity cracking (tight or <0.1 inch wide)
- Seal coat
- Rubberized seal coat and overlay

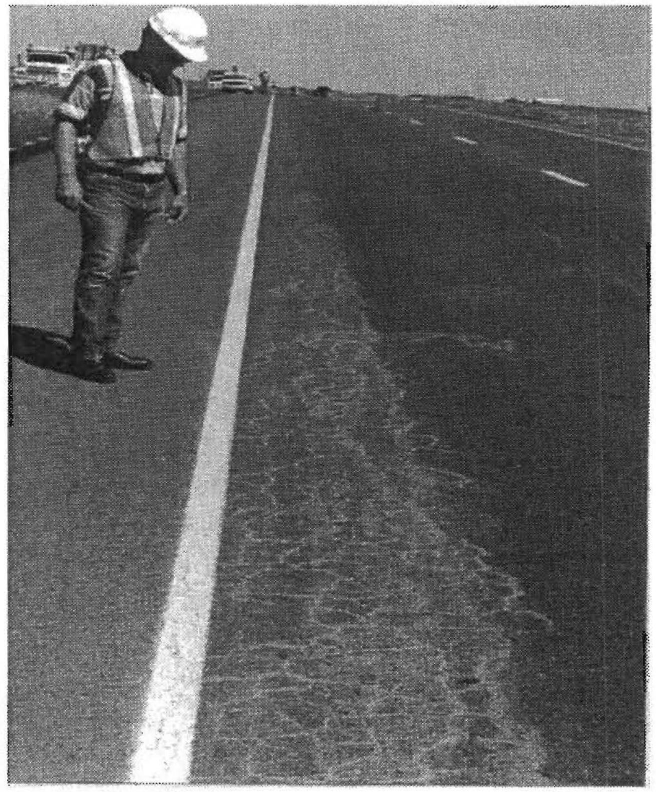
High severity cracking
- Crack seal
- Crack relief layer and structural overlay

High severity cracking with loss in ride
- Crack relief layer and structural overlay
- Flexible base overlay and resurfacing
- Surface removal and full depth base reclamation

DISCUSSION
Block cracking is almost always associated with shrinkage cracking from a stabilized base. It is very common with cement treated bases designed to meet TxDOT's 7-day strength criteria of 500 psi. Block cracking is not a problem provided that the cracks remain tight. However secondary deteriorations are also common when moisture enters the lower layers and when combined with heavy loads structural failure can occur. Wide, active cracks are almost impossible to stop from reflecting through the new surface. In some locations a rubberized chip seal followed by thick hot mix overlay have be found to be effective. In a few instances a crack pattern of small blocks has been found similar to alligator cracking. However, if the pattern covers more than the wheel paths then this is most probably related to construction problems rather than pavement fatigue. The most common cause is the use of too high a temperature during construction (burnt binder). This problem is easily verified by recovering the binder and running a penetration test. Under normal operations the penetration value of the binder recovered from relatively new surfacing should be higher than 35; values less than 25 are cause for concern. If this is found to be the case, the surfacing should be removed and replaced.

ALLIGATOR CRACKING





DESCRIPTION

Alligator cracks are a network of multi-sided blocks (polygons) resembling the skin of an alligator. This type of cracking is frequently referred to as fatigue cracking. It is associated with traffic and initially appears in the wheel path area. The blocks formed are less than 1 foot by 1 foot.

POSSIBLE CAUSES

Structural deficiency	(Weak or wet base or subgrade, thin layers)
Excessive air voids in HMA	
Asphalt cement properties	(Burnt binder)
Stripping of asphalt from aggregate	(Problems in an old buried HMA layer)
Layer debonding	(Poor construction practices)
Construction deficiencies	(Poor joints, segregation)

FAILURE INVESTIGATION

1. Traffic (per lane)
   a. Accumulated equivalent 18 kip axle loads to date
   b. Average of the 10 heaviest wheel loads daily (ATHWLD)
2. Construction Records
   a. Mix design - all stabilized materials
   b. Material properties - base, subbase, subgrade
   c. Pavement layer thicknesses
   d. Typical section (widening pavements with different base materials is a concern, in some instances this causes moisture be trapped)
   e. Field records from last overlay (condition of lower layers, date of construction, weather conditions)
3. Maintenance Activities
   a. Type
   b. Amount
   c. Effectiveness
4. Field Evaluation
   a. Condition survey, extent and severity of problem
   b. Drainage
   c. GPR survey (moisture in base, stripping in HMA, layer thickness, layer bonding)
   d. FWD survey (layer moduli)
   e. DCP survey (strength profile in base and subgrade)
   f. Samples (from cracked and uncracked areas, cores or bag samples)
   g. Bonding between layers (observation from coring, slab removal, Seismic Pavement Analyzer Test (SPA) test)
5. Laboratory Testing of Field Samples
   a. HMA properties
      Water susceptibility
      Asphalt content, asphalt penetration, air void content
      Aggregate properties (gradation, absorption, shape, surface texture)
   b. Base, Subbase, Subgrade
      Gradation, field moisture content
      Triaxial classification
      Tube suction test (moisture susceptibility)
   c. Stabilized Bases
      Asphalt treated (see item 5a above)
      Cement or lime - compressive strength before and after tube suction test

REHABILITATION ALTERNATIVES

A wide range of options are available but the optimal alternative can be selected only after the cause of the cracking has been determined.

Options include:

- Seal coat
- Replacement (remove and full depth HMA replacement in localized failed areas)
- Overlay of various thicknesses with or without special treatments to minimize crack reflection
- Recycle HMA (central plant or in-place if problem is near the surface)
- Full depth reclamation (treated existing with stabilizer and use as base or subbase)
- Reconstruction

DISCUSSION

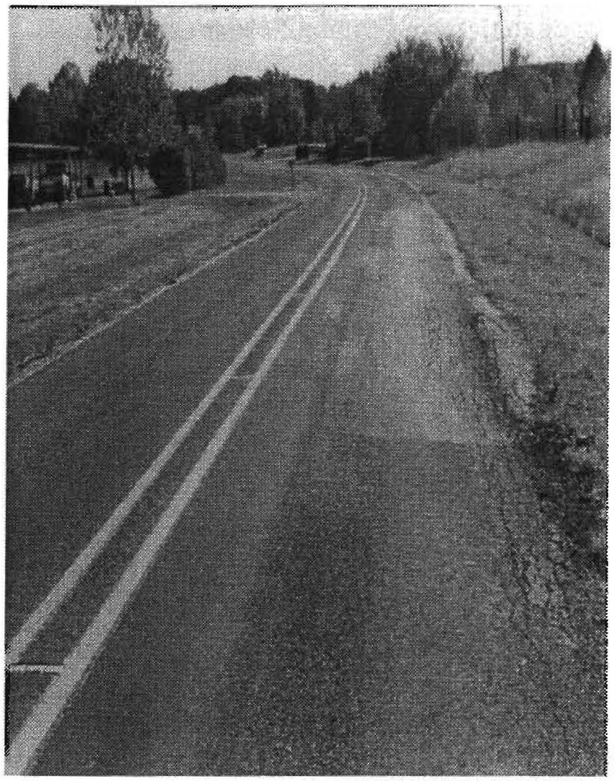
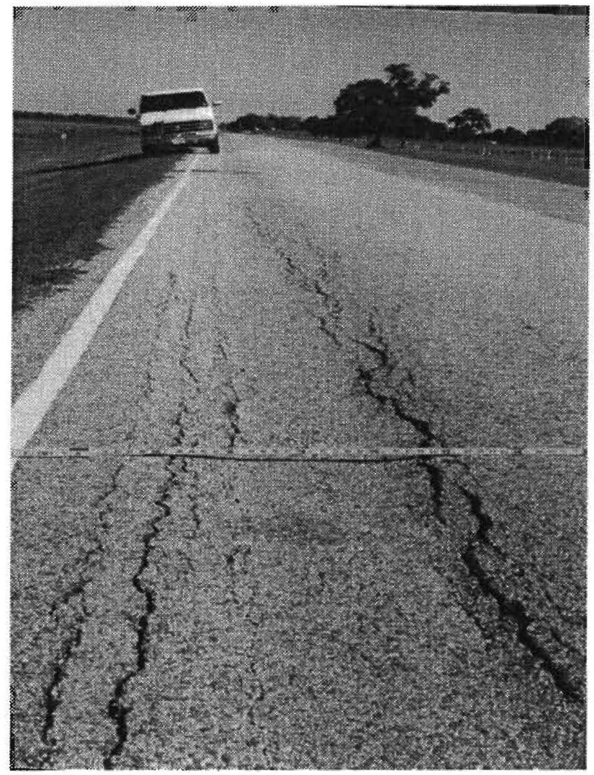
Alligator cracking is one of the most serious and complicated forms of pavement distress. Its occurrence can be influenced by all phases of design and construction as well as material properties and environment.

Alligator cracking is sometimes an indication that the pavement has been deflected (stressed) an excessive number of times and has formed a system of cracks as a result of such deflections. In most engineering materials alligator cracking would be referred to as fatigue cracking or the cumulative effect of multiple load applications.

The occurrence of alligator cracking is also accelerated by a host of materials and design issues, such as the state of bonding between layers, the presence of stripping in lower HMA layers, and moisture trapped within lower layers. Research has shown that trapping moisture can be caused by a range of design decisions such as placing stabilized shoulders on flexible base pavements, using different stabilized layers within the same stabilized base (for example a lower layer of stabilized recycled materials with an upper layer of stabilized new materials) and the use of impermeable fabrics with HMA layers. The presence of excessive moisture in lower layers can readily be detected by a GPR survey. Neither of the cases shown at the top of this section are classical fatigue cracking. The cracking in the photo on the left was caused by stripping of a lower HMA layer, and in the right photo by layer debonding.

The decision on which rehabilitation strategy to select is often straightforward once the cause of the alligator problem is found. However this is often a complex problem. A full structural investigation including FWD and GPR is recommended to assist in determining the cause.

FAILURES

DESCRIPTION

Failures are large areas of severe surface distress often caused by shear failure in the lower layers. Very deep ruts > 3 inches are classified as failures.

POSSIBLE CAUSES

Poor pavement edge support
Structural deficiencies (Thin layers, very weak base, severe stripping)
Trapped moisture

FAILURE INVESTIGATION

1. Traffic (per lane)
   a. Accumulated equivalent 18 kip axle loads to date and future 20 year estimates
   b. Average of the 10 heaviest wheel loads daily (ATHWLD)
2. Construction Records
   a. Mix design - all stabilized materials
   b. Material properties - base, subbase, subgrade
   c. Pavement layer thicknesses
   d. Date of construction and last overlay
3. Maintenance Activities
   a. Type
   b. Amount
   c. Effectiveness
4. Field Evaluation
   a. Condition survey, extent and severity of problem
   b. Drainage
   c. GPR survey (moisture in base, stripping, extent of problem)
   d. Ground-coupled GPR survey (if the problem is suspected to be caused by a deep problem such leaking pipes, subsurface springs, poor backfill)
   e. FWD survey (moduli of lower layers)
   f. DCP survey (strength profile in base and subgrade)
   g. Trenching (to locate or confirm subsurface problem)

REHABILITATION ALTERNATIVES

For localized structural failures

- Full depth patching

For major failures

- Reconstruction (with adding shoulder for lateral support)
- Pavement reclamation (with widening if required)

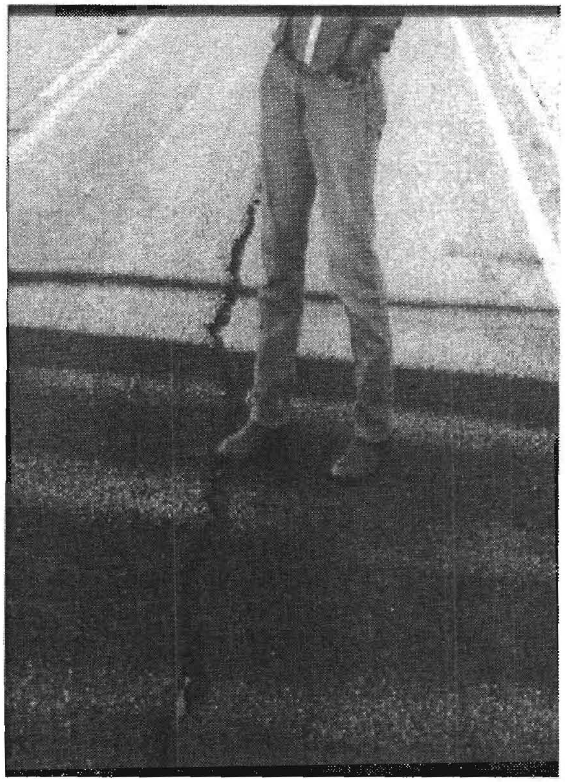
DISCUSSION

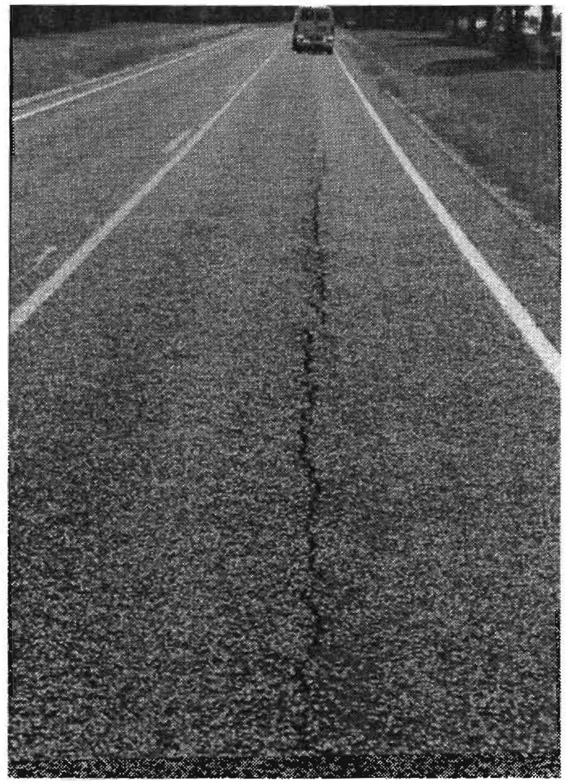
Failures are most often associated with overloading of thin farm to market (FM) roadways. They typically occur at the pavement edge where overloaded trucks, the lack of adequate shoulders, and thin layers are the main contributing factors. Severe base deterioration has also been found to cause failures; this may include trapped moisture, severe stripping of asphalt treated bases or loss of base density. An air-launched GPR survey can assist with identifying the extent and severity of major base problems.

In recent years, some areas of severe surface disintegration have been classified as failures. These often occur due to a lack of bonding between overlays or by moisture becoming trapped beneath a surface seal. These can be detected using a GPR survey.

If failures occur on high volume roadways then a full forensic investigation is often warranted. In urban areas the most common cause of dramatic sudden failures is leaking underground utilities. Failures due to spring activities and underground sinkholes have been documented but these are rare. Ground coupled radar surveys can help define the extent and possible cause of the problem. Soil investigations will also be required.

LONGITUDINAL CRACKING





DESCRIPTION

A longitudinal crack is a break or fracture of the pavement surface which is approximately parallel to the pavement centerline. Longitudinal cracks may be either traffic (load) associated or non-load associated.

Longitudinal cracking, such as edge cracks, construction joints, settlement, and possibly reflection cracking are non-load associated (not related to traffic) and usually result from some type of volume change in the paving or foundation materials.

Longitudinal cracks in the general vicinity of the wheel paths are indications of load-associated effects. Such cracks are usually the first indicator that alligator cracks will occur in the pavement surface.

POSSIBLE CAUSES	
Load Associated (wheel paths)	Non-Load Associated (not wheel paths)
Structural deficiency	Volume change of soil (edge drying)
Excessive air voids in HMA	Steep side slopes (high embankments)
Asphalt cement properties	Slope stability of fill materials
Stripping of asphalt from aggregate	Buried PCC (thermal movements)
Construction deficiencies	Segregation due to lay-down machine
Buried PCC or stabilized layers	Poor joint construction

FAILURE INVESTIGATION

Load Associated

See those factors enumerated in the section on Alligator Cracking

Non-Load Associated

1. Construction Records
   a. Properties of fill and subgrade materials (plasticity index, etc.)
   b. Lay-down and joint construction details
   c. Condition of underlying pavement (type and amount of cracks)
   d. Presence of buried slabs or stabilized bases
2. Maintenance Activities
   a. Type
   b. Amount
   c. Effectiveness
3. Laboratory Evaluation
   Soils
   a. Plasticity Index (PI) and volume change properties of fill and subgrade soils
   b. Density profile of fill
   c. Consolidation properties of in-place materials
   d. Shear strength of fill material
   HMA
   a. Segregation of HMA near crack
4. Field Evaluation
   a. Condition survey (extent and severity)
   b. Drainage
   c. Undisturbed samples of fill and subgrade foundation soils
   d. Geometric factors - width of lane, paved or unpaved shoulder, side slopes
   e. Other - presence of trees close to pavement edge

REHABILITATION ALTERNATIVES

- Crack sealing
- Seal coat or patch (applied to localized area with cracking)
- Replacement (dig-out and replace distressed areas)
- Thin overlay with special treatment to seal cracks and minimize reflection cracking
- Asphalt-rubber membrane with aggregate seal or thin overlay
- Heater-scarification with a thin overlay

To minimize longitudinal cracking in the design process consider the use of:

- Fabrics or grids between layers
- Vertical moisture barriers
- Adding sealed shoulders

DISCUSSION

Longitudinal cracking may occur as a consequence of either traffic- or nontraffic-related factors. It is important to recognize the difference. Detailed condition surveys will be required in order to reliably identify which category of distress is contributing to the problem.

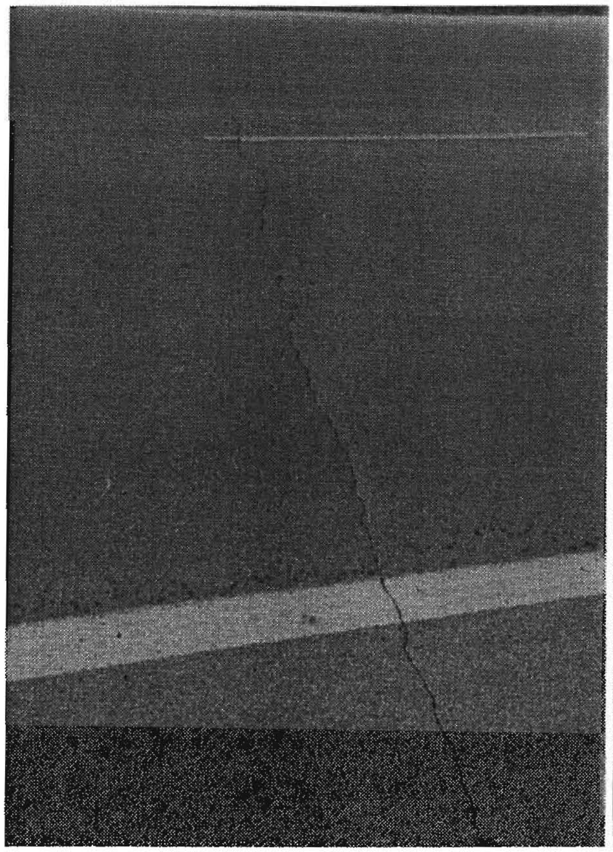
If the problem is nontraffic related it will not be necessary to implement a study comparable to that used for alligator cracking.

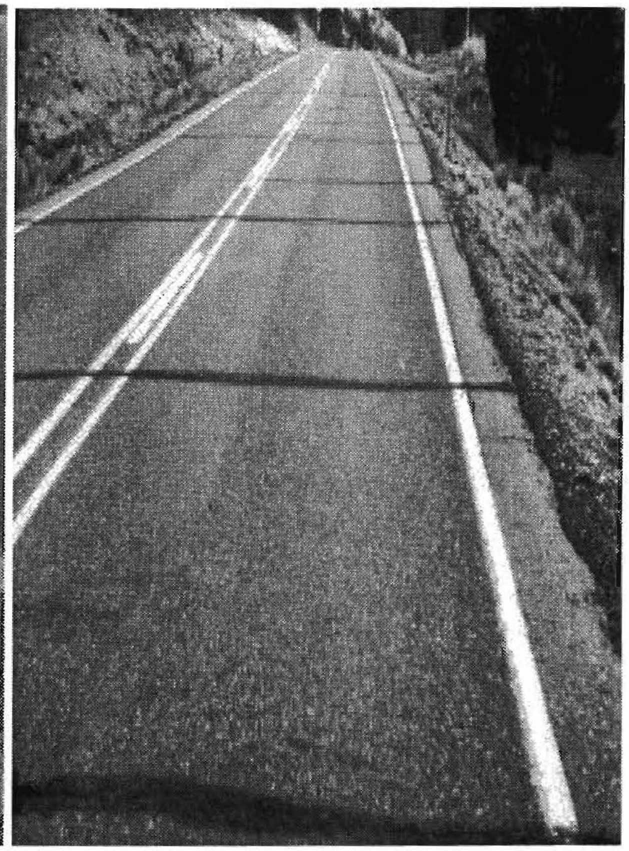
Cracking associated with settlement or slope stability becomes a foundation problem and can be analyzed by conventional procedures. Visual examination by experienced foundation engineers may be sufficient to identify slope stability and consolidation effects.

Edge cracking may be a function of volume changes in the foundation soil; it may also be a lack of shoulder support or lane width. In most cases the cause of edge cracking can be identified from field observation by a trained observer. With Texas conditions edge cracking may be impossible to eliminate and often a crack seal will prevent further deterioration.

Recent research studies have found that longitudinal cracking is a major concern in full depth pavement reclamation projects. This occurs where the soil has a PI > 35 and where edge drying is anticipated. This is most severe in locations with steep side slopes or where trees are close to the pavement edge. With stabilization projects the severity of the cracking is also a function of the stiffness of the stabilized layers. In some districts the use of horizontal fabrics over the treated layer has proven to be effective; low levels of stabilizer also minimize cracking. In extreme cases vertical moisture barriers may be considered to minimize drying of the soil.

TRANSVERSE CRACKING





DESCRIPTION

A transverse crack is a break or fracture of the pavement surface which is approximately perpendicular to the pavement centerline. Transverse cracks are normally non-load associated (not related to traffic) and usually result from some type of volume change in the HMA, base or subbase material. Transverse cracks may or may not extend across all lanes of contiguous paving and are often caused by reflection from underlying layers. In all probability most reflection cracking is a combination of traffic, materials and environmental effects.

POSSIBLE CAUSES

Hardness of asphalt cement
Stiffness of HMA
Volume changes in base and subbase (Particularly buried PCC slabs)
Poor base properties (Moisture susceptible base materials in colder climates)
Reflection cracks from lower layers

FAILURE INVESTIGATION

1. Construction Records
   a. HMA mixture design
   b. Materials properties - base (% stabilizer), subbase, (presence of buried PCC)
   c. Pavement layer thickness
   d. Condition of underlying pavement (type and amount of buried cracks)
2. Maintenance Activities
   a. Type
   b. Amount
   c. Effectiveness
3. Laboratory Evaluation of HMA
   a. HMA properties
      Asphalt content
      Asphalt properties (penetration, viscosity, temperature susceptibility, stiffness at low temperatures)
      Mixture properties (tensile strength, stiffness at low temperatures)
      Aggregate properties (gradation, absorption)
   b. Treated bases and subbases
      Shrinkage potential
      Tensile strength
   c. Untreated aggregate bases
      Gradation
      Mineralogy of clay fraction (Tube Suction Test performance)
4. Field Evaluation
   a. Condition survey (severity and extent)
   b. Roughness
   c. Cores of stabilized pavement layers
   d. FWD testing (stiffness of base and load transfer of cracks)
   e. GPR survey (detect excessive moisture in base, deterioration in lower HMA layers)
5. Climatological Information
   a. Minimum temperatures
   b. Rate of temperature drop
   c. Daily temperature change

REHABILITATION ALTERNATIVES

- Crack sealing
- Seal coat
- Overlay with special treatment to seal cracks and minimize reflection cracking
- Asphalt-rubber membrane with aggregate seal or thin overlay
- Heater scarification with a thin overlay

DISCUSSION

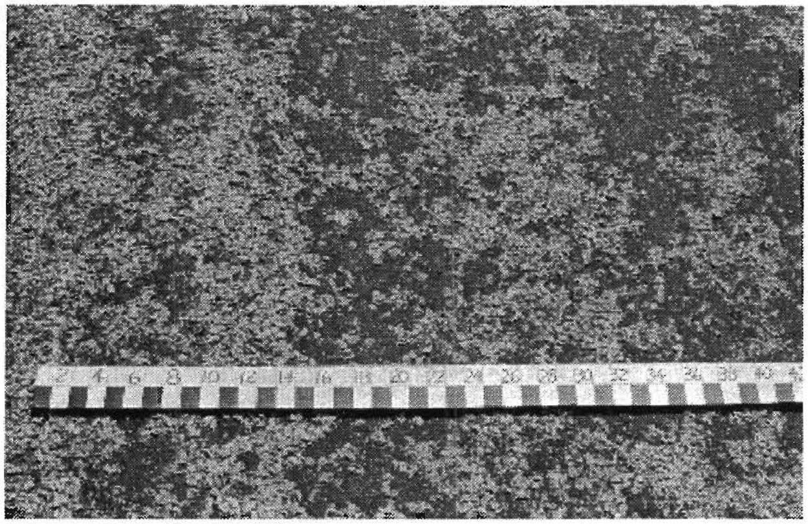
Transverse cracking is most commonly associated with either reflection cracking from stabilized bases (Portland cement and lime) or low-temperature cycling (thermal cracks).

The naturally occurring shrinkage cracks associated with Portland cement and lime stabilized materials are reflected through the asphalt concrete to the surface of the pavement. The higher the percentage stabilizer the more severe the cracking; sometimes overstabilization leads to block cracking rather than pure transverse cracking.

Low-temperature or thermal cracking can occur in asphalt concrete and may occur in untreated aggregate base. The low-temperature cracking in asphalt concrete is most frequently associated with the properties of the asphalt; stiffness and temperature susceptibility are the major contributing factors. Observations indicate that traffic volumes and weights may contribute to the frequency of low-temperature cracking.

Recent studies have related transverse cracking to problems with absorptive flexible base materials. Low-temperature cracking in untreated aggregates has been related to the mineralogy of the clay fraction of the base material. This is best detected by the Tube Suction Test (TST), which indicates a base's affinity for moisture. Materials which fail this test will perform poorly in colder climates. The TST is described in the appendix of this report. If the base is suspected to be the source of the surface cracking then a GPR survey should be conducted; base dielectrics of greater than 12 would be a cause for concern.

RAVELING



DESCRIPTION

Raveling is the progressive loss of surface material from HMA by weathering and/or traffic abrasion. Usually the fine aggregate as binder (matrix) will wear away first to be followed by the larger sized aggregate. As raveling progresses the pavement surface can become rough and in extreme cases can develop potholes. Seal coats can also exhibit a form of raveling in which the uniformly sized aggregate is lost from the surface resulting in an excess of asphalt on the surface of the pavement.

POSSIBLE CAUSES

Low asphalt content
Excessive air voids in HMA
Hardening of asphalt
Water susceptibility (Stripping)
Aggregate characteristics
Hardness and durability of aggregate

FAILURE INVESTIGATION

1. Traffic (per lane)
   a. Average daily traffic
   b. Percent trucks
2. Construction Records
   a. HMA mix design
   b. Asphalt content, source, type, and grade
   c. Aggregate characteristics
3. Maintenance Activities
   a. Type
   b. Amount
   c. Effectiveness
4. Laboratory Evaluation of HMA
   a. Asphalt content
   b. Asphalt properties (penetration, viscosity)
   c. Aggregate properties (gradation, absorption, shape, surface, texture, mineralogy)
   d. Water susceptibility
   e. Air voids
5. Field Evaluation
   a. Condition survey
   b. HMA cores for laboratory evaluation

REHABILITATION ALTERNATIVES

- Dilute emulsion or rejuvenating fog seal
- Seal coat with aggregate
- Slurry seal
- Thin HMA overlay

DISCUSSION

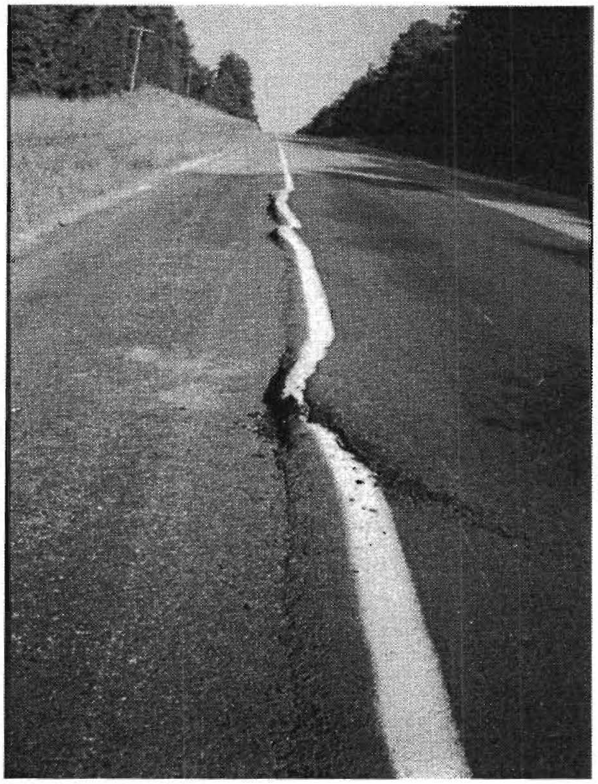
Raveling is usually associated with the asphalt in the HMA. The amount of asphalt actually incorporated in the HMA and the hardness (consistency) of the asphalt are prime considerations. Density of the HMA can also be a major factor contributing to raveling.

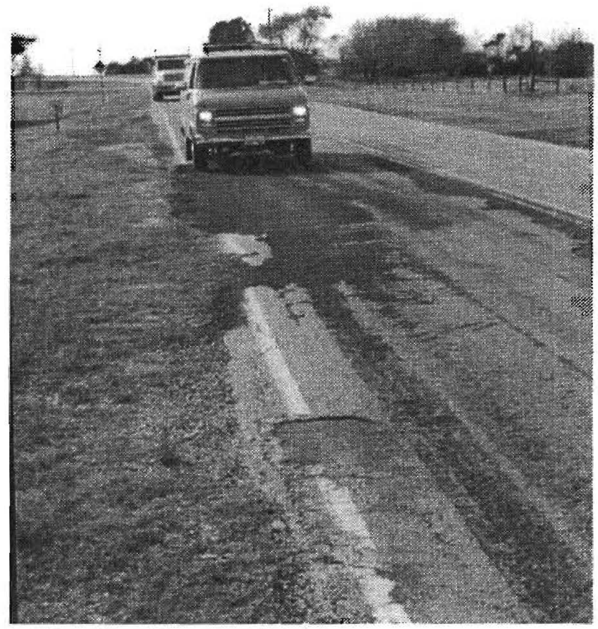
Raveling can also be caused by poor asphalt-aggregate adhesion in the presence of water (stripping). The identification of stripping as a probable cause will require careful evaluation and comparison with available distress criteria.

There are a number of ways to retard or correct raveling which are relatively inexpensive.

However, if there are systematic and identifiable causes of raveling, they should be corrected by changes in specification or construction requirements.

ROUGHNESS





DESCRIPTION

Roughness is the absence or lack of smoothness in the longitudinal or transverse surface profile, which causes poor vehicular riding quality.

POSSIBLE CAUSES

Presence of physical distress (cracking, rutting, corrugations, potholes, etc.)
Volume change in fill, subbase, or subgrade materials
Non-uniform construction
Lack of bonding between pavement layers (corrugations)

FAILURE INVESTIGATION

1. Traffic (per lane)
   a. Accumulated equivalent 18 kip axle loads to date
   b. Average of the 10 heaviest wheel loads daily (ATHWLD)
2. Construction Records
   a. Density - all layers and subgrade soil
   b. Site conditions - natural drainage locations, amount of cut and fill
   c. Pavement layer thickness
   d. Soil type
3. Maintenance Activities
   a. Type
   b. Amount
   c. Effectiveness
4. Laboratory Evaluation of Soil
   a. Volume change potential of subgrade material
5. Field Evaluation
   a. Condition Survey
   b. Roughness (Ride Score from PMIS)
   c. In-place density
   d. GPR survey, checking for subsurface defects
   e. FWD survey, checking overall structural adequacy
6. Climatological Information
   a. Minimum temperature
   b. Precipitation

REHABILITATION ALTERNATIVES

- Overlay
- Cold milling with or without overlay
- Heater scarification with overlay
- Heater planing with overlay (primarily for local areas and areas with corrugations)
- Recycle (central plant or in-place)
- Full depth reclamation

DISCUSSION

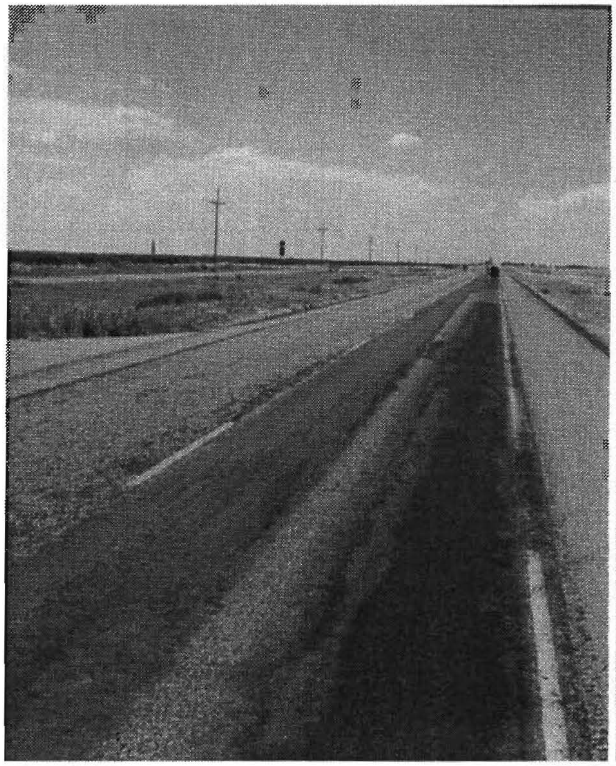
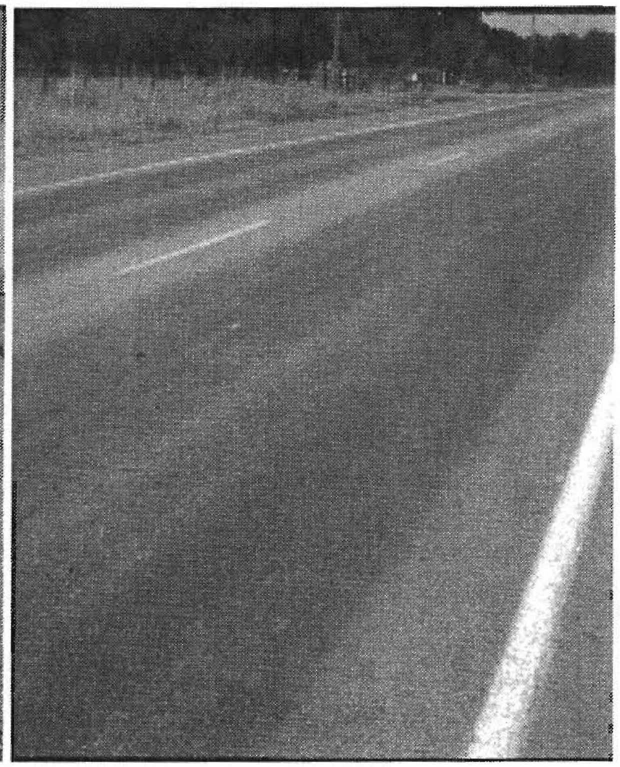
Since roughness can be a secondary result of any of the various forms of distress discussed in this manual, it will be necessary to first determine which of the three possible causes indicated in this section is the most probable cause.

If condition surveys indicate physical distress is the probable cause, the investigation would be directed to the form of distress identified; probably alligator cracking, rutting, or transverse cracking.

If physical distress is not the probable cause, expansive soils would be the next easiest to evaluate (eliminate). Knowledge of local soils would be the first clue; laboratory tests would follow. Recent research has indicated that large volume changes can occur with sulfate-rich soils treated with calcium-based stabilizers such as lime.

Non-uniform construction may be hard to determine; however, statistical evaluation of construction data and field tests would provide good indicators if this factor has contributed to roughness.

FLUSHING (BLEEDING)

DESCRIPTION

Flushing is the presence of an excess amount of asphalt on the surface of the pavement. Flushing or bleeding will be most prevalent in the wheelpath areas, although in some situations the entire pavement surface may exhibit flushing to some degree.

Seal coats can exhibit surface characteristics similar to flushing if there has been a significant loss of aggregate or if excess asphalt was utilized.

POSSIBLE CAUSES

High asphalt content
Excessive densification of HMA during construction or by traffic (low air void content)
Temperature susceptibility of asphalt (soft asphalt at high temperatures)
Excess application of a fog seal or rejuvenating materials
Water susceptibility of underlying asphalt-stabilized layers together with asphalt migration to surface

FAILURE INVESTIGATION

1. Traffic (per lane)
   a. Accumulated equivalent 18 kip axle loads to date
   b. Average of the 10 heaviest wheel loads daily (ATHWLD)
2. Construction Records
   a. HMA mix design
   b. Amount and type of prime and tack coat materials
   c. Asphalt content
3. Maintenance Activities
   a. Type
   b. Amount
   c. Effectiveness
4. Laboratory Evaluation of HMA
   a. Asphalt content
   b. Asphalt properties (penetration, viscosity, temperature susceptibility)
   c. Air voids
   d. Water susceptibility of asphalt stabilized base
5. Field Evaluation
   a. Condition survey
   b. HMA cores for laboratory evaluation
   c. Asphalt-stabilized base cores for laboratory evaluation
   d. Skid number

REHABILITATION ALTERNATIVES

- Overlay of open-graded friction course (none freeze areas only)
- Seal coat (well designed with good field control during construction)
- Cold milling with or without seal coat or thin overlay
- Heater-scarification with seal coat or thin overlay
- Heat surface and roll-in coarse aggregate
- Thin overlay

DISCUSSION

Flushing is generally associated with an excessive amount of asphalt in the HMA. Laboratory procedures have been developed to prevent use of too much asphalt; however, in some situations the laboratory procedures may not reliably estimate the effect of traffic (densification), weather (high temperature), and the temperature susceptibility of the asphalt cement.

There are a number of corrective treatments for flushing. One such treatment is to cover the surface with a seal coat. Extreme care should be exercised in using this treatment since the condition could be exaggerated by a poor seal coat application.

Flushing is a safety consideration relative to the performance of a pavement and should be corrected as soon as possible after being reported and evaluated.

CHAPTER 3

INTERPRETING NONDESTRUCTIVE TESTING DATA

The use of nondestructive testing (NDT) equipment has expanded considerably over the past 10 years. This chapter presents a summary of experiences with collecting and processing data from both Ground Penetrating Radar and Falling Weight Deflectometers. The goal is to describe the technology and summarize the types of output and interpretation schemes that are now available to TxDOT. The uses of the Dynamic Cone Penetrometer (DCP) will also be described as this is used by a growing number of TxDOT districts for localized failure investigations.

3.1 GROUND PENETRATING RADAR

This section will present an overview of how GPR technology can be used to help the TxDOT District Pavement Engineer with the investigation of pavement condition. The capabilities of the system will be described together with the basics of data processing. Typical examples of GPR data collected on Texas projects will be presented. The goal is to provide clear examples on how GPR can assist in the pavement evaluation process. Full details of GPR technology and a signal processing software (COLORMAP) can be found in other reports (*2, 3*).

In summary GPR has the following capabilities:

<u>Benefits</u>

1) Rapid evaluation of subsurface conditions in flexible pavements
2) Data collected at highway speed
3) Depth of penetration 24 inches
4) Signals sensitive to changes in moisture content, density, and layer thickness, all of which are critical factors in pavement performance investigations
5) With the integrated video system it is possible to pinpoint locations with suspected subsurface defects for validation coring (GPR will also minimize coring requirements)

Main Applications

1) Checking for uniformity of section (changes in surface thickness or base type)
2) Locating stripping in asphalt layer
3) Identifying moisture or density problems in base layers
4) Providing layer thickness for FWD analysis

TxDOT Contacts

For data collection: Carl Bertrand, Pavement Design Section

For data interpretation: Design Division

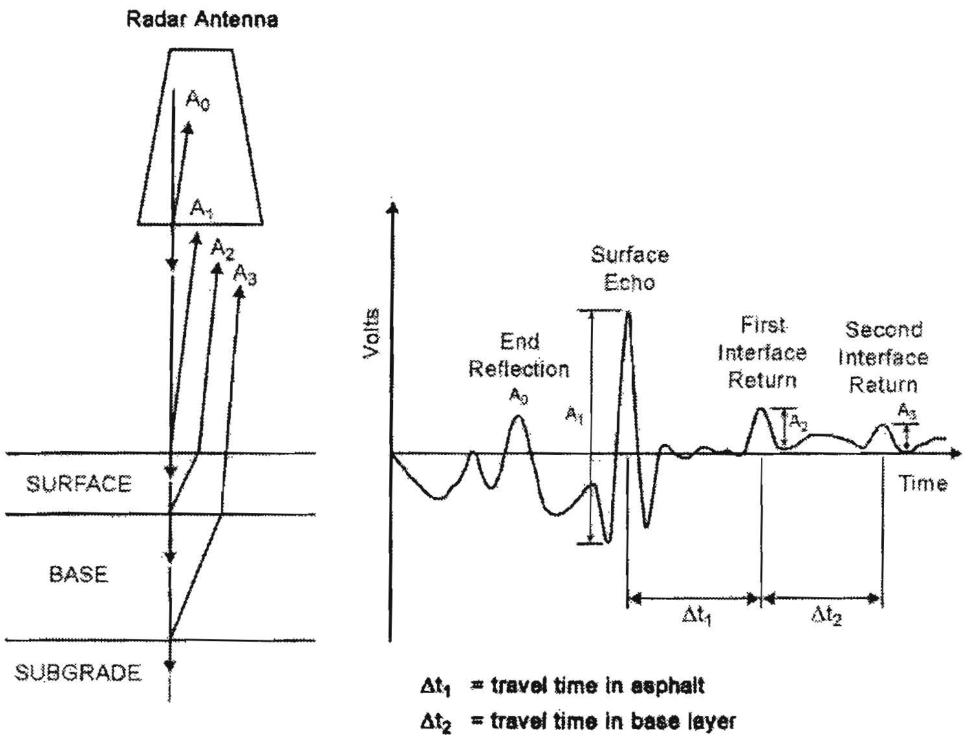
When to Request GPR Data Collection

1) On sections where plan sheets are not current
2) Sections with rapid deterioration or unusual distresses
3) Rehabilitation projects on all major highways. (TxDOT has four operating GPR units and plans to purchase additional units and conduct more training. This testing can be conducted at close to highway speed, so collection costs are minimal. Therefore GPR technology is available and relatively inexpensive. Its use on all major rehabilitation projects is therefore recommended.)

3.1.1. Basis of GPR Testing

Figure 1 shows the TTI Ground Penetrating Radar vehicle with the Pulse Radar antenna. The antenna transmits pulses of radar energy, with a central frequency of 1 GHz, into the pavement. These waves are reflected at significant layer interfaces in the pavement. The data acquisition system captures and displays these reflected waves as a plot of return voltage versus arrival time. As shown in Figure 1, the largest peak A_1 is the reflection from the pavement surface; the amplitudes before the surface reflection are internally generated noises known as the "end reflection". Although not related to the pavement structure, the time between the end reflection and the surface reflection is related to the height of the antenna above the ground; this measurement provides a means of accounting for antenna bounce as the system drives over rough highways. The reflections of major significance to pavement engineers are those that occur after

(a) TTI GPR Equipment



(b) Principles of GPR. The incident wave is reflected at each layer interface and plotted as return voltage against time of arrival in nanoseconds.

Figure 1. GPR equipment and principles of operation.

the surface echo. These represent significant interfaces within the pavement, and the measured travel time is related to the thickness of the layer.

The uniformity, density, and moisture content of each layer will impact these reflections. Below are listed examples of how changes in each of the reflections are related to the engineering properties of the pavement layers.

a. *The measured amplitude of reflection A_1 is constant for the test pavement*

Interpretation:

The section has a uniform density and moisture content. This is the ideal case. It is typically found in newly constructed pavements.

b. *The surface reflection A_1 has regular significant dips in amplitude*

Interpretation:

These dips are caused by low-density area in the mat. This could be caused by the presence of segregated areas in the mat or from poorly compacted construction joints. The amplitude of surface reflection A_1 has been correlated to the air void content of the HMA surfacing layer (*3*, *4*).

c. *The surface reflection A_1 has significant increases in amplitude*

Interpretation:

These are typically moisture related and indicate the presence of moisture trapped within the top HMA layer or the presence of moisture beneath a surface seal.

d. *The reflection from the base A_2 has a significant increase in amplitude*

Interpretation:

Normally associated with wet areas in the base.

e. *The reflection from the base A_2 has a significant decrease in amplitude*

Interpretation:

Normally associated with low-density areas in the base.

f. Significant reflections between A_1 and A_2

Interpretation:

Significant negative reflections are usually associated with areas of stripping within the HMA layer. However some naturally occurring aggregates can also give negative reflections. Moisture trapped at a lower interface will be a positive reflection followed by a negative reflection.

Examples of GPR data collected on Texas projects will be shown later in this section to illustrate each of these conditions.

3.2 VIEWING GPR DATA WITH COLORMAP

In 1995 TTI developed the COLORMAP software to process GPR data (*2*). During a typical GPR survey several thousand traces may be collected. These traces are color-coded and stacked side by side to develop an image of subsurface conditions for a section of highway. Figure 2 shows an example of the main COLORMAP display screen. These are GPR data taken from a section of newly constructed thick asphalt pavement over a thin granular base. A description of each of the denoted elements in this figure is given below.

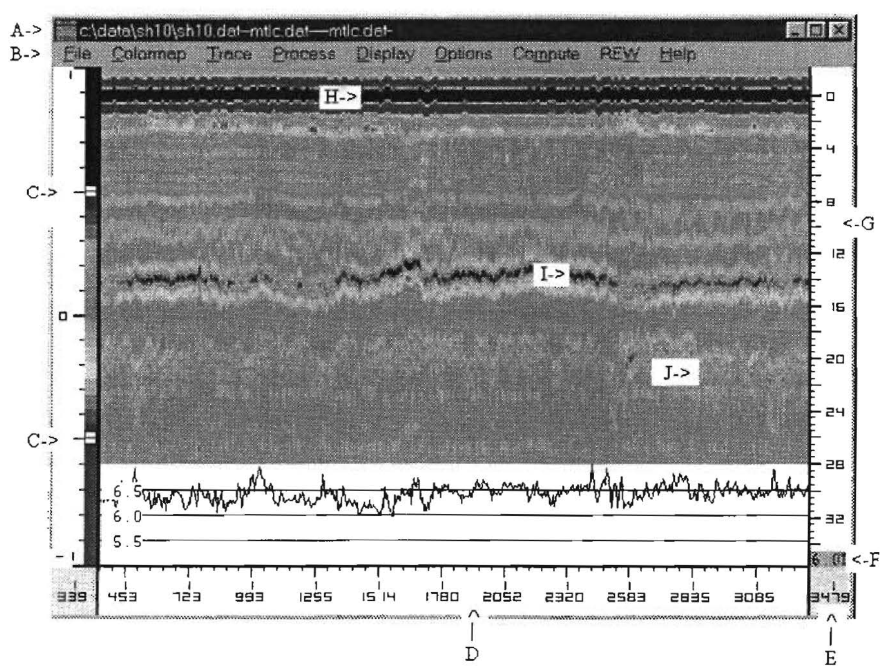


Figure 2. COLORMAP's Color-Coded GPR Traces.

A. Header information showing the directory in use and the list of files that have been opened for either data processing or for storing results.

B. Main pull-down menu bar in COLORMAP. The use of the pull-down options is described in TTI report 1702-5 (*2*).

C. User-defined color control buttons, the procedures by which the individual GPR waveforms are color coded and transformed into line scans.

D. Distance scale in miles and feet as collected by the GPR's distance measuring instrument (DMI).

E Total distance in miles and feet within the data file. If the metric option is chosen, this scale will be in kilometers and meters.

F. Default dielectric value used to generate the depth scale (G) at the right of the figure.

G. This is a depth scale in inches.

H. The surface of the pavement.

I. The reflection from the top of the flexible base, approximately 16 inches. below the surface.

J. The reflection from the top of the subgrade.

K. This is a plot of surface dielectric, which is automatically computed from the amplitude of surface reflection (peak A_1 in Figure 1).

3.3 USES OF GPR TECHNOLOGY IN PAVEMENT EVALUATION

In this section typical GPR results from a range of Texas pavements will be presented. These examples are intended to illustrate how GPR technology can be used to identify subsurface defects. The COLORMAP display is presented together with individual GPR reflections from specific locations. In some instances the "surface removal" technique has been used, where the surface reflection is aligned with the top of the COLORMAP display and the individual traces show two lines, one showing the raw GPR data and the other showing the residual subsurface reflections after surface removal. This technique is described in detail elsewhere (*2*); it is used when thin layers less than 3 inches thick are suspected. With 1 GHz systems, reflections from the top and bottom of layers less than 3 inches thick will overlap.

The following examples are provided:

<u>a) Good quality defect-free pavement</u> (Figure 3)

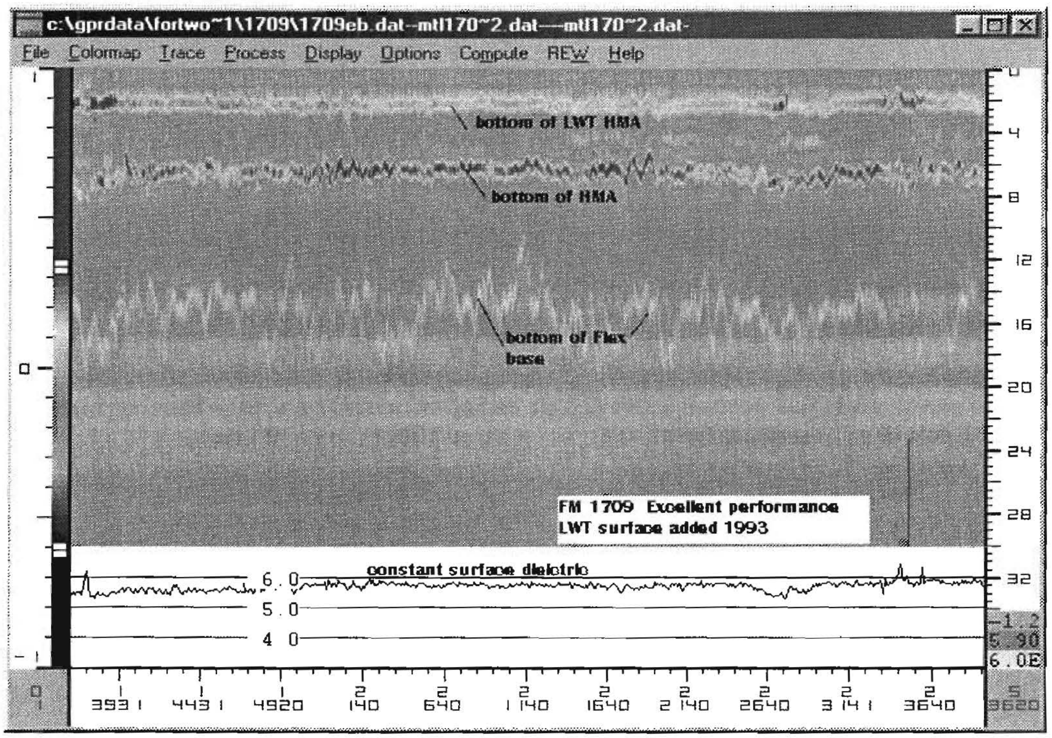
The COLORMAP display in the upper part of Figure 3 shows about 1 mile of data from a good performing pavement in Fort Worth. The lower graph shows a representative trace from a single location. The surface removal technique has been applied to this data set. The pavement surface is the top of the COLORMAP display shown in the upper part of Figure 3. The pavement consists of an 8 year old two-inch overlay over an existing flexible pavement. The pavement had been reported to be providing excellent performance. The important observations from the GPR data are the following:

- The layer interfaces are all clear and there are no significant reflections between these interfaces, therefore deterioration is not suspected within layers.
- The amplitude of reflection from the top of the base layer is relatively small. The calculated dielectric for the base layer is low (9.1 in lower figure). It has been proposed that top quality (dry) base layers will have a base dielectric of less than 10 (*5, 6*).
- The amplitude of surface reflection is constant (as indicated by the surface dielectric plot). This implies that the surface has uniform density and moisture content.

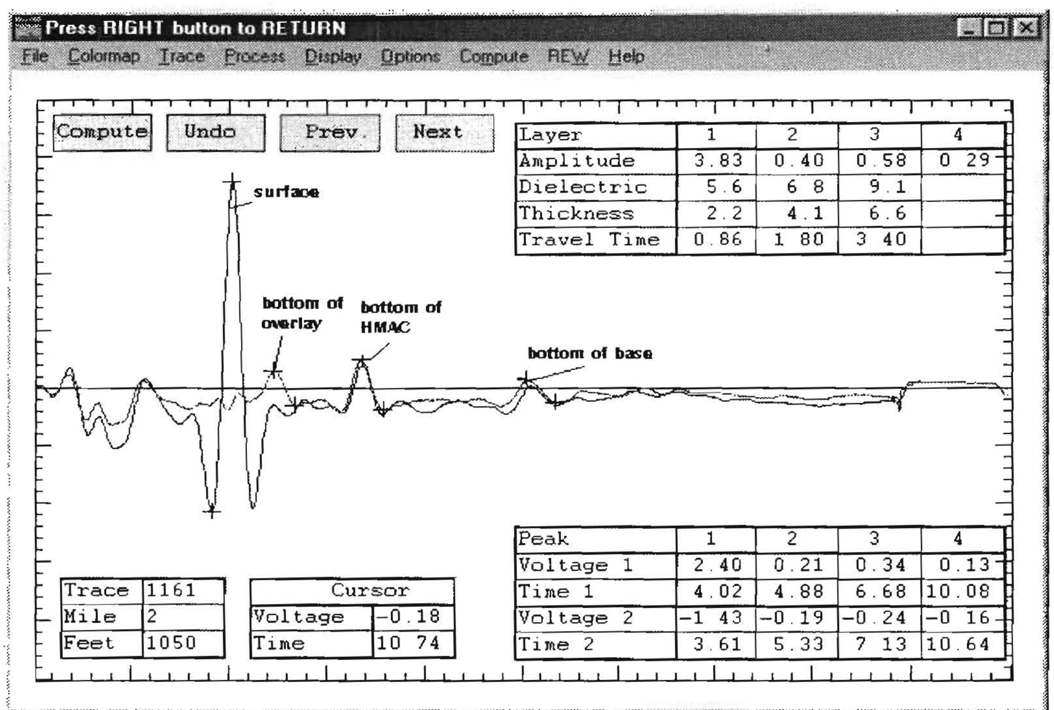
<u>b) Change in base moisture content</u> (Figure 4)

This figure shows the transition from a low dielectric to a higher dielectric base. The base with the higher dielectric (more moisture) is shown as a more intense reflection to the right of the COLORMAP display. If the base type does not change then this increase in reflection amplitude would indicate an increase in base moisture content. A proposed dielectric classification for flexible base materials is as follows:

• Less than 10	Top quality base materials (dry, not susceptible to environmental damage)
• 10 - 16	Marginal material
• Greater than 16	Poor material (large amounts of free moisture within the base indicating a lower shear strength and lower resistance to freeze-thaw cycling)

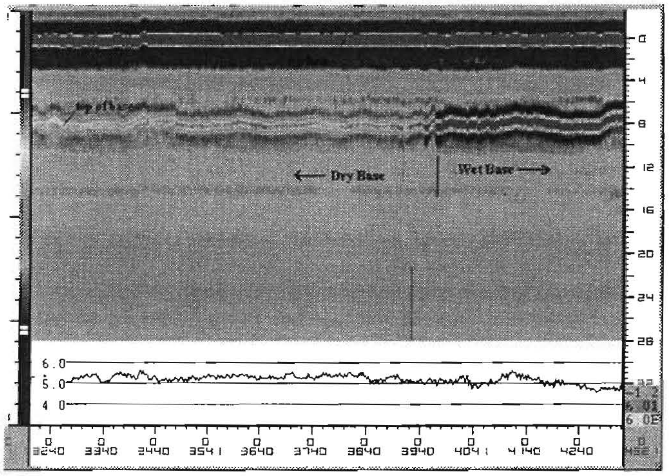


(a) COLORMAP Display (Surface at Top).

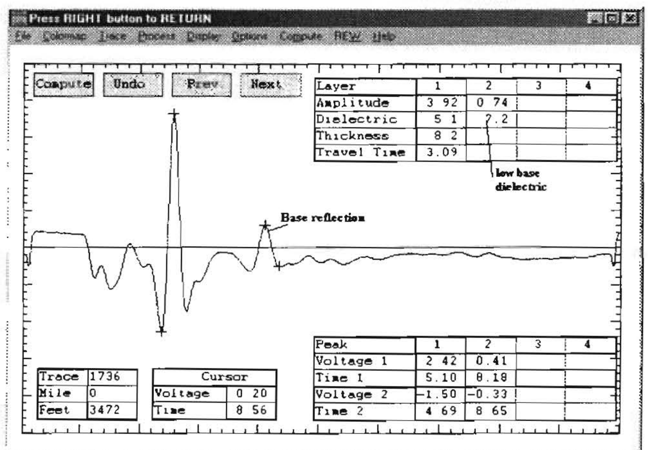


(b) Individual Trace (Raw Data Plus Reflections after Surface Removal).

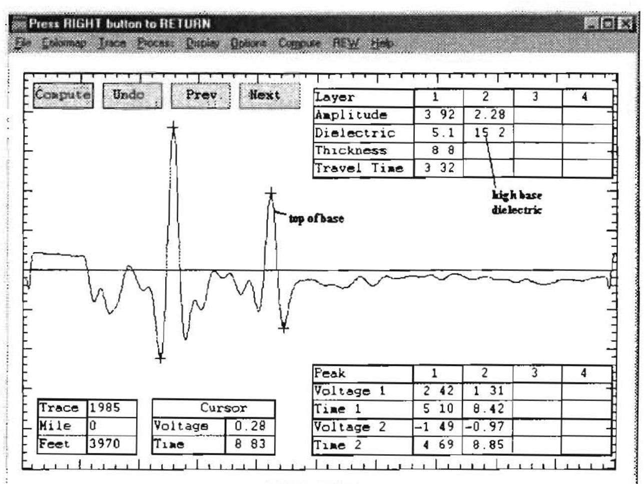
Figure 3. GPR Data from a Well-Performing Pavement. No Defects - Ideal Case.



Colormap Display



Low Dielectric Base (Dry)



Higher Dielectric Base (Wetter)

Figure 4. GPR Results Showing the Impact of Change in Base Moisture Content.

To convert base dielectric into base moisture content it is necessary to develop laboratory calibration curves. This procedure is described elsewhere (*3*).

<u>c) Moisture trapped beneath a surface seal</u> (Figure 5)

This figure shows a 2-inch thick HMA overlay (made with lightweight aggregates) on top of the concrete pavement. The overlay has recently received a microsurfacing seal. The point of interest in this figure is the highly variable surface reflection as demonstrated by the variable surface dielectric plot at the bottom of the COLORMAP display. This is also illustrated in the two GPR traces shown in the lower figures. The computed surface dielectric is observed to change from 3.9 to 8.1. The higher values indicate at that location there is a higher moisture content in the HMA below the surface seal. These areas should be cause for concern and could be problematic in freezing conditions.

<u>d) Near surface stripping in a lower HMA layer</u> (Figure 6)

Stripping is a very common occurrence in East Texas, where many pavements have lower HMA layers constructed with uncrushed river gravel aggregates. The upper figure shows the COLORMAP display for a 3000 ft section in the Dallas District. The surface of the pavement is at the top of this figure. The area of concern is the strong negative reflection (blue) which is observed from 2 to 5 inches below the surface. The individual GPR trace in the lower figure shows a very strong negative reflection closely following the strong positive surface reflection.

As described elsewhere (*2*), there are several naturally occurring pavement layers which can give similar patterns to those shown in Figure 6. These include drainage layers, plant mix seals and lightweight aggregate layers. The GPR interpretation should always include validation coring at a few selected locations.

<u>e) Variation in Pavement Structure</u> (Figures 7 and 8)

One basic concern when planning the rehabilitation of long sections of highway is the variability of the underlying structure. With older pavements the plans sheets are often of little use in defining what layers are beneath the surface. The layer types and thicknesses are key factors in interpreting the distress types found on the pavement surface. This is clearly demonstrated in the GPR data set shown in Figure 7, from a 2-mile section in the Bryan District.

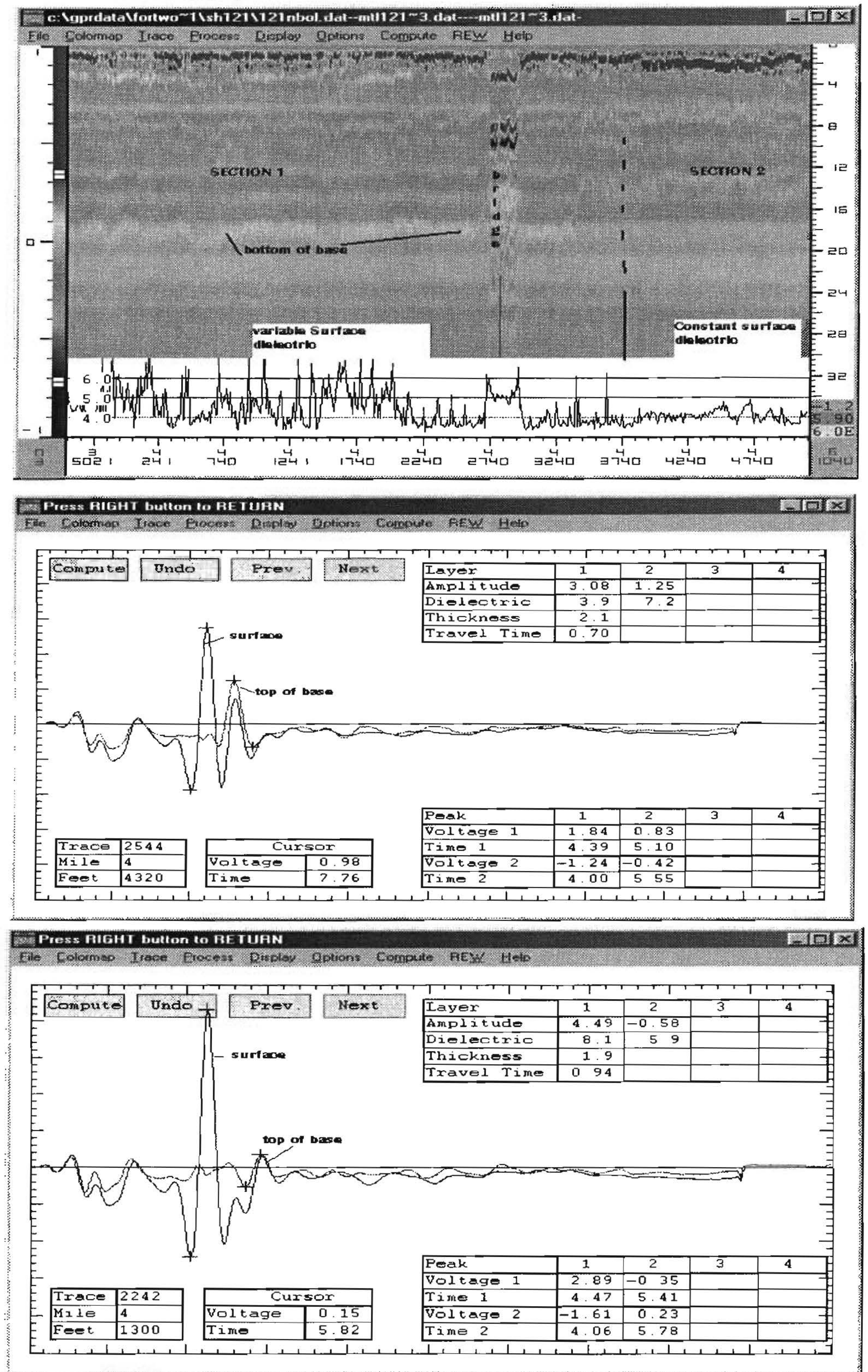


Figure 5. Moisture Trapped beneath a Surface Seal (Highly Variable Surface Reflection).

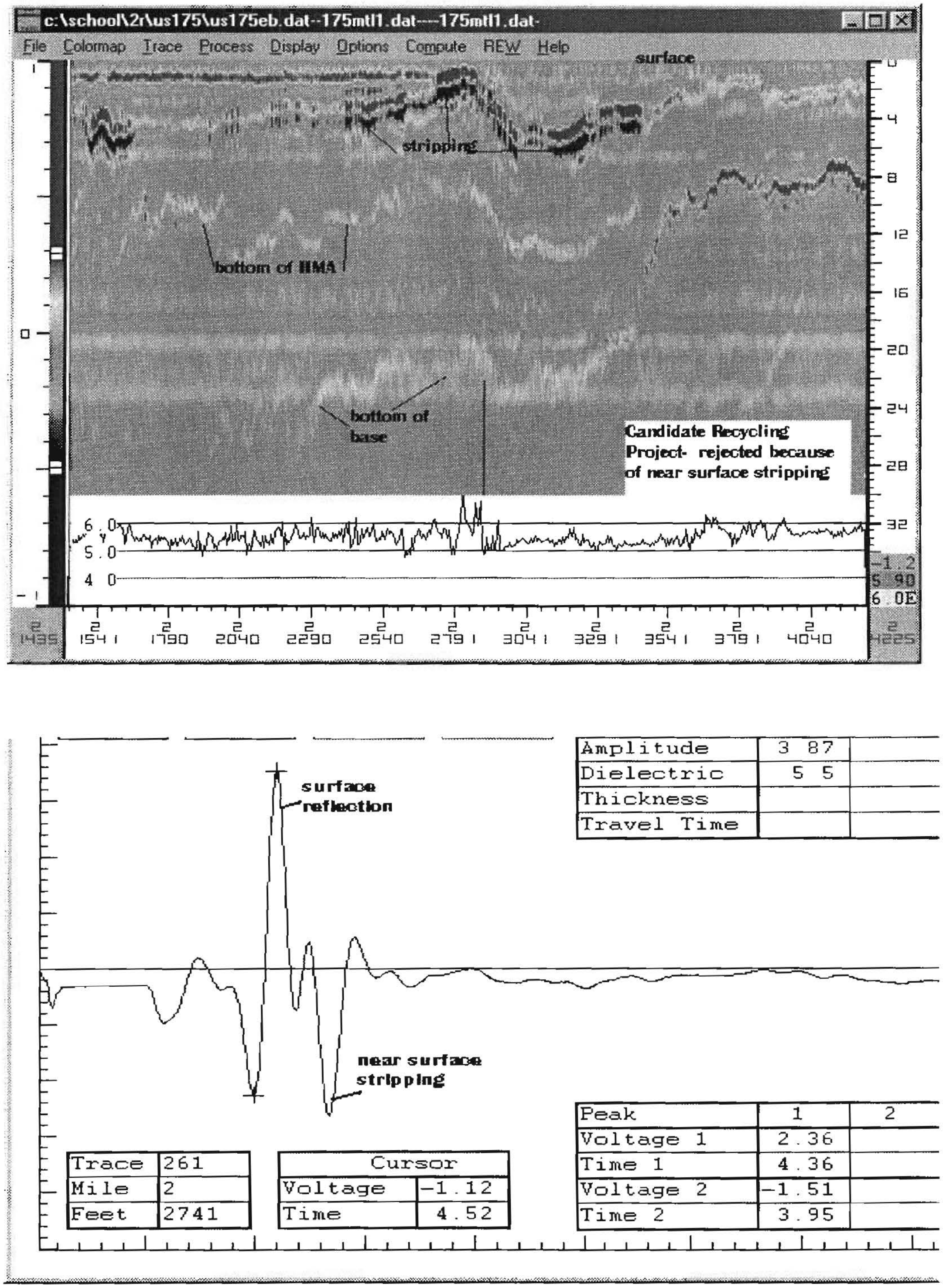


Figure 6. GPR Reflection Patters from a Pavement with Near Surface Stripping.

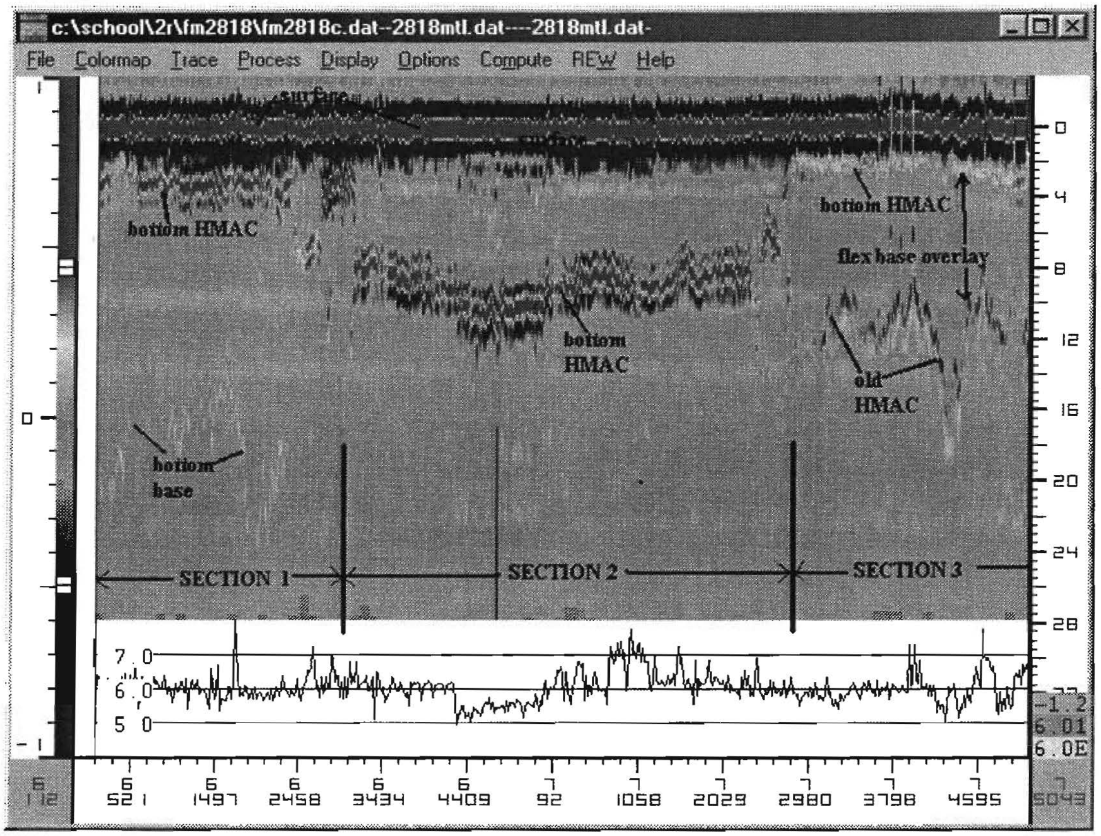
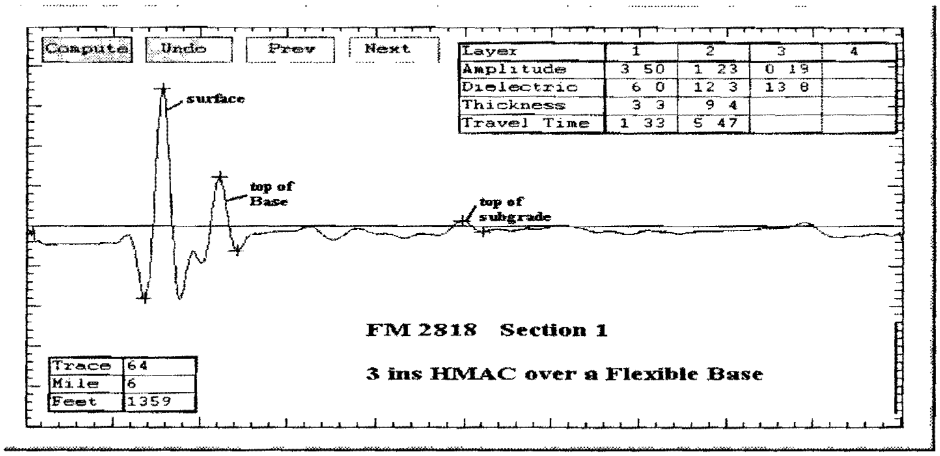
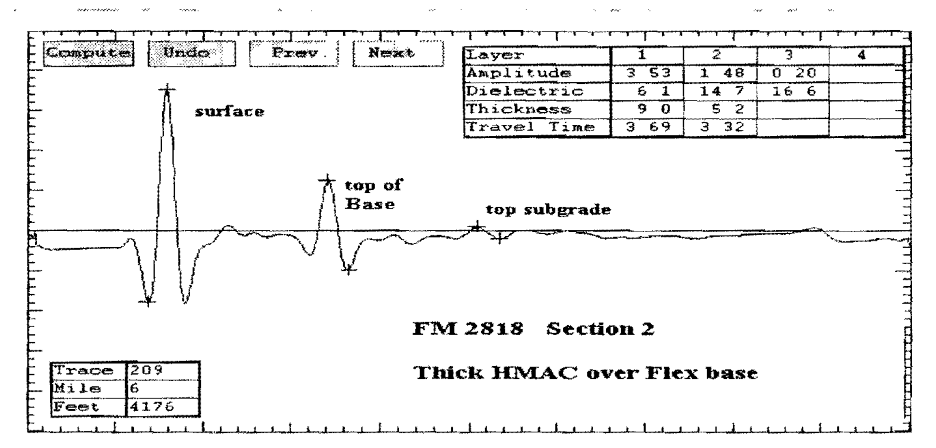


Figure 7. COLORMAP Display Highlighting Variable Pavement Structure on FM 2818 in Bryan District.





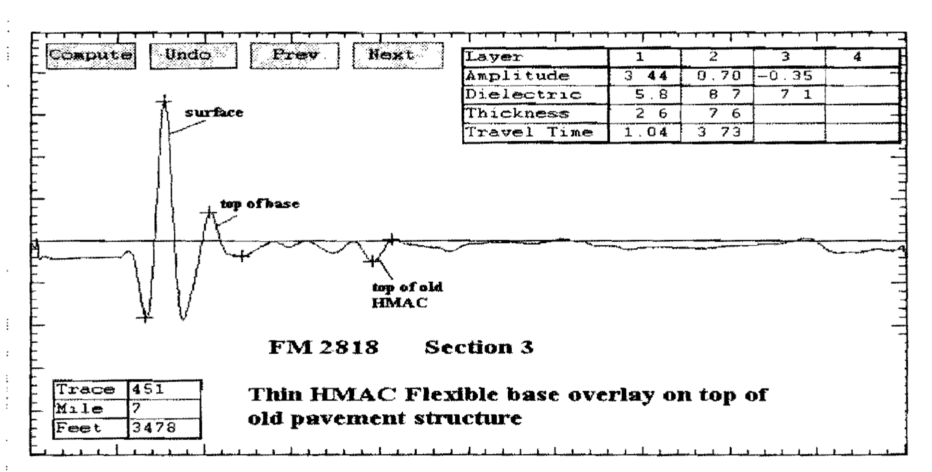


Figure 8. Representative GPR Traces from Each Section in Figure 8.

Three different pavement structures were found in this data. Individual GPR traces from each of these sections are shown in Figure 8. One clear benefit of GPR is that it can accurately identify where the pavement breaks occur so that future activities such as FWD data collection and interpretation, surface distress interpretation and coring can be more effective.

3.4 FALLING WEIGHT DEFLECTOMETER

TxDOT has a fleet of Falling Weight Deflectometers which are the main structural strength test device used in Texas to provide both structural evaluations and layer moduli for pavement design. The basis of the FWD and the significance of the surface deflections is shown schematically in Figure 9. In Texas it is standard to operate with the surface sensors mounted at 12 inch spacings. A critical issue in the interpretation is that the sensors farthest from the load plate are essentially measuring the stiffness properties of the lower layers (subgrade). The deflections measured at sensors closer to the load plate are influenced by the stiffness properties of the upper layers. In Texas the MODULUS 5.1 analysis program is used to process the FWD data *(7, 8)*. A summary of recommendations for collecting, processing and interpreting FWD data are presented in this section of the report.

3.4.1 Data Collection for Project-Level FWD Testing

- Sensor Locations, 1-foot spacing.
- Load Level - for design purpose the load level closest to 9,000 lb is recommended. This usually corresponds to FWD drop height 2. Taking multiple drops at the same location at different load levels is not recommended. Rather, collect only one or two drops at each location and use the time saved to collect data more frequently.
- Frequency of Data Collection - on any project a minimum of 30 drops should be taken. A maximum spacing of 500 ft should be used (unless GPR or core data indicate a closer test spacing is warranted).
- The FWD operator must monitor the temperature of the asphalt surface at the beginning and end of FWD data collection. The preferred method is by drilling a hole to a depth of 1 inch and measuring the temperature at that depth. As a minimum, surface temperature must be recorded.

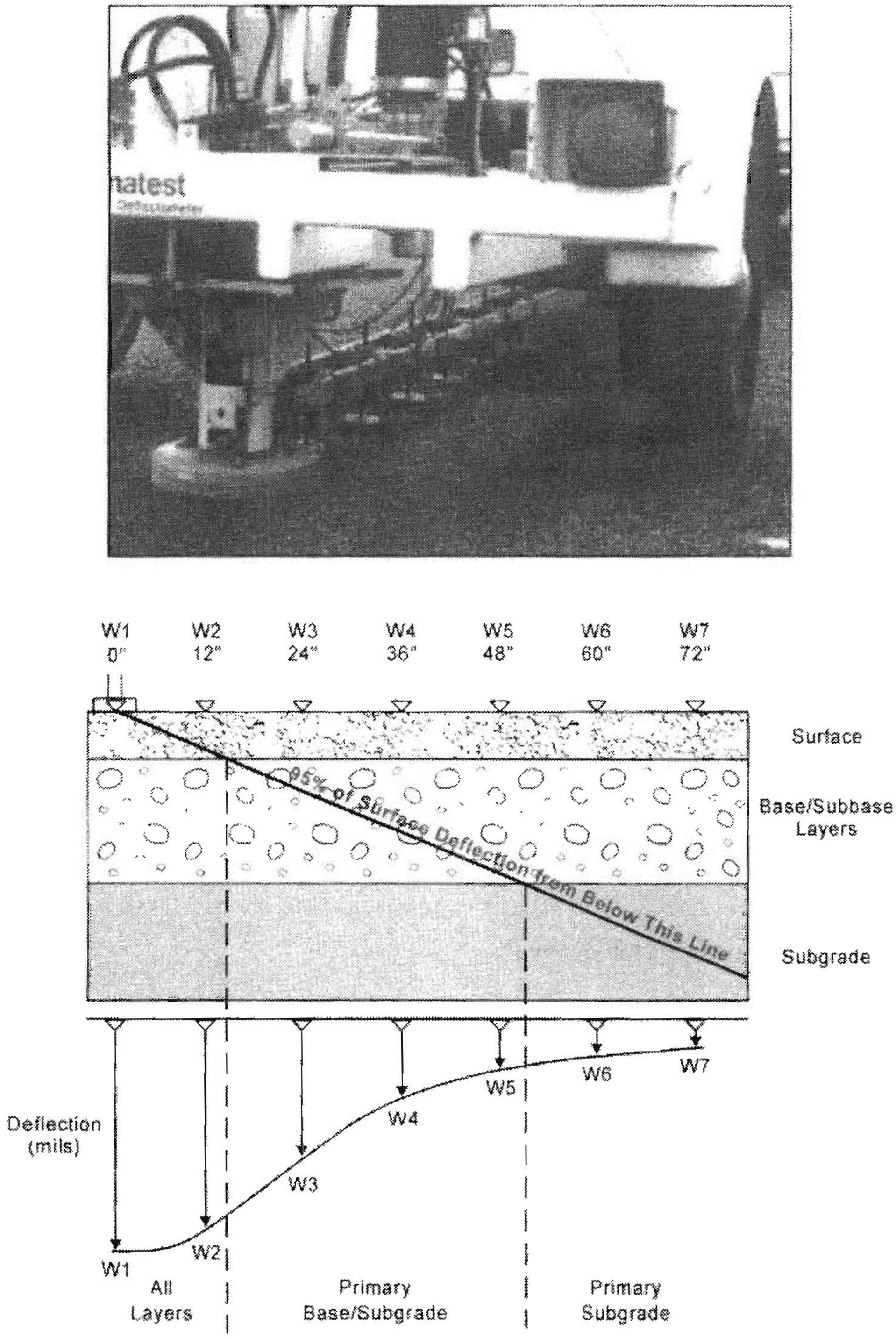


Figure 9. FWD Equipment and the Influence of the Various Layer on Surface Deflections.

These temperatures are used to correct the asphalt layer moduli and for inputs to the remaining life routine. However, temperature measurements are not required if the surfacing layer is less than 3 inches thick.

- The FWD operator should enter frequent comments into the FWD file, noting areas of structural damage (alligator cracking, rutting) or extensive maintenance (patching). Also note crests of hills, cut/fill sections, presence of standing water in ditches, etc. Avoid badly cracked areas when collecting data. While collecting data on any section, it is often beneficial to take the maintenance supervisor along so that any unusual features can be recorded as the FWD data is collected.
- The responsible engineer should estimate the average rut depth in the wheel paths and the percentage of alligator cracking in the wheel paths for each complete section. These will be required inputs in the Remaining Life computation in MODULUS. As a fall-back position, these values can be obtained from PMIS.

3.4.2 Data Analysis (Remaining Life Estimation)

The primary method of assessing layer strengths is by use of the backcalculation procedure discussed in section 3.4.3. However, to provide the engineer with basic information about the pavement's overall load-carrying capacity, the remaining life procedures were developed. Details of this procedure can be found elsewhere (*8, 9*).

As the first step in evaluating a pavement's structural strength option 2, the remaining life analysis program should be run. This step was intended as a rapid "first-cut" method of identifying the level of rehabilitation required. In many instances planning and budgeting for rehabilitation projects the question is often raised regarding the need for a structural overlay. The remaining life procedure was included in the MODULUS package to address this issue. This procedure requires the engineer to enter the thickness of the HMA of the section, the 20 year design traffic estimate (18-kip ESALs), and the level of structural distress in the section (alligator cracking and rutting). From the FWD data and the user inputs, the program calculates:

- the overall structural adequacy of the upper and lower layers in the pavement structure (5 categories from very strong to very weak),
- the remaining life classification (four categories from 0-2 years to greater than 10 years), and

- the uniformity of the section.

The layer strength classification will also be useful in validating the backcalculated moduli values.

3.4.3 Design Level Data Analysis (Backcalculation of Layer Moduli)

This routine is the main analysis tool within MODULUS 5.1. A step-by-step summary of how to process FWD data is as follows:

Assigning Layer Moduli

The following general guidelines should be used in the majority of cases when running the FULL ANALYSIS procedure:

- Always start with a three-layer analysis problem. For this first run, ignore possible stabilized subgrades. Guidelines will be given later on, when a four layer analysis should be run.
- Combine all asphalt layers into a single layer.
- Assign a nominal surfacing thickness of 12 mm (0.5 inches) to surface treated pavements (surface treatment over a granular base).
- Use the following default moduli values in the FULL ANALYSIS option:

Layer		Modular Range (ksi)	Poisson's Ratio
Asphalt		See Figure 10	0.35
Flexible Base		10-150	0.35
Lightly Stabilized Base < 3% Stabilizer		50-250	0.25
Heavily Stabilized Base > 3%		100-1000	0.25
Concrete		1000-7000	0.20
Subgrade	Weak Average Strong	5 10 15	0.40

When using Figure 10, the user inputs the average surface layer temperature at the time of FWD testing and uses the two outer curves to generate an acceptable moduli range. For a typical asphaltic layer, the backcalculated moduli value should fall in this range. If the value is outside of this range, then either the pavement is incorrectly modeled (probably wrong layer thicknesses) or there is a problem with the asphalt. A high number may indicate an aged stiff layer; a low number indicates a soft or cracked layer.

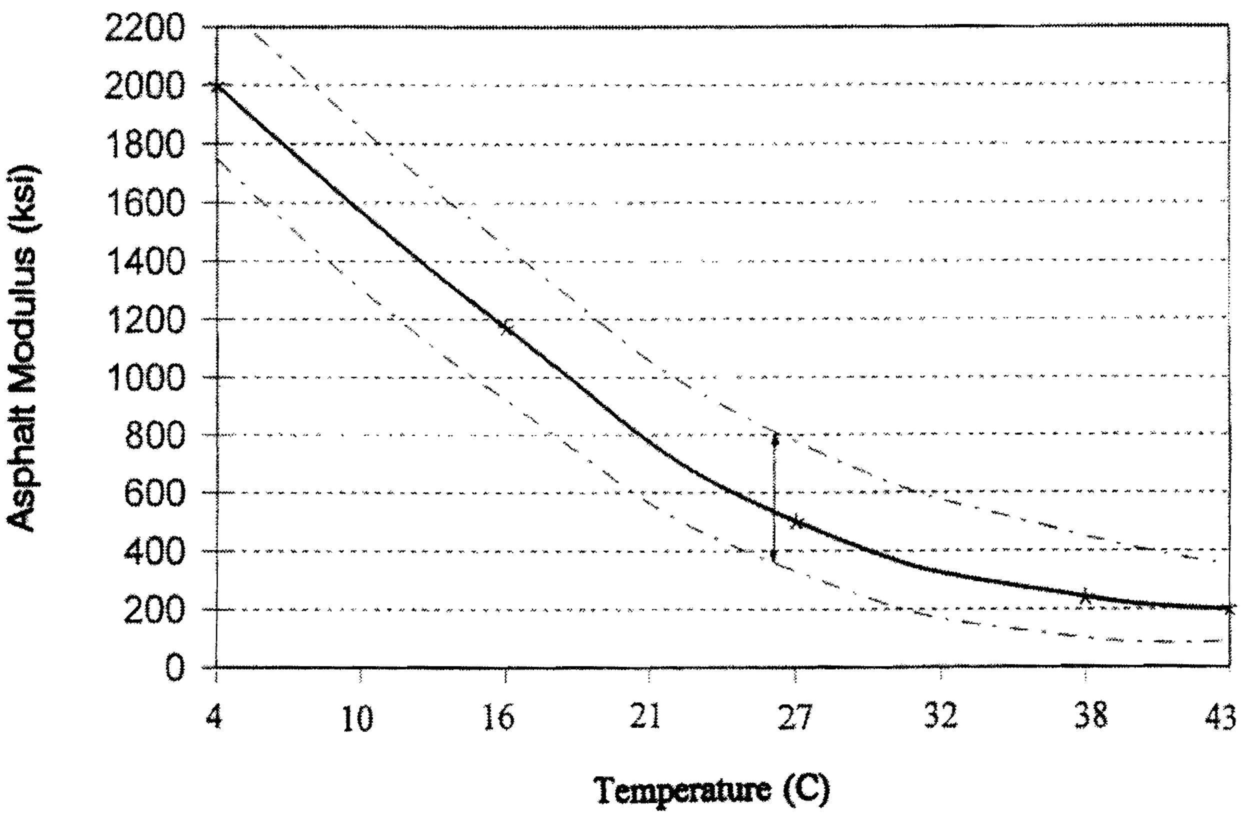


Figure 10. Influence of Temperature on Layer Moduli. The Outer Lines Represent the Acceptable Moduli Range. (For example at 26 °C (78 °F), the Low Value is 360 ksi and High Value is 810 ksi.)

- Backcalculation should not be attempted for surfacing layers less than 3 inches thick. A fixed modulus value should be entered for these thin layers. This input modulus value is calculated using Figure 10 and selecting the value from the middle curve. This value is entered for both the upper and lower range values in the MODULUS full analysis screen.
- The user must supply a recommended subgrade modulus and Poisson ratio to start the program. Typical values are 5, 10, and 20 ksi, based on knowledge of existing subgrade strengths. If this is unknown a value of 10 ksi should be used with a Poisson ratio of 0.4. (This input value is not critical, MODULUS will automatically rerun the analysis if its calculated value is substantially different from the user-supplied input value).
- The weighting factors should be left at 0.0, which means that MODULUS 5.1 automatically assigns weights to each sensor.
- If MODULUS is to be used to generate design moduli values for new paving materials, it is recommended that the pavement be at least 1 year old. This will provide sufficient time for the base and subgrade layers to reach a moisture equilibrium.

Typical Outputs

Figure 11 shows the main output value from MODULUS 5.1, from a section of IH 635 in Dallas with a total HMA surface thickness (Asphalt Stabilized Base (ASB) plus wearing surface) of 10 inches with a 24-inch thick treated base layer. Interpretation rules for these values will be discussed later. Items of interest to the pavement engineer are the following:

- average moduli values for each layer (144 ksi, 183 ksi and 14.7 ksi) for the surface, base, and subgrade respectively,
- variability of backcalculated moduli values (relatively uniform in this case as this was for a newly constructed section, for new pavements a 30 percent variability in layer moduli is common. With older pavements much higher variability is found),
- goodness of fit estimates (in terms of error per sensor); this was low in this case at 1.8 percent,
- the computed depth to a stiff layer which in this case was 93 inches below the pavement layer, and

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT)						(Version 5.1
District: 18			MODULI RANGE(psi)			
County: 0		Thickness(in)	Minimum	Maximum	Poisson Ratio Values	
Highway/Road: IH635	Pavement:	10.00	100,000	300,000	H1: % = 0.35	
	Base:	24.00	20,000	300,000	H2: % = 0.35	
	Subbase:	0.00	0	0	H3: % = 0.35	
	Subgrade:	93.00	20,000		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	9,056	7.19	3.73	2.61	2.02	1.56	1.26	1.02	196.	121.3	0.0	12.9	1.61	157.98
21.000	9,048	8.76	4.50	2.87	2.03	1.49	1.21	0.92	173.	75.0	0.0	15.5	3.03	187.74
41.000	9,033	8.43	3.98	2.60	1.91	1.50	1.22	0.94	152.	107.5	0.0	15.2	3.09	124.68
60.000	9,084	7.98	3.96	2.62	1.79	1.36	1.09	0.87	183.	88.7	0.0	17.2	2.75	161.61
80.000	9,037	7.74	3.34	2.32	1.72	1.35	1.12	0.90	147.	151.4	0.0	16.2	2.53	151.92
101.000	9,092	7.04	2.67	1.94	1.59	1.30	1.09	0.86	135.	296.7	0.0	15.5	2.26	116.54
120.000	9,052	6.98	2.82	2.07	1.61	1.31	1.09	0.87	145.	238.2	0.0	15.5	2.00	134.39
140.000	9,005	7.70	2.78	2.12	1.69	1.30	1.04	0.81	122.	250.3	0.0	16.0	1.06	116.90
162.000	8,993	7.02	2.91	2.13	1.67	1.31	1.09	0.85	149.	211.7	0.0	15.4	1.66	119.42
180.000	8,961	8.11	3.24	2.42	1.84	1.43	1.14	0.88	128.	177.7	0.0	14.8	1.44	121.93
200.000	8,933	8.33	3.49	2.49	1.82	1.43	1.13	0.89	133.	140.7	0.0	15.5	1.70	140.37
220.000	9,013	8.40	3.45	2.44	1.80	1.38	1.10	0.81	131.	143.0	0.0	16.3	1.68	104.26
240.000	8,973	8.39	3.58	2.52	1.89	1.42	1.12	0.85	137.	131.0	0.0	15.6	1.22	115.32
259.000	8,941	8.87	3.96	2.72	1.93	1.43	1.12	0.85	141.	99.6	0.0	16.0	1.83	122.42
280.000	8,969	7.87	3.15	2.34	1.89	1.54	1.24	0.94	125.	226.0	0.0	12.9	1.02	105.81
300.000	9,096	7.68	3.43	2.59	1.94	1.44	1.15	0.84	157.	143.0	0.0	14.4	1.80	192.66
320.000	9,021	6.89	2.47	1.90	1.48	1.19	0.97	0.77	135.	300.0	0.0	17.3	1.53	127.67 *
340.000	8,925	7.14	2.70	2.03	1.62	1.34	1.07	0.82	132.	275.5	0.0	15.1	1.01	108.13
360.000	8,941	7.20	3.09	2.20	1.74	1.43	1.16	0.91	147.	200.6	0.0	14.2	2.06	122.57
382.000	8,874	7.45	3.14	2.65	2.21	1.85	1.57	1.21	133.	266.7	0.0	8.9	1.91	113.48 *
402.000	8,917	7.37	2.87	2.13	1.77	1.47	1.22	1.00	126.	288.8	0.0	12.9	1.80	146.69
420.000	8,846	9.13	4.55	3.27	2.49	1.95	1.53	1.17	143.	100.3	0.0	10.2	0.85	126.17
Mean:		7.80	3.36	2.41	1.84	1.44	1.17	0.91	144.	183.4	0.0	14.7	1.81	126.97
Std. Dev:		0.67	0.58	0.33	0.22	0.17	0.14	0.11	19.	74.5	0.0	2.1	0.63	19.90
Var Coeff(%):		8.64	17.20	13.79	12.07	11.93	12.05	12.02	13.	40.6	0.0	14.0	34.60	15.67

Figure 11. Main Output from MODULUS 5.1, Computed Layer Moduli, Depth to Stiff Layer (93 inches), and Average Error per Sensor (1.81%).

- for each set of deflection data, the number of times the MODULUS solution was outside of the moduli range supplied by the user as input. This is denoted by the "*" in the far right column. In this example two bowls hit the limits.

Handling Shallow Bedrock Situations

MODULUS solutions have been found to be reasonable in most areas in Texas. However problems have been found with the results when the depth to a stiff layer is computed to be close to the surface (less than 60 inches). This happens in the following two different cases.

Case 1 Shallow Bedrock

This is common in West Texas particularly in the Odessa and San Angelo Districts. The rock layer directly beneath the pavement layers has a major impact on the deflections measured at the outer sensors. As a rule of thumb if the deflections measured at the W5 sensor (48 inches from the load) are less than 1 mil then bedrock is probably located close to the surface. With average depth to stiff layer of less than 60 inches the MODULUS solutions are not reliable. In these cases in the Full Analysis run the calculations should be repeated with a fixed H4 (subgrade thickness value) of 240 inches. The engineer should then compare the standard MODULUS solution with that calculated with the constant 240 inch subgrade and determine which is more reasonable in terms of both error per sensor and subgrade moduli values.

Case 2 Wet/Weak Subbases and Shallow Water Table

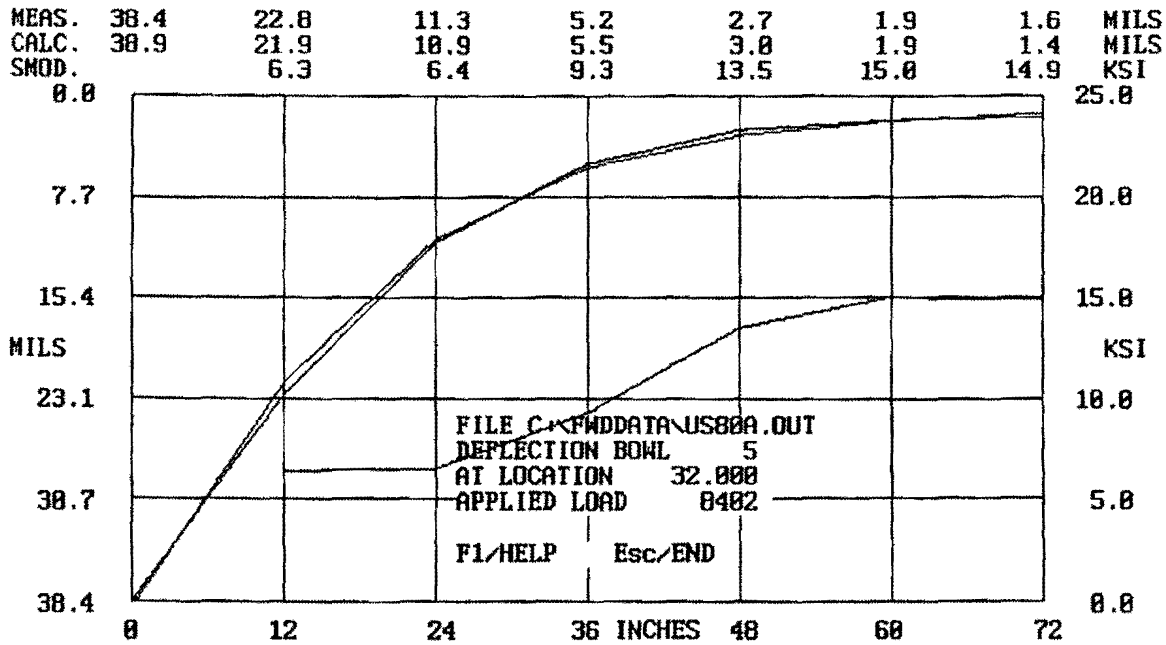
The current version of MODULUS will detect any stiffening of the subgrade from the measured surface deflections. It then assumes that the modular ratio between the subgrade and the underlying stiff layer is 100. This, particularly in East Texas, is not always the case. Problems have been found with very weak subbase or weak upper subgrade layers over stiffer clay layers or in areas where the water table comes close to the surface. Both of these instances are currently under investigation. Until these studies are complete, the following approach is recommended if one of these cases is suspected.

If the computed subgrade thickness is less than 60 inches and a shallow water table or weak upper subgrade is suspected, then review the "Display versus Measured" output within MODULUS. A typical output for a suspected weak subgrade section is shown in Figure 12. The lower line in the graph represents the composite MODULUS (SMOD values at top of figure) calculated using the Boussinesq equation (*10*). This is known as the Boussinesq single layer solution, and it provides a composite overall modulus for the entire pavement system. With reference to the discussion earlier (Figure 9) concerning the fact that the outer sensors are influenced by the subgrade layers only and that those sensors closer to the load become more influenced by the strength of the base and subbase layers. Clearly the shape of this curve can provide useful information concerning the strength profile in the upper subgrade. In this case the composite modulus calculated with the deflections measured 60 and 72 inches from the load are two to three times those calculated 24 inches from the load. This entire data set was reviewed and the average upper to lower subgrade modulus ratio was found to be approximately 3. The MODULUS 5.1 analysis was repeated with a new modular ratio of 3. This is done by hitting the <F9> button just prior to the <F10> button which runs the analysis. After hitting <F9> a window opens which allows the user to input a new modular ratio. For this section a ratio of 1 to 3 was input. The solutions obtained were documented in a technical memorandum to TxDOT (*11*), and the variation in subgrade strength was validated with field Dynamic Cone Penetrometer testing and by field sampling and compressive strength testing. The moduli obtained with the new modular ratio were judged to be "better" (lower error and more realistic moduli values) than the solution with the default modular ratio of 100.

When To Stop

Under normal conditions the user should attempt to reduce the average error per sensor to less than 6 percent per sensor. When rerunning the problem it may be necessary to:

- Adjust the acceptable moduli range for a layer if a limit is being hit.
- Remove non-representative deflection bowls from the data file. Often, data may be collected on sections where maintenance forces may have done extensive base repair work. These significantly lower deflection bowls should be excluded from the analysis.
- Break the highway into two or more sections if significant variations in strength are found based on delination graph from the MODULUS program.
- Adjust the weighting factors to eliminate a sensor which cannot be matched by the theoretical model. Setting the weighting factors to 1 for the sensors to be used and 0 for those to be dropped will eliminate these sensors from consideration. This option is to be used only if it is suspected that the sensor was malfunctioning.



Scale on Left - Deflection in mils

Scale on Right - Moduli (ksi) for Lower Line

Figure 12. MODULUS 5.1 Solutions for a Pavement with Weak Base and Weak Upper Subgrade. Note the Stiffening with Depth in Single Layer Modulus Solution (SMOD).

- Overwrite the computed depth to stiff layer value as discussed in the last section (for shallow bedrock an H4 value of 240 inches is recommended).
- Overwrite the default modular ratio of 100 as discussed in the last section (for stiff clay subgrades a value of 3 is recommended).

However, the 6 percent per sensor rule can be enforced with new pavements only in their first performance period. Maintenance activities which locally change layer thicknesses make setting error limits very difficult on older pavements. For older pavements no guidelines exist as to what error level is reasonable. The interpretation as to the "reasonableness of moduli values" is left in the hands of the pavement engineer. The guidelines presented in the next section are intended to aid in this assessment.

3.4.5 Interpreting the Moduli Values

The goal of any backcalculation analysis must be to provide realistic inputs for pavement design. Achieving a low error per sensor does not guarantee realistic answers. Backcalculated solutions are not unique; there may be several combinations of layer moduli that result in low error per sensor. Under Texas conditions, additional factors often occur which make it extremely difficult to reach the 6 percent per sensor goal; these include shallow bedrock situations where the outer sensor deflections are less than 1 mil. The presence of additional layers (often stabilized), the general poor information on layer thickness, and testing on repaired sections are also contributing factors.

With all of these uncertainties, how is the user to determine if the backcalculated moduli values are reasonable and of sufficient accuracy to be used for design? The answer is not simple. The user must gain experience with the backcalculated numbers and decide, based on his knowledge of the materials used and observed pavement performance, if the values are reasonable. To assist in this task, guidelines are given below on how to interpret these moduli values.

Subgrade Classification

Backcalculated Moduli Range (ksi)	Interpretation
1-4	Very Poor
4-8	Poor
8-12	Fair
12-16	Good
16+	Very Good

Flexible Base Classification

Modular Ratio E_{base} /$E_{subgrade}$	Interpretation
>3	Good
2-3	Marginal
<2	Poor

Stabilized Base

Base Modulus (ksi)	Interpretation
> 100	Light (<3%)
> 500	Intermediate (3-5%)
> 1000	Heavy (>5%)

Asphalt

Refer to Figure 10 to determine if the backcalculated moduli are within acceptable limits based on the surface temperature at the time of testing.

3.4.6 When to Use a Four-Layer Analysis

Four-layer analysis should only be performed if the three-layer results are unsatisfactory. Often in Texas, four layers are required primarily to handle stabilized subbases and subgrades. To check if a stabilized lower layer is present within MODULUS 5.1, review the plot of "Measured versus Computed" solutions. If a stabilized layer is present, a plot such as shown in Figure 13 is expected. This figure shows a pavement with a 5-inch asphalt surface, a 14-inch

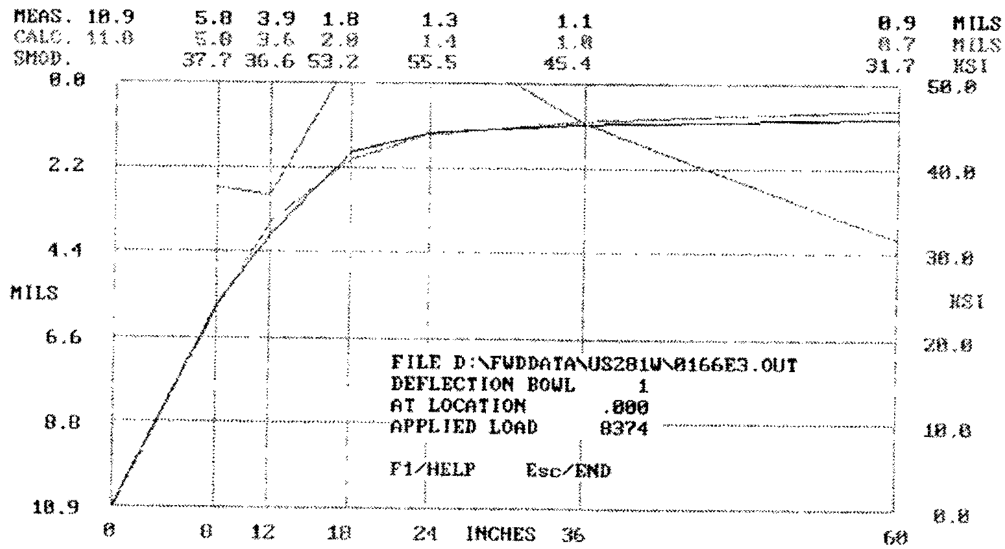


Figure 13. Evaluating the Need for a Four-Layer Analysis. The Composite Single-Layer Modulus Solution Shows a Definite Peak at the Sensor 24 inches from the Load.

flexible base, and a stiff 20-inch stabilized subbase. In this instance, field testing with DCP found that the penetration rates were higher in the base than in the stabilized subbase. This result is also observed in the single-layer solution (SMOD), where the highest moduli are computed with the sensor readings 18 and 24 inches from the load. In this instance a four-layer backcalculation analysis should be performed. If a stabilized subgrade layer is to be included as Layer 3, assign it a modular range from 25 to 250 ksi. If the MODULUS results are still unsatisfactory, then either coring or Dynamic Cone Penetrometer testing should be performed.

3.5 DYNAMIC CONE PENETROMETER

From an engineering viewpoint, one of the most important properties which a soil possesses is shearing resistance or shear strength. 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 Texas Triaxial Class (TTC) of a soil is used as a measure of shear strength. The DCP was originally designed and used for determining the strength profile of flexible pavements. It penetrates base and soil layers having TTC values of greater than 1. The DCP is a powerful, relatively compact, sturdy device that can be used by inexperienced personnel in pavement layer strength testing. The device does have problems with bases with large aggregates where the cone pushes aggregates, and it provides little useful information on heavily stabilized base layers.

Several districts within TxDOT have purchased DCPs for their subbase and base testing. Applications range from measuring the depth to a stiff layer, to estimating subgrade modulus for new pavement construction, to establishing if a stabilized subgrade is still effective.

DCP Device

The DCP is shown schematically in Figure 14 (*12*). The DCP used consists of a 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 17.6 lb. The angle of the cone is 60°, and the diameter of the base of the cone is 0.16 inches 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 17.6 lb sliding hammer from a height of 22.6 inches. 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 hammer. The DCP is designed to penetrate soils to depths of 36 inches. The strength of the soil is calculated from the rate of penetration of the cone.

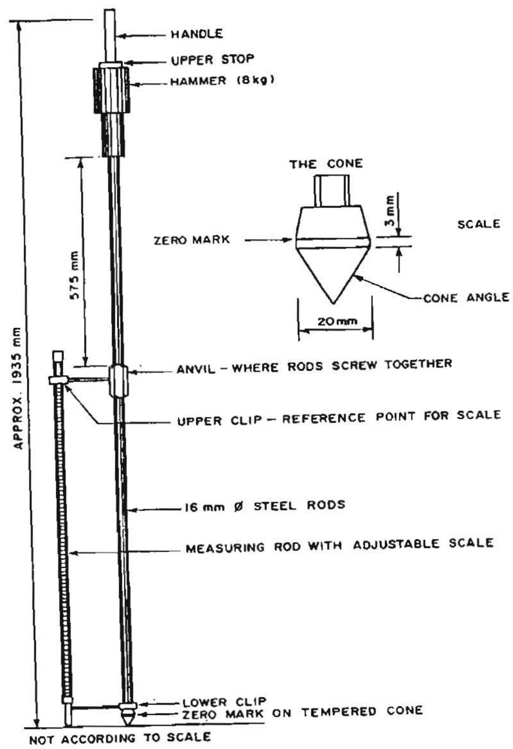
DCP testing is often conducted in existing pavements; therefore a small diameter access hole must be drilled. The complete DCP used in Texas is shown in the photograph in Figure 14. It includes the following additional items; a) a rotary percussion drill and long 1.25 inch diameter drill bit, b) a shop vacuum to clean out the hole, and c) a small portable generator.

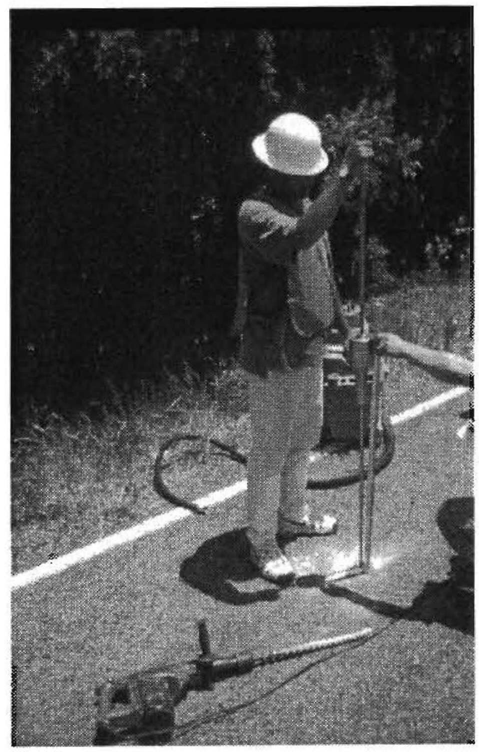
Extraction of the Cone

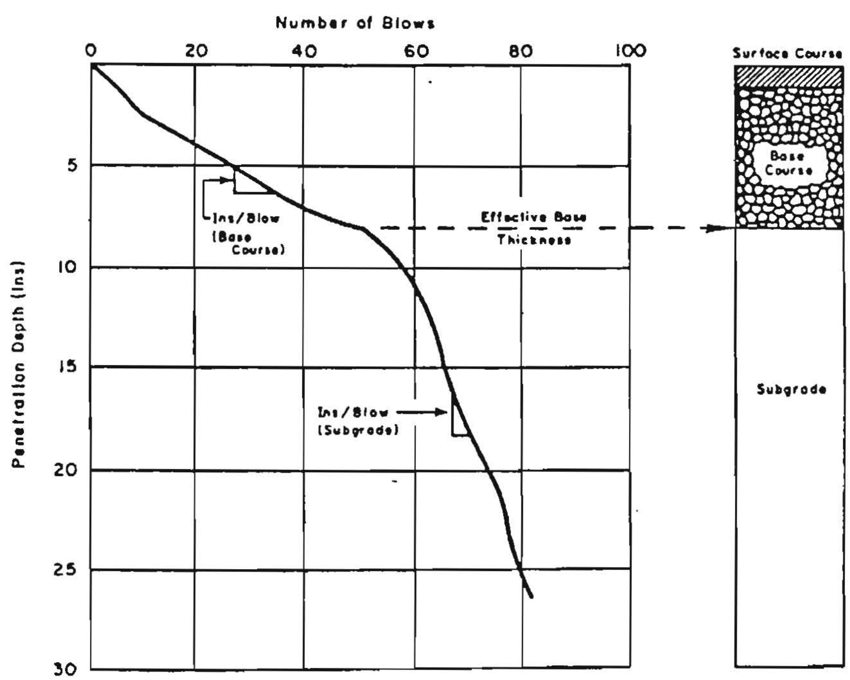
After the cone has been driven to the desired test depth, the cone or driving rod without disposable cone is extracted from the soil by driving the hammer against the top handle. Caution must be exercised during this operation in order not to damage the DCP device. The hammer must be raised in a vertical direction (rather than in an arching motion), or the rod may be bent or

A. Equipment

B. Field Operation







C. Typical Data

Figure 14. Dynamic Cone Penetrometer.

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 (*12*). The 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, the standard cone was difficult to remove. The disposable cone mounts on an adapter. At the conclusion of the test, the disposable cone 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

The correlation between the DCP penetration per blow and subgrade modulus is a two-step process. The DCP penetration rate in mm per blow is first converted to the CBR (California Bearing Ratio) using the following Corps of Engineers equation (*12*):

$$CBR = (292/PR)^{1.12}$$

where PR is the DCP penetration rate in mm/blow.

This can be converted to a subgrade modulus value with the use of the following equation:

$$E = 2500\ (CBR)^{0.64}$$

CHAPTER 4
PAVEMENT RECLAMATION

4.1 INTRODUCTION

Texas has many miles of thin surfaced flexible pavements which are structurally inadequate for the current level and intensity of truck loads. An increasingly popular rehabilitation strategy is full depth pavement reclamation wherein the existing pavement layers are milled, stabilized with lime, cement, fly ash, asphalt or combination of such materials and compacted as a new base or subbase layer. The two districts which have taken the lead in reclamation are the Bryan and Lubbock Districts. As of year 2000 the Bryan District has reclaimed over 500 miles of low-volume FM roadways. The stabilizers of choice in this district have been either cement or lime. The Lubbock District has also recycled many miles of low-volume roadways primarily using fly ash as the stabilizing agent.

From studies conducted to date few guidelines are available to assist districts in selecting the optimum stabilizer type and content. The current strength-based specifications for cement treated bases are based on achieving a 7 day unconfined compressive strengths of 500 psi. However, in many cases this has been shown to result in layers which do not perform satisfactorily in terms of shrinkage cracking. This is cracking of the base layer itself or cracking of the stiff base due to movements of the underlying subgrade layers. Accordingly many districts have developed their own rationale for selecting stabilizers. For example, the Childress and Lubbock Districts' goal of adding sufficient stabilizer to improve the existing material to meet the Class 1 flexible base triaxial criteria, TxDOT method 117-E (*13*). Other districts still base their requirements on 7 day unconfined compressive strengths, but values in the range of 200 to 300 psi are specified.

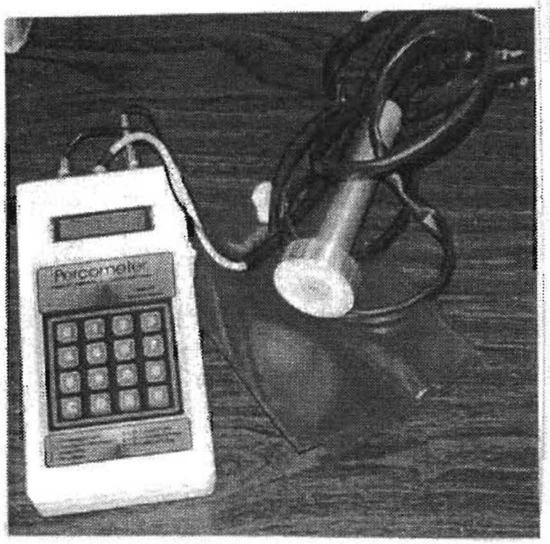
4.2 NEW DESIGN APPROACH

The selection of the optimal stabilizer content for any layer is indeed a balancing act. The layer must be strong enough to withstand the forces applied by the design truck loads. It must be volumetrically stable and not shrink or crack excessively, and it must also withstand the impact of moisture and environmental cycling. Increasing the stabilizer content will usually increase both strength and durability, but it will also produce more shrinkage cracking.

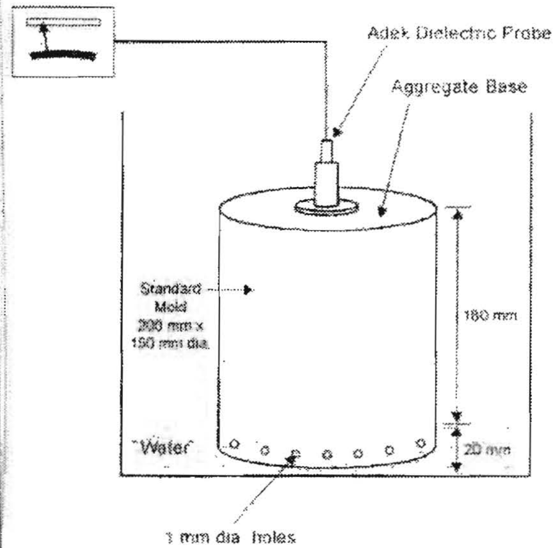
As part of this project new criteria were developed for selecting stabilizer content. These involve selecting the minimum stabilizer content to meet the requirements of strength and moisture susceptibility. A new test called the Tube Suction Test was proposed to assess moisture susceptibility of stabilized materials. This rationale behind this test is presented in the next section of this report.

4.3 DEVELOPMENT OF THE TUBE SUCTION TEST

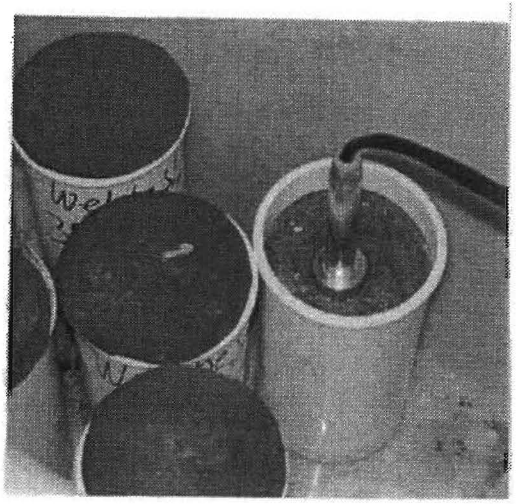
In the past five years, the Texas Transportation Institute has been evaluating a new test procedure, the tube suction test, to identify poorly performing unstabilized base materials (*5, 14, 15*). In the TST the moisture susceptibility of any material is evaluated in terms of its capillary rise characteristics and the resulting surface dielectric values. The test set up for the TST is shown in Figure 15. It involves compacting a 6 by 8-inch sample at optimum moisture content in a concrete cylinder mold with a series of small diameter holes drilled in its base. The sample is dried in a 40 °C room for several days, and then the capillary rise of moisture is monitored with a dielectric probe, which measures the dielectric properties of the surface of the sample. High surface dielectric measurements indicate suction of water by capillary forces. The mineralogy of the fines contributes to the affinity for water. The dielectric itself is the measure of the unbound moisture within the base. It is this "free" moisture rather than the bound moisture which is thought to be responsible for poor performance under applied vehicle loads and poor resistance to freeze-thaw cycling. Studies have been underway in both Texas and Finland to relate the laboratory and field performance of materials classified by the TST (*6, 15*). These laboratory studies found that materials ranked poorly by the TST also had poorer load bearing capability as measured by their resilient modulus and permanent deformation properties. They also reported that poorly ranked materials were also highly susceptible to frost damage. Based on Texas and Finnish studies, tentative criteria have been established for unstabilized base material. If the surface dielectric exceeds a value of 10, then that material may not perform well under heavy traffic loads in areas which are subject to freeze-thaw cycling. A failure level of 16 has been proposed. Materials exceeding this value should be considered for chemical stabilization.



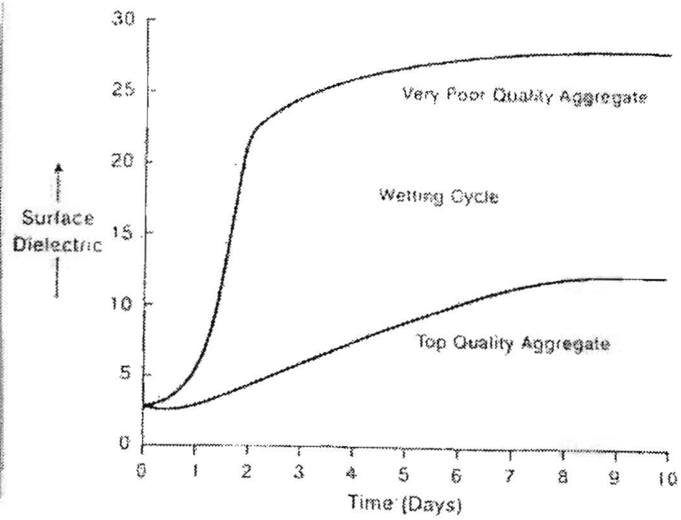
a. Dielectric Measuring Equipment



b. Test Set-up



c. Surface Measurement



d. Typical Results.

Figure 15. Tube Suction Test (*5*).

Extensive failure investigations in Texas have indicated that the durability problems with stabilized bases are often associated with moisture problems. The TST has recently been extended and modified for testing stabilized base materials. Full details of the test procedure for stabilized materials are given in the Appendix. In this modified procedure, the specimens of stabilized materials are first cured for 7 days prior to the dryback. They are then placed in 0.25 inches of deionized water. Recent work at TTI has linked the TST results to poorly performing stabilized bases in Texas (*16*). If moisture can flow into the stabilized layer from surface cracks or from wet subgrades, then deterioration may occur. Due to the metastable nature of many of the mineral phases associated with chemical stabilization, water movement can also leach alkali metals and alkaline earth metals, decreasing the strength of the stabilized material.

4.4 RECOMMENDED DESIGN CRITERIA

The initial work with the TST was performed on unstabilized base materials. However recently this has been extended to assisting with selecting the optimal stabilizer type and stabilizer content. The proposed testing procedure for stabilization studies is shown schematically in Figure 16. In summary at each stabilizer content under evaluation the samples are compacted using standard TxDOT specification at optimum moisture content. The samples are placed in a sealed bag and placed in a 100 percent humidity room for 7 days. After 7 days the unconfined compressive strength of one of the samples is measured. The remaining sample is then subjected to the TST cycle which includes a 4 day dryback and then a 10 day capillary rise. The surface dielectric of the sample is measured periodically over the 10 days. The unconfined compressive strength of the TST sample is measured at the completion of the TST.

For the Fort Worth materials tested as part of this study the following criteria were used:

- Minimum Unconfined Compressive Strength (UCS_a)= 250 psi (7 day cure strength),
- Final Dielectric less than 10, and
- UCS_c after TST sample at least 80 percent of UCS_a (7 day strength).

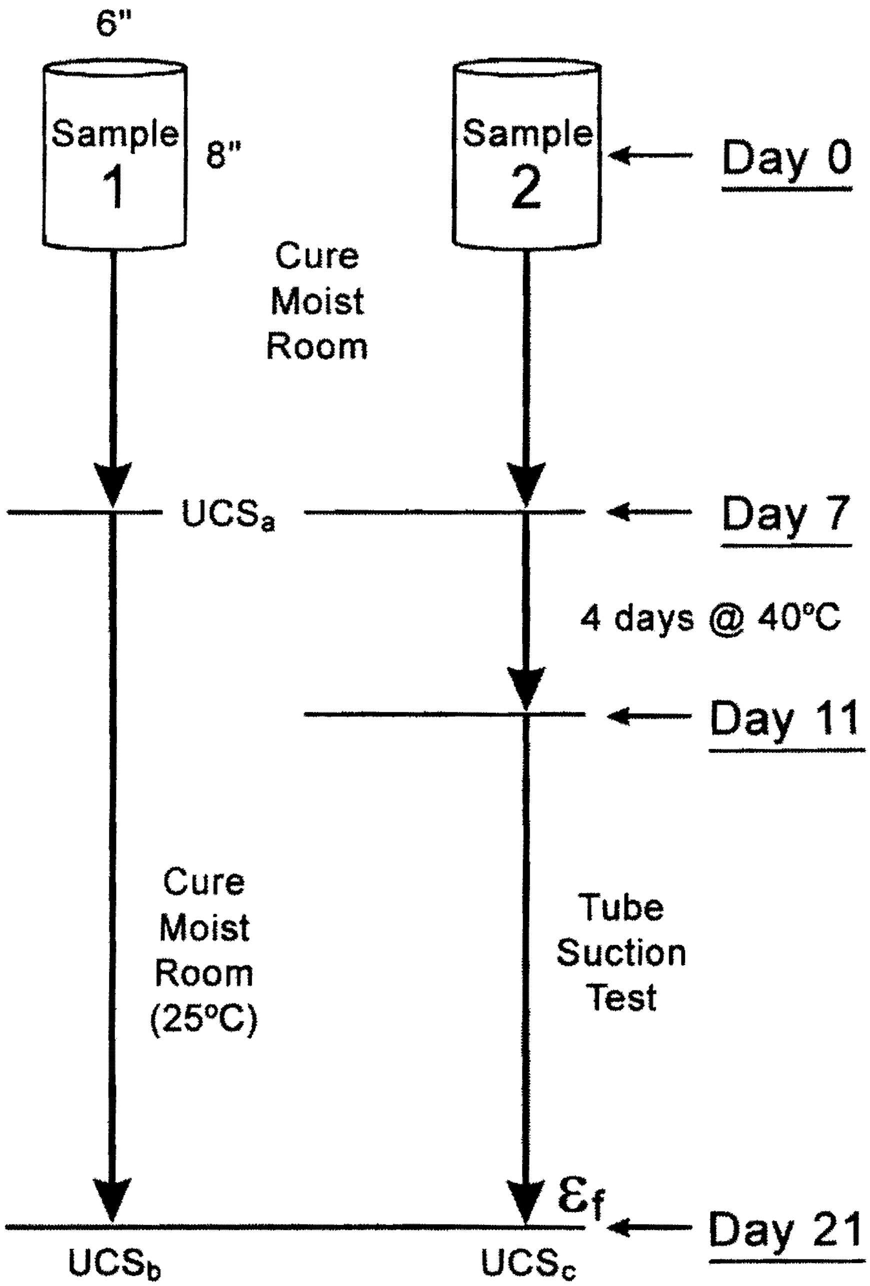


Figure 16. Recommended Testing Procedure for Selecting Optimal Stabilizer Content.

Although not part of the test criteria it has become standard practice to place the sample used in the 7 day break back in the moisture room to check for rehealing of the material. In project 0-1712 three full depth reclamation projects were evaluated for the Fort Worth District. Samples of the existing surface and base were milled from the pavement section. The samples were screened so that all materials passed the 1-inch sieve. A standard Optimum Moisture Content (OMC) test was performed followed by the test sequence described above. The district has already decided that they were going to use cement, so the test sequence was run at a range of cement content. The results from the FM 2331 are shown below in Table 1.

Table 1. Selecting the Optimum Stabilizer Content for FM 2331, Fort Worth District.

Cement Content (%)	**7 day UCS_a (psi)**	**21 day UCS_b (psi)**	**TST UCS_c (psi)**	**Final Dielectric (ε)**
0	73	117	58	5.8
2	225	255	270	4.8
3	339	380	283	4.9
4	342	385	336	4.5

Based on the results shown above the 3 percent cement content was selected for field use. All of the materials passed the tube suction test dielectric criteria, even the untreated raw material. This indicates that the existing base is good quality with little or no deleterious fines.

The important feature of these new criteria is that optimum stabilizer content is selected based on both reduced unconfined compressive strength and moisture susceptibility, rather than strength alone. The goal is to reduce stabilizer content while maintaining durability to hopefully avoid excessive shrinkage cracking. The current strength-based criteria set a high unconfined compressive strength criteria of 500 psi, with the underlying intent that this high strength will automatically guarantee durability. With many marginal materials this leads to high stabilizer contents which often do not give satisfactory long-term performance.

TxDOT has recently purchased additional TST test equipment and full-scale implementation is scheduled for 2001.

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APPENDIX
TUBE SUCTION TEST

APPLICATION

Based on the principles of soil suction, moisture ingress, and dielectric permittivity, the tube suction test is designed to evaluate the moisture susceptibility of aggregates used as base materials in pavements. The Adek Percometer™, a 50 MHz surface dielectric probe, is employed in the TST to measure the dielectric values of compacted aggregate samples during a monitored exposure to capillary rise conditions in the laboratory. The interpretation of test results is founded on an empirical relationship between the final dielectric value and the expected performance of aggregate base materials in the field. The Adek Percometer is shown in Figure A1.

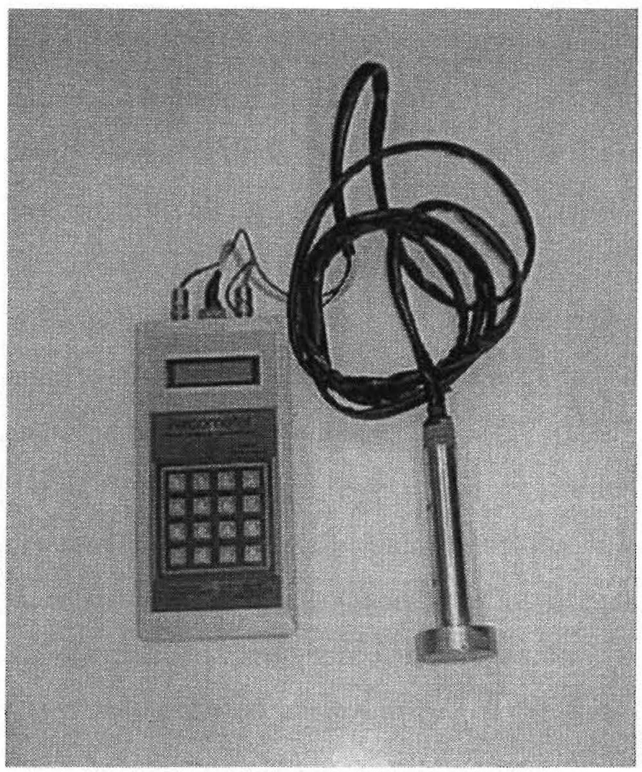


Figure A1. Adek Percometer.

Since the stabilized materials may be base or subgrade, the TST protocol for stabilized materials only includes sample testing and data analysis procedures. Standard procedures for sample preparation specify the use of 6 by 8 inch specimens for base material and 4 by 6 inch specimens for subgrade, using the appropriate TxDOT method for sample preparation. The specimens are allowed to cure for 7 days then dried back 4 days. Base material is dried in a 104 °F environment and subgrade is air dried to prevent excessive cracking.

Sample Testing

1. After the 4 day dry back, record the weight of each sample to the nearest gram.
2. Use the Adek Percometer to take six dielectric readings on the surface of each sample. Five should be around the perimeter of the sample, and the sixth should be in the center. Press down on the probe with a force of about 20 pounds to ensure adequate contact of the probe on the sample surface. Some twisting can also be used to seat the probe. Follow this pattern for each sample each time dielectric readings are made. See the attached typical TST data collection form.
3. Place each sample in the empty soaking basin. Base materials are placed directly into the basin. Subgrade specimens should be placed on a 4-inch diameter porous stone.
4. Use distilled water at 77 °F to fill the soaking basin. For subgrade specimens, the water level should be maintained at a depth equal to the height of the porous stone. Previous experience with subgrade soils in this test has indicated the specimens tend to "melt" over time when placed in direct contact with water. The setup for testing subgrades is shown in Figure A2. For base samples, a water depth of 1 inch is maintained. A picture of the setup for testing base specimens is shown in Figure A3.

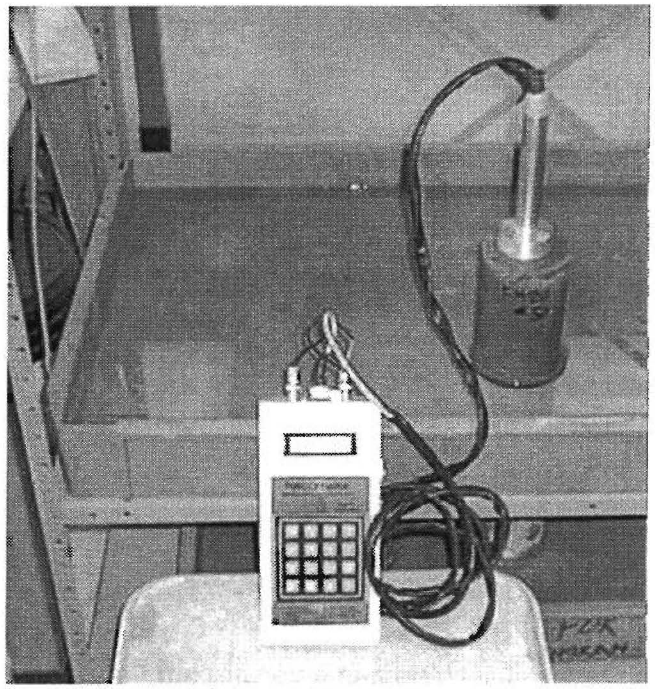


Figure A2. Setup of the TST for Subgrade Soils.

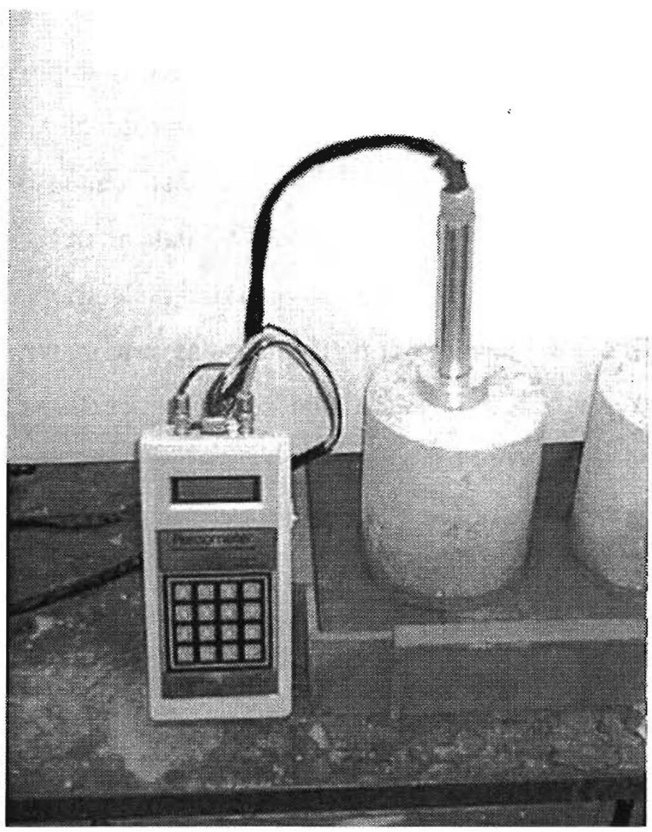


Figure A3. Base Specimens in the TST.

5. Take additional dielectric readings at time intervals of approximately 24 hours until testing is completed.
6. If the water content is also to be monitored through time, the weight of each sample should be recorded at the same or similar time intervals.
7. The test is completed when the elapsed time exceeds 240 hours. Take final dielectric readings, and record the final weight.
8. If the actual dry density of each sample is to be calculated, place the samples in an oven for drying to constant weight. The oven should be maintained at a temperature of 230 °F. Record the dried weight.

Data Analysis

1. For each sample at each time interval, discard the highest and lowest dielectric readings. Average the remaining four readings, and plot these against time.
2. If the weight of each sample was also recorded at each time interval, the water content through time may be calculated for each sample and also plotted against time.
3. Other information such as dry density, relative density, and residual moisture within each sample before placement in the water bath can also be calculated and shown in the analysis. See the attached typical TST data analysis sheet.
4. Base aggregates whose final, or asymptotic, dielectric values are less than 10 are expected to provide superior performance as base materials in the presence of water. If these aggregates have reasonable strength characteristics, they should be able to withstand both heavy truck loads and environmental stresses. Aggregates with final dielectric values between 10 and 16 are expected to provide marginal performance as base materials. For inadequately drained base layers, moisture available in the subgrade soils or from the pavement surface may cause poor freeze-thaw resistance and reduced shear strength. Aggregates with dielectric values exceeding 16 are expected to be especially moisture susceptible. In the field, these aggregates are expected to be moist or near saturation and susceptible to forming ice lenses upon freezing. Under a dynamic load, plastic deformation of these materials may occur because of high pore water pressure and low shear strength.

The criteria for soil aggregates is a new area, as subgrade soils often have dielectric values close to or above 10 even when at very low moisture contents. A stabilized subgrade is deemed a good performer if:

- Moisture does not reach the top of the specimen, and the final surface dielectric value after 10 days is not significantly different from the original surface dielectric value, and
- The unconfined compressive strength (UCS) of the specimen after undergoing the TST is greater than or equal to 80 percent of the UCS of a corresponding specimen that was cured in a 100 percent relative humidity environment for 21 days.

Because little research has been done on dielectric testing of subgrade soils, the most weight in evaluating the soil performance is put on the strength after the TST. In practice, however, most subgrade soils we have tested have not met condition number 2, even when moisture does not rise through the entire specimen. However, the two evaluation parameters, when used together, are useful for identifying the better performer when comparing different stabilizer types and/or contents.

Figure A4 shows a typical data collection form used during the TST. Figure A5 is a typical summary of TST data.

TUBE SUCTION TEST

Data Collection Form

SAMPLE:

Year:

Dielectric Value

Date (mm/dd)																		
Time																		
Mass (g)																		
No. 1																		
No. 2																		
No. 3																		
No. 4																		
No. 5																		
No. 6																		

Comments																		

Figure A4. Typical TST Data Collection Form.

TUBE SUCTION TEST
Data Analysis Report

SAMPLE: **Pharr Caliche Untreated**

SAMPLE PREPARATION

Mold Mass (g)	283
Sample Diameter (in)	6.0
Sample Height (in)	8.1
Sample Volume (ft^3)	0.133

Wet Total Mass (g)	8240
Wet Soil Mass (g)	7957
Dry Total Mass (g)	7517
Dry Soil Mass (g)	7230

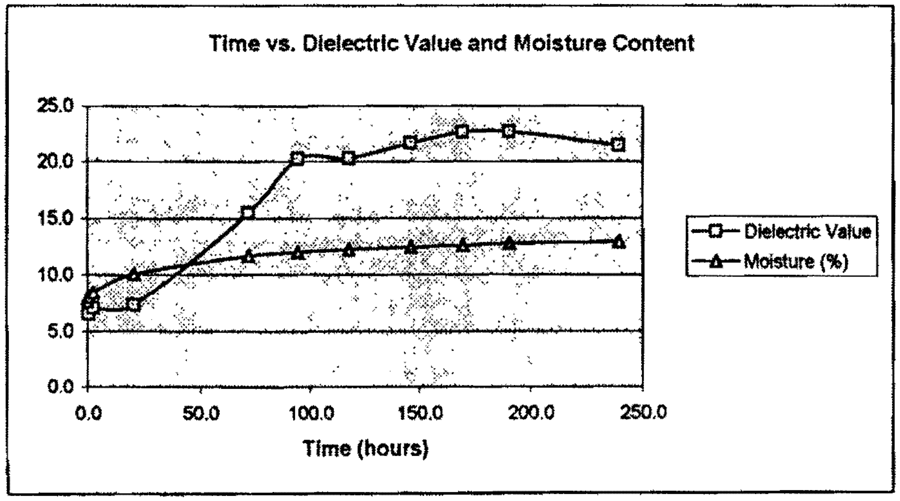
Desired Compaction Moisture (%)	10.0
Actual Compaction Moisture (%)	10.1
Desired Dry Density (lb/ft^3)	123.5
Actual Dry Density (lb/ft^3)	120.3
Relative Density (%)	97.4

SAMPLE TESTING

Time (hr)	0.0	0.5	2.5	20.5	72.0	94.3	117.9	146.4	170.2	190.6	239.5						
Total Mass (g)	8064	8096	8122	8242	8358	8378	8397	8414	8426	8434	8446						
Soil Mass (g)	7781	7813	7839	7959	8075	8095	8114	8131	8143	8151	8163						
Moisture (%)	7.6	8.1	8.4	10.1	11.7	12.0	12.2	12.5	12.6	12.7	12.9						

Dielectric Values

No. 1	6.2	6.6	7.3	7.8	16.7	20.8	20.8	20.8	20.1	24.9	22.1						
No. 2	6.4	5.7	6.2	6.6	14.1	19.9	20.7	22.0	23.7	20.6	23.6						
No. 3	6.7	7.3	7.4	7.4	15.9	19.0	19.9	19.9	20.8	24.2	22.6						
No. 4	7.3	6.4	7.4	7.5	15.2	21.3	19.8	23.9	25.8	20.9	17.3						
Average	6.7	6.5	7.1	7.3	15.5	20.3	20.3	21.7	22.6	22.7	21.4						



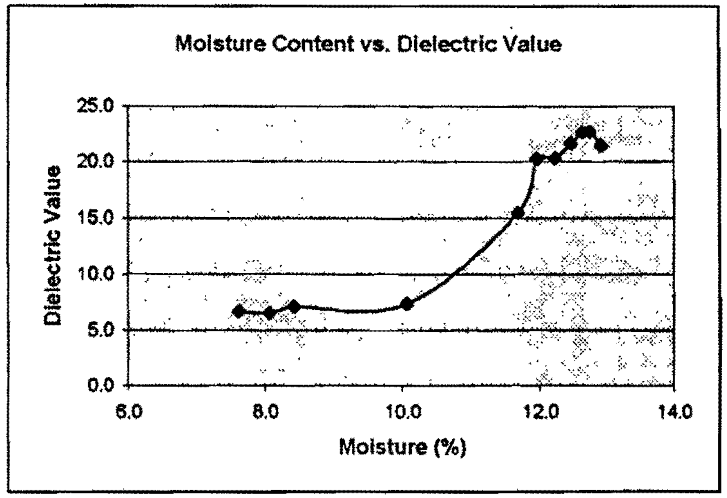


Figure A5. Typical TST Data Analysis Sheet.