Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/TX-03/0-4358-1		

4. Title and Subtitle	5. Report Date
MATERIALS, SPECIFICATIONS, AND CONSTRUCTION TECHNIQUES FOR HEAVY-DUTY FLEXIBLE BASES: LITERATURE REVIEW AND STATUS REPORT ON EXPERIMENTAL SECTIONS	October 2002 Resubmitted: July 2005
	6. Performing Organization Code
7. Author(s) Arno Hefer and Tom Scullion	8. Performing Organization Report No. Report 0-4358-1
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. 0-4358
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 Technical Research: September 2001 – August 2002 14. Sponsoring Agency Code

15. Supplementary Notes

Project performed in cooperation with the Texas Department of Transportation and the Federal Highway Administration.
Project Title: Materials, Specifications, and Construction Techniques for Heavy-Duty Flexible Bases
URL: http://tti.tamu.edu/documents/0-4358-1.pdf

16. Abstract

For the majority of the Texas highways the granular base layer is the main structural component of the pavement system. Project 0-4358 was initiated to provide TxDOT guidelines to improve its base specifications in order to better withstand the demands of future traffic loads.

This report provides the results of a literature search and a survey of the performance of experimental heavy-duty base pavements recently constructed in Texas. Base specifications and construction practices from eight U.S. DOTs and two overseas countries were compared with TxDOT current and proposed specifications. Currently, TxDOT is the only agency that does not control the amount of fines (minus 200 fraction) in its bases. Research studies have indicated that high levels of minus 200 can severely impact both moisture susceptibility and cold weather performance. The newly proposed TxDOT specifications with limits on the fines content are in line with the practices of other agencies in similar climates.

The results from three experimental sections on US-281, US-77 and FM-1810 provide several important observations about current base performance. The use of small amounts of stabilizer to upgrade marginal materials does not always guarantee a top-quality base. Better methods of selecting optimal stabilizer content are needed. Furthermore, the flexible bases with high fines content appear to have high initial field stiffnesses that decease rapidly with time.

17. Key Words	18. Distribution Statement
Flexible Base, Heavy-Duty Base, Specifications, Performance	No restrictions. This document is available to the public through NTIS: National Technical Information Service Springfield, Virginia 22161 http://www.ntis.gov

19. Security Classif.(of this report)	20. Security Classif.(of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	154	

Form DOT F 1700.7 (8-72) **Reproduction of completed page authorized**

MATERIALS, SPECIFICATIONS, AND CONSTRUCTION TECHNIQUES FOR HEAVY-DUTY FLEXIBLE BASES: LITERATURE REVIEW AND STATUS REPORT ON EXPERIMENTAL SECTIONS

by

Arno Hefer
Graduate Research Assistant
Texas Transportation Institute

and

Tom Scullion, P.E.
Research Engineer
Texas Transportation Institute

Report 0-4358-1
Project Number 0-4358
Project Title: Materials, Specifications, and Construction Techniques for Heavy-Duty Flexible Bases

Performed in cooperation with the
Texas Department of Transportation
and the
Federal Highway Administration

October 2002
Resubmitted: July 2005

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

DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Federal Highway Administration (FHWA) or the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation. The engineer in charge was Tom Scullion, P.E., (TX, # 62683).

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

ACKNOWLEDGMENTS

This project has been funded by TxDOT's pavement and materials research management committee (RMC 1) and FHWA. Caroline Herrera is the project director for this project, and her guidance and direction are greatly appreciated. The staff of several Districts assisted with the field data collection; special thanks to Richard Williammee and Andrew Wimsatt from Fort Worth, Danny Gonzalez and Jeremy Hinojosa from Corpus Christi, and Rene Castro from the Pharr District.

TABLE OF CONTENTS

Page

LIST OF FIGURES

Figure **Page**

LIST OF TABLES

Table **Page**

CHAPTER 1

INTRODUCTION AND BACKGROUND

The quality of the base layer has a significant impact on the life of a pavement, especially a flexible pavement. Table 1.1 demonstrates the contribution of the base layer to the overall health of flexible pavements (National Cooperative Research Program [NCHRP], 2000). Pavement distress, such as alligator cracking, can be minimized if the base layer is constructed properly using appropriate aggregates that maintain their integrity throughout the life of the pavement. Proper material selection and construction techniques are necessary for achieving desired performance. This matter is of even greater importance for heavy-duty flexible pavements currently being designed by TxDOT to withstand large volumes of heavy traffic.

Modern investigators agree that some of the benchmark tests used to qualify aggregates for specific uses do not demonstrate sufficient correlation between test and field performance (Powell, 1999, 2001; Langer, 2001). The selection of appropriate base materials is currently, for the most part, based on empirical tests that characterize aggregates. Given their widespread use and experience of Texas Department of Transportation (TxDOT) personnel with these tests, they may be a reasonable first attempt to ensure durable pavements. However, the results from these tests do not appear to relate to the performance of a given base layer. It is not uncommon for a district to categorize several base materials with vastly different field modulus and moisture retention properties as Class I base. Strength-related tests, such as the Texas triaxial test, are frequently not performed because of time or personnel constraints. Texas needs other simple tests that provide a clear relationship to performance for important projects.

The proper processing and compaction of the base layer during construction has a tremendous impact on the performance of a pavement. Currently, TxDOT uses the density of the base layer at the completion of the compaction to judge the quality of the layer. Achieving proper density is necessary, but other parameters are also important: stiffness, moisture retention, potential moisture variation from the top to the bottom of the layer, and the amount of time the layer is exposed before it is sealed or covered are also important. Virtually every base layer placed in the state of Texas passes the density test, but premature failures due to the other parameters mentioned are not uncommon. The thickness and number of lifts are also quite important.

Research on some of these issues has been documented in the past. As part of Study 4358, Texas Transportation Institute (TTI) researchers conducted a comprehensive independent literature review. A recent national study, NCHRP (2000), focused on identifying performance-related tests for aggregates used in base and subbase layers. An understanding of the research approach and findings under the NCHRP project is of great importance in the current study, and frequent reference to this work will be made where appropriate. The objectives of the current study are as follows:

- to compare current performance-related specifications of selected states in the U.S. and other countries for high-quality granular base, complemented by a review of background literature on the test methods and specified parameters;

- to give an overview and summarize the findings of the recent NCHRP study: "Performance-Related Tests of Aggregates for use in Unbound Pavement Layers";
- to identify important elements of the specified construction process and to give an overview of the implementation of product quality control through sampling and testing;
- to collect and summarize data from the three heavy-duty/Class 1 base experimental projects in Texas; and
- to evaluate performance trends and to correlate initial laboratory strengths with observed field response and performance.

Chapters 2 through 4 of the report are a literature review addressing the first three objectives of the study. Chapter 5 includes construction and performance-related information obtained from actual constructed experimental sections where heavy-duty flexible bases were incorporated. These sections are located on US-281 in the Pharr District, US-77 in the Corpus Christi District, and FM-1810 in the Fort Worth District.

Table 1.1. Summary of Flexible Pavement Distress and Contributing Factors (NCHRP, 2000).

Distress	Description of Distress	Base Failure Manifestation	Contributing Factors
Fatigue Cracking	Fatigue cracking first appears as fine, longitudinal hairline cracks running parallel to one another in the wheel path and in the direction of traffic. As the distress progresses the crack will interconnect, forming many-sided, sharp angled pieces (resulting in the commonly termed alligator cracking); eventually cracks become wider and in later stages some spalling occurs with loose pieces prevalent. Fatigue cracking occurs only in areas subjected to repeated traffic loading.	Lack of base stiffness causes high deflection/strain in the asphalt concrete surface under repeated wheel loads, resulting in fatigue cracking of the asphalt concrete surface. Alligator cracking only occurs in areas where repeated wheel loads are applied. High flexibility in the base allows excessive bending strains in the asphalt concrete surface. The same result can also be caused by inadequate thickness of base. Changes in base properties with time can render the base inadequate to support loads.	Low modulus Improper gradation High fines content High moisture level Lack of adequate particle angularity and surface texture Degradation under repeated loads and freeze-thaw cycling
Rutting/ Corrugations	Rutting appears as long surface depressions in the wheel path and may not be noticeable except during and following rains. Pavement uplift may occur along the sides of the rut. Rutting results from permanent deformation in one or more pavement layers or subgrade, usually caused by consolidation and/or lateral movement of the materials due to load.	Inadequate shear strength in the base allows lateral displacement of particles with applications of wheel loads and results in a decrease in the base layer thickness in the wheel path. Rutting may also result from consolidation of the base due to inadequate initial density. Changes in base properties with time due to poor durability or frost effects can result in rutting.	Low shear strength Low density of base material Improper gradation High fines content High moisture level Lack of adequate particle angularity and surface texture Degradation under repeated loads and freeze-thaw cycling
Depressions	Depressions are localized low areas in the pavement surface caused by settlement of the foundation soil or consolidation in the subgrade or base/subbase layers due to improper compaction. Depressions can contribute to roughness and can cause hydroplaning when filled with water.	Inadequate initial compaction or non-uniform material conditions results in additional reduction in volume with load applications. Changes in material conditions due to poor durability or frost effects may also result in localized densification with eventual fatigue failure.	Low density of base material
Frost Heave	Frost heave appears as an upward bulge in the pavement surface caused and may be accompanied by surface cracking, including alligator cracking with resulting potholes. Freezing of underlying layers resulting in an increase volume of material cause the upheaval. An advanced stage of distortion mode of distress resulting from differential heave is surface cracking with random orientation and spacing.	Ice lenses are created within the base/ subbase during freezing temperatures, particularly when freezing occurs slowly, as moisture is pulled from below by capillary action. During spring thaw large quantities of water are released from the frozen zone, which can include all unbound materials.	Freezing temperatures Source of water Permeability of material high enough to allow free moisture movement to the freezing zone

CHAPTER 2

CURRENT PERFORMANCE RELATED SPECIFICATIONS FOR HIGH QUALITY GRANULAR BASES

2.1 INTRODUCTION

In order to review current national specifications for high-performance granular bases, researchers focused on states with climatic conditions similar to Texas. On an international level, South Africa and Australia have been identified as countries that have similar climates and are known for building pavement structures similar to those in Texas. In addition a number of other U.S. state departments of transportation (DOTs) were randomly selected to complement the search for other innovative concepts in practices related to high-quality aggregate base layers.

Figure 2.1, Texas climatic regions, was obtained from the well-known presentation of nine climatic regions in the U.S., introduced in the 1970s (NCHRP, 1974; AASHTO, 1993). These zones were based on both the Thorntwaite Index (TI) and number of freeze/thaw cycles. The TI is an indication of moisture balance between rainfall, evaporation, and plant moisture demands. A positive TI indicates a wet climate with free water available. The following table provides a summary of the climatic regions relevant to Texas and their descriptions, as well as other selected states represented by these regions. Seven of the nine regions are found in Texas to a larger and lesser extent, ranked in Table 2.1 from 1 to 7, respectively.

Table 2.1. Selected States Representative of the Climatic Regions Found in Texas.

Texas Climatic Region		Description	Selected States of Similar/Partially Similar Climatic Regions
Extent Rank No.	Classification		
1	Dry-Freeze-Thaw (III-B)	- Freeze-thaw cycles in surface and base. - Very little moisture in the pavement structure during the year.	**New Mexico** Arizona California (southeast) Nevada Oklahoma (southwest)
2	Dry-No Freeze (III-C)	- Low temperatures are not a problem. - Very little moisture in the pavement structure during the year.	**Arizona (south)** California (southeast & central)
3	Intermediate-No-Freeze (II-C)	- Low temperatures are not a problem. - Seasonal variability of moisture in the pavement structure.	California
4	Wet-No Freeze (I-C)	- Low temperatures are not a problem. - High potential for moisture presence in the entire pavement structure throughout the year.	**Florida** California (north coast)
5	Intermediate-Freeze-Thaw (II-B)	- Freeze-thaw cycles in surface and base. - Seasonal variability of moisture in the pavement structure.	**Oklahoma (southeast)** Arizona (central)
6	Wet-Freeze-Thaw (I-B)	- Freeze-thaw cycles in surface and base. - High potential for moisture presence in the entire pavement structure throughout the year.	**Arkansas** Oklahoma (east)
7	Dry-Freeze (III-A)	- Severe winter with high potential for frost penetration of appreciable depths. - Very little moisture in the pavement structure during the year.	**Nevada** Idaho (south)

Researchers attempted to select at least one other state in the US that represents a climatic region in total, or at least to a large extent. These states are highlighted with bold text in Table 2.1. Figure 2.1 shows that Dry-Freeze Thaw is represented by approximately 40 percent of the state, while Dry and Intermediate-No Freeze are roughly 30 percent and 20 percent, respectively. Wet/No Freeze and Intermediate-Freeze/Thaw between 5 percent and 10 percent while the remaining regions are estimated to cover to be less than 5 percent. Despite the fact that most states are exposed to more than one climatic condition, it is uncommon for states to have different specifications that relate to these regions.

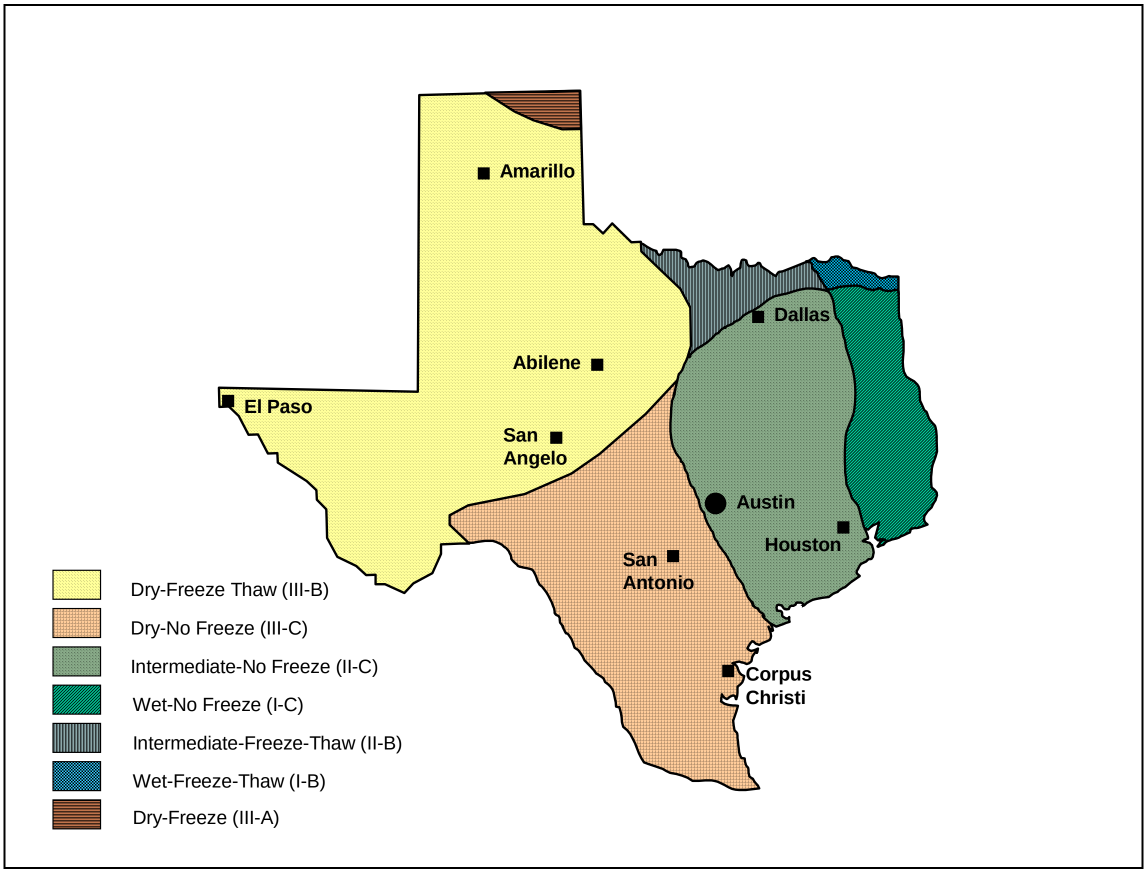


Figure 2.1. Conceptual Presentation of Climatic Regions in Texas.

2.2 COMPARISON OF SPECIFICATIONS USED BY DIFFERENT ROAD AGENCIES

Table 2.2 summarizes specifications for high-quality granular bases currently used by the selected road agencies. Six broad specification categories control the quality of the base layer, i.e. strength, gradation, fines, degradation and soundness, characteristics of the crushed particles, moisture susceptibility, and compaction. Following sections introduce different tests and discuss the significance of the test's parameters and specified values under the categories mentioned above based on a review of the literature. The focus of this project is on aggregate obtained from naturally occurring sources.

The type of materials required for the base differ greatly among the states and depend on the resources available. Some states have clearly defined classes based on geological group, while other classifications are based on the state of weathering, such as crushed solid rock versus natural gravel. In a number of cases the description of the aggregate to be used is very wide or not defined at all and depends only on the specified test parameters. Aggregates are commonly specified to consist of clean, hard, and durable particles, free from frozen lumps, deleterious matter, or harmful adherent coatings. Specifications on foreign and deleterious matter are in most cases region specific and are therefore omitted from this comparison.

Table 2.2 contains both the current TxDOT flexible base specification (1993) and the newly proposed draft specification (proposed), together with specifications from eight other DOTs in the U.S. and four international highway agencies with climates similar to Texas. While most of the specifications are similar in many respects, there are several key differences:

- Texas is the only agency that does not regulate the amount of minus 200 material. The maximum permitted in the other agencies is 12 percent, with most having an upper limit of 10 percent.
- The Texas allowable PI requirement of 10 appears high; most other DOTs specify 8 or below. Only one other agency (Oklahoma) has similar plasticity index (PI) specifications.
- Most of the agencies have requirements for crushed faces.

Table 2.2. Comparison of Specifications for High-Quality Granular Base Courses.

Road Authority	Texas (1993)	Texas (Draft 2003)	Arkansas (1996)	Arizona (1996)	California (1999)	Florida (2000)	Idaho (2001)
Designation	Flexible Base	Flexible Base	Aggregate Base	Aggregate Base	Aggregate Base	Graded Aggregate Base	Aggregate Base
Class or grade representing high performance specification	Type A, Grade 1: Crushed stone from a single source	Type A, Grade 1 to 3: Crushed stone or crushed concrete produced from oversized aggregate, from single naturally occuring source.	Classes 7 & 8: Crushed natural solid rock	Class 1 & 2: Stone, gravel, or other approved inert material	Class 2 or 3: May include up to 50% reclaimed material	Group 1: Limestone, marble, or dolomite Group2: Granite, gneiss, or quartzite	Aggregate base may include up to 15% crushed glass
Strength	**Triaxial Class 1** Min. compressive strength, psi: 45 at 0 psi lateral pressure and 175 at 15 psi lateral pressure	**Texas Triaxial Test**: Angle of internal friction (ϕ), degrees Min. 50 Cohesion (c), psi: 10 > C > 5	-	-	**R-value** Min. 78	**Limerock Bearing Ratio** Min. 100	**R-value** Min. 75
Maximum aggregate sizes	1-3/4"	2"	3/2", 1"	1", 3/4"	2", 1"	2"	3/8", 1/2", 3/4", 1", 2"

Gradation		Texas (1993)	Texas (Draft 2003)	Arkansas (1996)		Arizona (1996)		California (1999)		Florida (2000)	Idaho (2001)	
		Type A, Grade 1	Type A, Grade 1	Class 7	Class 8	Class 1	Class 2	37.5 mm	19 mm	Group 1 and 2	3/4"	1" (A)
English	**Metric**	retained (passing)	retained (passing)									
3"	**75 mm**											
2"	**50 mm**		0 (100)					100		100		
1-3/4"	**45 mm**	0 (100)										
3/2"	**37.5 mm**		0 - 10 (90 - 100)	100			100	90 - 100		95 - 100		100
1"	**25 mm**				100	100	90 - 100		100		100	90 - 100
3/4"	**19 mm**			50 - 90	65 - 100	90 - 100		50 - 85	90 - 100	65 - 90	90 - 100	
7/8"	**22 mm**	10 - 35 (65 - 90)	15 - 40 (60 - 85)									
1/2"	**12.5 mm**											60 - 80
1/4"	**6.70 mm**											
3/8"	**9.5 mm**	30 - 50 (50 - 70)	35 - 55 (45 - 65)							45 - 75		
No. 4	**4.75 mm**	45 - 65 (35 - 55)	60 - 70 (30 - 40)	25 - 55	25 - 55			25 - 45	35 - 60	35 - 60	40 - 65	35 - 60
No. 8	**2.36 mm**					35 - 55	35 - 55				30 - 50	25 - 50
No. 10	**2.00 mm**									25 - 45		
No. 16	**1.18 mm**											
No. 30	**0.600 mm**							10 - 25	5 - 35			10 - 30
No. 40	**0.425 mm**	70 - 80 (20 - 30)	70 - 85 (15 - 25)	10 - 30	10 - 30							
No. 50	**0.300 mm**									5 - 25		
No. 200	**0.075 mm**		90 - 100 (0 - 10)	3 - 10	3 - 10	0 - 8	0 - 8	2 - 9	2 - 9	0 - 10	3 - 9	2 - 9

Road Authority	Texas (1993)	Texas (Draft 2003)	Arkansas (1996)	Arizona (1996)	California (1999)	Florida (2000)	Idaho (2001)
Fines	Max. PI 10 Max. LL 35 Max. BLS 2	Max. PI 8 Max. LL 25	Max. PI 6 Max. LL 25 No. 200 < 2/3 (No. 40)	Max. PI 3	Sand Equivalent Min. 22	**Group 1:** Max. PI 4 (NP) Max. LL 25 (35) ( ) for Limerock No. 200 < 2/3 No. 40 **Group 2:** Sand Equivalent Min. 28	Sand Equivalent Min. (No. 200 > 5) 30 Min. (No. 200 < 5) N/A
Degradation and soundness	**Wet Ball Mill** Max. loss 40% **Increase in passing No. 40** Max. 20%	**Wet Ball Mill** Max.loss 30% **Increase in passing No. 40** Max. 10%	**Los Angeles Test** Max. loss 45%	**Los Angeles Test** **At 100 revolutions** Max. loss 9% **At 500 revolutions** Max. loss 40%	**Durability Index** Min. 35	**Loss Angeles Test** **Group 1:** Max. loss 45% **Group 2:** Max. loss 65% **Soundness, Sodium Sulfate**. Loss 15%	**Los Angeles Test** Max. Loss 30%
Moisture susceptibility	-	**Dielectric Value** Max. 8	-	-	-	-	-
Shape, angularity, and surface texture	-	-	Crushed material retained on No. 4 Min. 15%	Material on No. 4 Min 30% with at least one rough and angular surface.	-	-	Material on No. 4: Min. 60% with at least one fractured face.
Compaction	100% Tex-113-E	100% Tex-113-E	98% AASHTO T-180, Method D	100% Arizona Test Methods 225, 226, 227	95% California Test 216	100% AASHTO T-180	95% of Idaho T-74 or AASHTO T-180

Table 2.2. Comparison of Specifications for High-Performance Granular Base Courses. (Continued)

Road Authority	Nevada (2001)	New Mexico (2000)	Oklahoma (1999)	South Africa (1998)	Queensland (1999)	New South Wales (1997) (Based on Particle Size Distribution)	New South Wales (1997) (Based on Shear Strength)
Designation	Aggregate Base	Aggregate Base	Aggregate Base	Crushed Stone Base	Unbound Base	Unbound Base	Unbound Base
Class or grade representing high performance specification	Type 1 Class A & B, Type 2 Class A & B, Type 1 Class B normally used under hot mix asphalt surfaces	Type I & II: Crushed stone, crushed or screened gravel, caliche, sand, reclaimed asphalt pave. (RAP), or combination	Type A, B, & C: Coarse part of gravel, stone, mine chats, disintegrated granite, crushed concrete; fine aggregate of sand, stone dust, or other finely divided mineral matter	G1: Crushed stone from sound rock; all fractions from the same parent rock	Type 1 Base, unbound pavement: Subtype 1.1 & 1.2 Crushed stone	Unbound Material: (DGB20) 20 mm nominally seized densely graded base. Traffic Category 1: >107 MESALs	Unbound Material: (DGB20) 20 mm nominally seized densely graded base. Traffic Category 1: >107 MESALs
Strength	**R-value** Min. 70	-	-	-	-	Strength inferred from compliance with specified particle size distribution	**Modified Texas Triaxial No.** Max. 2
Maximum Aggregate sizes	3/2", 1"	1"	2", 3/2"	37.5 mm (3/2")	26.5 mm (1.05")	20 mm (0.79")	37.5 mm (3/2")

Gradation		Nevada Class 1	Nevada Class 2	New Mexico Type I (target values)	New Mexico Type II (target values)	Oklahoma Type A	Oklahoma Type C	South Africa Type G1	Queensland Type 1	NSW DGB20	NSW Unbound Base
English	**Metric**										
3"	**75 mm**										
2"	**50 mm**						100				
1-3/4"	**45 mm**										
3/2"	**37.5 mm**	100				100	90 - 100	100	100		100
1"	**25 mm**	80 - 100	100	100	100		80 - 100	84 - 94 (26.5 mm)	85 - 100 (26.5 mm)	100 (26.5 mm)	
3/4"	**19mm**		90 - 100	90	92	40 - 100		71 - 84	75 - 100	95 - 100	
7/8"	**22mm**										
1/2"	**12.5mm**						60 - 80	59 - 75 (13.2 mm)		70 - 90 (13.2 mm)	
1/4"	**6.70mm**									50 - 70	30 - 55
3/8"	**9.5mm**					30 - 75			58 - 80		
No. 4	**4.75 mm**	30 - 65	35 - 65	45	55	25 - 60	40 - 60	36 - 53	45 - 62		
No. 8	**2.36 mm**								33 - 45	35 - 55	
No.10	**2.00mm**			32	37	20 - 43	25 - 45	23 - 40		**-0.425/-2.36**	
No. 16	**1.18 mm**	15 - 40	15 - 40							35 - 55%	
No. 30	**0.600 mm**									**-0.075/-0.425**	
No. 40	**0.425 mm**					8 - 26	15 - 30	11 - 24	14 - 22	35 - 55%	
No .50	**0.300 mm**									**-0.135/-0.075**	
No. 200	**0.075 mm**	2 - 12	2 - 10	6	8	4 - 12	0 - 5	4 - 12	5 - 10	35 - 60%	

Road Authority	Nevada (2001)	New Mexico (2000)	Oklahoma (1999)	South Africa (1998)	Queensland (1999)	New South Wales (1997) (Based on Particle Size Distribution)	New South Wales (1997) (Based on Shear Strength)
Fines	**No. 200 / Max. PI** 0.1 - 3.0 / 15 3.1 - 4.0 / 12 4.1 - 5.0 / 9 5.1 - 8.0 / 6 8.1 - 11.0 / 4 11.1 - 15.0 / 3 Max. LL 35	Max. PI 6 Max. LL 25	Max. PI 10 Max. LL 35 No. 200 < 2/3 (No. 40)	**- 0.425 mm fraction** Max. PI 5 Max. LL 25 Max. LS 2% **- 0.075 mm fraction** Max. PI 12	**Subtype 1.1** Max. PI 4 Max. LL 25 Max. LS 2.5 **Subtype 1.2** Max. PI 6 Max. LL 28 Max. LS 3	**LL (if non-plastic)** Max. 20 **PL (if plastic)** Max. 20 PI Max. 6	**LL (if non-plastic)** Max. 20 **PL (if plastic)** Max. 20 PI Max. 6
Degradation and Soundness	**Los Angeles Test At 500 revolutions** Max. loss 45%	**Aggregate Index (AI)** Max. 35 AI = f(LA, SL, A): LA: Los Angeles Loss SL: Soundness Loss A: Absorption	**Los Angeles Test** Max. loss 45% **Durability Index** Min. 40	**10% Fines Value** Min. Dry 110-200 kN Min.Wet 125 kN Min. Dry/Wet 75% **Agg. Crush'g Value** Max. 21% - 29%	**Subtype 1.1** **10% Fines Value** Min.Wet 130-150 kN Wet/Dry Variation Max. 30% - 40% **Degradation Factor** Min. 40 - 50	**If PI $\leq$ 1:Dry Comp. Strength on -19mm** Min. 1.7 MPa **Agg. Wet Strength** Min. 70 kN **Wet/Dry Variation** Max. 35%	**Dry Compressive Strength on -19mm** Min. 1.7 MPa **Agg. Wet Strength** Min. 70 kN **Wet/Dry Variation** Max. 35%
Moisture Susceptibility	-	-	-	-	-	-	-
Shape, angularity and surface texture	**Crushed Particles** Class 1B: Min. 15% fractured faces	**Crushed Particles** Material on No. 4: Min. 50% at least two fractured faces.	**Crushed Particles** **Type A,B:** Min. 40% on No. 4, one or more fractured faces **Type C:** 100% on No. 4 with two or more fractured faces	**Crushed Particles** All faces shall be fractured faces. **Flakiness Index** Max. 35% on -26.5 +13.2 mm fraction	**Crushed Particles** Minimum of 70% crushed particles. **Flakiness Index** Max. 35% on coarse fraction.	**Particle Shape by Proportional Caliper (% mishapen)** Max. 35	-
Compaction	95% T-101	96% AASHTO T-180, Method D	100% AASHTO T-99, Method C or D	88% ARD (Apparent Relative Density)	102% RDD (Relative Dry Density)	102% RTA-111	102% RTA-111

Notes: PI = Plasticity Index
LL = Liquid Limit
BLS = Bar Linear Shrinkage

2.3 STRENGTH OF AGGREGATE MASS

Strength of the aggregate mass for crushed stone is often not specified, and in such cases it is inferred that compliance with other properties, normally gradation, will be sufficient to ensure adequate load-bearing capacity. Florida uses the limerock bearing ratio (LBR), a modification of California bearing ratio (CBR), and California, Idaho and Nevada uses the R-value (resistance value). Of the U.S. states selected for the study, Texas is the only one that uses a triaxial test based specification; however, New South Wales, Australia uses a modified Texas triaxial number.

2.3.1 Resistance (R-Value)

The R-value, or resistance value of a soil, is determined by the Hveem Stabilometer (American Association of State Highway and Transportation Officials [AASHTO] T-190, American Society for Testing Materials [ASTM] D2844). This is a closed-system triaxial test developed by the California Division of Highways and basically measures the internal friction of the material or its ability to resist lateral deformation when acted upon by a vertical load. The compacted specimen, 4 in. in diameter and 2.5 inches in height, is encased in a rubber membrane and tested in a metal chamber at full moisture saturation. A vertical pressure of 160 psi is applied to the sample while lateral pressures of the fluid between the membrane and metal chamber are monitored. The R-value ranges from 0 (water) to 100 (steel) (Huang, 1994). Soils and aggregates used in road building typically range from less than 5 to 85. The minimum value specified by California and Nevada for bases is 78, while Idaho specifies a minimum value of 75.

2.3.2 Limerock Bearing Ratio (LBR)

Florida has used LBR test (FM5-515) since the 1960s for determining the strength of subgrade materials, and for base course materials since the mid-1970s. The LBR test is a modification of the widely used CBR test and likewise measures the load required to cause a standard circular plunger to penetrate a standard specimen at a specified rate. The pressure required to penetrate the plunger to a depth of 0.1 in. (2.54 mm) into the material under consideration is expressed as a percentage of the pressure required by the same plunger to penetrate the standard sample to the same depth. The standard pressure is based on a typical limerock found in Florida that has a standard pressure lower than the CBR, which is based on crushed stone found in California. The LBR test was essentially adapted to represent field densities and moisture conditions of pavement materials typically found in Florida. Because of these modifications, the LBR test would seem to give less conservative results if compared directly with the CBR. The CBR can be related to the LBR by applying a factor of 1.25 (Ping and Ge, 1996). The LBR, like the CBR, is an empirical-based test. The literature study also revealed that the CBR test is not commonly specified for high-quality crushed stone base material but often for base layers of a lower-quality granular material, for granular material used in the lower pavement structure, and subgrade soils. NCHRP (2000) researchers abandoned the CBR test during the second stage of laboratory testing because of poor correlation with performance obtained during the first stage.

2.3.3 Texas Triaxial Test

The triaxial test system is used widely in research applications to determine both strength and stiffness properties of granular materials. This concept can approximate actual conditions of a specimen under loading resulting from the three principal known and controlled stresses as well as its repeated load capability. In the context of this specification the static triaxial test measures shear strength properties of the base material, i.e., the cohesion (c) and angle of internal friction (ϕ), that reflect resistance to permanent deformation.

The Texas triaxial test (Tex-117-E) differs from the standard triaxial compression test in that the confining pressure on the specimen is induced through compressed air between a metal triaxial cell and its thick rubber lining which is in contact with the specimen. The failure envelope obtained from the 'Mohr's Diagram' is transferred onto a standard material classification chart shown in Figure 2.2. The current specification for a Class 1 base is the ultimate compressive strength at 0 and 15 psi lateral pressure, which is 45 and 175 psi, respectively. The Draft 2003 specification is based on the shear strength parameters c and ϕ, measured in a conventional traixial cell as described in Tex 143E, the required minimum ϕ, of 50° and c between 5 and 10 psi.

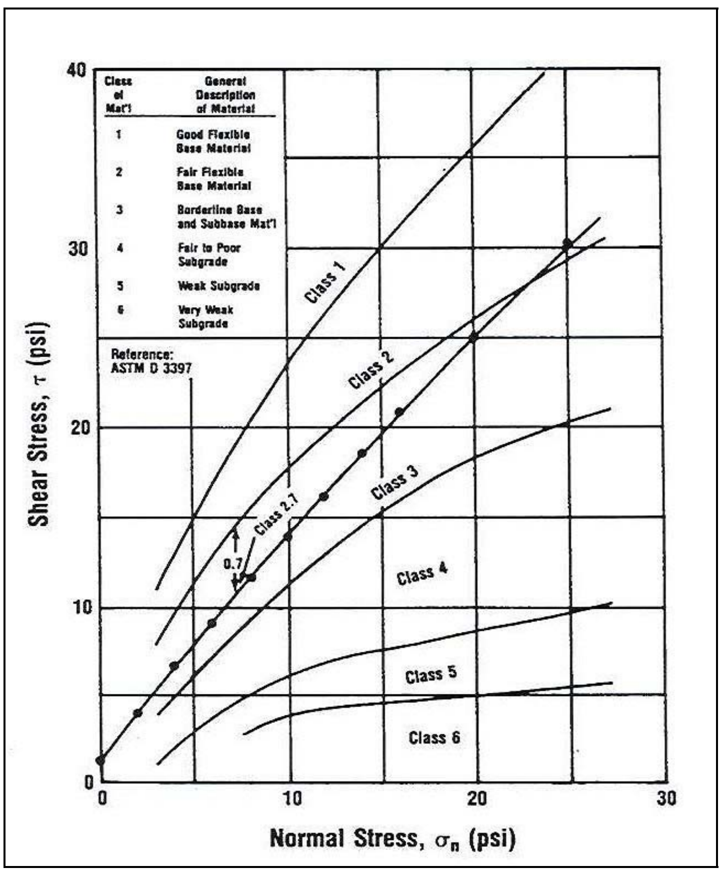


Figure 2.2. Material Classification Chart for Texas Triaxial Test Results (NSA, 1991).

Static triaxial tests were conducted on a variety of unbound base materials during the NCHRP (2000) study in different stages of the materials testing program. Dry and wet strengths were obtained at confining pressures of 5, 10, and 15 psi. Specimens were compacted at optimum moisture content at 100 percent of AASHTO-T180. Dry values represented optimum moisture content, and wet specimens represented 90 percent of saturation. The following table summarizes the results obtained during Stage I and Stage II testing. Only the results of the specimens that represented high-performance base materials are presented here. It is difficult directly compare these strength results with the current Texas Triaxial criteria described earlier. The moisture conditioning and test configuration are somewhat different. However, at 15 psi confining the specified strength (6) would be 175 psi. In the dry condition only the Gabbro and Basalt materials would meet this requirement. This could indicate that the current Texas criteria are conservative. The NCHRP report points out that the standing time of the triaxial samples after soaking, one hour, was too long. This is confirmed by the insignificant difference between dry and wet results. This effect could therefore not be inferred from the test data.

Table 2.3. Static Triaxial Test Results from the NCHRP (2000) Study.

ID	Rock Type	Dry				Wet			
		c (psi)	ϕ°	σ_d 5 psi	σ_d 15 psi	c (psi)	ϕ°	σ_d 5 psi	σ_d 15 psi
I-1	Limestone	5	53	76	155	-	-	-	-
I-2	Limestone	-	-	-	-	0	51	38	109
II-7	Sandstone	6	48	63	116	7	39	51	85
II-8	Granite	11	48	96	154	8	53	94	130
II-12	Gabbro	10	53	98	176	7	55	88	151
II-14	Dolomite	6	52	84	152	11	50	92	143
II-16	Basalt	12	50	102	169	0	59	102	169
Average		8	51	87	154	5	51	78	131
Std. Deviation		2	2	15	20	4	6	26	30

Notes:
ID Specimen identification number used in NCHRP study
σ_d Deviator stress from static triaxial test
c Cohesion
ϕ Angle of internal friction

Gray (1962) selected and used the Texas triaxial test in a comprehensive study on the influence of various properties of crushed stone on the strength and deformation characteristics of these materials. The results obtained are fundamental to the objectives of the current study and will be reported in the relevant sections that follow.

Several studies (e.g., O'Malley and Wright, 1987) have shown that the Texas triaxial classification may not be directly related to the long-term performance of the base. These researchers pointed out problems with the test procedure, namely the additional confining provided by the thick latex membrane and the limitations on the amount of lateral deformation permitted in the test configuration. Recognizing this limitation, CSTM is gearing towards a new triaxial test procedure, provisionally Tex-Method 143 (Fernando et al., 2001). This procedure will use a "true" triaxial configuration, and thin latex membranes will replace the thicker membranes. The material strength will also be related to conventional triaxial results of cohesion and angle of internal friction. However, research is required to ensure that the results from the new procedure better relate to the long-term performance of the base. The approach followed by the NCHRP researchers, described in Chapter 3 of this report, found that the deviator stress obtained at a confining pressure of 5 psi for the standard triaxial test and the repeated load triaxial test result at a confining pressure of 15 psi were the parameters that correlated the best with performance ratings of their study materials.

The NCHRP recommendation of using confining pressures of 5 and 15 psi is significant when compared to the current Texas use of 0 and 15 psi. Some researchers have commented that achieving a strength of 45 psi with 0 psi confining is difficult unless the materials have some cohesion. They have observed that several excellent performing materials fail the 0 psi strength requirement. Many districts have commented that in their efforts to upgrade a material to Class 1 with the addition of low levels of stabilizer they often waive the 0 psi requirement. Indeed it has been commented that the only way to make some of the materials pass the 0 psi requirement is to add some “clay” to the base, clearly an undesirable practice. If revisions are to be made to the current Item 247, it is recommended that TxDOT consider replacing the 0 psi requirement with a 5 psi requirement. Based on the extensive testing conducted by Scrivner and Moore (1967), a Class 1 material will require a compressive strength of more than 105 psi at 5 psi confining, and a Class 2 material will require a compressive strength of more than 70 psi.

Based on our interviews with TxDOT personnel, many districts avoid Texas triaxial tests as a part of specifications because of a lack of time and personnel necessary to conduct them. Under Item 247, triaxial tests can be removed from the specifications by specifying a Grade 3 through Grade 6 base material. If districts avoid the Texas triaxial tests, it is probable that they will also avoid the new triaxial protocol (Tex-Method 143) will also be avoided. Therefore, it is imperative that new test methods that are easier to perform and that are better correlated to the performance of the base are identified and incorporated in the material selection process.

2.4 GRADATION

Gradation is a measure of the relative size distribution of particles making up the aggregate mass. Dense graded materials have traditionally been considered to provide high shear strength and stiffness, although there is a trend toward more free-draining open-graded materials that have greater resistance to excessive moisture and frost action. Among other properties, the amount passing the No. 200 sieve is almost universally specified to limit frost susceptibility and to ensure sufficient permeability, preventing development of excess pore pressures. The influence of the aforementioned gradation characteristic can have a significant effect on the performance of the base (NCHRP, 2000).

Some states have more than one gradation specification, while others, such as Idaho, have a range of gradations based on maximum aggregate size. The motivation for including different gradations based on maximum aggregate size has primarily been practical, although it is known that larger stone contributes to improved load-bearing capacity. Since the strength and durability of flexible bases depend on the grain-to-grain contact, it may be feasible to design the gradation of the base material to maximize interlocking between the aggregates. This can be achieved by developing gradations that better follow the 0.45 power Fuller curve, similar to the procedure followed in the mix design for hot-mix asphalt. In that manner, one can optimize the strength and stiffness of the material and, as such, minimize the potential for permanent deformation and cracking due to loss of strength in the layer. This may increase the initial cost of the construction, but it will certainly improve the performance of the pavement. The finest sieve used in the current Texas specifications is a No. 40 sieve. Texas has virtually no specifications for controlling the amount of fine-grained sand and silts or clays. Under Grade 1 specifications, anywhere from 15 percent to 30 percent by weight of the base materials can be fine sand or silt and clay. The proposed TxDOT specification provides for 0 to 10 percent passing the No. 200 sieve, which from Table 2.2, is in line with most of the other agencies' specifications.

2.4.1 Maximum Aggregate Size

In a comprehensive study on the effect of various material properties on the strength and deformation of dense graded base course aggregate, Gray (1962) included the effect of maximum size aggregate and of the fines passing the No. 200 sieve. Gray used the Texas triaxial test in this study. Based on experience, good quality crushed stone was associated with a continuous grading, which could be described by Talbot's Equation, i.e., $P = (d/D)^n$, in which P is the percentage passing any given sieve *d*, and *D* is the maximum size. The value for n was usually between 1/3 and 1/2, but approached 1/3 for practical construction. Gray tested crushed stone with different maximum sizes complying with Talbot's Equation for n of 1/3 and 1/2. Results are presented in Figures 2.3 to 2.6.

Study results indicate that:

- The Mohr failure envelope intercepts the shear strength axis on different points indicating that "apparent" cohesion exists in well-graded, moist mixes. Also, cohesion increased with larger maximum sized aggregate.
- The slightly greater slope of the curve, angle of internal friction, with increase in maximum size is intuitively expected due to better aggregate interlock and therefore resistance to shear provided by larger maximum aggregate size.
- The ultimate strength also increases as the maximum aggregate size increases.

The significance of these findings was proven by an increase in percent strain with an increase in maximum aggregate size for a given normal load and lateral pressure. Gradations with n = 1/2 with a low mortar content essentially produced stone-to-stone contact with higher consequent resistance to deformation. However, the results showed that gradations with n = 1/3 could sustain higher normal pressure at any given lateral pressure (Figure 2.5). The study results recommended that such a grading would better serve bases with thin surfacings.

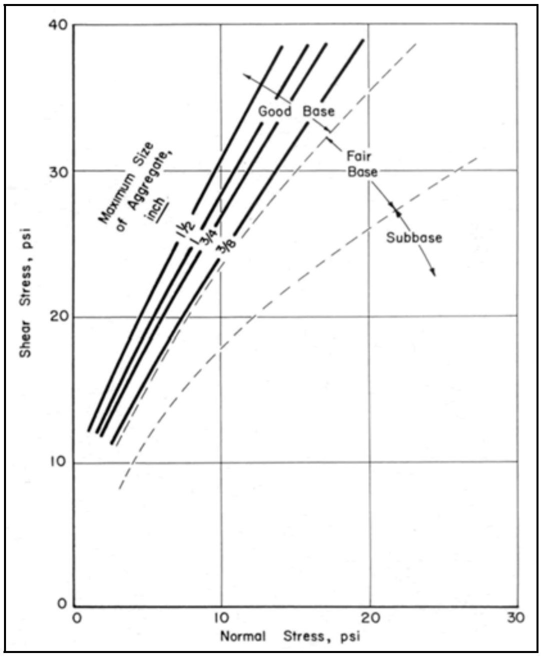


Figure 2.3. Effect of Maximum Size Aggregate on Shear Strength for Gradings with n = 1/3.

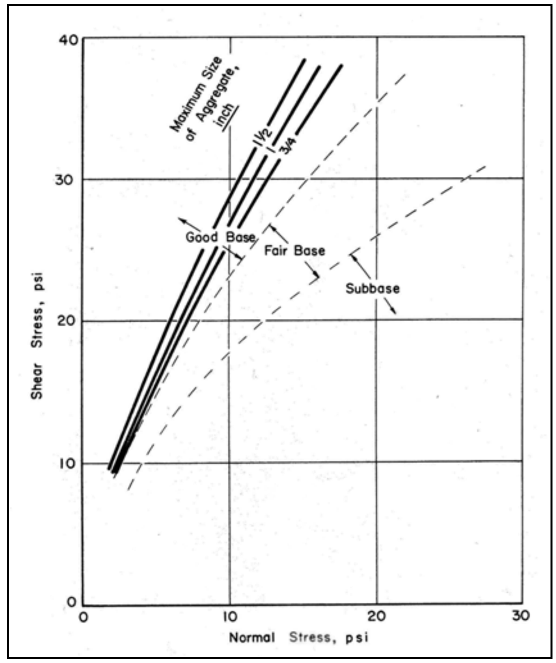


Figure 2.4. Effect of Maximum Size Aggregate on Shear Strength for Gradings with n = 1/2.

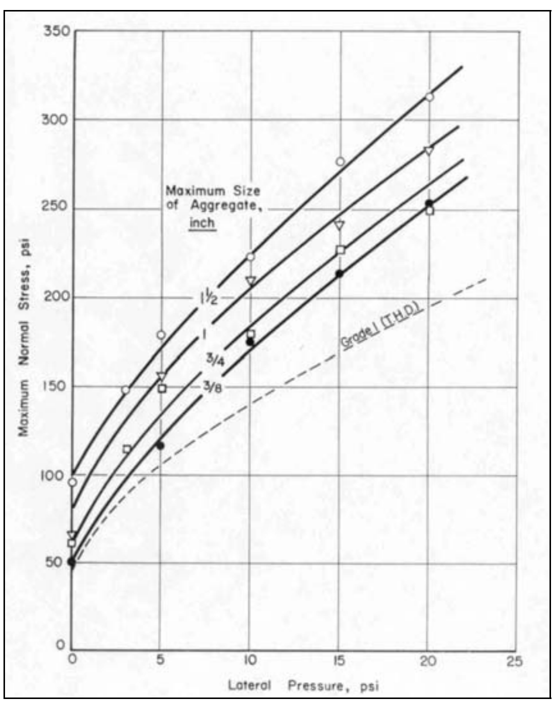


Figure 2.5. Relationship of Principal Stresses for Gradings with n = 1/3.

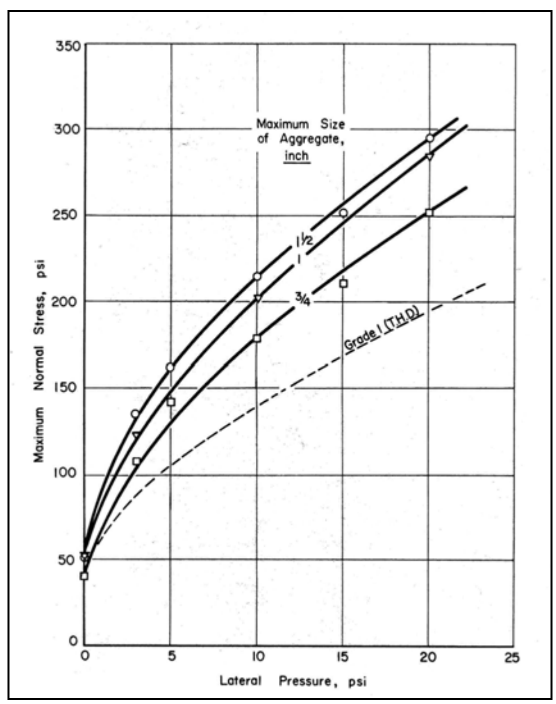


Figure 2.6. Relationship of Principal Stresses for Gradings with n = 1/2.

2.4.2 Fines Passing the No. 200 Sieve

Gray (1962) also investigated the influence of the addition of fines (minus No. 200) on the strength and density achieved with a dense graded crushed limestone base material with .75 in. maximum size. The non-plastic fines passing the No. 40 sieve were also produced from limestone. The compaction effort was held constant, and the fines ranged from 2 to 20 percent. The data plotted in a classification chart format, Figure 2.7, shows the ranking of the gradations based on shear strength. Although all conform to good base classification criteria, Figure 2.8 indicates that there is a significant difference in the load carrying capacity of the aggregate depending on the percentage passing the No. 200 sieve. Figure 2.9 illustrates a significant trend of maximum strength obtained at an optimum fines content of 9 percent when tested at the different levels of lateral pressure. Gray found that the optimum amount of minus No. 200 material decreases as the maximum size is increased, i.e. 8 percent for 1 in., 7 percent for 1.5 in., and 6 percent for 2 in. maximum aggregate size.

The curves further show that high density would be difficult to achieve with a low percentage of fines (less than 4 percent) but may be readily achieved with a wide range of fines up to 20 percent. However, an excessive amount of fines, greater than 9 percent, reduces the maximum bearing capacity of the crushed stone. In addition, excessive fines produces frost-susceptible bases.

The influence of the fraction passing the No. 200 sieve was also studied by other researchers. Barksdale and Itani (1989) determined the resilient and permanent deformation characteristics of river gravel, granitic gneiss, shale, limestone, and quartzite aggregates using a cyclic load triaxial test. They illustrated the tendency of aggregates to undergo increased permanent deformation with the addition of more fines. The resilient modulus of the granitic gneiss was determined at different fines contents. As fines were increased from 0 to 10 percent, the resilient modulus decreased by about 60 percent for the samples tested (Jorenby and Hicks [1986]).

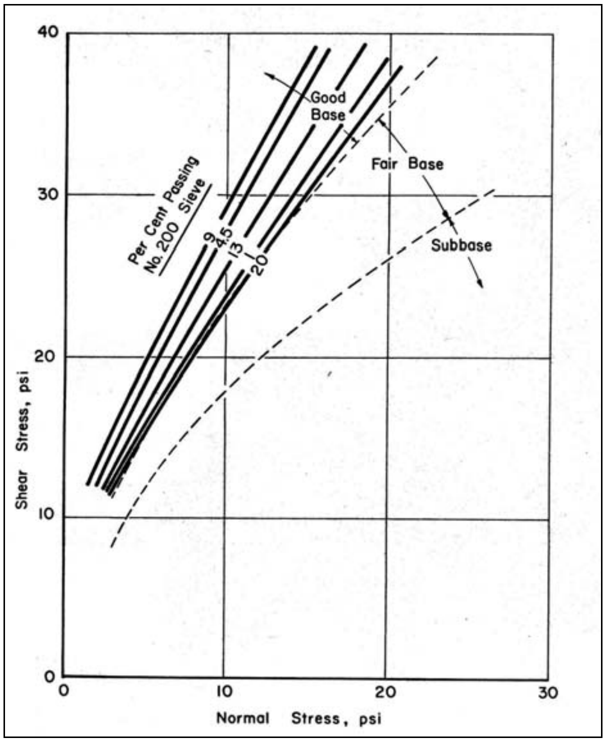


Figure 2.7. Effect of Amount Passing No. 200 Sieve on Shear Strength, where 9% is the Optimal Fines Content.

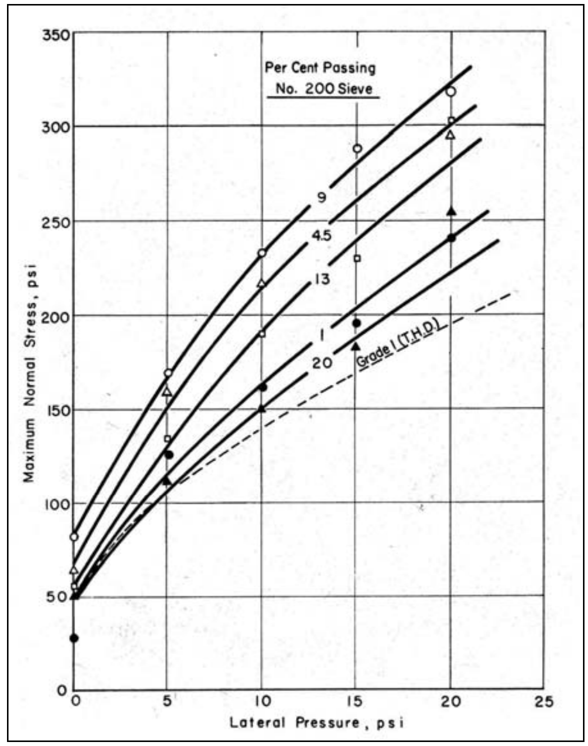


Figure 2.8. Relationship of Principal Stresses for Materials with Different Amounts Passing No. 200 Sieve, where 9% is Optimal Fines Content.

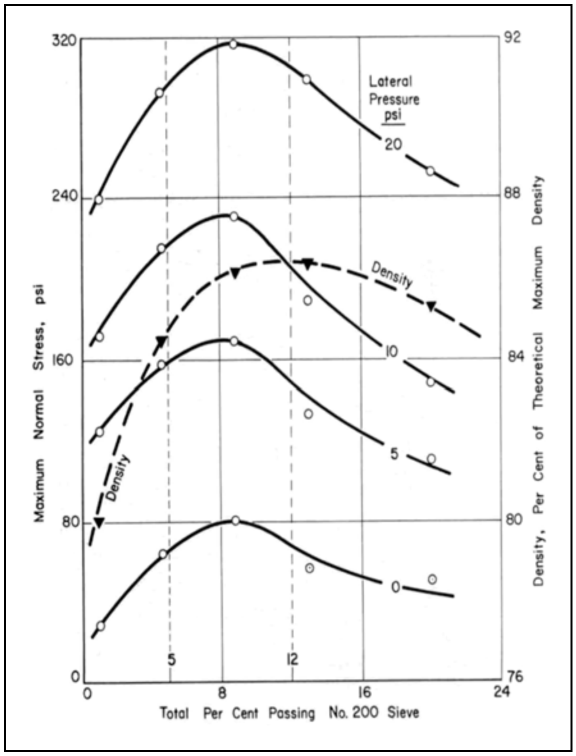


Figure 2.9. Relationship of Strength and Density for Material with Different Amounts Passing No. 200 Sieve.

2.5 PLASTICITY OF THE FINES

The effect that water will have on the performance of aggregate is dependent upon the plasticity of the fines fraction. Greater quantities of water are attracted and retained by fines of high plasticity which causes loss of shear strength and stiffness. Table 2.2 indicates the Atterberg limits and sand equivalent specified by most agencies. It is of interest to note that Nevada specifies different PIs depending on the percentage passing the No. 200 sieve. South Africa also associates a specified PI with grain size.

The Atterberg limit is most frequently used as a measure of control and includes liquid limit (LL), plastic limit (PL), plasticity index (PI), and linear shrinkage (LS). LL and PL are determined in accordance with AASHTO T-89 and T-90, respectively, and are performed on the fraction finer than the No. 40 (0.425 mm) sieve. Liquid limit represents the moisture content, in percent, at which the transition from the liquid to the plastic state of the material is reached, whereas the plastic limit represents the moisture content at which the transition from plastic to a semi-solid state is reached. The moisture content at which the transition from a semi-solid to solid state takes place is defined as the shrinkage limit. PI is simply the difference between LL and PL. Most agencies specify a maximum PI of 6, while Arizona specifies a maximum value of

3 and Texas currently specifies a maximum value of 10 (reduced to 8 in the proposed year 2003 specification). Nevada allows a PI of up to 15 if the fraction finer than the No. 200 sieve is smaller than 3 percent, but limits it to a maximum of 3 if the fines fraction exceeds of 11 percent.

A few agencies use the sand equivalent (ASTM D 2419-91) as a measure of the relative portions of claylike or plastic fines and dust in granular soils. This test is simple to perform and is designed to provide rapid results in the field. A measure of the fraction finer than the No. 4 (4.75 mm) sieve is placed in a graduated, transparent cylinder filled with a mixture of water and calcium chloride, which acts a flocculation agent. The total height of the sand and the flocculated clay is obtained visually after agitation followed by 20 minutes of settling. The sand equivalent is the ratio of the height of the sand alone (with a weighted foot on top) to the height of the clay plus sand (without a weighted foot on top) multiplied by 100. A higher sand equivalent value indicates cleaner fine aggregate with empirically derived values between 25 and 60 typically specified for Hot Mix Asphalt (HMA) (NCHRP, 2000). California, Florida, and Idaho specificied minimum sand equivalent values of 22, 28, and 30, respectively.

The effect of plasticity on graded crushed stone was investigated by Gray (1962) and will be referred to in some detail in the following paragraphs. Barksdale and Itani (1989) also studied the influence of plasticity of the fines on resilient modulus and permanent deformation by substituting a portion of the granitic gneiss used with kaolinite or bentonite. They showed that the performance of aggregate bases could be influenced detrimentally by the presence of plastic fines.

Gray (1962) tested six different gradations complying with Talbot's Equation with n = 1/3. One set with a maximum aggregate size of 3/8 in. and gradations with plasticity indices of non-plastic, 4.8, and 8.9; and another set with maximum aggregate size of 1 1/2 in. and gradations with plasticity indices of non-plastic, 3.6, and 7.2 were tested. The triaxial test results in the format of Mohr's failure envelope are presented in Figures 2.10 and 2.11. Figures 2.12 and 2.13 show the ultimate strength against lateral pressure for the two sets of gradations. For the 3/8 in. maximum size, only the non-plastic gradation can be classified as good base material, while the non-plastic and 3.6 PI gradations can be classified as good base material for the 1 1/2 in. maximum size. These data clearly demonstrates the detrimental effect that high plasticity has on base course performance and the pronounced countereffect that large aggregate, i.e., frictional resistance and aggregate interlock, can have. A rapid increase in percent strain with increase in plasticity index also illustrated this effect.

Gray states that the best solution to overcome performance problems related to high PI materials is to avoid the use of weathered rock and to require that the fines portion be produced from the parent rock. He concludes that while a PI of 6 or less is widely accepted, the lower the value the better, and non-plastic material is the best.

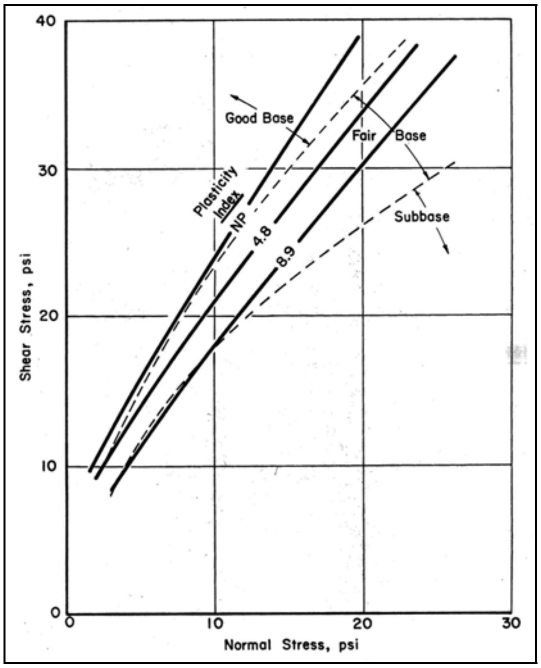


Figure 2.10. Effect of Plasticity Index on Shear Strength for Gradings with n = 1/3 with 3/8 in. Maximum Size.

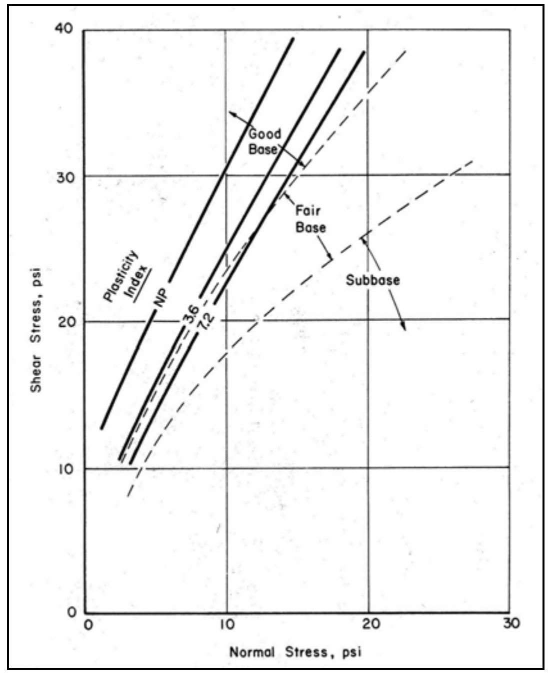


Figure 2.11: Effect of Plasticity Index on Shear Strength for Gradings with n = 1/3 with 1 1/2 in. Maximum Size.

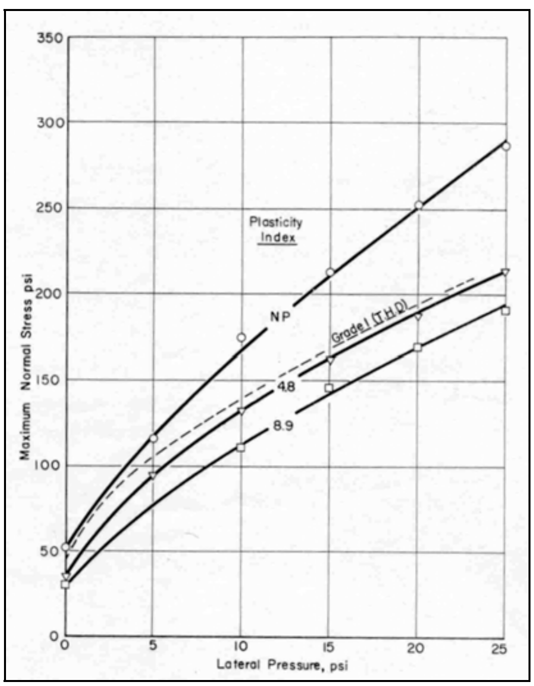


Figure 2.12: Effect of Plasticity Index on the Relationship of Principal Stresses for Gradings with n = 1/3 with 3/8 in. Maximum Size Aggregate.

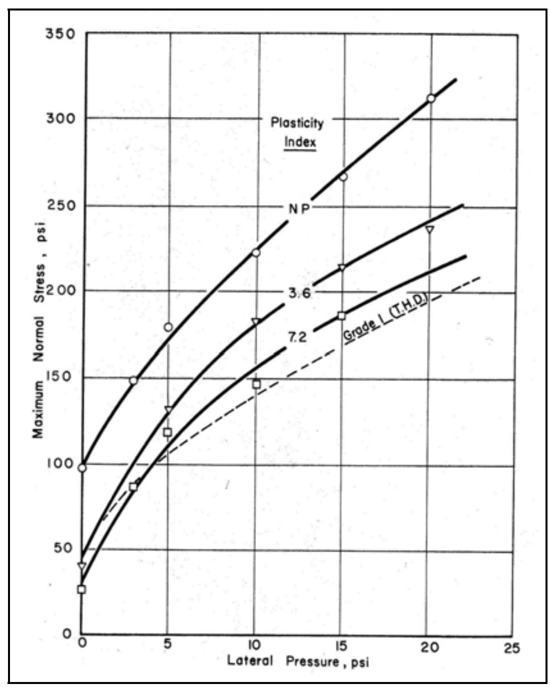


Figure 2.13. Effect of Plasticity Index on the Relationship of Principal Stresses for Gradings with n = 1/3 with 3/8 in. Maximum Size Aggregate.

2.6 DEGRADATION AND SOUNDNESS

Langer (2001) states that the influence of rock properties and their suitability has been recognized, but the importance thereof has not received the same emphasis as other engineering properties. He encourages the broader use of geology in the development of tests and establishment of specification limits based upon such tests. Langer also pointed out that hardness (resistance to load), strength (resistance to abrasion), and toughness (resistance to impact) of aggregates are the component properties that resist degradation. Mineralogy of rock particles, the packing or cementation, orientation and cleavable minerals, abundance of pores or fractures, and amount of weathering generally control hardness, strength, and toughness. These petrographic features of a rock are related to origin of the rock and subsequent weathering or alterations and can be observed and described by traditional geological methods. Fundamentally, physical properties that affect soundness are the size, abundance, and continuity of pores, channels, and fractures, and water saturation, which can be obtained from petrographic analysis.

Tests for degradation and soundness are generally incorporated into most specifications for aggregate used for base course construction as indicated by Table 2.2.

2.6.1 Degradation

Degradation refers to the mechanical breakdown of aggregates due to action of construction equipment and/or traffic (Langer, 2001). Table 2.2 indicates that among the selected road agencies, the Los Angeles test is the most popular test for determining degradation. Texas uses the wet ball mill test, while South African and Australian agencies tend to use crushing tests based on the British standards such as the aggregate crushing value and the 10 percent fines aggregate value.

2.6.1.1 Parameters from Crushing Tests

The aggregate crushing value (ACV) test and 10 percent fines aggregate crushing test (10 percent FACT) both involve the application of a large static compressive pressure that is transmitted onto an aggregate sample. Although both tests intrinsically measure the same material property, the ACV is the smaller and less expensive test to perform. In this test the resulting fines passing the 2.4 mm sieve, are measured and expressed as a percentage of the initial sample weight. The lower the ACV the higher the resistance to crushing. However, the ACV results become unreliable for values exceeding of 30 percent because of “clogging-up” of the mould with crushed particles, which prevents further crushing. South Africa specifies a maximum value for ACV between 21 and 29 percent. The ratio of the wet to dry values is usually obtained to assess the strength of the material under wet conditions.

The 10 percent FACT is a more complex test to perform but provides a more reliable identification of materials with poor crushing characteristics, due to the material being crushed only until 10 percent fines have been produced. In this test, the load required to obtain 10 percent crushed particles is recorded, and higher loads would therefore be preferred for good resistance against crushing. South Africa (1998) specifies a minimum dry 10 percent fines value of 110 to 200 kN and a minimum wet value of 125kN, with a wet-dry ratio of not less than

75 percent. Queensland (1999) only specifies a minimum wet value of between 130 and 150 kN with a maximum variation between wet and dry values not exceeding 40 to 50 percent.

2.6.1.2 Parameters from Impact-Based Tests

Many states measure degradation based on the Los Angeles (LA) test (AASHTO T-96; ASTM C 535). The LA test is a measure of the degradation of mineral aggregates of standard gradations resulting from a combination of actions including abrasion or attrition, impact, and grinding. A sample of specific weight and grading is rotated in a steel drum containing a specific number and size of steel spheres. As the drum with mounted blades rotates, it creates an impact-crushing effect is created. After the required number of revolutions, the material is sieved to measure the degradation as a percent loss.

Although the test is relatively simple, practical, and inexpensive, ASTM states that the results should only be used to indicate the relative quality of aggregate from sources having similar mineral composition. The variation of maximum specified limits, i.e., from 40 to 65, raised concern in the past (Powell, 1999). Different publications have indicated that the LA test is a good predictor of susceptibility of an aggregate to mechanical breakdown during construction, but not of field performance.

NCHRP (2000) pointed out that a comparison of the LA test values with ACV for the same aggregate shows a relatively large scatter, which indicates that at least one of the tests, or both, is not a reliable indicator of field performance. NCHRP also stated that both these tests have inherent discrepancies if the mechanism of loading in the field is considered, i.e., the ACV involves a single large static pressure, while the LA test subjects particles to severe impact. NCHRP (2000) refers to correlation studies done with the LA test where the addition of water (modified LA test) yielded a better correlation with more fundamental petrographic analyses.

The correlation matrix developed for aggregate toughness during the NCHRP (2000) study indicated that the percent loss of the coarse fraction obtained by the Micro-Deval test correlated significantly with performance. This test was consequently selected to represent the category in further analysis. The Texas wet ball mill test did not form part of the correlation study. Although similar in concept, both the Micro-Deval and the Texas wet ball mill tests (Tex 116E) differ from the traditional LA test in that aggregate is tested in a soaked condition.

2.6.2 Soundness and Durability

Soundness relates to durability and is defined by Langer (2001) as the ability of aggregate to resist breaking down because of repeated cycles of wetting and drying or freezing and thawing. The sodium sulfate soundness test is used by Florida and forms part of the aggregate index used by New Mexico. California specifies a durability index, while the ratio between a wet strength and dry strength serves as a measure of durability in countries such as South Africa and Australia.

The durability index (AASHTO T-210), used primarily by the western states, is similar in concept to the sand equivalent test and is done on the coarse and fine fractions. Aggregate index, used by New Mexico, is a function of the LA loss, soundness loss, and water absorption.

The sodium sulfate of magnesium sulfate soundness test (AASHTO T-104; ASTM C 88) is commonly used to measure soundness. In order to simulate the expansion of water on freezing, the test involves alternate cycles of immersion in saturated solutions of sodium sulfate or magnesium sulfate and drying to precipitate salt in permeable pore spaces expanding upon re-hydration of the salt. Different sources expressed concern about the poor reproducibility of the test and that it is not very useful as a specification test for accepting or rejecting material. Despite the criticism, NCHRP (2000) found that this test showed a good correlation with performance, and it was incorporated into the proposed aggregate performance evaluation chart. In Texas the soundness test is used for aggregates to be used in hot mix not in base.

2.7 MOISTURE SUSCEPTIBILITY

Base layers of structures constructed with moisture susceptible aggregates are prone to rapid development of permanent deformation during rainy seasons and periods of freeze-thaw. Texas began implementing the tube suction test (TST) in 2001, and it is included in the Draft 2003 specifications. The TST was developed in a cooperative effort between the Finnish National Road Administration and the Texas Transportation Institute (TTI) for assessing the moisture susceptibility of granular base materials. In this test compacted specimens are soaked by capillary action in the laboratory for a period of 10 days. The surface dielectric values (DEV) are measured on a regular basis to assess the rate at which certain criteria are reached. The lower the DEV the better the potential performance of the material under consideration. Their findings suggest that aggregate base materials with dielectric values less than 10 may be confidently ranked as neither moisture nor frost susceptible (Guthrie and Scullion, 2001). As this test is an indication of the behavior of a material in a certain environmental setting, the generalization of such a specification is not recommended. NCHRP (2000) adopted the proposed criteria of less than 10 for good material, 10 to 16 for fair material, and more than 16 for poor material. They found that the classification of the TST correlates well with known field performance of aggregates where such information was available.

Kolisoja et al. (2002) demonstrated the significant contribution of the TST in selecting base course aggregates that correspond to seasonal moisture variation in roads. It is stated that the applicability of the suction theory stems from the fact that permanent deformation originates from excess pore water pressure in the aggregate caused by dynamic axle loading. Increased pore water pressure reduces the effective stress between soil particles and leads to plastic deformation after only a limited number of axle load applications. Using the variation of Gibb's free energy, the suction theory explains the function of effective stress between soil particles and the impact of water in the aggregate during different seasons.

A series of projects was conducted mainly by the Tampere University of Technology (TUT) in Finland during 1996 to 2000 that concentrated on investigating the mechanical, electrical, chemical, and thermodynamic factors that affect the seasonal variation of strength and deformation behavior of base course aggregates. The test procedure involved several stages of mechanical testing, including normal as well as long-lasting cyclic loading triaxial tests, where specimens were exposed to treatments simulating dry, moist, and post freeze-thaw cycle seasonal conditions. Dielectric and electrical conductivity measurements were taken on the surface of the specimens during the drying and water adsorption stages using a Percometer, corresponding to the TST suggested by Scullion and Saarenkento (1996) and adopted by Texas.

During the first stage of the research several crushed gravel and rock aggregates over a spectrum of known field performances were tested. The fines contents of selected aggregates were systematically varied during the second stage to investigate the sensitivity of these aggregates to the amount of No. 200 (0.075 mm) material included. The maximum grain size in all of the test materials was 18 mm, while the fines content varied from 2.6 percent to 10.7 percent, and percent passing the 2 mm sieve varied from about 18 percent to 46 percent. The effect of addition of different amounts of emulsified bitumen to problem materials was also investigated during the third stage but will be omitted from this review.

This research indicated that suction properties result foremost from the fines content, but also from chemical properties of the aggregate. Figure 2.14 presents the relationship obtained between fines content and dielectric value measured on gravels and rock aggregates in the TST. It is shown that as the fines content exceeds 5 percent, the aggregate absorbs so much water that the dielectric limit value of 9, suggested for problematic aggregates, is exceeded. For poor-quality rock aggregates the limit value can be exceeded even at fines contents less than 4 percent. This research further illustrated that increase in compaction is associated with increased capillary forces, indicated by an increase in dielectric value, which corresponds to the volumetric water content in the material. Gravel aggregate samples absorbed water at various void contents less than rock aggregates, in which the chemical processes on newly crushed surfaces caused an osmotic suction.

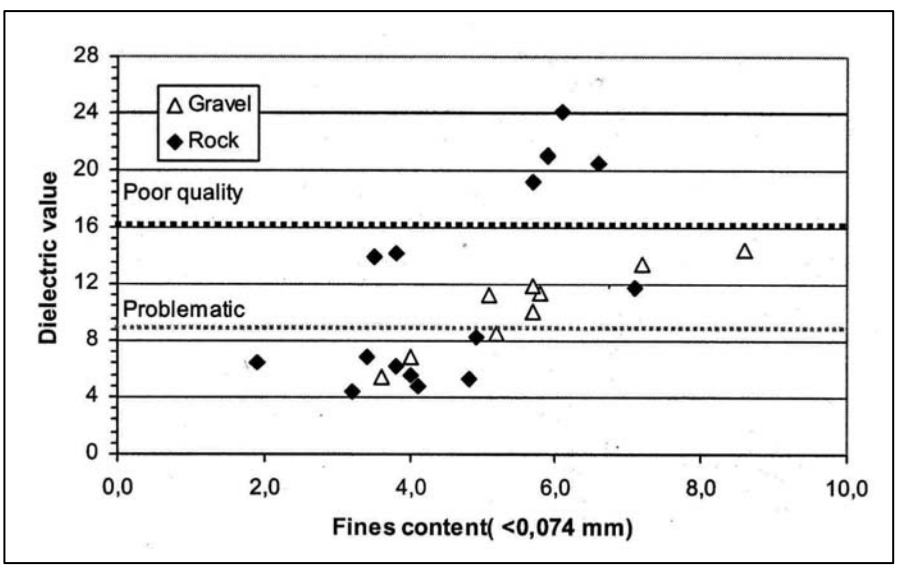


Figure 2.14. Relationship between Fines Content and Dielectric Value Measured from Examined Gravel and Hard Rock Aggregates in the TST Test.

The relationships between resilient modulus and fines content and permanent deformation and fines content were also established. The resilient moduli of dry samples increased as the fines content increased, while the moduli decreased as the fines content increased for specimens representing moist and post freeze-thaw cycle conditions. In addition, although the lowest moduli were obtained in the thaw condition, the values of all materials tested were still reasonable, i.e. on the order of 200 MPa. However, although the resilient moduli values were little changed by fines content, the Finnish researchers concluded that a rise in fines content from 3.9 percent to 10.7 percent resulted in a significant increase in permanent deformation.

The relationship of dielectric value on the surface of the specimens after the water adsorption stage and permanent axial deformation of the test specimen in series of 100,000 load cycles performed after a freeze-thaw cycle is shown in Figure 2.15. A large-scale cyclic triaxial testing facility was used that required specimens with a diameter of 8 in. (200 mm) and height of 16 in. (400 mm). The surface dielectric values measured on these specimens were therefore lower than those values that would normally be measured on 8 in. samples during the TST.

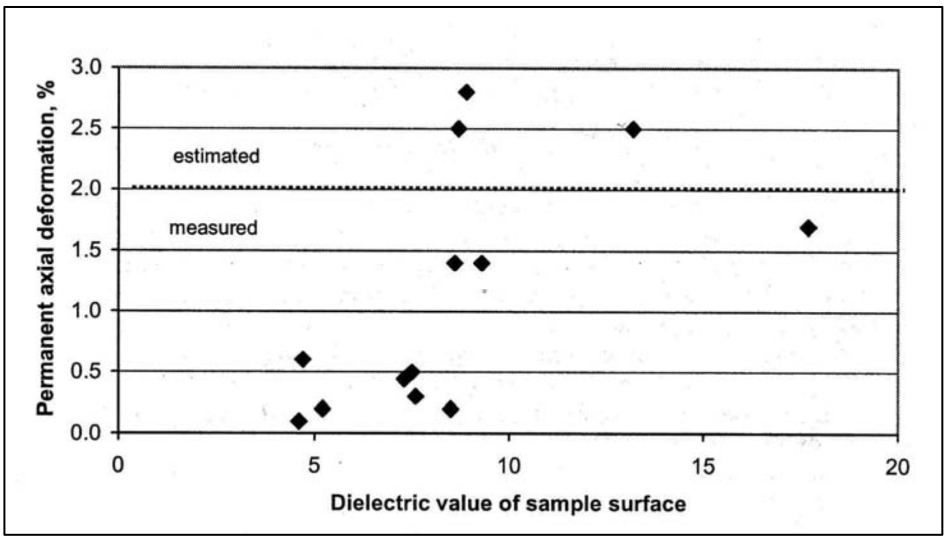


Figure 2.15. Relationship between Dielectric Value of Test Specimen Surface after Water Adsorption Stage and Permanent Deformation Measured after Freeze-Thaw Cycle in the TST Test.

Permanent deformation of 2 percent represented failure considering that this was achieved after only a few hundred load applications. Figure 2.15 shows that large permanent deformation was obtained for specimens with dielectric values higher than 8, recognizing the fact that these values would be higher for specimens of normal height.

A significant conclusion drawn by the researchers is that although the permanent deformation of the aggregates was considerably different, resilient modulus values were not significantly lowered even during the thawing phase. Results further show that suction properties of aggregates seem to have a significant effect on permanent deformation behavior. Kolisoja, et al. (2002) suggested the following applications of the TS test in evaluating aggregates:

- to estimate if an aggregate with a given grain size distribution is frost susceptible and if it is susceptible to permanent deformation;
- to evaluate how the grain size distribution of a crushed aggregate should be changed so that it would be suitable for use as an unbound base course material; and
- whether it can be used to assess the effectiveness of binder type and amount of binder required in cases where stabilization is considered.

The results from this study clearly show the impact of fines on the moisture susceptibility of granular base materials. However, the authors did not report any of the minerological properties of the fines. This will be investigated in the laboratory testing phase of study 4358. For bases which perform poorly in the TST it is proposed to identify the composition of the fines and the fine clay fraction in particular. It is proposed that it is the type of fines rather than the amount of fines which dictate engineering properties. This will be reported on in later reports in this project.

2.8 SHAPE, ANGULARITY, AND SURFACE TEXTURE

Numerous methods exist to measure and express particle shape, angularity, and surface texture in terms of some individual parameter or a combined index (NCHRP, 2000). These characteristics are influenced significantly by the geological formation and mineral composition of the parent rock, which causes the aggregate to fracture in a certain way during crushing. The percentage fractured faces is commonly specified world wide, while flakiness index is a shape parameter used in some overseas countries. Where the triaxial test is performed as part of the strength specification, the angle of internal friction (ϕ) can also give an indication of the particle angularity.

Table 2.2 indicates that the percentage of fractured faces is commonly used by agencies included in this investigation. However, the specifications differ in terms of the percentage specified, the fraction under consideration, the definition of a fracture, and the definition of a fractured face. According to ASTM D5821, the percentage of fractured particles in the size fraction is performed on the No. 4 sieve. A fractured or crushed fragment is defined as one having one or more fractured faces, while a fractured face is defined as a face that exposes an interior area of more than 25 percent of the cross-sectional area of the particle. This method relies on visual inspection to count the crushed particles manually so that they can be expressed as a percentage of the total aggregate mass.

Both South Africa and Queensland, Australia, specify flakiness index as a measure of the shape of the aggregate. This parameter is similar in concept to the flat and elongated particles expressed as a percentage of the coarse aggregate described in ASTM D4791. The flat and elongated particles are defined in terms of the ratio of width to thickness or length to width greater than a specified value. The ratio is usually measured as the minimum dimension to the maximum dimension of the aggregate particle specified as a maximum value typically between 1:3 and 1:5. The NCHRP (2000) study investigated flat and/or elongated particles, a particle shape index, and uncompacted voids and found that the mass-based flat or elongated (FOE) parameter with a ratio of 1:5 correlated the best with performance and was consequently used for further analysis.

Researchers have documented the influence of shape, angularity, and texture on performance in the past, and general findings are presented here to illustrate the effect of the individual parameters on performance. Barksdale and Itani (1989) determined the resilient and permanent deformation characteristics of different aggregates using a cyclic load triaxial test and introduced an aggregate influence factor (AIF) to illustrate the combined effects of shape and surface characteristics on permanent deformation and resilient modulus. Cheung and Dawson (2000) conducted a study on the effects of particle and mix characteristics on the performance of some granular materials. Three granular material types were selected, representing different physical properties of stone particles. Angularity, shape, surface, and strength characteristics formed part of the particle examination. Large shear box tests, large-diameter load tests, and a trafficking trial followed to investigate the performance of the materials under wheel loading. Results indicated that angularity and roundness had the greatest effect on permanent deformation and strength of aggregates. Stiffness was influenced by the surface friction and surface roughness of the unbound aggregate particles. Researchers drew the following conclusions:

- Stiffness of the material increases with surface roughness and surface friction. Rough surfaced crushed stone is therefore expected to produce higher stiffness than granular material with smooth surfaced particles.
- Aggregate with lower angularity and high roundness tended to produce mixes with lower shear intercept values (cohesion) and higher plastic strains as well as lower compressive strength under repeated loading.
- Resistance to permanent deformation increased with an increase in the apparent friction angle.
- No relationship existed between the strength of the stone particles and performance of the mix in terms of stiffness, permanent deformation, and strength.

TxDOT is embarking on additional research in the measurement of aggregate shape and angularity. Study 1707 will be using the AIMS (Aggregate Imaging System). This automated procedure based on advanced image analysis hold much potential. This work is just initiating and the initial focus will be on aggregates for hot mix rather than base.

2.9 COMPACTION

Several researchers reported on the significance of compaction in relation to good base performance. It has been shown that the degree of compaction has a dominant influence on the behavior of granular materials under various in-service conditions. Marek and Jones (1974) refer to compaction as the process of forcing particles together in order to maximize interparticle friction. Density is measured as an indicator of the compaction achieved and is simply the amount of solids in a unit volume of material, hence is inversely related to the voids between the aggregate particles. The degree of compaction is typically evaluated in terms of relative density. Relative density refers to the material's density after compaction in the field relative to the "maximum" attainable, or reference, density for the same material utilizing specific equipment and procedures in the laboratory.

Maximum dry density (MDD) is used world wide as reference density. Table 2.2 shows that most states use the modified Proctor density (AASHTO T-180; ASTM D 1557-70), or equivalent methods such as Tex-113-E, for establishing a reference density for compaction control. Compaction of 100 percent AASHTO T-180 is commonly specified. A limited number of states still use the standard Proctor density (AASHTO T-99; ASTM D 698-70). The compactive effort for the T-180 methods includes a 10 lb (4.54 kg) rammer and an 18 in. (457 mm) drop, while the T-99 method includes a 5.5 lb (2.5 kg) rammer and a 12 in. (305 mm) drop. The degree of compaction achieved at 100 percent of the density established using the AASHTO T-99 method is therefore substantially less than that achieved using the AASHTO T-180 method. New South Wales, Australia, specifies a slightly higher relative compaction of 102 percent RTA111, which is essentially the same as the standard Proctor density.

Table 2.4 shows the compactive effort currently specified by TxDOT in procedures 113-E and 114-E. Note that this effort is less than the modified proctor used in other states.

Table 2.4. Compactive Effort in Procedures 113-E and 114-E.

Compaction Effort	Approximate Compaction Energy (ft-lb/ft^3)	Procedure for Molding 6 in Diameter, 8 in Tall Sample
Standard Proctor (Tex-114-E is equivalent to Standard Proctor)	12400	4 lifts of 74 blows per lift with a 5.5 lb hammer and 12 inch drop height (or 4 lifts of 27 blows per lift with the 10 lb hammer and 18 inch drop height)
Tex-113-E	22920	4 lifts of 50 blows per lift with 10 lb hammer and 18 inch drop height
Modified Proctor	56000	4 lifts of 122 blows per lift with 10 lb hammer and 18 inch drop height

Most specifications require placement of base and subbase materials at optimum moisture content and at some relative density. Researchers recognize that the use of such density specifications causes the following concerns (NCHRP, 2000):

- Optimum moisture content varies as relative density varies.
- Laboratory impact compaction does not reflect field compaction where pneumatic and vibrating rollers are used. This may be attributed to significant edge effects introduced by laboratory equipment, and ignoring the results of the influence of large particles.

2.10 STIFFNESS

The Texas draft 2003 standard specification for granular materials makes provision for the inclusion of "Young's Modulus," which is a measure of stiffness. Layer moduli are commonly used in design applications and are generally related to pavement performance. It would therefore be ideal if this parameter could be specified to help bridge the gap between design and the product required during construction. Currently, the resilient modulus laboratory test of a granular material is generally used as a research tool because of high cost and complexity associated with these tests.

Although layer moduli are widely used in production design applications, laboratory-determined values typically differ significantly from values obtained from field tests. Apart from inherent errors in the field and laboratory test procedures, quantification of the effect of a wheel load moving over an element of pavement structure is much more complex. The non-linear stress versus strain behavior exhibited by granular materials was pointed out by Hicks and Monosmith (1971). Allen and Thompson (1974) reported the effect that changing lateral and vertical stresses may have on resilient modulus. The anisotropic resilient properties of unbound granular materials have gained the renewed interest of researchers such as Audo-Osei et al. (2000) and Tutumueler and Seyhan (2000).

Three significant parameters related to the stress-strain curve of granular pavement materials under a load are depicted in Figure 2.16 (Nazarian et al., 1998a). The strength of the material (S_{max}) is indicated by the horizontal line asymptotic to the curve. The resilient modulus from laboratory testing and the backcalculated modulus from falling weight deflectometer (FWD) measurements in the field normally correspond to a secant modulus (E1, E2, E3), which is strongly affected by the magnitude of strain experienced by the material. The initial modulus (E_{max}), or tangent modulus, can be measured directly or backcalculated using a seismic source and is therefore also termed the seismic modulus. This high-frequency, low-strain modulus also corresponds to Young's modulus. This modulus is directly affected by the initial state of stress and density of the material and is difficult to obtain with the resilient modulus test because of equipment limitations.

For stiffness to be incorporated as part of material product specifications, simplified laboratory tests or practicable laboratory testing procedures need to be developed. The following paragraphs present proposed tests and procedures.

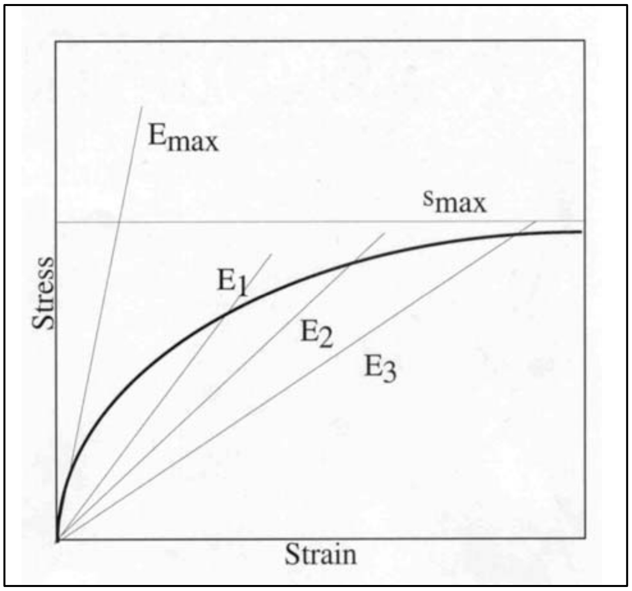


Figure 2.16. Typical Stress-Strain Curve for Pavement Material (Nazarian et al. [1998a]).

2.10.1 Resilient Modulus

The repeated load triaxial test is commonly used to obtain the resilient response of a sample where repeated or pulse loads are applied axially to a cylindrical specimen with the confining pressure held constant. The resilient modulus, M_R, is the maximum deviator stress divided by the maximum recoverable axial strain. The resilient modulus test used widely in the U.S. was developed under the Strategic Highway Research Program (SHRP) (AASHTO T 294-94; SHRP Protocol P46).

Based on the fact that the same aggregate properties that influence shear strength also influence stiffness, NCHRP (2000) researchers proposed an aggregate performance evaluation procedure that would combine performance ratings based on stiffness with ratings based on shear strength. This performance evaluation procedure, or decision chart, is presented in Chapter 3, Table 3.4. In this approach, tests are conducted in sequence from simple screening tests to more robust and expensive tests, such as resilient modulus, while checking results for conformance with appropriate criteria. At each step of the process, the aggregate is either rejected or it is advanced to the next phase of tests and eventually accepted if it meets all the criteria. The acceptance criteria are based on traffic and climatic or regional factors and therefore optimize the testing in such a way that high-level testing would not be required where it is not warranted by traffic conditions.

During the NCHRP (2000) study, the range of resilient moduli obtained was between 26 ksi for poor base aggregate to 90 ksi for good base aggregate tested at optimum moisture content. Proposed stiffness criteria for different operational conditions are presented in Table 3.4. The researchers, however, suggest that the user agency adjust these values to suit their conditions.

2.10.2 Seismic Modulus

The time-consuming nature of triaxial tests make, them impractical to use on a routine basis. Seismic tests are simpler tests to perform and can be used to rapidly determine the modulus of a material.

Researchers have promoted the free-free resonant column test for use as a means of material quality control. Young's modulus, obtained by performing this test, is also considered for inclusion into the Texas 2003 standard specifications. The specimen is suspended from two wires. A hammer with a load cell is used to impact the specimen on one end of the specimen to induce seismic waves, while an accelerometer is securely placed on the other end. This test has also been conducted with success by placing the specimen in an upright position on an insulated frame (Guthrie and Scullion, 2000). The length-to-diameter ratio of the specimen must be less than two. The signals from the accelerometer and load cell are used to determine the resonant frequency, (f). Young's modulus (E) can be determined once the frequency, mass, density (ρ), and length of the specimen, (L), are known:

$$E = \rho(2fL)^2$$

The shear modulus can also be determined if the accelerometer is placed in the radial direction and the specimen impacted in the radial direction. The shear and Young's modulus can then be combined to calculate Poison's ratio.

It is recommended that soft specimens be supported by a cylindrical polyvinyl chloride (PVC) pipe during testing. The researchers state that these tests have been performed successfully on subgrade, granular base, stabilized base, asphalt (AC) and Portland cement concrete (PCC) cores. The method is nondestructive and is claimed to produce repeatable results (Nazarian et al., 1998).

Since seismic moduli from field or laboratory measurements correspond to very small external loads, they are typically 1.7 times greater than moduli determined under actual wheel loads for unbound base course materials. In order to render this parameter useful, a constitutive model that relates the seismic modulus to the nonlinear modulus under any other loading regime was developed. The details of this model and its implementation with the use of computation algorithms, representing several structural models, were documented by Nazarian et al. (2002). The researchers suggest that seismic testing methods hold the key to developing mechanistic pavement design procedures that contain performance-based specifications, where the same engineering properties that are used for designs are used to determine the suitability of the material for construction.

Nazarian et al. (2002) report values for Young's modulus obtained for five different base courses. These values were collected in the field with the seismic pavement analyzer (SPA) and analyzed using the spectral analysis of surface waves (SASW) method. Details on these testing and analysis techniques are reported elsewhere.

The following table summarizes average values obtained for these bases together with modulus values obtained from traditional backcalculation of deflection basins measured with a FWD.

Table 2.5. Average Values for Young's Modulus Obtained from Seismic Field Testing.

US-281 Section ID	Base Description	Young's Modulus (ksi)	Modulus from Fitted Deflection Basin (ksi)
480113	Dense Graded Aggregate	139	50
480114	Dense Graded Aggregate	123	48
480161	Limerock Asphalt	225	144
480162	Crushed Limestone	118	34
480167	Caliche	117	31

2.11 SUMMARY

This chapter has provided an overview of both the base specifications in use in several national and international agencies, together with a general discussion of the recent efforts to develop new performance related tests. With regard to the objectives of developing high performance granular bases the following are important observations:

1. The current Item 247 granular base specifications differ from all other agencies in that TxDOT is the only agency which does not regulate the amount of fines. Other agencies typically limit the amount passing the No. 200 sieve to less than 10%. In Texas it is possible to have more than 20% fines.
2. Grey (1962) found that each base has an optimal fines content. Problems with shear strength and frost susceptibility will be fond if large increases or decreases in the amount of fines are permitted. The TxDOT recommended range of from 5 to 10% appears reasonable.
3. The work of Kolisoja (2002) clearly demonstrated the contribution of the tube suction test in selecting aggregates. Materials ranked poor in the TST did very poorly in permanent deformation tests. The TST will be used extensively in later stages of this study.

CHAPTER 3
RESEARCH FINDINGS FROM THE RECENT NCHRP STUDY

3.1 INTRODUCTION

"Performance Related Tests of Aggregates for use in Unbound Pavement Layers" was research performed under NCHRP Project 4-23, completed in August 2000. The purpose of this project was primarily to

- evaluate existing tests,
- identify new tests that relate to performance, and
- develop better procedures for testing and selecting aggregates for use in unbound pavement base and subbase layers.

This section will provide an overview of the NCHRP research project, summarize the findings, as well as suggest future research. The following flow diagram outlines the research approach followed to achieve the stated objectives.

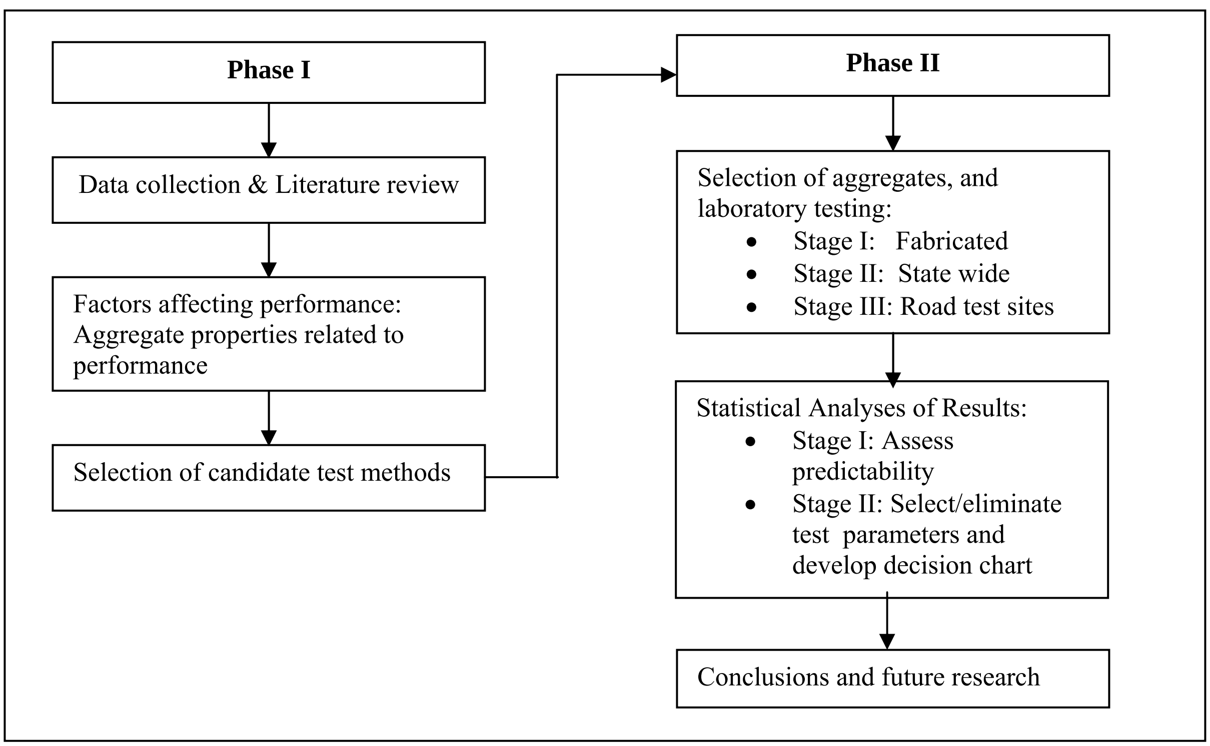


Figure 3.1. NCHRP (2000) Research Approach.

3.2 SELECTION OF CANDIDATE TEST METHODS

The Phase I research concentrated on the selection of test methods for use in the Phase II research, where these methods would be evaluated based on actual test results and correlation with performance. Existing data were obtained from a literature review, research findings, performance data, current practices, and other sources. In order to complement the objectives of this research project, it was important to know what the factors influence performance, as well as aggregate properties related to these factors. The following table (Table 3.1) was compiled as a starting point to assess the relationship between performance parameters, aggregate properties, and test parameters.

Table 3.1. Relationship between Aggregate Properties and Performance for Flexible Pavements.

Performance Parameter	Related Aggregate Property	Test Parameter That May Relate to Performance
Fatigue Cracking	Stiffness	Resilient modulus, Poison's ratio, gradation, fines content, particle angularity and surface texture, frost susceptibility, degradation, density
Rutting, Corrugations	Shear Strength	Failure stress, angle of internal friction, cohesion, gradation, fines content, particle texture, and shape and angularity, density, moisture
Fatigue Cracking, Rutting, Corrugations	Toughness	Particle strength, particle degradation, particle size, gradation, high fines
Fatigue Cracking, Rutting, Corrugations	Durability	Particle deterioration, strength loss
Fatigue Cracking, Rutting, Corrugations	Frost Susceptibility	Permeability, gradation, minus 0.02 mm fraction, density, fines type
Fatigue Cracking, Rutting, Corrugations	Permeability	Gradation, fines content, density

The evaluation of test methods was based on a subjective qualitative rating of each test method in each category (aggregate property), based on the research team's experience and judgment. The following rating factors were considered in the evaluation and selection process:

- performance predictability,
- accuracy,
- practicality,
- complexity,
- precision, and
- cost.

The candidate tests selected from this evaluation for use in research Phase II are summarized in Table 3.2.

3.3 DEVELOPMENT AND VALIDATION OF AN AGGREGATE PERFORMANCE PREDICTION PROCEDURE

The second phase of the research was executed in three stages. The selection of aggregates, laboratory testing, and statistical analysis of the results were based on the objectives set for each stage. The following paragraphs present a description of each stage and significant findings.

3.3.1 Stage I

The purpose of this stage was primarily to standardize and calibrate the test scheme based on fabricated samples. The samples were blended to represent the full shear strength range, i.e., from poor to good quality material. The materials were rated on anticipated performance by the research team. The fabricated samples were subjected to all the tests outlined in Table 3.2; a correlation analysis and multivariable regression analysis were then performed on the data to assess the ability of parameters to predict performance.

The following significant conclusions were drawn:

- From correlation analysis on data from this preliminary testing, static triaxial deviator stress at confining pressure of 10 psi (D10), repeated load triaxial deviator stress at confining pressure of 15 psi (RLTT), California bearing ratio (CBR), and dielectric constant value (DCV) were selected for multiple regression analysis. Four different combinations (models) were investigated.
- Multivariable regression analysis indicated that all the models had a high regression coefficient, R^2, although only the model that included the D10 and RLTT parameters was statistically significant at a 5 percent level.
- Shear strength was the single variable most strongly related to relative order of performance.
- The CBR was discarded from Stage II testing because of poor correlation with performance.

3.3.2 Stage II

The Stage II aggregate samples were provided by state DOTs across the U.S. The DOTs also rated these materials based on known performance history when used in base or subbase layers. On a scale of 1 to 5, a rating of 1 was used for materials with a poor performance history, while a rating of 5 was used for materials with an excellent performance history. Rating criteria were provided to the DOTs and are documented in the project report. In addition to performance ratings, traffic categories and climatic data formed part of the information provided by the different DOTs.

Table 3.2. Selected Tests for the Laboratory Test Program.

Aggregate Property	Test Method	Test Reference	Test Parameter
Screening Tests	Sieve Analysis	T-27/11[a]	Particle size distribution
	Atterberg Limits	T-89/90[a]	PL, LL, PI
	Specific Gravity	T-84/85[a]	Specific gravity
	Moisture-Density Relationship	T-99/180[a]	Maximum dry density
	Flat & Elongated Particles	D4971[b]	F or E, F and E
	Uncompacted Void Content	TP 33[a]	
	Shape and Texture	D 3398[b]	Particle shape and texture index
Shear Strength	Static Triaxial Shear	T-296[a]	c, ϕ, shear strength
	Repeated Load Triaxial Shear		Deviator stress to cause rapid failure
	California Bearing Ratio	T-193[a]	CBR
Stiffness	Resilient Modulus		
Frost Susceptibility	Tube Suction Test		Dielectric constant
	Index Method		F categories
Toughness	LA Abrasion	C 131[b]	% loss, passing No. 12 sieve
	Agg. Impact Value	BS 812[c]	% loss, passing 2.4 mm sieve
	Agg. Crushing Value	BS 812[c]	% loss, passing 2.4 mm sieve
	Micro-Deval Test	TP-58-99[a]	% loss, passing No. 16 sieve
	Gyratory Degradation		Before and after gradation
Durability	Sulfate Soundness	T-104[a]	Weighted average loss
	Agg. Durability Index	T-210/176[a]	Durability index

Notes: a: AASHTO reference test method.
b: ASTM reference test method.
c: British reference test method.

The primary objective of the Stage II data analysis was to finalize the set of tests that could be used to predict aggregate performance in unbound pavement layers. Well-established screening tests that provide test results in the form of a single test parameter did not form part of the correlation analysis of Stage II data, i.e. sieve analysis, Atterberg limits, moisture-density relationship, and specific gravity and absorption. Correlation analysis was performed on data obtained from the Stage II laboratory tests to determine which tests provided the best relationship with performance. The selected test parameters are summarized in Table 3.3.

Table 3.3. Description of Test Parameters Selected for Performance Prediction.

Test Category	Property	Description	Range
Screening Tests	Cu	Coefficient of uniformity to represent aggregate gradation	2 $\geq$ 10
	D	Maximum dry density (psi)	126 – 143
	FAE 5:1 m	Flat and elongated particles in ratio of 5 to 1, based on the mass of the sample (%)	0.0 – 4.1
	Uc	Composite uncompacted voids value (%)	13.9 – 52.2
Toughness	MDI	Micro-Deval test result for 19.0 to 9.5 mm fraction (% loss)	5.0 – 42.7
Shear Strength Tests	σ_d dry	Repeated load triaxial test result, tested at optimum moisture content (OMC) at 15 psi confining stress (psi)	98 – 190
	σ_d wet	Repeated load triaxial test result, tested when saturated at 15 psi confining stress (psi)	95 – 190
	σ_{d5} dry	Standard triaxial test deviator stress at 5 psi confinement tested at OMC (psi)	26 – 103
	σ_{d5} wet	Standard triaxial test deviator stress at 5 psi confinement tested when wet (psi)	59 – 184
Stiffness	M_R	Resilient modulus when tested dry (psi)	26 – 90
Durability	S	Aggregate soundness value for coarse fraction	1.0 – 43.0
Frost Susceptibility	DCV	Dielectric constant value from the tube suction test	30.0

The selected test parameters were utilized together with performance ratings, traffic categories, and climatic conditions to develop a decision chart for selecting aggregates based on predicted performance. The decision chart is presented as Table 3.4. Tests are conducted in sequence, while results are checked for conformance with the suggested criteria. In each step of the testing program, the aggregate is either rejected or advanced to the next level of testing. If the full range of criteria is met, the material is accepted.

3.3.3 Stage III

The aggregate performance prediction procedure developed in Stage II was validated using Stage III laboratory data. Materials with known field performance were used and included one sample from the Ohio Test Road and two samples from the Minnesota Road Research Facility (MnROAD). The materials were subjected to the tests proposed in the decision chart and the traffic, moisture, and temperature conditions under which the aggregates were expected to provide good performance. Actual and predicted performance compared well.

Table 3.4. Recommended Tests and Test Parameters for Assessment of Aggregate Performance Potential (NCHRP, 2000).

Tests												
Traffic [a]	High		Medium		High		Low	Medium		Low		
Moisture [b]	High	Low	High	Low	High	Low	High	High	Low	Low	High	Low
Temperature [b]	F	F	F	F	NF	NF	F	NF	NF	F	NF	NF
Screening Tests [c]												
Gradation, Cu	≥6			≥6				≥2				≥2
Max. Aggregate Size	≥¾”			≥¾”				≥¾”				≥¾”
Minus #200, %	≤5			≤8				≤10				0 ≤% ≤12
Atterberg Limits [d]	Non Plastic			Non Plastic				Non Plastic				Non Plastic
Uncompacted Voids, Uc	<35			<45				<55				<65
Flat & Elongated 5:1m	<0.10			<0.10				<0.32				<0.32
Toughness/ Abrasion												
Micro Deval, MD	≤5			≤15				≤30				≤45
Durability												
Sulphate Soundness, S	≤ 3			≤30				≤30				≤45
Frost Susceptibility [e]												
Tube Suction Test, DCV	≤7			≤10				≤15				≤20
F- Category	F-1			≥F-2				≥F-2				≥F-3
Shear Strength [f] (psi)												Not Required
Std. dry, σ_c=5 psi, σ_d	≥100			≥60				≥40		≥25		
Std. wet, σ_c=5 psi, σ_d	≥180			≥135				≥90		≥60		
Rep. dry, σ_c=15 psi, σ_d	≥180			≥160				≥30		≥90		
Rep. wet, σ_c=15 psi, σ_d	≥180			≥160				≥125		≥60		
Stiffness (ksi)												
Res. Modulus, M_R	≥60			≥40				≥ 32		≥ 25		

Notes:
a. (ESALs/year < 100,000) = Low (100,000 – 1,000,000 ESALs/year) = Medium (ESALs/year > 1,000,000) = High
b. Based on standard AASHTO definitions.
c. Screening tests should also include specific gravity and moisture-density relationship tests.
d. Most DOTs allow some plastic fines in their base/subbase layers; all test samples were non-plastic.
e. Frost susceptibility tests are not required in non-frost areas.
f. Triaxial tests are optional for the low-traffic category.

3.4 FUTURE RESEARCH SUGGESTED BY THE NCHRP STUDY

The ranges for the selected test parameters for different traffic and climatic conditions were based on laboratory tests and current state DOT specifications. The researchers suggest that user agencies adjust these values to suit their conditions.

NCHRP researchers proposed modifications to the triaxial procedure (AASHTO T-296) used in this study. These include better control over drainage during testing and modification to the load frame or adjusted confining stress in order to accommodate the full shear stress range.

A field evaluation plan has been suggested to further validate the performance-related tests of identified aggregate. This procedure includes accelerated pavement testing (APT) of specially constructed pavement sections and validation of the procedure in actual practice. The latter would provide for monitoring of the pavement construction and performance as well as testing the adaptability of the highway department methods of aggregate evaluation and comparing the test results with current procedures.

SUMMARY

Table 3.4 shows significant recommendations resulting from this project. Of particular importance are the following items shown in the table:

- an upper limit of 10% minus #200;
- use of the tube suction test for cold weather performance;
- testing using the standard triaxial test at 5 and 15 psi; currently TxDOT specifies tests at 0 and 15 psi, and several studies has shown that results from 0 psi tests can eliminate very good materials.

These recommendations will be incorporated into later studies in project 0-4358.

CHAPTER 4

ASPECTS OF MATERIAL AND CONSTRUCTION QUALITY CONTROL

4.1 INTRODUCTION

Material and construction quality control schemes can essentially be subdivided into control of the process and control of the product. Three general types of control points are encountered that indicate the time frame and nature of acceptance measures required. These are not often clearly defined or incorporated in a functional way. Queensland (1999), Main Roads Standard Specifications, incorporates these control points throughout the text in an attempt to clarify and thus simplify the implementation of the quality control requirements. The three terms used with associated definitions are as follows:

- Hold point: An identified point in a construction process after which the contractor is not allowed to proceed without a direction from the engineer.
- Witness point: An identified point in a construction process at which an activity is observed.
- Milestone: A point within a project which marks the occurrence of an important activity, or where progress is verified by the completion or start of an activity.

The first part of this chapter is concerned with the construction process specifications, which are essentially monitored by observation. The objectives are to identify all the elements in the process of constructing high-performance granular bases and to highlight important construction attributes addressed by different road agencies.

Material process control *inter alia* includes acceptance of sources and certification, transport of materials, and stockpiling. Details of the material process control do not form part of this study, although some aspects thereof feature in the second part of this chapter. Comparing various material product specifications is one of the major objectives of this study, and details have been provided in previous chapters.

Quality control measures on specified material properties typically include tolerances of 5 percent on individual sieve sizes of the target gradation, while some agencies also apply tolerances to other parameters such as the Atterberg limits. Construction product tolerances are widely applied to layer thickness, level, cross-fall, and grade.

Various road agencies typically use a variety of analysis procedures for the acceptance evaluation of material and construction lots. These include comparison with single specified target values, use of simple statistical procedures, and use of more comprehensive statistical analysis procedures taking into account variation induced by materials and the construction process. Tolerances, acceptance limits, and acceptance evaluation procedures are outside the scope of this study.

Compliance testing forms an integral part of product quality control. The second part of this chapter is focused on the frequency of sampling or minimum number of samples required for compliance testing by different agencies.

4.2 CONSTRUCTION PROCESS SPECIFICATIONS

A detailed summary of the construction process specifications for base courses of 10 road agencies is presented in Appendix A. The following paragraphs outline the key elements of the construction process and highlight important construction attributes obtained from the literature review.

4.2.1 Supporting Layer

Most agencies specify an acceptable subgrade or subbase for adequate support of the base course. Most specifications only refer to the applicable clauses for performing the work. Proof rolling followed by repair of soft spots is often provided as an alternative to density requirements (Texas, Draft 2003; New Mexico, 2000).

4.2.2 Trial Sections

South Africa (1998) requires a trial section before the crushed stone layer may be constructed. Approval of the base material is granted only after the successful construction of the trial section. The trial section is specified to be between 150 and 200 m in length and the width as ordered by the engineer.

Florida (2000) only requires a trial section in the event of the compaction of more than 150 mm in a single lift. The length of the section may vary between 90 and 300 m. Once approved, the compactive effort remains unchanged and a new source of material requires a new trial section.

4.2.3 Spreading and Mixing

Texas (Draft 2003) and Arkansas (1996) require spreading and mixing of material on the same day it is delivered, or as soon as possible. Specifications on spreading and mixing are commonly concerned with maintaining a uniform mixture during this activity. Alternatively or in addition, the direct consequences of non-uniformity of material such as segregation and the formation of hard nests are addressed by many agencies. Illinois (1997) specifies that minimum blading or manipulation is required to prevent segregation.

As mixing forms part of the process by which the moisture is uniformly distributed, Florida (2000) specifies that the entire width and depth of the layer be manipulated as a unit during wetting and drying operations. Florida further requires that the addition of water be carried out by uniform mixing-in through disking to the full depth of the course.

Although most specifications have general equipment requirements, some agencies have more detailed requirements for spreading and mixing. Different requirements for central plant mixing or mixing in the field either form part of the immediate construction specifications (Illinois,

1997) or are handled in different reference clauses (South Africa, 1998). Queensland (1999), for example, has specific requirements for equipment used during spreading and mixing. The latter include self-propelled, purpose-built spreading machines with capacity to spread material in one pass to the necessary uncompacted depth and one-half the pavement width, or 3 m, whichever is less.

4.2.4 Moisture Content

Specifications on water content are often not explicit but inferred through phrases such as "material shall have sufficient moisture content to obtain the required compaction," or "compact when proper moisture conditions are attained," or "maintain substantially at optimum moisture during compaction." In the latter phrase, "optimum" is not defined but is open to interpretation. The use of trial sections, mentioned above, may be utilized to determine the field optimum moisture content for the type of material and compaction sequence used before construction.

Some specifications require compaction at optimum moisture content, with reference to the laboratory obtained value. Issues with such a specification were mentioned in Section 2.9 and essentially stem from the fact that the optimum moisture content in the field is different from the optimum moisture content (OMC) obtained from laboratory results, associated with the maximum dry density (MDD). This difference is not only induced by a unique field density required for a specified relative compaction (unless compacted at 100 percent of MDD), but also by the method of compaction employed.

In addition to the moisture content being distributed uniformly and adjusted to attain the specified compaction, New South Wales (1997) also requires the moisture content to be within 60 percent to 90 percent of OMC. Queensland (1999) allows adjustment of the moisture content to suit the compaction process, but requires that the layer conform to a specified maximum degree of saturation before it is covered by the next layer or by the surface. A maximum of 65 percent saturation is specified for crushed stone, while 70 percent is specified for natural gravels. These moisture requirements are related to those often specified before the placement of a prime coat, as discussed in Section 4.6.

4.3 LAYER THICKNESS OF SINGLE LIFTS

The concept of maximum specified layer thickness for material compacted in a single lift is based on the restrictions imposed by static rollers. With the advent of the vibratory roller, many of these specifications remained unchanged, while others were altered to provide for the benefit of obtaining deeper compaction with these types of rollers.

4.4 COMPACTION

Texas (Draft 2003) specifies two methods of compaction, i.e., "ordinary compaction" and the "density control" method. The aforementioned specifications make use of proof rolling with correction of weak spots until compaction is secured. Compaction specifications of all the other agencies for base courses are based on density requirements.

Illinois (1997) and New South Wales (1997) require, respectively, compaction to start immediately after placement of the aggregate and compaction to be completed promptly after spreading. Terms such as "blade," "shape," and "trim" are used to describe the actions during the compaction process to obtain the required grade and cross section of the typical section. New South Wales addresses potential problems with uncompacted material that breaks away on the edges by specifying that compaction should start at the low side, or on the sides, and progress to the high point.

While specifications often require "suitable equipment" to obtain the specified density, Illinois specifies the details of the type of compaction equipment and sequence to some extent. Illinois specifies compaction with a tamping roller, or pneumatic-tired roller, or vibratory roller, or a combination, and final rolling with a three-wheel roller.

South Africa (1998) considers preparation of the surface by "slushing" after initial compaction to be an important part of obtaining the required density. This process will be discussed in the following paragraph. Density requirements of various agencies are summarized in Table 4.1.

Table 4.1. Layer Thickness Specifications for Single Lifts.

Road Agency	Minimum Thickness	Maximum Thickness	Increased Allowed
Texas	N/A	N/A	N/A
Arizona	N/A	6" (150 mm)	If approved by engineer
Arkansas	N/A	6" (150 mm)	8" (200 mm) if vibratory roller used
California	N/A	6" (150 mm)	No
Florida	3" (75 mm)	6" (150 mm)	8" (200 mm) after test section.
Illinois	N/A	4" (100 mm)	8" (200 mm) if proved by testing
New Mexico	N/A	6" (150 mm)	N/A
New South Wales	4" (100 mm)	6" (150 mm)	N/A
Queensland	3" (75 mm)	10" (250 mm)	N/A
South Africa	N/A	6" (150 mm)	If approved by engineer

4.5 SURFACE PREPARATION AND FINISH

The compacted layer is typically required to be stable and free from surface laminations, areas of segregation, corrugations, or contamination. In addition, New South Wales (1997), Queensland (1999), and South Africa (1998) describe the required finished surface, respectively, as "tight dense," "with coarse particles slightly exposed," and "firm, stable, with closely knit surface of aggregate exposed in mosaic."

The Queensland specifications point out that the surface may be further watered, drag-broomed and rolled. The process to obtain the surface finish described in the South African specifications is called "slushing and brooming". The process specification requires that slushing commence immediately after compaction. Short sections should be slushed at a time and finished in one continuous process. The sections should be thoroughly watered, rolled, and slushed by the use of steel-wheeled rollers of not less than 12 tons each or pneumatic-tired rollers. The process should continue until all excess fines in the mixture have been brought to the surface and the specified density reached. The damp surface should be swept, or broomed, to remove excess fines and loose aggregate and then left to dry out. After slushing and brooming, the surface-dry section should finally be rolled with a steel-wheeled roller. This process should produce a completed layer that is firm, stable, and with a closely knit surface of aggregate exposed in mosaic.

4.6 PRIMING AND MAINTAINING

Texas (Draft 2003) requires curing of the base before placing the surfacing; the new specification states that the base should be at least 2 percent below optimum moisture content before priming. Florida (2000) specifies the moisture content of the top half of the base to be less than 90 percent of optimum at the time of priming, while South Africa (1998) requires the base to be primed as soon as the moisture in the base drops to 50 percent of optimum. The Queensland (1999) specifications on the moisture allowed in the base at the time of an overlay were discussed in Section 4.2.4.

The general specification of protection and maintenance of the base in a condition that is satisfactory for the placement of a surface are commonly specified. The Illinois (1997) specifications restrict any operation that may cause subgrade material to work into base, such as hauling over the completed base with a soft subgrade after inclement weather. Illinois further requires a minimum maintenance period of 10 days or proof rolling for shorter periods.

4.7 COMPLIANCE TESTING

Information from seven road agencies was used to compile a summary of schedules for sampling and testing, which outline the time or location of sampling and minimum frequency of sampling required for different specified tests or measurements. The schedules are provided in Appendix B. These requirements are often not published as part of the standard specification documents, but in reference construction handbooks or testing manuals published by the agencies. Only general requirements are specified by some agencies, such as Arkansas (1996), while others, such as California (1999), implement more comprehensive requirements for sampling and testing of granular base courses.

The frequency of sampling is generally based on fixed lot sizes, expressed in terms of the number of samples per lot, e.g. one sample per 2000 tons. The California specifications allow for a reduction in the frequency of sampling if the material is uniform and well within specification limits. New South Wales (1997) implements a reduction in the frequency of testing if process control of the material achieved a consistent product. The requirements for initial process control are summarized in the following table.

Table 4.2. Prerequisite for Reduced Frequency of Sampling (New South Wales, 1997).

Test	Process Control Requirements of Material under Production
Plasticity Index and Liquid Limit	All results from the previous six consecutive lots being non-plastic with a lower liquid limit consistently 20 or less.
Modified Texas Triaxial	All results from the previous six consecutive lots being satisfactory. A minimum of one test to be carried out for each 4000 t produced.
Wet/Dry Strength	All results from the previous six consecutive lots tested being satisfactory.
Note: Only more common tests are included in table.	

4.8 SUMMARY

This chapter shows that similar construction practices are used in the different highway agencies surveyed. Little innovation was found in the way base materials are placed. The one exception being Queensland (1999) which has progressed to automated base spreading equipment.

CHAPTER 5

PERFORMANCE MONITORING OF EXPERIMENTAL TEST SECTIONS CONSTRUCTED AS HEAVY DUTY FLEXIBLE PAVEMENTS

5.1 INTRODUCTION

Three experimental road sections that were constructed in Texas between 1997 and 1999 under different projects are included in this study. The base courses of these pavements were constructed to carry high traffic loads and are therefore good candidates for a field study on heavy-duty flexible bases. One of the important features of these sections is that they have been subdivided into smaller test sections with the objective to investigate the performance of different granular bases.

The following table gives introductory information about the three projects, i.e., district and county, year completed, and base types represented.

Table 5.1. Information on Experimental Sections.

Project	District and County	Year Completed	Base Types
SHRP/FHWA SPS-1 Project 4801 US-281	Pharr District, Hidalgo County	1997	Crushed Limestone (CSAB) Dense Graded Aggregate (DGAB) Lime Rock Asphalt (LRA) Caliche (CAL) Crushed Concrete (CCAB)
TxDOT Project 1869 US-77	Corpus Christi District, Nueces County	1997	Yucatan Limestone Lime Rock Asphalt Caliche + lime Caliche + cement
TxDOT Project 7-3931 FM-1810	Fort Worth District, Wise County	1999	Regular Grade 1 Large Stone Gradation

This part of the report presents the processed information gathered during the past few years on these test sections and assesses the performance parameters obtained in relation to those currently used for design. This information will be used to complement the following stages of this project, which will include the development of prototype specifications and laboratory and field validation studies.

5.2 US-281 TEST SECTIONS IN THE PHARR DISTRICT

5.2.1 Background

This project was originally part of the Long-Term Pavement Performance (LTPP) program developed by the Strategic Highway Research Program (SHRP) and Federal Highway Administration (FHWA). This LTPP program includes 16 projects located across the U.S. referred to as Specific Pavement Studies (SPS), designed to study the factors that affect pavement performance.

This experimental section is located in the southbound lanes of US-281 in Hidalgo County north of McAllen in the Rio Grande valley. The project site falls in the dry no-freeze climatic zone as depicted in Figure 2.1. The rainfall in this area is very low, averaging less than 12 inches per year. The natural soil is sand; therefore, with the low rainfall and free-draining subgrade soils, moisture intrusion from below is not a concern.

The project section is a four-lane divided highway with 12-foot wide lanes and 10-foot wide outside shoulders. The final report on the SPS-1 project indicates that the estimated traffic at that time included 32.8 percent heavy trucks with annual average daily traffic in two directions of 10,180 vehicles. The designs were based on a total of 10 million 18 kip equivalent single axles over a structural design period of 20 years. The road was opened to traffic in April 1997.

A total of 20 smaller test sections formed part of the original project, of which eight were considered to be supplemental at the time, as they had been included specifically for evaluating various base materials by the Texas Department of Transportation. These eight experimental sections are the focus of the current study (FWHA/LTPP, 1997).

5.2.2 Construction

Table 5.2 outlines the detailed locations and structures of the sections under consideration. Additional information can be obtained from the final project report, FWHA/LTPP (1997).

Table 5.2. Positions and Pavement Structures of US-281 Test Sections.

Section (Cell ID)	Sequence (South)	Pavement Structure	Begin Station (foot × 100)	End Station (foot × 100)
480113	1	4” AC & ACB 8” Dense Graded Aggregate Base (DGAB) 12” Subgrade stabilized with 2% lime	1558 + 00	1551 + 00
480114	5	7” AC & ACB 12” DGAB 12” Subgrade stabilized with 2% lime	1522 + 00	1515 + 00
480160	9	5” AC & ACB 10.5 “ Lime Rock Asphalt (LRA) 12” Subgrade stabilized with 2% lime	1460 + 00	1453 + 00
480161	8	5” AC & ACB 8.5” LRA 12” Subgrade stabilized with 2% lime	1468 + 00	1461 + 00
480162	7	5” AC & ACB 8.5” Crushed Limestone Aggregate Base (CSAB) 12” Subgrade stabilized with 2% lime	1482 + 00	1475 + 00
480163	6	5” AC & ACB 10.5” CSAB 12” Subgrade stabilized with 2% lime	1490 + 00	1483 + 00
480164	3	5” AC & ACB 10.5” Crushed Concrete Aggregate Base (CCAB) 12” Subgrade stabilized with 2% lime	1542 + 00	1535 + 00
480165	2	5” AC & ACB 10.5” CCAB 12” Subgrade stabilized with 2% lime	1550 + 00	1543 + 00
480166	10	5” AC & ACB 14” Caliche + ½% lime (CAL) 12” Subgrade stabilized with 2% lime	1452 + 00	1445 + 00
480167	4	5” AC & ACB 14” CAL 12” Subgrade stabilized with 2% lime	1530 + 00	1523 + 00

5.2.3 Materials

Table 5.3 summarizes the tests on samples from the different base course materials incorporated during construction.

Table 5.3. Base Material Properties for Test Sections on US-281.

Parameter Description		DGAB	LRA	CSAB	CCAB	CAL
Gradation		Typ. D, Grade 6 (Valley Caliche)[1]	Typ. A, Grade 1 (Vulcan)	Typ. A, Grade 1 (Global/ Vulcan)	"Flexbase" (Frontera)	"Flexbase" (Guerra)
English	Metric					
4"	102 mm					
3"	75 mm					
2"	50 mm				0 (100)	0 (100)
1 ¾"	45 mm		0 (100)	0 (100)		
3/2"	37.5 mm	0 (100)[2]				
7/8"	22.4 mm	17 (83)	32 (28)	22 (18)		
½"	12.5 mm				49 (51)	48 (52)
3/8"	9.5 mm	52 (48)	51 (49)	32 (28)		
No. 4	4.75 mm	65 (35)	58 (42)	63 (37)	66 (44)	70 (30)
No. 40	0.425 mm	83 (17)	83 (17)	83 (17)	78 (22)	78 (22)
No. 200	0.075 mm	85 (15)				
Fines	PI	2.9	0.6	3.8	13	12
	LL	18.1	24	21.6	34.7	35.2
Wet Ball Mill, %		-	23	28.8	30	27
Increase in % fines (No. 40)		-	15.8	10.8	-	-
Wet Ball PI		-	-	-	11	9.6
Strength (psi) at 0 psi lateral pressure		-	60.4	90.7	-	-
Strength (psi) at 15 psi lateral pressure		-	184.5	198.3	173.2	171
Strength (psi) at 15 psi lateral pressure after modification		-	-	-	233.5	204.1
Maximum Dry Density, MDD (pcf)		133.9	117.4	120.9	122.5	115.6
Optimum Moisture Content, % OMC		4.4	8.1	8.4	10.3	15

Notes:
1) Brackets indicate source.
2) Brackets indicate % passing.
3) Modification includes addition of 0.5% lime.

Performance problems were encountered early in the life of these experimental sections. Many of the sections rutted very badly in the first few years. A full forensic study was conducted by TxDOT in 2000, under the direction of Dr. DarHao Chen (Chen, 2001). During that study tests were conducted on all pavement layers in several of the test sections. It was determined that the rutting was primarily coming from the top 1.5 inches of asphalt: it contained too much asphalt, and changes had been made to the aggregates used. In 2001 this layer was milled off and replaced with new surfacing. No problems were encountered with either the base or subbase layers. During trenching and sampling of these sections it was determined that:

- The subbase layer was a lot thicker and stiffer than indicated on the plans; in some cases the layer was stiffer than the base. The average subbase thickness was 20 inches.
- The moisture contents of the bases were substantially less than OMC.

- Dynamic cone penetrometer tests were conducted on three sections (CSAB, DGAB, and LRA). Using these data the following estimates for modulus were made: 40, 72, and 105 ksi, respectively.

Base samples were obtained from trenches excavated on selected sections of US-281 in 2000. These samples have been stored at TTI. A preliminary investigation was conducted to evaluate the moisture susceptibility of these materials. Dielectric values were determined following the proposed Texas tube suction test procedure. Preliminary results on the three aggregates tested show potentially high moisture susceptibility for both the dense graded aggregate (DGAB) and lime rock asphalt (LRA) material, with dielectric values in excess of 15. For all three aggregates the final moisture content after 10 days was above the optimum moisture content of the material, indicating that if moisture is available then these materials may have a problem. Despite the magnitude of these values, moisture is not expected to be a major concern in this dry area located in the Rio Grande Valley. The significance of these results will be discussed in detail later in this section.

Table 5.4. Laboratory Dielectric Constant Values for Base Samples from US-281.

Base Aggregate	Sample	Time (days)	0.0	1.6	2.6	3.6	4.7	5.9	9.8
DGAB	1	Moisture (%)	3.0	5.4	5.5	5.6	5.7	5.7	5.8
		DEC	5.8	16.7	16.4	15.6	18.0	15.4	16.7
	2	Moisture (%)	2.9	5.4	5.5	5.5	5.6	5.6	5.7
		DEC	5.2	11.9	13.3	14.3	12.7	15.3	13.2
CSAB	1	Moisture (%)	3.9	8.9	9.1	9.2	9.3	9.5	9.6
		DEC	4.1	7.9	9.9	8.1	11.1	9.4	11.3
	2	Moisture (%)	3.9	9.7	9.8	10.0	10.1	10.2	10.4
		DEC	5.3	13.4	17.0	13.5	14.6	11.3	13.9
LRA	1	Moisture (%)	4.1	7.9	8.1	8.2	8.3	8.5	8.6
		DEC	6.2	14.2	16.4	17.3	17.9	16.7	16.1
	2	Moisture (%)	4.8	7.5	7.7	7.9	8.1	8.3	8.5
		DEC	9.0	12.1	12.6	12.8	17.6	16.2	16.8

5.2.4 Post-Construction Condition Surveys

No distress at time of survey.

5.2.5 Ground Penetrating Radar

Table 5.5 summarizes thicknesses and dielectric constant values for base layers of different sections. The June 2000 dielectric constant (DEC) values are generally higher than the August 2000 values. Despite the fact that the caliche sections have the highest DEC values, they are not considered critical. The 2002 values for the DGAB are exceptionally low. Detailed processed results are presented in Appendix B.

Table 5.5. Base Dielectric Constants and Layer Thicknesses for US-281.

Section	Statistic	June 2000	August 2002	
		Dielectric Constant	Dielectric Constant	Thickness (in)
113 (DGAB)	Average	6.0	5.1	9.4
	CoV	0.05	0.04	0.11
	Minimum	5.4	4.7	7.0
	Maximum	7.0	5.8	12.0
114 (DGAB)	Average	6.6	5.6	14.3
	CoV	0.03	0.05	0.07
	Minimum	6.1	4.8	11.1
	Maximum	7.2	6.2	16.1
160 (LRA)	Average	8.9	7.1	12.9
	CoV	0.06	0.05	0.08
	Minimum	7.8	6.2	10.5
	Maximum	10.3	8.7	14.9
161 (LRA)	Average	8.7	7.3	10.5
	CoV	0.06	0.05	0.07
	Minimum	7.5	5.4	8.8
	Maximum	9.8	8.2	12.3
162 (CSAB)	Average	8.4	7.3	10.8
	CoV	0.08	0.08	0.05
	Minimum	6.8	5.1	9.2
	Maximum	10.2	9.6	12.3
163 (CSAB)	Average	8.5	7.0	12.8
	CoV	0.08	0.07	0.07
	Minimum	7.2	5.7	9.0
	Maximum	10.6	8.8	15.2
164 (CCAB)	Average	8.0	6.6	9.0
	CoV	0.14	0.15	0.12
	Minimum	6.7	5.4	6.2
	Maximum	13.9	11.5	13.2
165 (CCAB)	Average	7.5	6.6	8.2
	CoV	0.11	0.10	0.11
	Minimum	6.2	5.4	6.5
	Maximum	11.1	9.9	11.2
166 (CAL)	Average	12.2	11.2	11.1
	CoV	0.09	0.14	0.07
	Minimum	9.8	6.3	9.5
	Maximum	14.5	14.6	13.7
167 (CAL)	Average	10.4	8.5	6.9
	CoV	0.14	0.12	0.07
	Minimum	8.1	4.8	5.6
	Maximum	15.0	11.8	8.1

5.2.6 Deflection Surveys

Deflections have been measured with falling weight deflectometer (FWD) devices since 1997. Backcalculated layer moduli are presented graphically over time in Figure 5.1. The ranking of the bases in terms of stiffness can easily be distinguished from the graph. The caliche (CAL) and limestone bases (CSAB) show little variation and are on the low end of the stiffness scale, 40 to 60 ksi. The limerock asphalt (LRA) gradually increased with time from an average value of 220 ksi in 1997 to a value on the order of 380 ksi in 2002. The crushed concrete bases (CCAB) produced relatively high moduli, which appear to have stiffened with time. The stiffening with time for the CCAB and the LRA base is attributed to self-cementing of the aggregate particles. Indeed, during trenching of the LRA section a diamond saw was used and the face of the saw cut looked very similar to that observed with asphalt stabilized bases. A similar trend is observed for the dense graded aggregate base. Graphical presentations of variation of moduli over each section are provided in Appendix B.

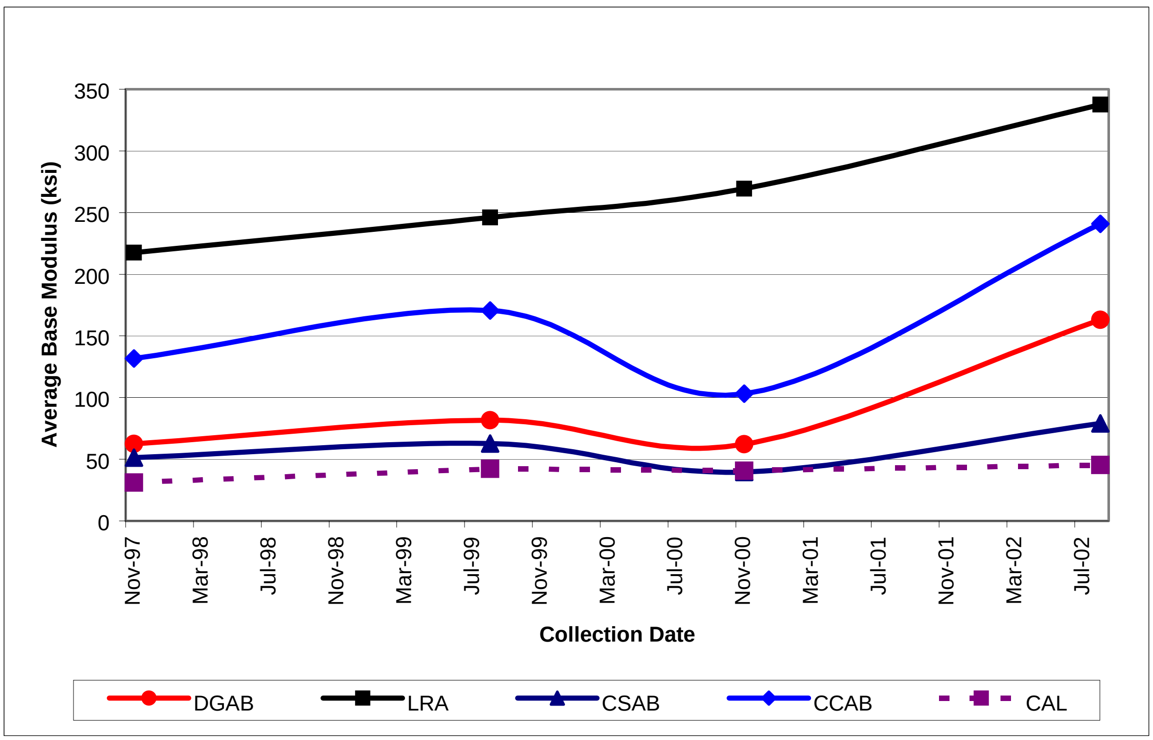


Figure 5.1. Base Layer Moduli for Sections on US-281 from 1997 to 2002.

5.2.7 Summary

TxDOT milled and resurfaced these experimental sections in 2001, so there is no visual surface distress on any of the sections. Currently, the evaluation of the performance of these bases has to be inferred from the nondestructive test data. The intention of the project designers was that all of these bases would be TxDOT Class 1 bases and that the district would be able to compare the performance of the locally available and widely used caliche bases with that of alternate and more expensive base materials. Based on the results obtained, the following conclusions are drawn:

- The Caliche appears to be the worst performing base; it has the lowest in-place modulus and highest dielectric value. In many areas of the state, designers use 70 to 80 ksi for Class 1 bases when placed on stabilized layers; however, the average value for the caliche is in the 40 to 50 ksi range.
- Both the LRA and crushed concrete provide moduli values well above those typically used for flexible base material. The maximum value ever found for normal flexible bases in Texas is in the 100 to 125 ksi range. This supports the conclusion that these bases are "setting up" themselves.
- All of these bases were classified as Class 1 materials but the insitu performances are substantially different. The most probable explanation for this is the self-cement action of two bases and the very low insitu moisture contents of all the bases especially the DGAB. These results are clearly impacted by the following factors: (a) the favorable subbase support (very thick stiff layer), (b) climate (little or no rainfall), and (c) free-draining subgrade (pure sand).
- The laboratory dielectric results are also interesting. The TST was unable to rank these bases in terms of performance. In fact, all of the materials did poorly on the TST. The lack of available moisture in this experiment and the thick pavement layers above and below the base are clearly factors. Several interesting theories exist to explain some of the variations. One thought was that some of the difference may have been caused by the differences between laboratory molding at 70 degrees and field compaction at temperatures in excess of 100 degrees. For LRA material it was theorized that the field conditions could activate the free asphalt within the sample and that this could coat and waterproof the fines in the LRA bases. No validation of this has been performed. The main conclusion here is that TxDOT needs to review how best to implement the TST criteria in the new 2003 specification; clearly, in some areas of the state poor performance on the TST does not translate to poor performance on the roadway.

5.3 US-77 TEST SECTIONS IN THE CORPUS CHRISTI DISTRICT

5.3.1 Background

The Corpus Christi District initiated the construction of experimental test sections on US-77 to establish design parameters for pavement structures historically built in this district and to investigate the cost effectiveness of alternative structures with different base types (Sebesta and Scullion, 2000). These test sections would also represent heavy-duty flexible pavements capable of accommodating loads associated with interstate highways. A control section and three treatment sections were constructed and have been monitored since 1997.

The four sections are located on US-77 in Nueces County near Robstown. Although this experimental site is within the dry no-freeze climatic zone, it is located near the Nueces and Corpus Christi Bays. It is therefore considered that the influence of moisture may be more pronounced on a micro climatic scale in comparison with the US-281 site.

Each section is approximately 1000 feet in length and includes the northbound and southbound lanes of the four-lane divided highway. The experimental sections were designed to carry an estimated 15 million equivalent standard axles (ESALs) over a design life of 20 years.

5.3.2 Construction

Information on the locations and pavement structures of these sections is provided in Table 5.6.

Table 5.6. Positions and Pavement Structures of US-77 Test Sections.

Section (No.)	Sequence (North)	Pavement Structure	Begin Station (feet × 100)	End Station (feet × 100)
4	1	8" AC Surface 18" Caliche stabilized with 2% lime 12" Subgrade stabilized with 4% lime	160 + 00	170 + 00
1	2	8" AC Surface 18" Caliche stabilized with 4% cement 12" Subgrade stabilized with 4% lime	170 + 00	180 + 00
2	3	8" AC Surface 18 " Yucatan Limestone 12" Subgrade stabilized with 4% lime	180 + 00	190 + 00
3	4	8" AC Surface 18" Lime Rock Asphalt 12" Subgrade stabilized with 4% lime	150 + 00	160 + 00

Notes:
- All sections across northbound and southbound lanes.
- Section numbers are the original numbers as shown on the plans.
- Section 4 was originally the control section.

5.3.3 Materials

At the time of writing this report, laboratory test information was available on only one of the four materials used in this project; this is shown below in Table 5.7. No field samples have been taken from this section. The base widely used in this district is caliche with 2% lime. The other three bases are viewed as experimental.

Table 5.7. Base Material Properties for Test Sections on US-77.

Parameter Description Gradation English	Metric	Caliche + 2% Lime	Caliche + 4% Cement	Yucatan Limestone	LRA
4”	102 mm				
3”	75 mm				
2”	50 mm				
1 ¾”	45 mm				0 (100)
3/2”	37.5 mm				
7/8”	22.4 mm				14 (86)
1/2”	12.5 mm				
3/8”	9.5 mm				42 (58)
No. 4	4.75 mm				59 (41)
No. 40	0.425 mm				84 (16)
No. 200	0.075 mm				
Fines	PI				Non-Plastic
	LL				-
Wet Ball Mill, %					33.5
Increase in % fines (No. 40)					11.0
Wet Ball PI					-
Strength (psi) at 0 psi lateral pressure					46.0
Strength (psi) at 15 psi lateral pressure					190.1
Strength (psi) at 15 psi lateral pressure after modification					-
Maximum Dry Density, MDD (pcf)					115.6
Optimum Moisture Content, % OMC					6.0

5.3.4 Post-Construction Condition Surveys

No distress at time of survey.

5.3.5 Ground-Penetrating Radar

Table 5.8 indicates that the dielectric constant (DEC) values obtained from the 2002 survey are not significantly different from those obtained in 1997. It is evident that higher DEC values are encountered on the caliche sections, indicating the affinity for moisture of these type of bases. It is of interest to note that the cement-stabilized section tends to show higher values, with an average DEC on the northbound section of 13.3 and a maximum DEC in excess of 15 on the southbound section. Detailed plots of the base dielectric values are presented in Appendix C.

Table 5.8. Base Dielectric Constants and Layer Thicknesses for US-77.

Section[1]	Statistic	July 1997		August 2002	
		Northbound	Southbound	Northbound	Southbound
No .4 CAL+2% lime	Average	9.9	12.3	11.0	9.6
	CoV	0.18	0.08	0.11	0.12
	Minimum	7.7	9.7	6.9	7.2
	Maximum	13.9	15.1	15.1	14.9
No. 1 CAL+4% cement	Average	10.2	12.2	13.3	12.0
	CoV	0.18	0.09	0.08	0.13
	Minimum	4.9	4.9	8.0	8.8
	Maximum	14.8	15.2	14.9	17.9
No. 2 (Yucatan)	Average	7.6	8.1	8.5	8.6
	CoV	0.13	0.14	0.07	0.16
	Minimum	6.2	4.3	7.1	5.6
	Maximum	13.2	12.3	11.1	13.8
No. 3 (LRA)	Average	6.6	7.1	7.8	8.0
	CoV	0.10	0.10	0.07	0.09
	Minimum	5.8	5.8	6.5	6.5
	Maximum	10.7	10.0	10.3	13.2

Note: 1) Results for slow lane presented.
2) Base thickness not detected by ground penetrating radar (GPR), value fixed at 18 in.

5.3.6 Deflection Surveys

As part of the economic evaluation of the different bases constructed, deflection data have been collected frequently since the pavement was opened to traffic in 1997. Gonzalez and Hinojosa (2000) backcalculated the moduli and compiled a graph showing the variation of moduli over time. The latter is presented in Figure 5.2. The graph indicates that a clear difference exists between the stiffness values of the base courses investigated. The control section, constructed with caliche stabilized with 2% lime, is on the lower end of the spectrum, while the limerock asphalt course is on the higher end of the spectrum. The moduli of both the stabilized caliche sections and the limestone section tend to vary marginally with time. The limerock asphalt course, however, appears to stiffen with time.

Figure 5.3 presents moduli determined from deflections collected between 2000 and 2002. The stabilized caliche sections exhibit a similar low-variation, relatively constant pattern. The average modulus of the limestone course seems to have decreased over the last 2 years. Although the limerock asphalt modulus shows large variations, it maintained the gained modulus of more than 150 ksi over this period of time. A detailed presentation of the variation of modulus in each section is presented in Appendix C.

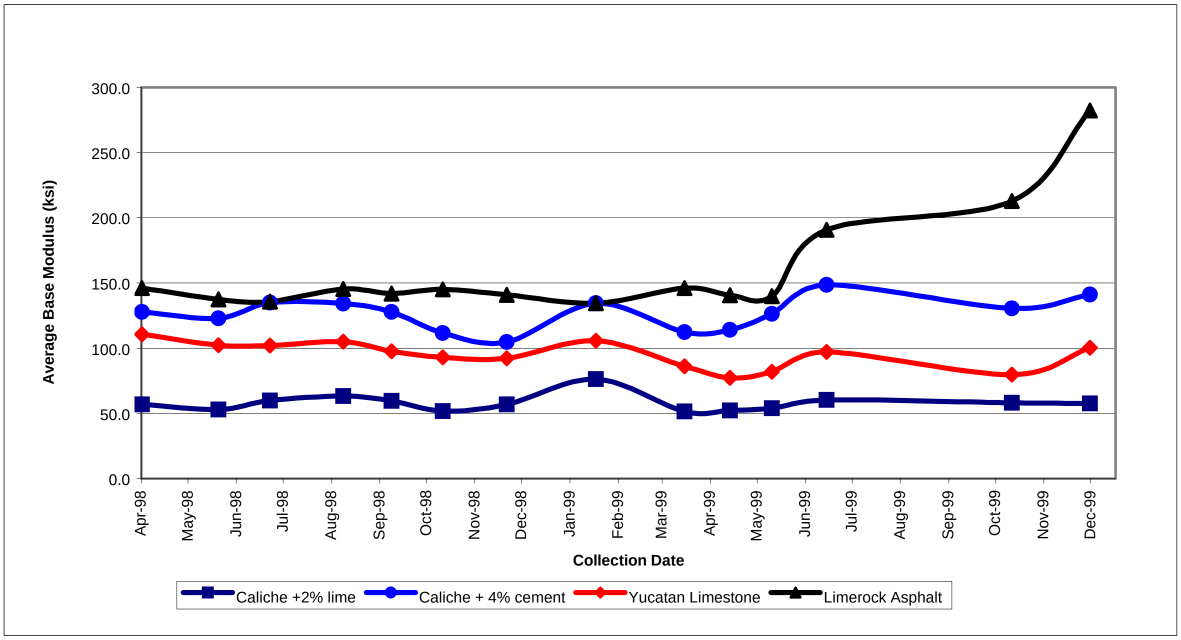


Figure 5.2. Base Layer Moduli on US-77 from 1998 to 1999 (Gonzalez and Hinojosa, 2000).

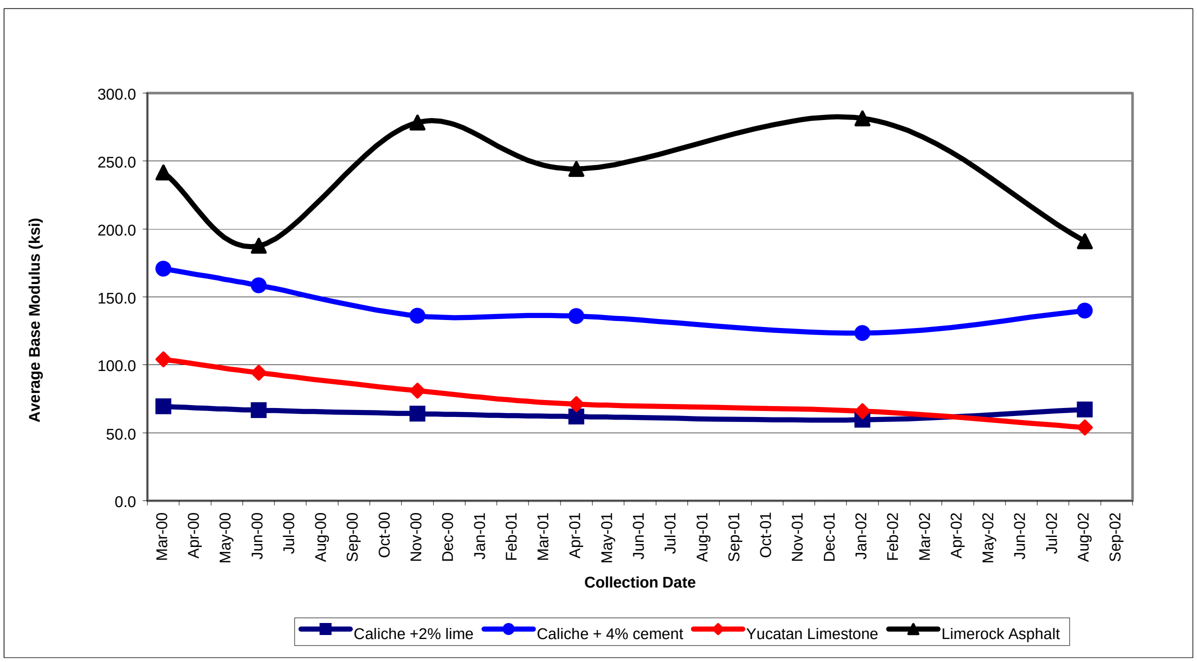


Figure 5.3. Base Layer Moduli on US-77 from 2000 to 2002.

5.3.7 Summary

- The main conclusions from the nondestructive data presented above are as follows:

- The caliche materials stabilized with lime have the lowest base moduli of all of the materials used.
- The LRA and caliche stabilized with cement have moduli values that are consistently higher than those typically found for flexible base materials in Texas.
- The reason for the reduction in field moduli for the Yucatan limestone is not known at this time. This should be the subject of further study. The section has developed some longitudinal cracks that are not thought to be load related but may be associated with edge drying. However, these cracks may permit moisture to enter the base layers. The GPR dielectric values of all of the bases have increased over time, but the values are still thought to be reasonable. Clearly, it would be beneficial to take base samples from these sections.

5.4 FM-1810 TEST SECTIONS IN THE FORT WORTH DISTRICT

5.4.1 Background

Research Project 3931 was conducted in-house by the Texas Department of Transportation in cooperation with the Federal Highway Administration from September 1998 to December 2000 (Williammee et al., 2000). Several roadway failures were reported in the Fort Worth District that were thought to have originated in the flexible base course. This district had not adopted the triaxial classification as part of the specifications of base course materials, and in recent years it was recognized that their standard flexible base requirement did not provide adequate supporting structures for overlying courses. Although the district used the gradation specification of the 1993 Texas Standard Specifications, this situation revealed the detrimental effect that a high fines content can have on the strength of the aggregate mass if it is not indirectly controlled by another means, such as a triaxial test parameter. The current specification allows a variation of up to 35 percent of the fines, which includes the material passing the No. 200 sieve (0.075 mm).

The main objective of this research was to investigate the influence of fines on strength and to propose a new gradation envelope. A proposed large stone gradation and a regular gradation were used in the base courses of two experimental sections constructed in August 1999. These sections of FM-1810 are located in Wise County, northwest of Decatur, near Chico. Within the broad climatic regions defined in Chapter 2, this project can be categorized under intermediate freeze-thaw.

The experimental sections of FM-1810 carry a large amount of heavy trucks, as FM-1810 serves as an access route to the Pioneer quarry. The design was based on an average annual daily traffic of 5280 vehicles in 2000 and 8480 vehicles in 2020, with 29.3 percent trucks (which is a low estimate). The 20-year design ESAL was 6.38 million. The road was opened to traffic in September 1999, and researchers estimate that the total ESALs carried to date is approximately 500,000.

5.4.2 Construction

Table 5.9 summarizes the pavement structures and positions of the experimental sections.

Table 5.9. Positions and Pavement Structures of FM-1810 Test Sections.

Section (North)	Pavement Structure	Begin Station	End Station
1	7.5" ACP Surface 12" Large aggregate crushed stone 12" Cement stabilized subgrade	1 + 1000	1 + 900
2	7.5" ACP Surface 12" Regular graded crushed stone 12" Cement stabilized subgrade	4 + 000	4 + 800
Note: All sections are located across northbound and southbound lanes.			

5.4.3 Materials

Researchers performed laboratory tests on aggregate mixes prepared to represent the limits currently used, and on modified gradations. Aggregate from three local producers, i.e., Pioneer-Bridgeport, Vulcan Gilbert, and Arnold Blum, was used. Based on these results, a new flexible base gradation envelope was proposed with improved shear strength as indicated by its triaxial class.

The results obtained during the first stage of this laboratory investigation showed a triaxial classification range between 1.9 and 3.5 for aggregate obtained from the local producers. The second stage of this investigation involved modification of the gradations, which produced a narrower classification range of between 2.1and 2.5. A new large stone gradation was proposed that would narrow the wide strength range, improve strength, and potentially lower cost because of less crushing. The proposed gradation, with maximum aggregate size of 4 inches, was tested and rendered a triaxial classification of 1.0. Research committee members agreed that this gradation would be used in the demonstration test section. The Pioneer-Bridgeport source was selected to produce material for construction. Table 5.10 summarizes the results obtained for the two bases used.

Table 5.10. Base Material Properties for Test Sections on FM-1810.

Parameter Description		Section 1: Station 1 + 000 to Station 1 + 900 (meter)		Section 2: Station 4 + 000 to Station 4 + 800 (meter)		
Gradation		Proposed Large Stone Gradation		Regular Type A, Grade 6		
English	Metric (mm)	Specification	Constructed	Specification	Lower limit	Upper limit
4”	100	< 100	100	-	-	-
3”	75	80 – 100	99	-	-	-
	45	50 – 75	70	95 – 100	95	100
3/2”	37.5	-	-	-	-	-
	22.4	-	-	65 – 95	65	95
3/8”	9.5	15 – 40	54	-	-	-
No. 4	4.75	-	-	25 – 60	25	60
No. 40	0.425	0 - 10	9	20 – 35	20	35
No. 200	0.075	-	-	-	18	28
Fines	PI	Max. 12 Min. 0	NP	Max. 12 Min. 4	6	6
	LL	Max. 45	NP	Max. 45	22	22
Wet Ball Mill, %		Max. 50	-	Max. 50	-	-
Increase in % fines (No. 40)		Max. 20	-	Max. 20	-	-
Texas Triaxial Class		-	1.0	-	1.9	3.5
Strength (psi) at 0 psi lateral pressure		-	82.7	-	56.5	9.0
Strength (psi) at 15 psi lateral pressure		-	253.4	-	158.2	90.9
Maximum Dry Density, MDD (pcf)		-	138.1	-	126.3	130.2
Optimum Moisture Content, % OMC		-	6.4	-	5.9	4.9

5.4.4 Post-Construction Condition Surveys

No distresses at time of survey.

5.4.5 Ground Penetrating Radar

A GPR survey was carried out in July 2002, and results are summarized in Table 5.11. Detailed color subsurface images of the pavement structures are presented in Appendix D. The average dielectric values are generally between 7.5 and 9.0, which is indicative of good base conditions. The color images also show no problem with the bases. The HMA surface, however, shows signs of defective areas in the westbound lane of the large stone aggregate base section. These problems are thought to be associated with longitudinal joint compaction problems. A few longitudinal cracks are observed on the surface, but these currently do not extend into the base.

Table 5.11. Base Dielectric Constants and Layer Thicknesses for FM-1810.

Section	Statistic	Eastbound		Westbound	
		Dielectric Constant	Layer Thickness (in)	Dielectric Constant	Layer Thickness (in)
No. 1 (Large Stone)	Average	7.6	11.6	8.8	12.2
	CoV	0.07	0.12	0.05	0.05
	Minimum	6.5	7.7	7.2	10.2
	Maximum	10.5	15.4	10.1	14.1
No. 2 (Regular)	Average	8.3	11.7	7.6	12.2
	CoV	0.06	0.06	0.06	0.15
	Minimum	7.0	9.9	7.0	4.1
	Maximum	10.2	13.7	9.3	16.9

5.4.6 Deflection Surveys

Backcalculated elastic moduli for the two experimental sections on FM-1810 were reported by Williammee et al. (2000), determined from FWD collected in September 1999 and October 2000. Deflections were measured again in July 2002. The average moduli for the two directions are plotted in Figure 5.4. Although the initial modulus of the regular graded base was initially higher, it decreased significantly with time to an average value of 61 ksi, while that of the large stone aggregate base increased gradually with time and seems to stabilize at an average value of 83 ksi.

Data for all years show that the moduli of both sections in the westbound lanes are higher than those in the eastbound lanes. The 2002 results are 93 and 74 ksi, respectively, for the large stone base, and 73 and 49 ksi, respectively, for the regular graded base. The 2002 moduli are presented in more detail in Appendix D of this report.

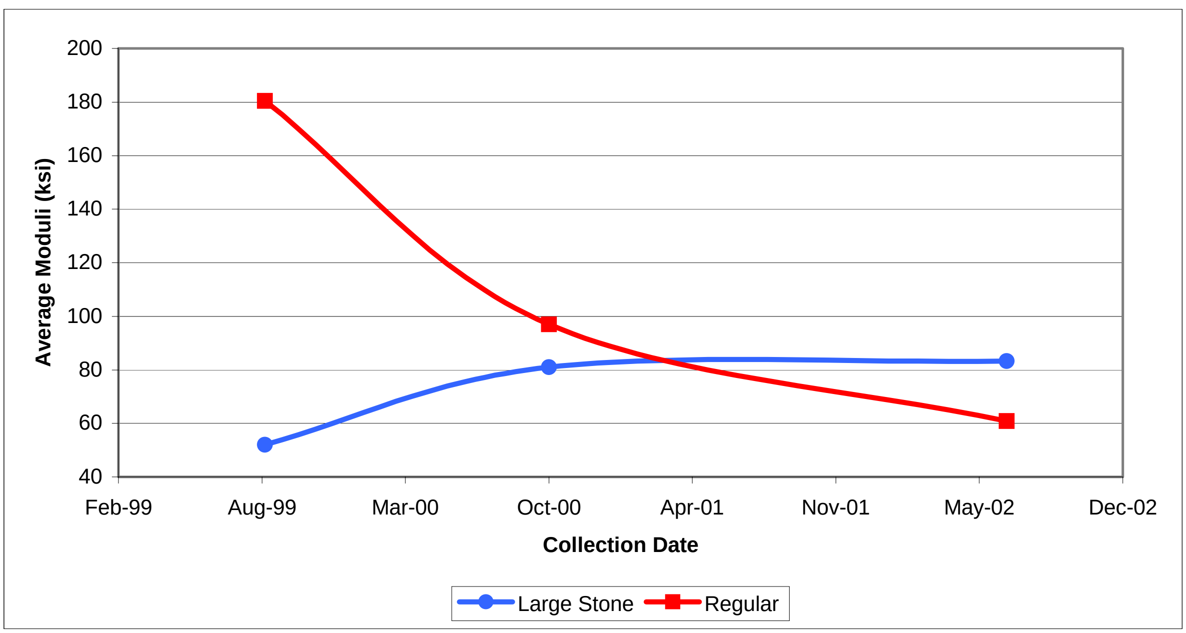


Figure 5.4. Base Layer Moduli on FM-1810 from 1999 to 2002.

5.4.7 Summary

The trends shown in Figure 5.4 are very interesting. The regular base with high minus 200 content (>20 percent) had a very high initial pavement stiffness. It is theorized that this may be related to the practice of "slush rolling," whereby the process of watering and rolling with a steel wheel roller, causes the excess fines to migrate to the upper base. When they dry they create a dense stiff layer, which was measured with the FWD to have a backcalculated modulus of 180 ksi. However, over time this modulus drops. After almost 3 years in service the average backcalculated modulus has dropped to around 60 ksi. The large stone base shows a different trend. The initial modulus is low, on the order of 50 ksi, but over time this increased gradually to a value of over 80 ksi after almost 3 years. This could indicate that these bases need trafficking to consolidate, and with time they will provide a dense stiff support layer.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

Chapters 2, 3, and 4 of this report present an extensive literature review of heavy-duty base specifications, construction practices, and recent research findings from several DOT's in the U.S. and from highway agencies in South Africa and Australia. Conclusions and recommendations are made in the following areas.

Improvements to the Current Texas Triaxial Procedure (Method 117E)

- The current Class 1 specification requires a material to meet a strength requirement at both 0 and 15 psi confining strength. TxDOT should consider replacing the 0 psi requirement with a 5 psi requirement. Class 1 materials will require a compressive strength of greater than 105 psi at 5 psi confining. This is consistent with recent NCHRP recommendations and district experience. The current 0 psi requirement can eliminate excellent materials.
- A 10-day capillary rise requirement should be mandatory for all heavy-duty base materials.

Comments on the New Draft Specification (Item 245 – Year 2003)

- Currently, TxDOT is the only agency that does not limit the amount of minus 200 material in heavy-duty bases. The new specification, with a maximum value of 10 percent, is in line with other agencies. One observation is that the colder the climate the lower the minus 200 limit. In many areas in the northeast the minus 200 must be less than 5 percent. Experience has shown that the higher the minus 200 the more potential exists for freeze thaw problems.
- Texas is the only agency proposing to use cohesion and angle of internal friction in specifications. Other agencies rely on empirical values such as CBR or R-values. TxDOT should carefully evaluate the use of two parameters instead of one.
- The statewide implementation of the dielectric test should include allowances for climate, rainfall, and subgrade type. In many areas the suction properties of a flexible base will have a major impact on pavement performance, in others, the impact will not be as important.

The specification requiring that the base be at least 2 percent below OMC before sealing will potentially eliminate some costly failures like those that have occurred in recent years. Moisture testing should be mandatory if the project experienced significant rainfall prior to sealing.

Conclusions from the US-281 Experiment Sections

The US-281 project is the largest Class 1 base project placed in Texas. Five different bases were used in duplicate sections. The sections were constructed in 1997, and each has carried identical traffic.

- The caliche appears to be the worst performing base; it has the lowest in-place modulus and highest dielectric value. In many areas of the Texas pavement design, engineers use 70 to 80 ksi for Class 1 bases when placed on stabilized layers; however, the average value for the caliche is in the 40 to 50 ksi range.
- Both the LRA and crushed concrete provide moduli values well above those typically used for flexible base material. The maximum value found for normal flexible bases in Texas is in the 100 to 125 ksi range. This supports the conclusion that these bases are "setting up" themselves.
- With regard to this research study, all of these bases on US 281 were classified as Class 1 materials, but the in-situ performances are substantially different. The most probably explanation for this is the self-cement action of two bases and the very low in-situ moisture contents of all the bases especially the DGAB. These results are clearly impacted by the following factors; (1) the favorable subbase support (very thick stiff layer), (2) climate (little of no rainfall), and (3) free draining subgrade (pure sand).
- The laboratory dielectric results are also interesting. The TST was unable to rank these bases in terms of performance. In fact, all of the materials did poorly on the TST. The lack of available moisture in this experiment and the thick pavement layers above and below the base are clearly factors. Several interesting theories exist to explain some of the variations. Some of the difference may have been caused by the differences between laboratory molding at 70 degrees and field compaction at temperatures in excess of 100 degrees. With the LRA material it was theorized that the field conditions could activate the free asphalt within the sample and that this could coat and waterproof the fines in the LRA base. No validation of this theory has been performed. The main conclusion here is that TxDOT needs to review how to best implement the TST criteria in the new 2003 specification; clearly, in some areas of the state poor performance on the TST does not translate to poor performance on the roadway.
- Samples of these bases are already stored at TTI. A testing plan to further identify and explain the differences in field performance should be undertaken in year 2 of Study 4358.

Conclusions from the Experimental Sections on US-77

The US-77 experiment was placed to compare the performance of the widely used caliche base treated with 2% lime to three alternatives. The goal was to identify bases that could be used with confidence in future high-volume facilities.

- The caliche materials stabilized with lime have the lowest base moduli of all of the materials used.
- The LRA and caliche stabilized with cement have moduli values that are consistently higher than those typically found for flexible base materials in Texas.
- The reason for the reduction in field moduli for the Yucatan limestone is not known at this time. This reduction should be the subject of further study. The section has developed some longitudinal cracks that are not thought to be load related but associated with edge drying. However, these cracks may be permitting moisture to enter the base layers. The GPR dielectric values of all of the bases have increased over time, but the values are still thought to be reasonable. Clearly, it in would be beneficial to take base samples from these sections.

- Samples should be taken from these sections and incorporated into the testing plan for Study 4358.

Conclusions from of the Experimental Sections on FM-1810

- The experimental large stone base had a relatively low modulus value directly after construction. From FWD analysis a value of around 50 ksi was calculated from field deflection data. However over time this has increased to a value of over 80 ksi.
- The regular flexible base with the high minus 200 content (>20 percent) initially had a very high base modulus of over 180 ksi. This has gradually decreased with time, and after almost 3 years it has fallen to less than 60 ksi.
- The large stone base appears to have provided an excellent pavement foundation layer.

REFERENCES

Allen, J.J. and Tompson, M.R. "Resilient Response of Granular Materials Subjected to Time-Dependent Lateral Stresses". *Transportation Research Record 510*, Transportation Research Board, National Research Council, Washington, D.C., 1974.

American Association of State Highway and Transportation Officials (AASHTO). *AASHTO Guide for Design of Pavement Structures*, Washington, D.C., 1993.

Arizona Department of Transportation, *Standard Specifications for Road and Bridge Construction*. Arizona, USA, 1996.

Arkansas State Highway and Transportation Department, *Standard Specifications for Highway Construction*. Arkansas, USA, 1996.

Audo-Osei, A., Little, N., and Lytton, R.L. "The Anisotropic Resilient Properties of Unbound Granular Materials." *ICAR, 8th Annual Symposium Proceedings*, Denver, Colorado, USA, April 12 – 14, 2000.

Barksdale, R.D., and Itani, S.Y. "Influence of Aggregate Shape on Base Performance." *Transportation Research Record 1227, Rigid and Flexible Design and Analysis*, Transportation Research Board, National Research Council, Washington D.C., 1989, pp. 173 –182.

California Department of Transportation, *Standard Specifications*, 1999. http://www.dotca.gov/

Chen D.H., Bilyeu J., Scullion, T., Lin, D., and Zhou, F. "Forensic Evaluation of Premature Failures of the Texas SPS1 Sections," ASCE Journal of Infrastructure Testing, Vol. 1, 2001.

Cheung, L.W., and Dawson, A.R. "The Effects of Particle and Mix Characteristics on the Performance of some Granular Materials." *ICAR, 8th Annual Symposium Proceedings*, April 12–14, Denver, Colorado, 2000.

Draft Standard Specifications for Construction of Highway, Streets and Bridges, Draft. Texas Department of Transport, TX, USA, 2003.

Fernando, E.G., Liu, W., Taehee, L., and Scullion, T. "The Texas Modified Triaxial (MTRX) Design Program." *Report 1869-4*. Texas Transportation Institute, The Texas A&M University System, College Station, Texas, 2001.

Florida Department of Transportation, *Standard Specifications for Road and Bridge Construction*. 2000. http://www11.myflorida.com/

Gonzalez, D., and Hinojosa, J. "*Research.xls*." TxDOT Internal Research Summary of US-77 Base Layer Moduli: Electronic format. Texas Department of Transportation, Corpus Christi District, Texas, 2000.

Gray, J.E. "Characteristics of Graded Base Course Aggregates Determined by Triaxial Tests." *National Crushed Stone Association, Engineering Bulletin No. 12*, National Crushed Stone Association, Washington, D.C., 1962.

Guthrie, W.S., and Scullion, T. "Assessing Aggregate Strength and Frost Susceptibility, Characteristic of the Tube Suction Test." Proc. of the Texas Section of the American Society of Civil Engineers, Fall Meeting, El Paso, Texas, 2000. pp. 197-206.

Hicks, R.G., and Monosmith, C.L. "Factors Influencing the Resilient Response of Granular Materials." *HRB 345*, National Research Council, Washington, D.C., 1971.

Huang, Y.H. *Pavement Analysis and Design*, Prentice-Hall Inc., Eaglewood Cliffs, New Jersey, 1994.

Idaho Transportation Department. *Standard Specifications for Highway Construction*, 2001. http://www2.state.id.us/

Illinois Department of Transportation. *Standard Specifications for Road and Bridge Construction.* Update, 1997. http://dot.state.il.us/

Jorenby, B.N., and Hicks, R.G. "Design and Performance of Flexible Pavements." *Transportation Research Board, Record 1095*, Washington, D.C., 1986.

Kolisoja, P., Saarenketo, T., Peltoniemi, H. and Vuorimies, N. Laboratory Testing of Suction and Deformation Properties of Base Course Aggregates. *Transportation Research Board, 81st Annual Meeting*, Washington, D.C., January 2002.

Langer, W.H. Geological Considerations Affecting Specifications. *ICAR, 9th Annual Symposium Proceedings*, Austin, Texas, April 22–25, 2001.

Marek, C.R., and Jones, T.R., Jr. Compaction – An Essential Ingredient for Good Base Performance. *Proceedings from Conference on Utilization of Graded Aggregate Base Materials in Flexible Pavements*, Oak Brook Hyatt House, Oak Brook, Illinois, 1974.

National Cooperative Highway Research Program (NCHRP). Roadway Design in Seasonal Frost Areas. Synthesis of Highway Practice No. 26, Washington, D.C., 1974.

National Cooperative Highway Research Program (NCHRP). Performance Related Tests of Aggregates for Use in Unbound Pavement Layers. Final Report of Project 4-23, Transportation Research Board, National Research Council, Washington, D.C., 2000.

National Stone Association (NSA). *The Aggregate Handbook.* National Stone Association, Washington, D.C., 1991.

Nazarian, S. Abdallah, I., Yuan, D. and Ke, L. "Design Modulus Values Using Seismic Data Collection." *Technical Memorandum 1780-1*, The Center for Highway Materials Research, The University of Texas at El Paso, El Paso, TX, February 1998.

Nazarian, S. Abdallah, I., Yuan, D. and Ke, L. A Sensitivity Study of Parameters Involved in Design with Seismic Moduli. *Research Report 1780-2*, Center for Highway Materials Research, The University of Texas at El Paso, El Paso, TX, January 2002.

Nevada Department of Transportation. *Standard Specifications for Road and Bridge Construction*. Nevada, USA, 2001.

New Mexico State Highway and Transportation Department. *Standard Specifications for Highway and Bridge Construction*, 2000. http://www.nmshtd.state.nm.us/

RTA Conditions of Contract and Quality Assurance Specifications, *Rev. 2001*. Roads and Traffic Authority of New South Wales, 1997. http://www.rta.nsw.gov.au/

Oklahoma Department of Transportation. *Standard Specifications for Highway Construction*, Oklahoma, USA, 1999.

O'Malley, E.S., and Wright, S.G. "Review of Undrained Shear Strength Testing Methods Used by the Texas State Department of Highways and Public Transport." *Research Report 446-1F*, Center for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin, Texas, 1987.

Ping, W.V.,Yu, Z., and Ge, L. "Evaluation of Pavement Bearing Characteristics Using the Florida Limerock Bearing Ratio Test." *Transportation Research Record 1547*, Transportation Research Board, National Research Council, Washington, D.C., 1996 pp 53-60.

Powell, J.D. "Aggregate Properties-Do We Know What We Want?" *ICAR 7th Annual Symposium Proceedings*, Austin, Texas, 1999.

Powell, J.D. "Characterization vs. Specification." *ICAR, 9th Annual Symposium Proceedings*, Austin, TX, April 22 – 25, 2001,

Standard Specifications Roads, 3rd Edition. Department Main Roads. http://www.mainroads.qld.gov.au/, 1999.

Scrivner, F. H., and Moore, W.M., "Some Recent Findings in Flexible Pavement Research," *Research Report 32-9*, Texas Transportation Institute, Texas A&M University, College Station, Texas, 1967.

Scullion, T. and Saarenketo, T. Using Suction and Dielectric Measurements as Performance Indicators for Aggregate Materials. *Transportation Research Record 1577*, Transportation Research Board, NRC, Washington, D.C., 1996, pp. 24.

Sebesta, S., and Scullion, T. “An Engineering/ Economics Evaluation of the Class 1 Bases Materials Used by the Corpus Christi District in the US-77 Experimental Test Sections.” *TxDOT Study 1869*. Texas Transportation Institute, The Texas A&M University System, College Station, TX, 2000.

South African Institution of Civil Engineering (SAICE). *Standard Specifications for Road and Bridge Works for State Authorities*. Committee of Land Transport Officials (COLOTO), Halfway House, South Africa, 1998.

“Strategic Study of Structural Factors For Flexible Pavements US-281 Southbound, Hidalgo County, Texas.” *SPS-1 Project 4801*, Copy of Final Report. FHWA/LTPP Southern Region Coordination Office, Austin, Texas, December 1997.

Texas Department of Transportation. *Standard Specifications for Construction of Highway, Steets and Bridges*, TX, USA, 1993.

Thompson, M.R. and Smith, K.L. Repeated Triaxial Characterization of Granular Bases. *Transportation Research Record 1278*, Transportation Research Board, National Research Council, Washington, DC., 1990 pp. 7 – 17.

Tutumueler and Seyhan. “Directional Dependency of Aggregate Stiffnesses: An Indicator of Granular Base Performance.” *ICAR, 8th Annual Symposium Proceedings*, Denver, Colorado, April 12 – 14, 2000.

Williammee, R.S., Thomas, G.L., and Wimsatt, A.J. “Re-Evaluation of Flexbase in the Fort Worth District.” *Report 7-3931*, Texas Department of Transportation, 2000.

APPENDIX A
ASPECTS OF MATERIALS AND CONTRUCTION QUALITY CONTROL

Sections	Process Elements	Construction Attributes
Texas (Draft, 2003)		
1	Subgrade or Existing Roadbed Preparation	• Shape to specified typical sections. • Perform work in accordance with provisions of the applicable bid item. • Proof roll and correct soft spots as directed.
2	First Course	• Deliver required amount of material to each 100 ft station. • Spread and shape material uniformly on same day as delivered, or as soon as possible. • Blade and mold material to conform to specified typical sections. • Remove and correct segregated areas by replacement with well graded material.
3	Succeeding or Finish Course	• Same as for "First Course." • Surface to be smoothed in accordance with specified typical section after completion of spreading, blading, shaping, and compaction. • Establish lines and grades. • Completed base to be cured before surfacing is placed, as directed.
4	Compaction Method	• Water used for compaction to be clean and free from industrial wastes and other objectionable materials. • "Ordinary Compaction" Method: - Sprinkle and roll until uniform compaction is secured, - Correct all irregularities, depressions, or weak spots by scarification and addition of suitable material. Reshape and recompact by sprinkling and rolling, as directed. • "Density Control" Method: - Sprinkle and compact to not less than the specified density. - If completed layer fails to meet or loose its required density, stability, or finish before placing the next course, rework and retest.
Arizona (1996)		
1	Placement (Layer Thickness)	• Water added, mixed, and processed to produce a uniform blend before final placement. • After processing, material to be placed and spread on prepared support in a uniform layer not exceeding 6" in compacted depth, unless approved by engineer. • Method of dumping or spreading to be determined by contractor.
2	Compaction	• *Specified density the only requirement.*
3	Finishing	• To be finished with equipment capable of shaping and grading the surface within tolerances specified.
4	(Maintenance)	• Compacted layer to be maintained to condition satisfactory to receive surfacing when required.
5	(Correction)	• Areas not within tolerance to be corrected by scarifying, placing additional material, and remixing.
Arkansas (1996)		
1	(Supporting layer)	• To comply with requirements and free from excess moisture, and not frozen.
2	(Shaping)	• To depth and lines that conform to thickness, width, and cross section after compaction.
3	(Layer thickness)	• Two or more layers if >6" (150 mm), except if vibratory roller used, then 8" (200 mm).
4	(Spreading and Mixing)	• Same day as hauled, no segregation, no hard nests, no mixing in of subgrade material. • No dumping on pavement surfaces that will not be overlayed under same contract. • Mixing to be carried out before placement.

Sections	Process Elements	Construction Attributes
5	(Restricted Work Space)	• Mix to full depth and compact using any method to obtain density requirement.
6	(Compaction and Moisture Content)	• Maintain substantially at optimum moisture during compaction. • Maintain grade and cross-section during operation by blading.
7	(Correcting Deficiencies)	• Correct deficiencies by scarification, addition of material, mixing, reshaping, and recompaction.
8	(Maintenance)	• Contractor to maintain completed base until accepted.
California (1999)		
1	Subgrade	• To comply with specifications, and free from loose or extraneous material.
2	Adding Water	• Material to have sufficient moisture content to obtain required compaction at time of spreading. • Moisture to be uniformly distributed throughout material.
3	Spreading (& Layer Thickness)	• Aggregate to be delivered as uniform mixtures. Uniformity to be maintained during spreading. • Max. compacted thickness for one layer is 150 mm. If more than 150 mm, two or more layers of equal thickness to be constructed. • Inaccessible areas: Spreading in one or more layers by any means to obtain required results • Subgrades of cohesionless sand may be stabilized by dumping base aggregate in piles and spreading of material ahead of piles.
4	Compaction (& Correcting)	• Aggregate bases to be watered after compaction. • Surface of compacted material not to vary more than 15 mm from specified grade. Layer to be reshaped, reworked, and recompacted to specified density in case of non-conformance.
Florida (2000)		
1	Spreading	• Spread uniformly, remove all segregated material, and replace with properly graded material.
2	Number of Courses (Test Section)	• When specified thickness >6 in. (150 mm), construct in multiple courses of equal thickness. • Individual courses not to be compacted in layers of less than 75 mm. • Thickness of first course may be increased to protect subgrade from damage by equipment. • Max. lift of 200 mm may be accepted based on densities achieved on a test section. - Test section to be 300 ft (90 m) to 1000 ft (300 m) in length, full width. - Acceptance based on density achieved at bottom 6 in. of compacted layer of test section. - Once approved, the compactive effort remains unchanged and a new source of material requires a new test section. - Density of bottom 6 in. (150 mm) of thick lift operations to be verified periodically.
3	Compaction and Finishing [General]	• Single course: Scarify after spreading, shape to required grade and cross section. • Multiple course: Clean first course of foreign material, shape approximately to requirements. Density test to be carried out before proceeding with upper course. Finish and shape material for top course to produce grade and cross section free of scabs and laminations, after compaction.
4	[Moisture content] (& Compaction)	• Manipulate entire width and depth of layer as a unit during wetting or drying operations. • Addition of water to be carried out by uniform mixing in through disking to full depth of course. • Compact when proper moisture conditions are attained.

Sections	Process Elements	Construction Attributes
5	[Correcting Defects]	• Contamination of base material: If subgrade material mixed in with base, dig out and replace. • Cracks and cheks: If required by engineer, remove by rescarifying, reshape, and recompact.
6	[Widening Strips]	• Special equipment to be used which will achieve specified density.
7	Priming and Maintaining	• Prime only when base meets specification, is firm, unyielding, not prone to undue distortion. • Moisture content of top half of base not to exceed 90% of optimum at time of priming. • Maintain true grade and template, with no rutting or distortion during surfacing operation.
Illinois (1997)		
1	Subgrade	• Subgrade to be prepared in accordance with reference specifications.
2	General	• Aggregate to be delivered as uniform mixtures.
3	(Moisture Content)	• Wetting of aggregate in cars, bins, stockpiles, or trucks are not permitted. • Water added at central plant (Type A), or in field as required (Type B).
4	(Thickness of Layers)	• Max. layer thickness 100 mm (4 in.). Can be adjusted to max. 200 mm if proved by testing.
5	(Placing)	• To be deposited in full-lane width, directly on preceding compacted layer, with spreader and free from segregation. Minimum blading or manupilation is to be required. • Uniformity to be maintained during spreading.
6	(Compaction)	• Compaction to commence immediately after placement of aggregate, with tamping roller, or pneumatic-tired roller, or vibratory machine, or combination. • Final rolling with three-wheel roller required. • Engineer to approve compaction process.
7	(Contamination)	• Any operation which may cause subgrade material to work into base may be restricted by engineer, such as hauling overcompleted base with soft subgrade after inclement weather. • Material in affected areas is to be removed and replaced.
8	Maintaining	• Until entire section is accepted, with min. period of 10 days, or less if proof rolled.
New Mexico (2000)		
1	Preparation of Foundation	• Surface to be cleaned of loose, deleterious, and frozen materials. • Top 150 mm to meet density requirements for that layer. • Subgrade may be proof rolled with 30-ton roller upon request, and soft areas repaired.
2	Mixing and Placing (& Layer Thickness)	• Homgeneous mixture of unseregated uniformly dispersed material to be provided. • Spread and compact in layers so that density is obtainable, not exceeding 150 mm.
3	Plan Base Course Depth	• If paid by square meter, depths to be monitored throughout placement operation.
4	*Stockpiled Base Course (MaterialControl)*	• *To be stockpiled at locations shown on plans.* • *Segregation and unnecessary loss of material at stockpile loaction to be prevented.* • *A pad of 150 mm to be constructed of stockpile material, equipment capable of stacking stockpile in neat and regular shape to be used.* • *Contaminated or unsatisfactorily material to be replaced.*

Sections	Process Elements	Construction Attributes
5	Removing and Processing	• Care taken during removing and stripping of base course, contamination kept to minimum. • Stripped base course material to be processed to meet requirements.
6	Contractor Process Control	• Contractor to develop and administer a quality control plan addressing all elemets which affect the quality of the base course (minimum required elements to address listed).
New South Wales (1997)		
1	Spreading (& layer thickness)	• Spreading and compaction in uniform layers, to provide thickness specified, after trimming. • Each compacted layer not more than 150 mm, not less than 100 mm.
2	Compaction and Moisture	• Moisture uniformly distributed at time of compaction, adjusted to attain specified compaction, and within 60% to 90% of optimum moisture content. • Compaction to be completed promptly after spreading. • Compaction to commence at low side, or sides, and progress to high point to prevent uncompacted material breaking away. Particular attention given to outer edges of layers.
3	Trimming	• Layer surface to be trimmed during compaction, and corrected, to produce tight dense surface parallel with finished pavement surface.
Queensland (1999)		
1	Layer Thicknesses	• Individual compacted thickness chosen to suit construction process and requirements. • Compacted layer thickness less than 250 mm and more than 75 mm.
2	Moisture Content (& Compaction)	• Moisture may be adjusted to suit compaction process to obtain required density. • Layer should conform to max. specified degree of saturation before covered by next layer or surface.
3	Surface Finish	• Uniform surface, free from loose, segregated, and contaminated areas, with coarse particles slightly exposed. May be trimmed, lightly watered, drag-broomed, and rolled.
4	Construction Equipment (Spreading)	• Self-propelled, purpose-built spreading machines with capacity to spread material in one pass to necessary uncompacted depth and half pavement width, or 3 meters, whichever one is the lesser.
South Africa (1998)		
1	Trial Section	• 150 m to 200 m to be constructed to prove suitability of process with material to obtain required product.
2	Supporting Layer	• Supporting layer to comply with specifications, free from excess moisture.
3	Spreading and Mixing	• Contractor may select to mix and spread by mixing plant or paver. • Material dumped uniformly to ensure sufficient quantities to ensure thickness, cross section, and level of layer will be properly formed. • Dumps to be spread out to layer with thickness suitable for mixing. • Required quantity of water to be added and material mixed to obtain homogeneous mixture • Max. individual compacted layer thickness 150 mm unless otherwise permitted.
4	Compaction	• Spread and mixed material to be compacted throughly using suitable equipment to obtain specified density throughout entire layer after slushing. • Compacted layer to be free from surface laminations, areas of segregation, corrugations.

Sections	Process Elements	Construction Attributes
5	Surface Preparation (Watering and Slushing)	• Immediately after compaction, short sections to be thoroughly watered, rolled, and slushed by means of steel-wheeled rollers (not less than 12 tons), or pneomatic-tyred rollers. • Continue process until all excess fines in mixture have been brought to surface and density has been reached. • Swept, broom, excess fines and loose aggregate from damp surface and allow to dry out. • Care to be taken not to roll surface out of shape during operation. • Operation to be continuous and completed before equipment used on next section. • After slushing and brooming, wind-dried section to be finally rolled with steel-wheeled roller. • Completed layer to be firm, stable, with closely knit surface of aggregate exposed in mosaic, free from nests of segregated material, laminations, and corrugations.
6	Other	• Junctions with existing surfaces and layers. • Reconstruction of existing crushed stone layers. • Work in restricted areas. • Watering and rolling the floor of a pavement excavation.
7	Protection and Maintenance	• Base to be primed as soon as possible and as soon as moisture dropped to 50% of optimum. • Protect and maintain until surfacing applied.
Notes: • Section descriptions and structure of the original specification are retained in this summary. • Construction process specifications are the main focus of this summary. Where necessary to retain the original structure, product (such as material or density) specifications are indicated in *italics*. • (Round brackets) used to introduce descriptive words to highlight process elements. • [Square brackets] used to indicate existing subsections.		

Test	Minimum Acceptance Testing			Remarks
	Location or Time of Sampling	Frequency of Sampling	Reduced Frequency of Sampling	
Texas (Draft 2003)				
Gradation	During stockpiling operations, from stockpile, or from windrow	Each 4000 C.Y. or 6000 tons	N/A	Any one or combination of three locations can be selected provided that one of 10 tests will be sampled from windrow.
Liquid Limit				
Plasticity Index				
Wet Ball Mill		Each 20,000 C.Y. or 25,000 tons		For stockpiles not built in horizontal layers, sampling is to be 1 test per 4500 C.Y. or 6000 tons.
Triaxial				For stockpiles not built in horizontal layers, sampling is to be 1 test per 12,000 C.Y. or 16,000 tons.
Compaction	As designated by the engineer	Each 3000 lin. ft. per course per travel		
Thickness		1 depth per per 3000 lin. ft. per course per travel		If payment is by the S.Y. frequency shall be as called for in the governing specification.
Arkansas (1996)				
Gradation	From stockpile	1 test per 1000 tons	N/A	
Plasticity Index			If first 5 tests show that material is non-plastic, further testing for PI may be waved.	
Moisture Content	Location randomly selected by Engineer		N/A	
Compaction				
Thickness				
California (2001)				
% Crushed Particles	Materials site or stockpile	As necessary for acceptance	N/A	Minimum 1 acceptance test per project.
Sieve Analysis		Every 2500 t or 1500 m^3	1 per day unless source is changed	
Durability Index		If initial source changes or new source developed	N/A	

R-Value		Every 2500 t or 1500 m^3	1 per day unless source is changed. May be waved if test records show that material from same source, having comparable grading and sand equivalent values, meets min. R-value.	
Sand Equivalent			1 per day unless source is changed.	
California (2001)				
Test	**Minimum Acceptance Testing**			**Remarks**
	Location or Time of Sampling	**Frequency of Sampling**	**Reduced Frequency of Sampling**	
Moisture	Materials site or stockpile	2 times daily if paid for by weight	N/A	
Relative Compaction		As necessary for acceptance		
Dimensions				
New Mexico (2001)				
Minimum Process Control				Process control tests can be incorporated with acceptance control tests if statistically approved.
Gradation	During process as needed to control operations	1 per 1000 tons	N/A	
Fractured Faces				
Sand Equivalent				
Atterberg Limits				
Moisture Content		As needed to control operations		
Minimum Acceptance Control				
Gradation	Roadway before compaction	1 per 2000 tons/ 10,000 tons lot	N/A	
Density	Roadway after compaction			
New South Wales (1997)				
Gradation	At point of delivery, within 3 days of completing stockpile.	2 per 1 to 500 t lot 3 per 500 to 1000 t lot 4 per 1000 to 2000 t lot 5 per 2000 to 4000 t lot	N/A	
Liquid Limit			1 per 1 to 500 t 2 per 501 to 400 t	
Plasticity Index				

Mod. Texas Triaxial		1 per 1 to 1000 t lot 2 per 1001 to 2000 t lot 3 per 2001 to 4000 ton lot	1 per 4000 t	
Particle shape				
Aggregate wet strength			Frequency depends on magnitude of wet/dry result	
Wet/Dry variation				
Relative compaction >100%	Site, random sampling locations	1 per lot ≤ 50 m^2 3 per 50 to 500 m^2 lot 4 per 500 to 1000 m^2 lot 1 per 500 m^2/lot >1000 m^2 1 per1000 m^2/lot >5000 m^2	N/A	
South Africa (1989)				
Material tests	Before incorporation into work and after compaction.		N/A	A lot is a sizeable portion of work or quantity of material which is assessed as a unit for the purpose of quality control and selected to represent material or work produced by essentially the same process and from essentially the same materials.
Relative compaction	Locations in random stratified pattern.	Min. 6 per lot		
Thickness		Min. 30 per lot		

APPENDIX B US-281

Summary of 2002 GPR Results

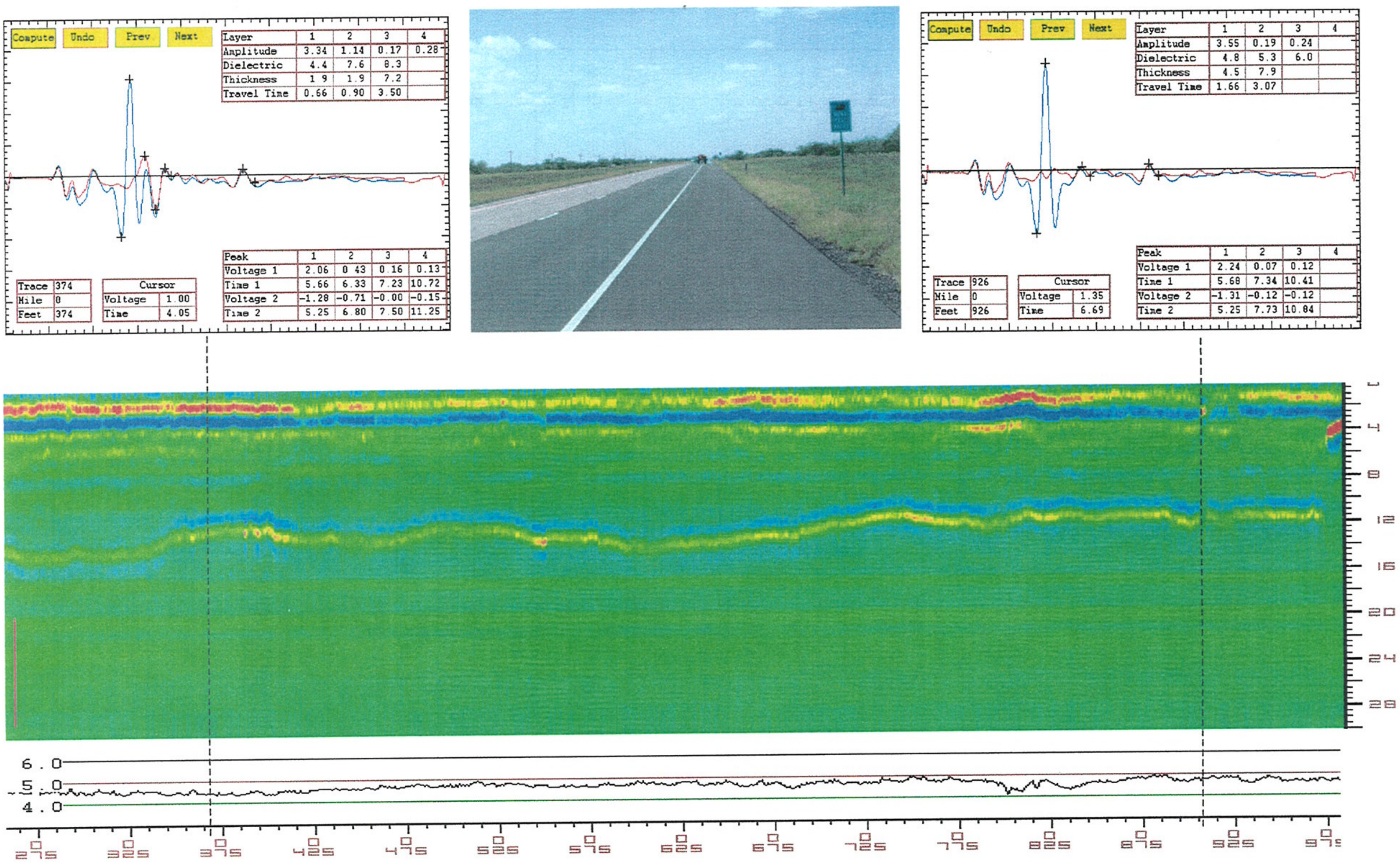
Detailed GPR Color Scheme Outputs

Detailed FWD and GPR Data Plots

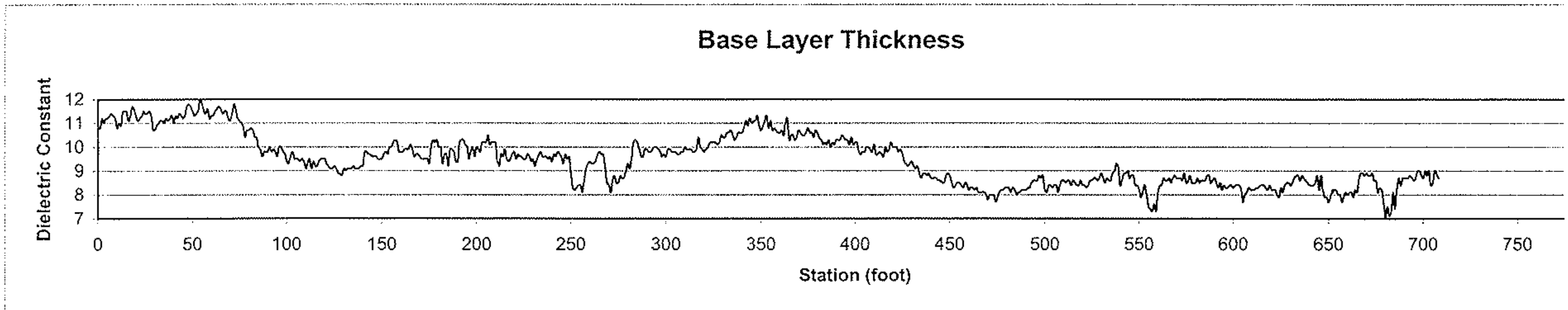
US-281: SUMMARY OF 2002 GPR RESULTS

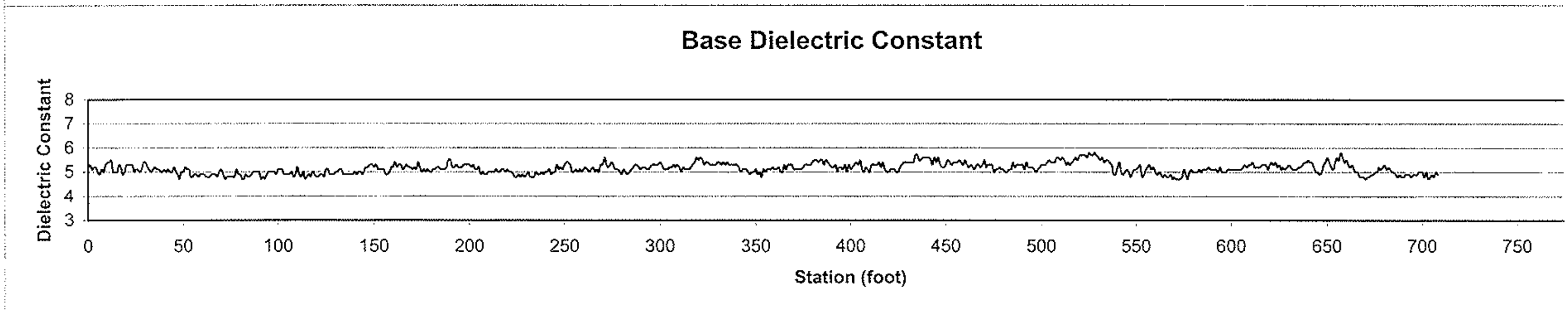
Section	Statistic	HMA Thickness (in.)	HMA Dielectric Constant	Base Thickness (in.)	Base Dielectric Constant	Subgrade Dielectric Constant
113 DGAB	Average	4.3	4.7	9.3	5.2	6.0
	CoV	0.06	0.03	0.11	0.04	0.06
	Minimum	3.6	4.1	7.3	4.7	5.1
	Maximum	5.6	4.9	11.3	5.8	7.2
114 DGAB	Average	7.5	4.9	14.3	5.6	5.8
	CoV	0.06	0.04	0.07	0.05	0.05
	Minimum	5.9	4.1	12.8	4.8	5.0
	Maximum	8.3	5.2	16.9	6.2	6.5
160 LRA	Average	5.2	4.9	13.3	7.0	7.4
	CoV	0.06	0.04	0.07	0.04	0.05
	Minimum	4.7	4.6	10.5	6.2	6.4
	Maximum	6.3	5.1	14.9	7.6	8.3
161 LRA	Average	4.8	5.1	10.4	7.4	8.0
	CoV	0.07	0.03	0.07	0.04	0.05
	Minimum	4.1	4.8	9.0	5.4	5.5
	Maximum	5.8	5.3	12.3	8.2	8.9
162 CLSB	Average	4.9	4.9	10.7	7.2	7.8
	CoV	0.07	0.03	0.06	0.08	0.08
	Minimum	4.2	4.3	9.2	5.1	5.2
	Maximum	6.2	5.2	12.3	9.6	10.4
163 CLSB	Average	5.0	4.9	12.9	7.1	7.4
	CoV	0.1	0.03	0.06	0.07	0.07
	Minimum	4.1	4.5	9.0	6.2	6.3
	Maximum	7.3	5.1	15.2	8.8	9.4
164 CCAB	Average	5.2	4.6	8.8	6.4	6.8
	CoV	0.08	0.03	0.12	0.14	0.15
	Minimum	3.9	4.3	6.2	5.4	5.5
	Maximum	6.3	5.1	11.3	10.8	12.0
165 CCAB	Average	5.0	4.7	8.4	6.6	6.9
	CoV	0.08	0.03	0.13	0.10	0.10
	Minimum	4.0	4.2	6.5	5.6	5.6
	Maximum	5.9	5.2	11.2	9.9	10.5
166 CAL	Average	5.0	5.5	10.9	11.7	11.8
	CoV	0.08	0.02	0.05	0.11	0.14
	Minimum	4.4	5.1	9.5	8.8	6.7
	Maximum	6.9	6.9	12.4	14.6	15.6
167 CAL	Average	5.9	4.6	7.0	8.3	8.8
	CoV	0.08	0.04	0.07	0.13	0.13
	Minimum	4.6	4.3	5.8	6.5	6.7
	Maximum	7.0	5.1	8.1	11.2	11.9

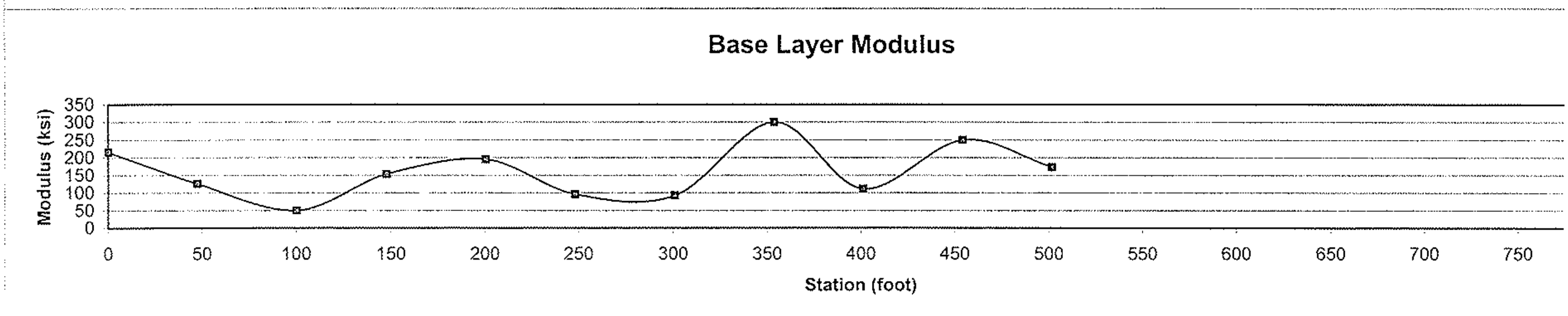
US-281 Southbound, Slow Lane, Section 480113: Dense Graded Aggregate Base



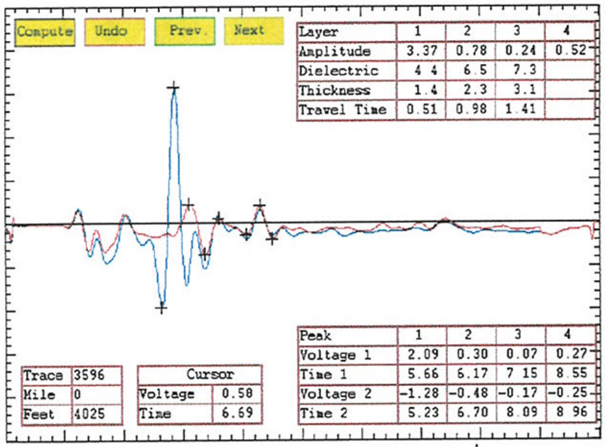
US-281 Southbound, Slow Lane, Section 480113: Dense Graded Aggregate Base

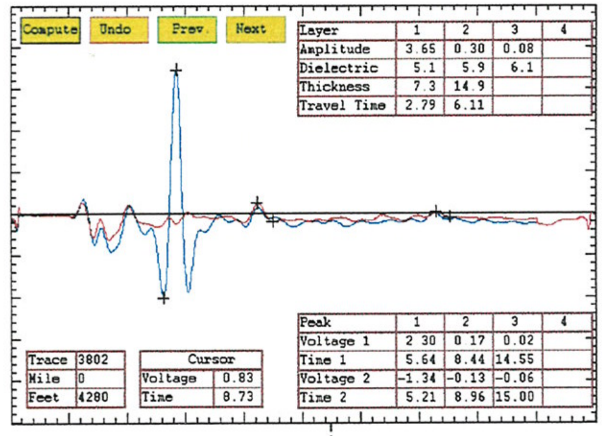


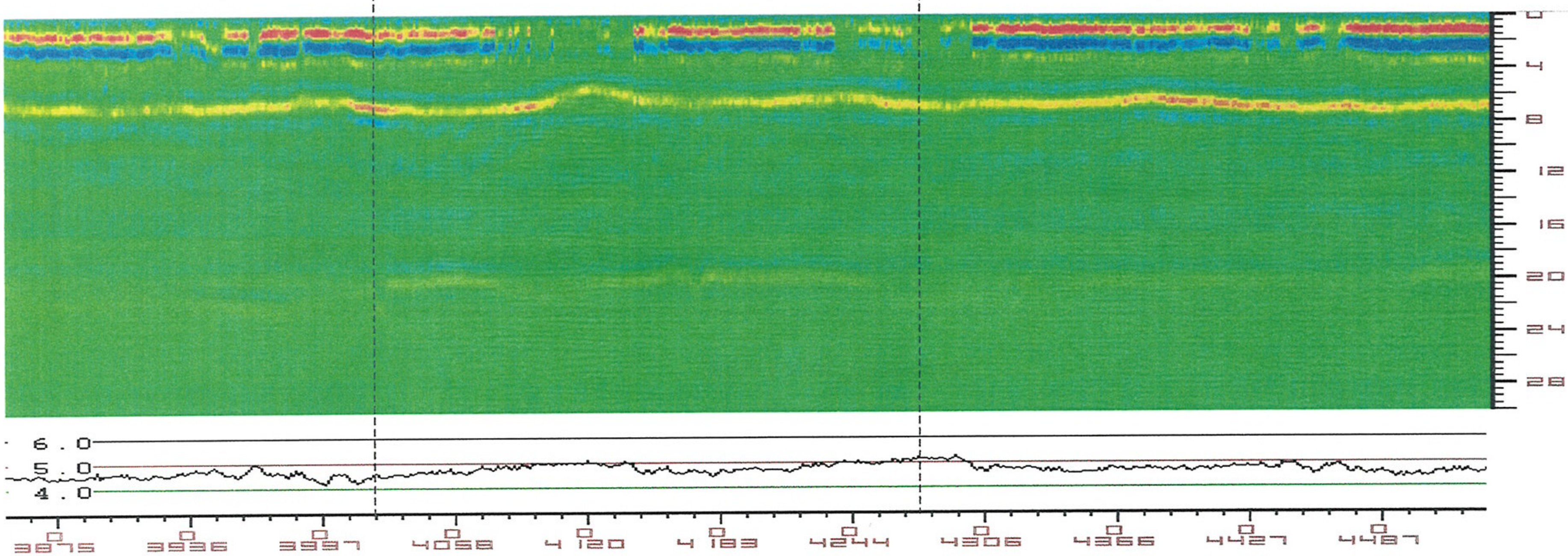




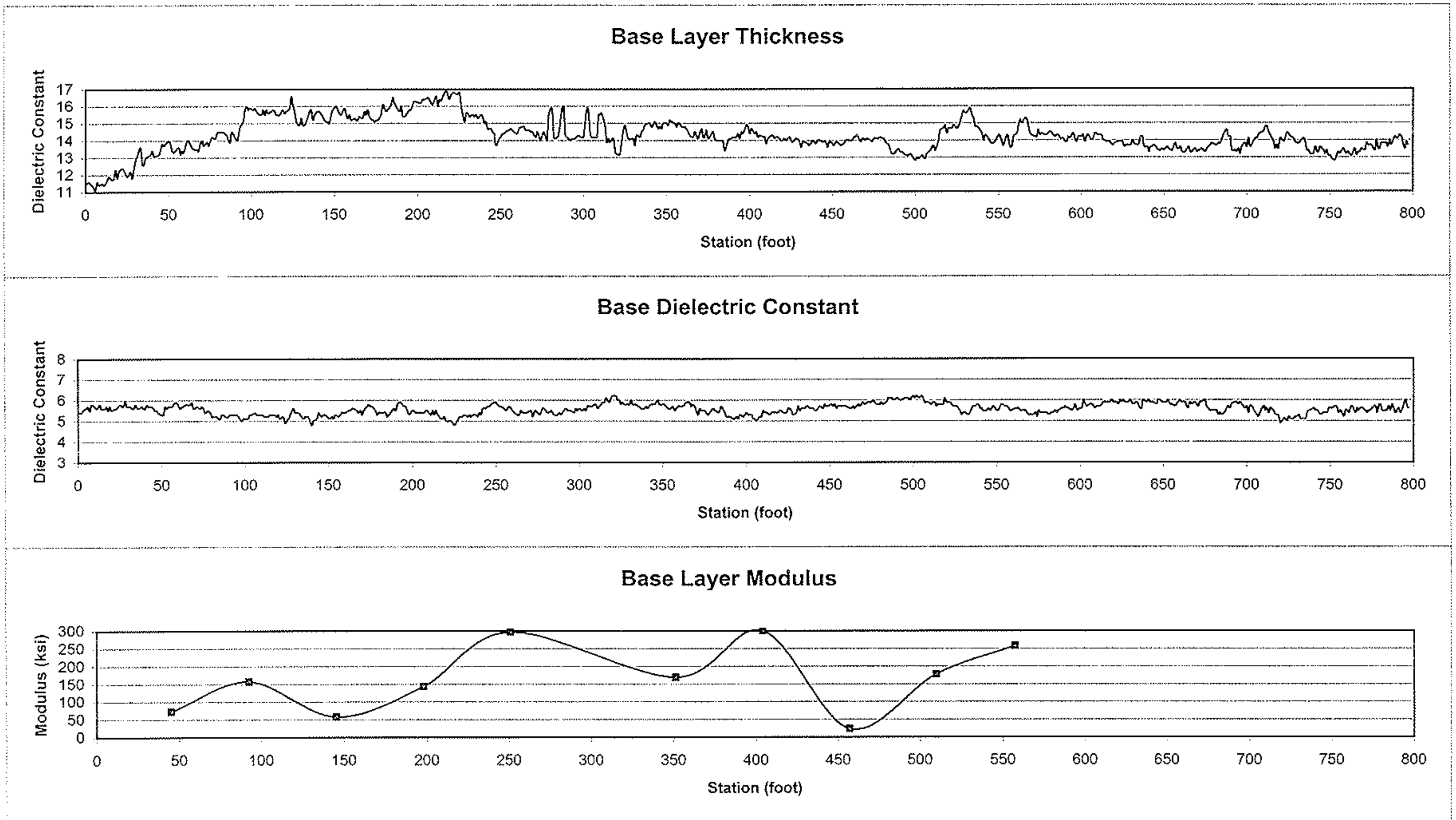
US-281 Southbound, Slow Lane, Section 480114: Dense Graded Aggregate Base



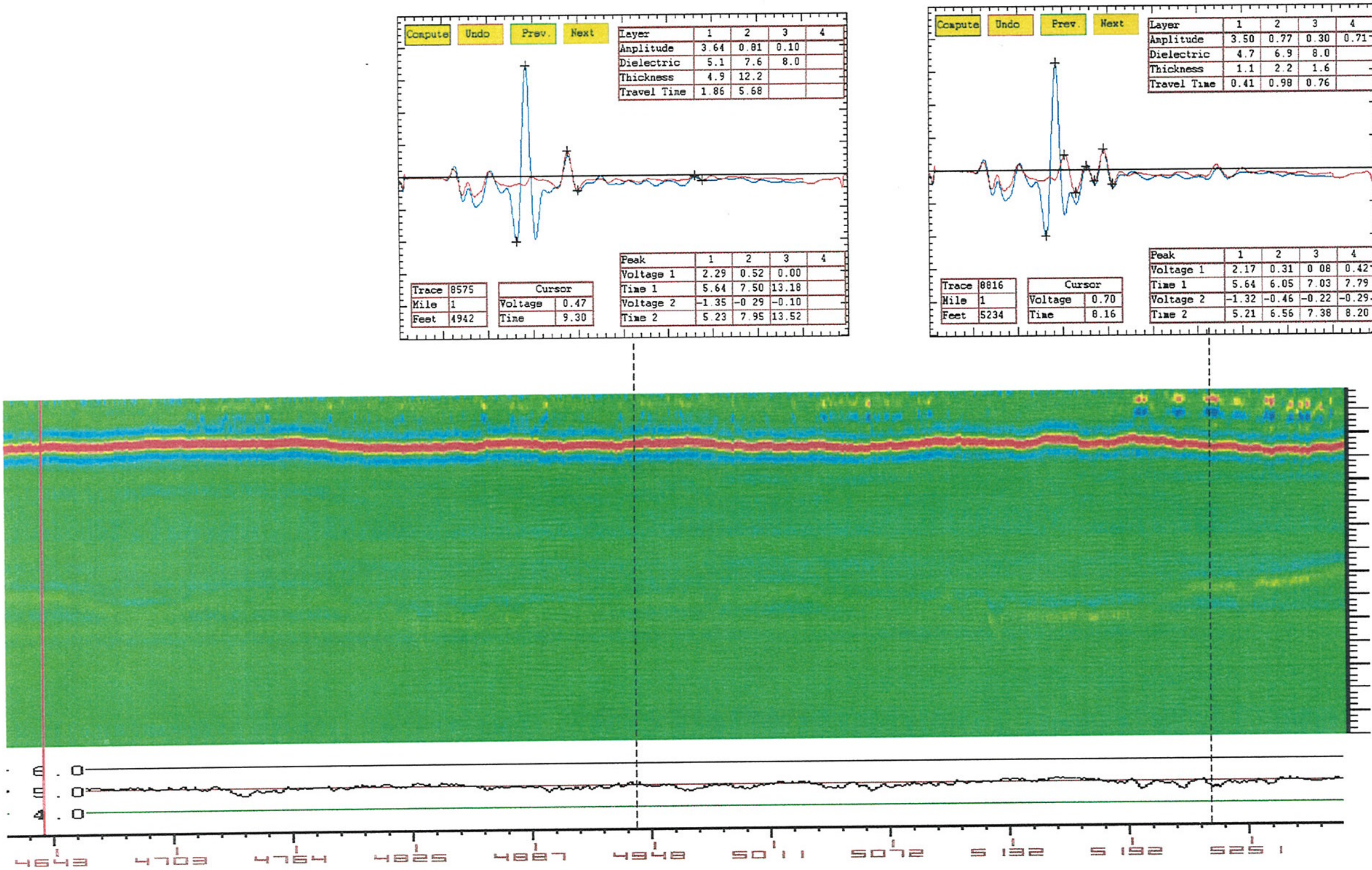




US-281 Southbound, Slow Lane, Section 480114: Dense Graded Aggregate Base

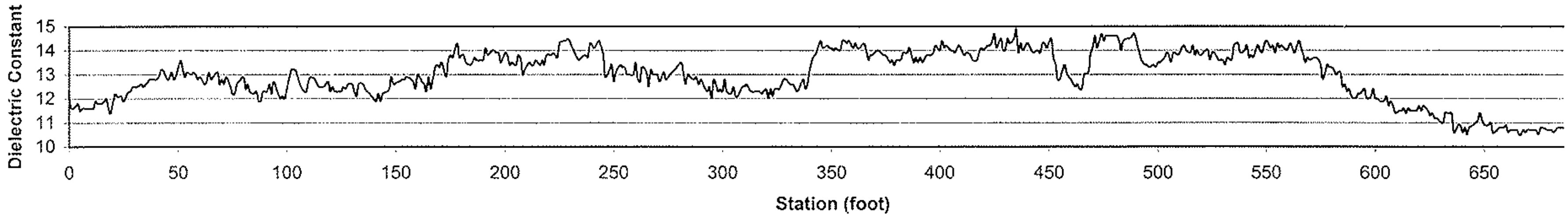


US-281 Southbound, Slow Lane, Section 480160: Lime Rock Asphalt Base

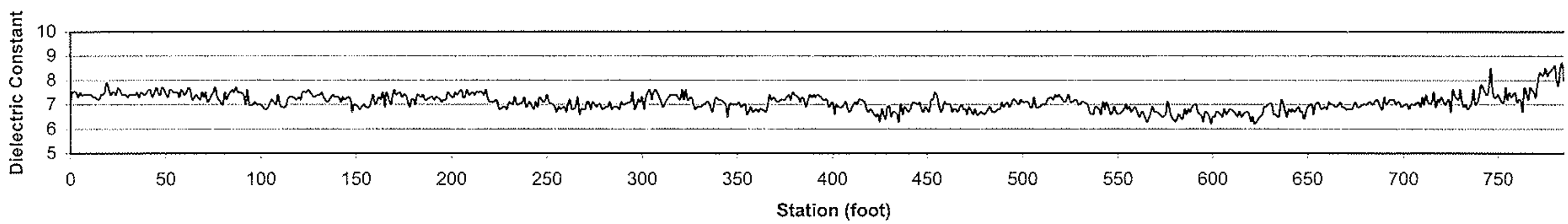


US-281 Southbound, Slow Lane, Section 480160: Lime Rock Asphalt Base

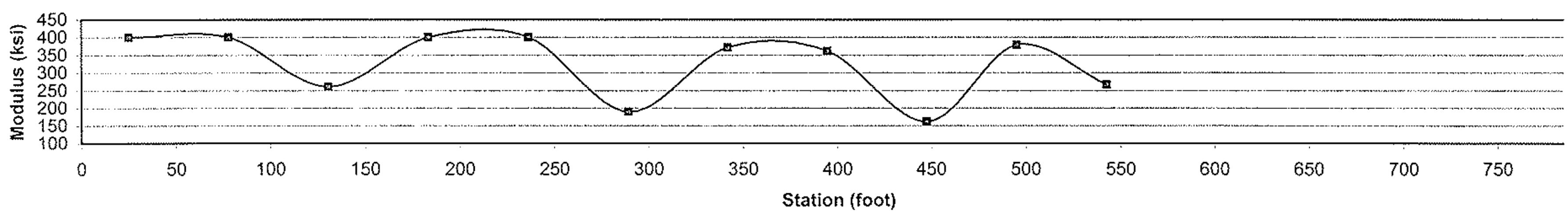
Base Layer Thickness



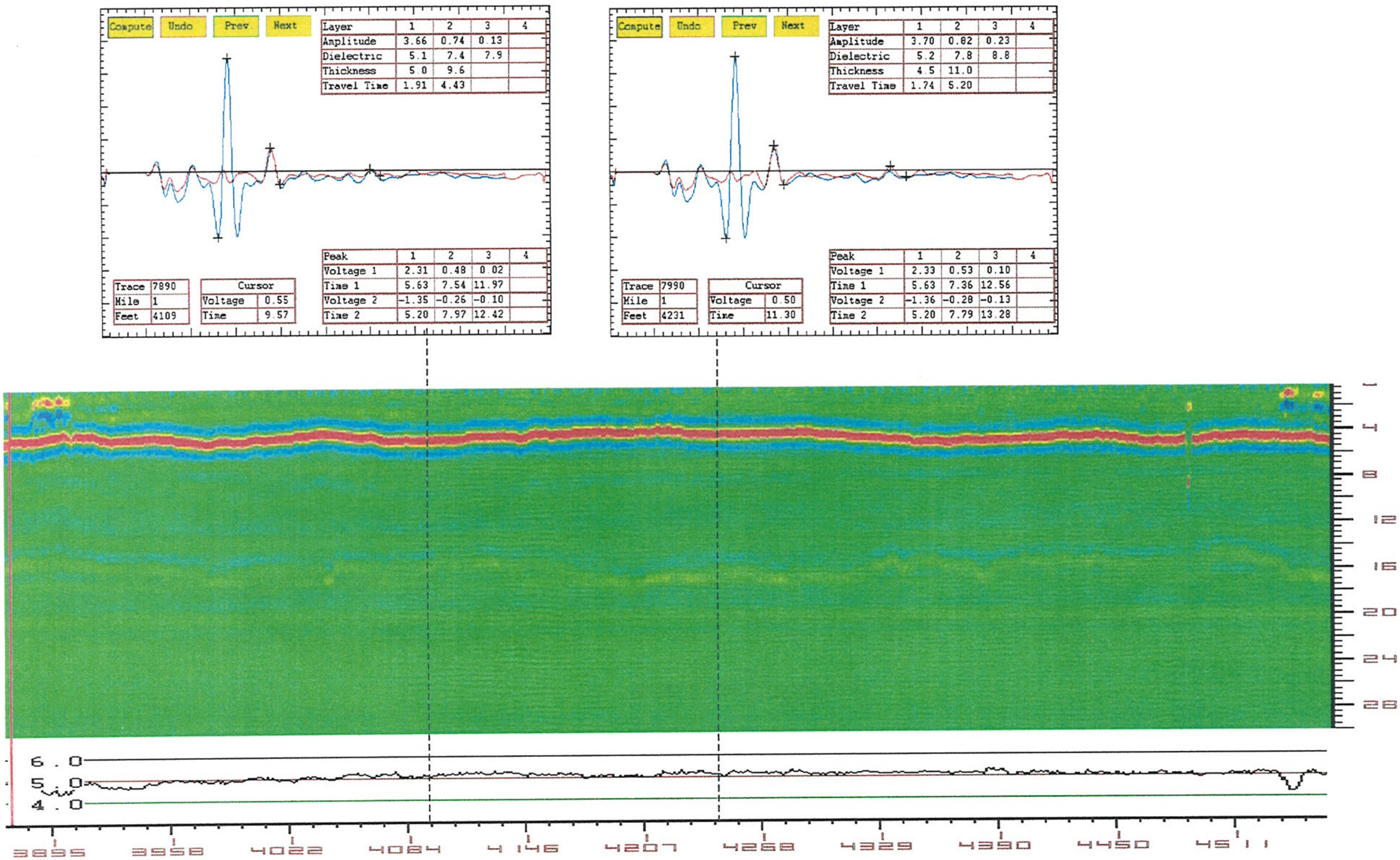
Base Dielectric Constant



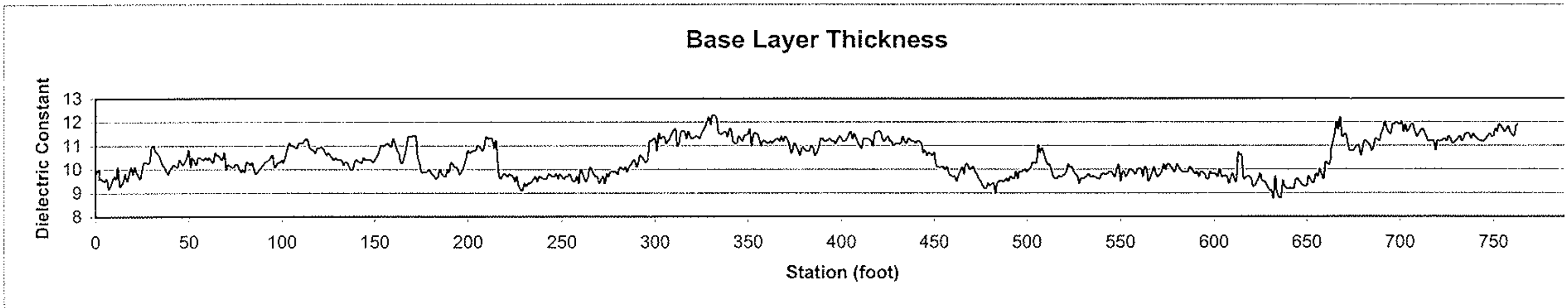
Base Layer Modulus

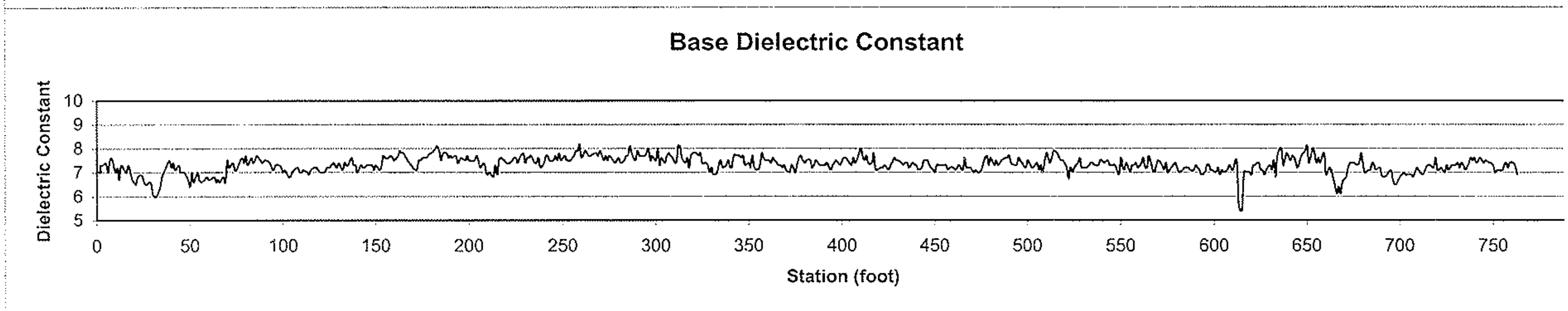


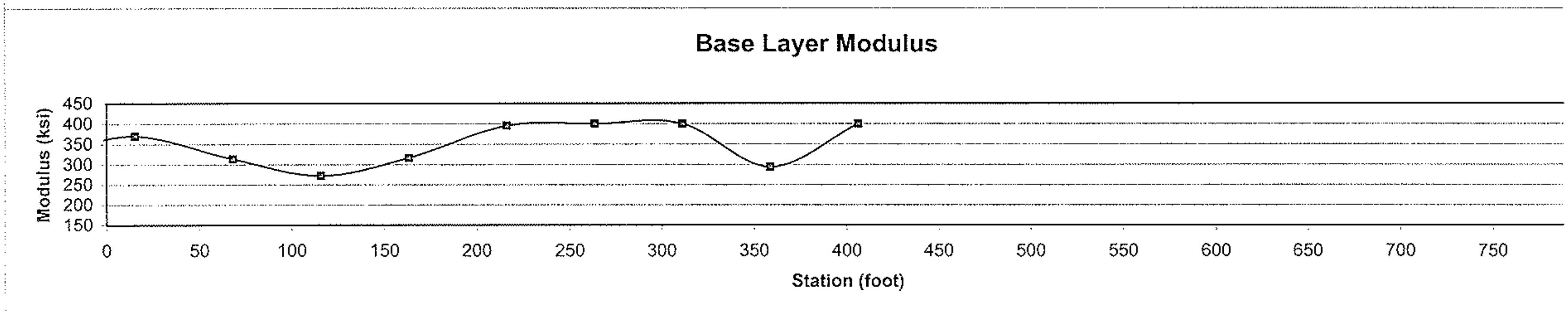
US-281 Southbound, Slow Lane, Section 480161: Lime Rock Asphalt Base



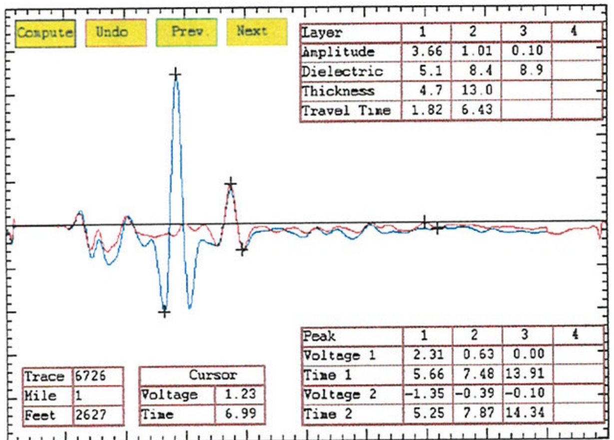
US-281 Southbound, Slow Lane, Section 480161: Lime Rock Asphalt Base

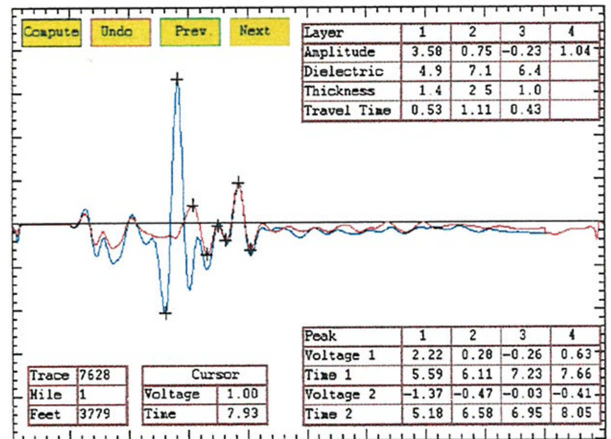


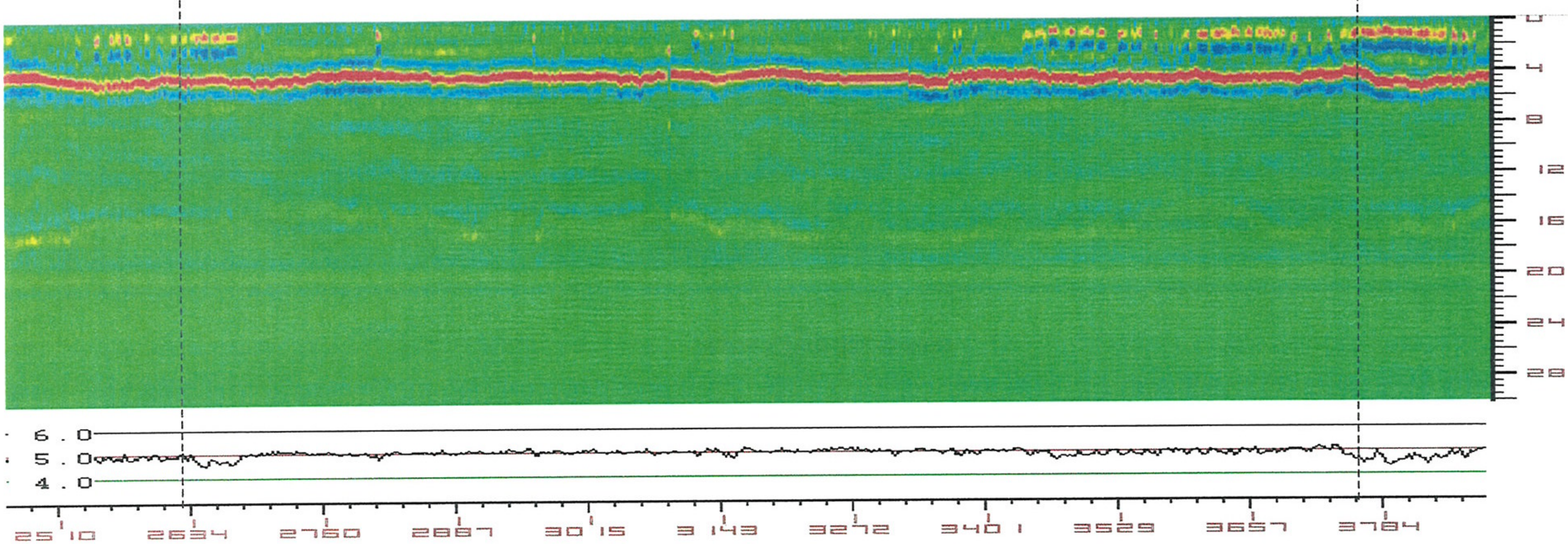




US-281 Southbound, Slow Lane, Section 480162: Limestone Crushed Aggregate Base

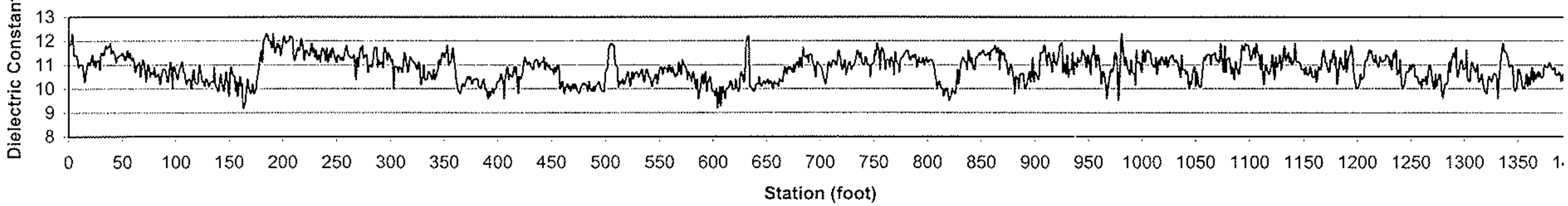




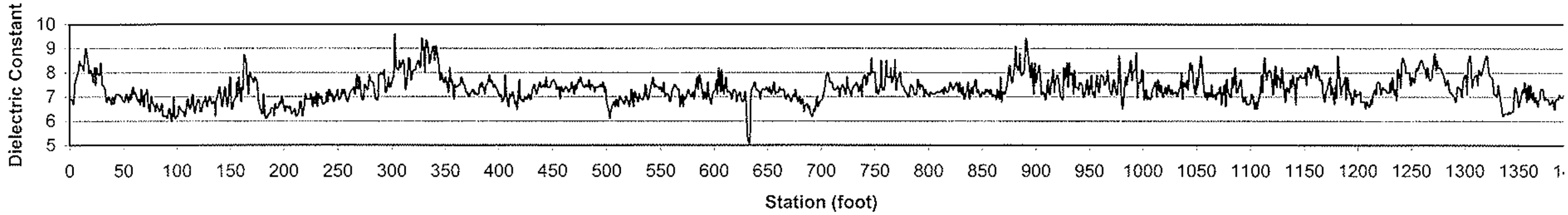


US-281 Southbound, Slow Lane, Section 480162: Crushed Limestone Aggregate Base

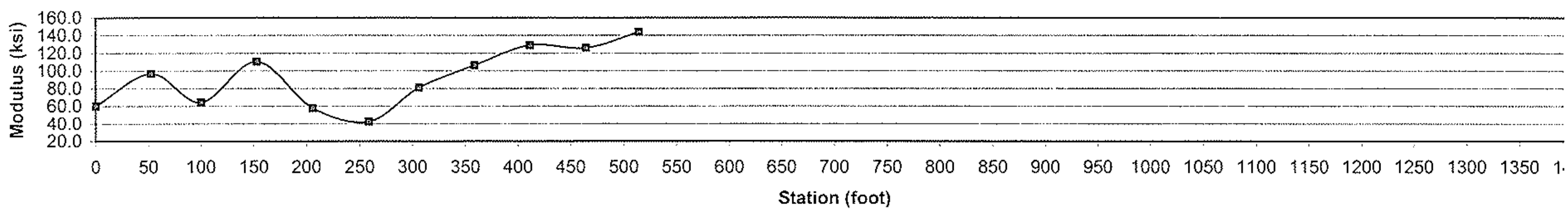
Base Layer Thickness



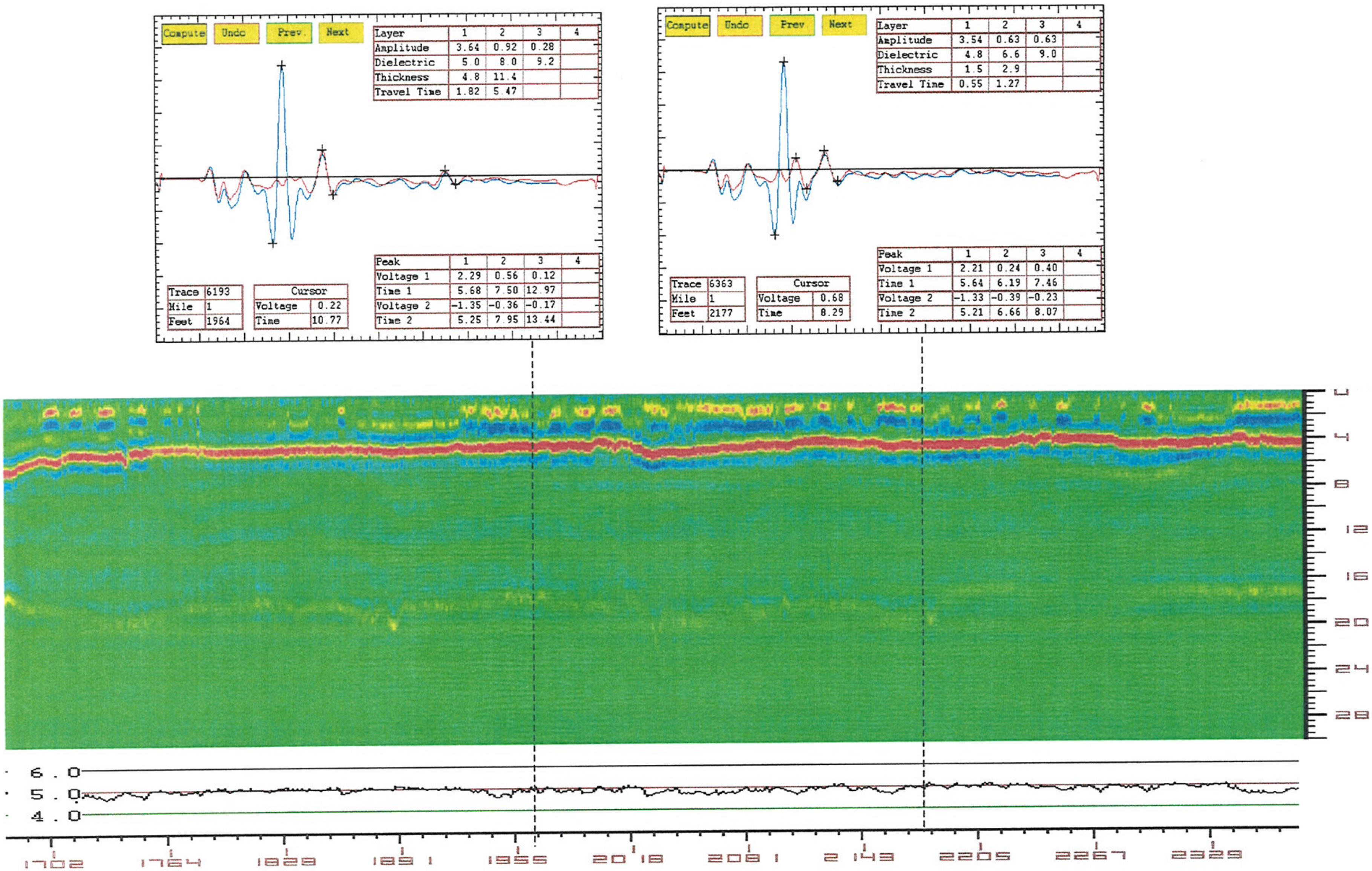
Base Dielectric Constant



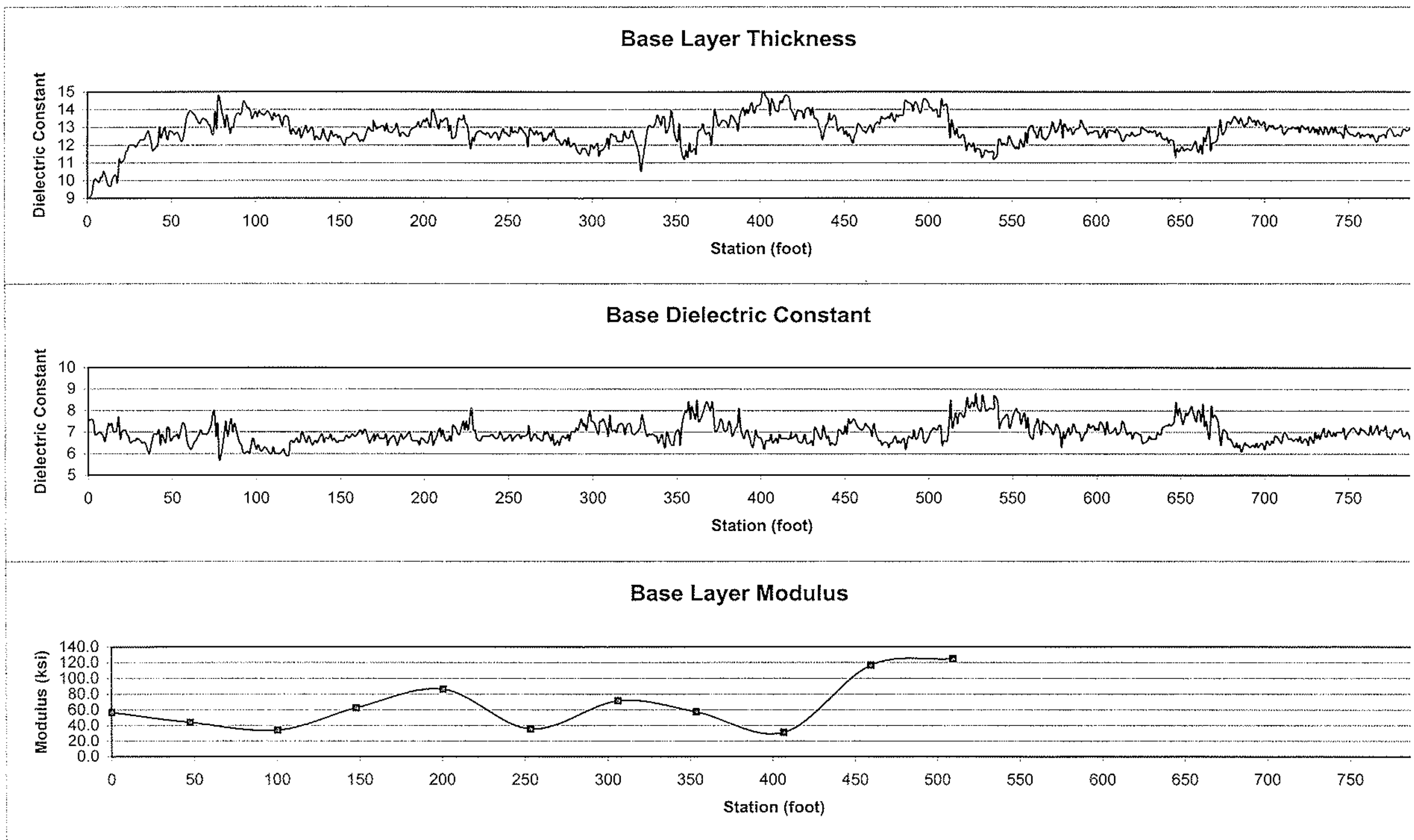
Base Layer Modulus



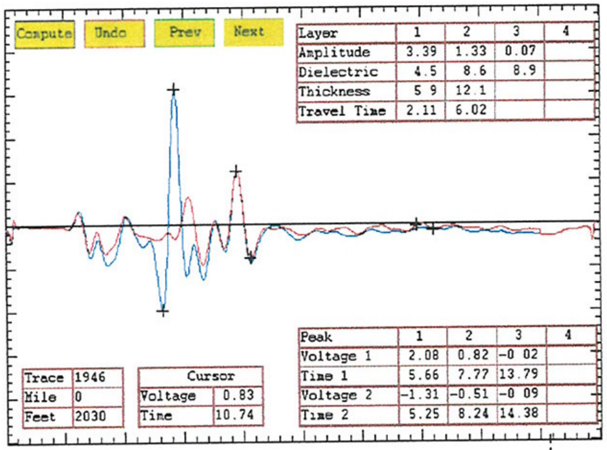
US-281 Southbound, Slow Lane, Section 480163: Limestone Crushed Aggregate Base

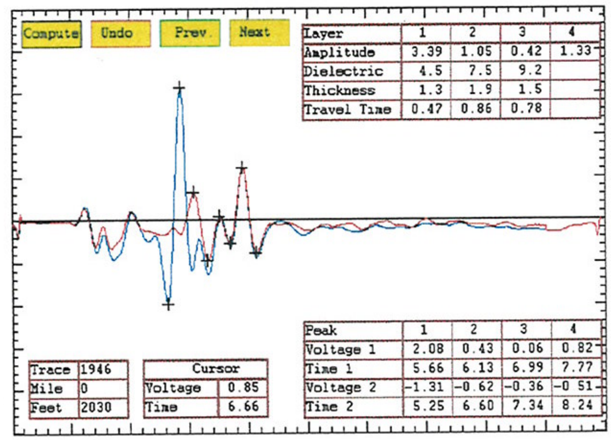


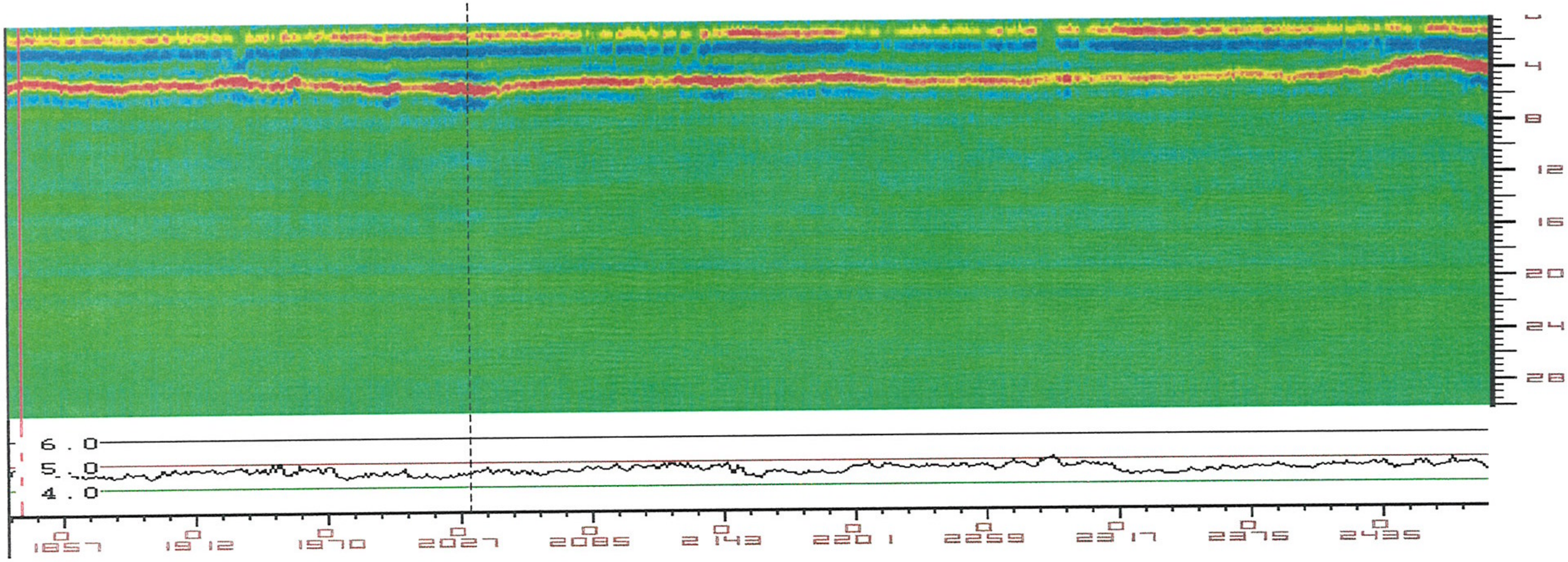
US-281 Southbound, Slow Lane, Section 480163: Crushed Limestone Aggregate Base



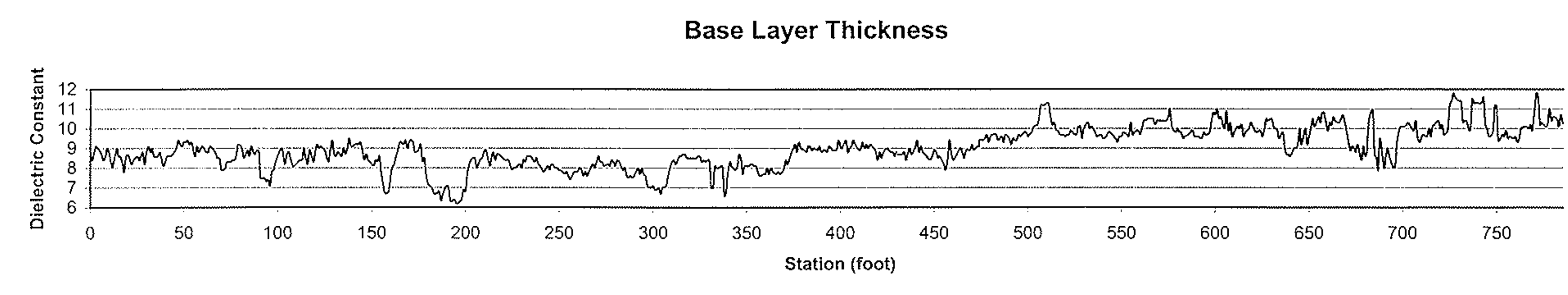
US-281 Southbound, Slow Lane, Section 480164: Crushed Concrete Aggregate Base

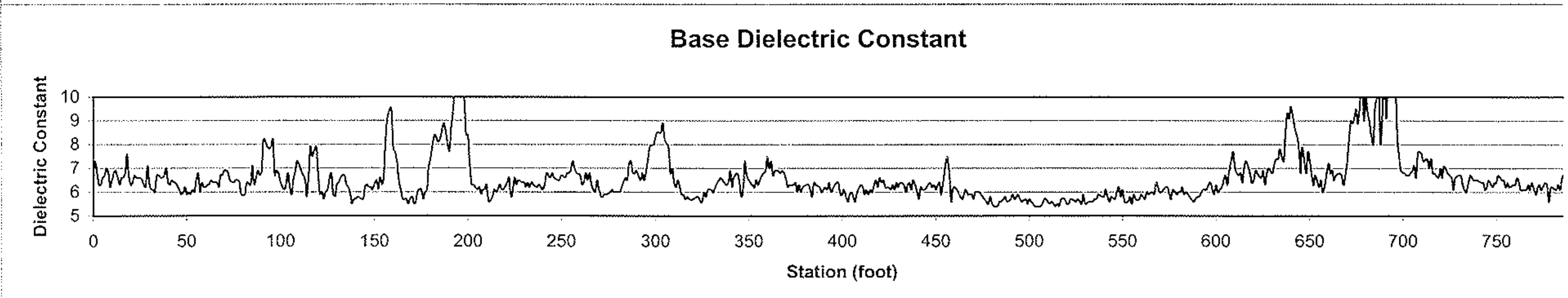


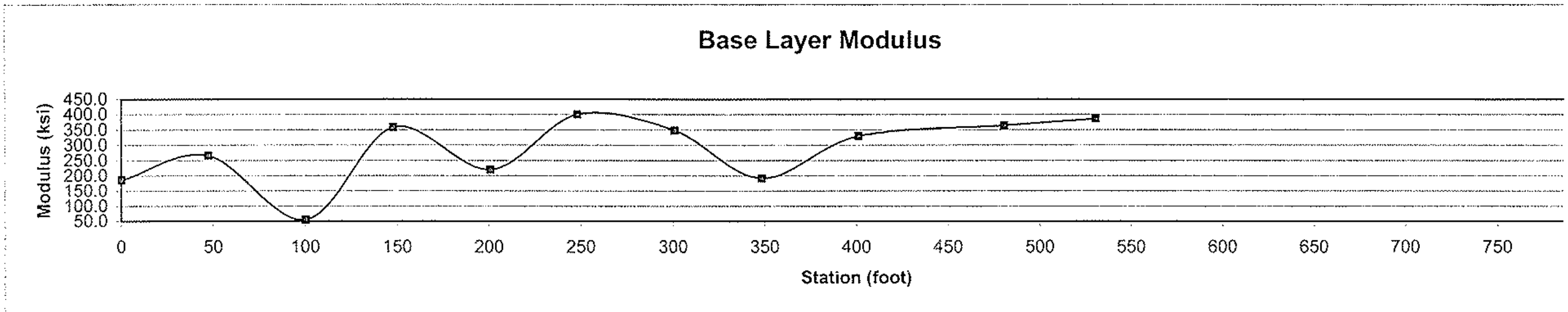




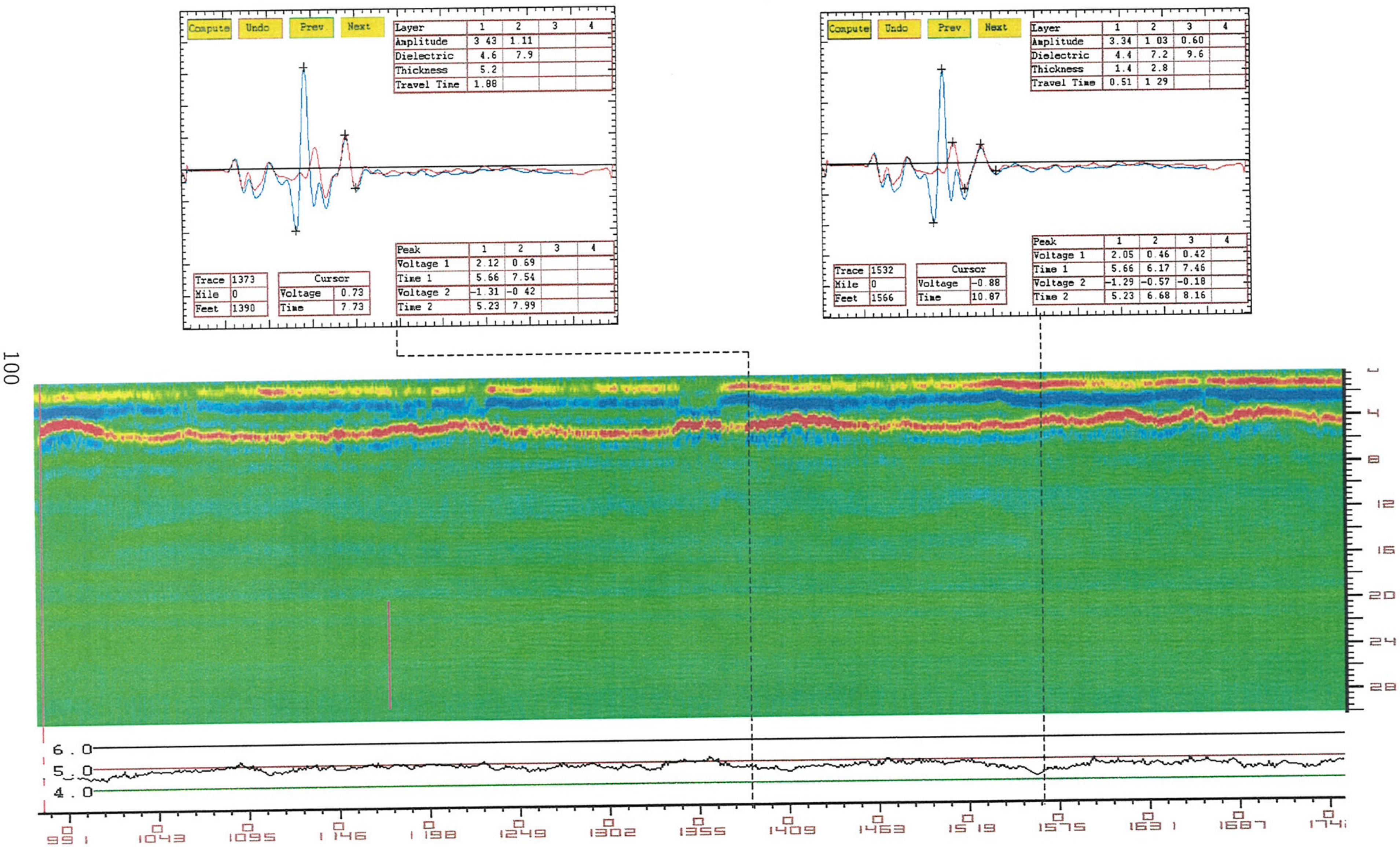
US-281 Southbound, Slow Lane, Section 480164: Crushed Concrete Aggregate Base



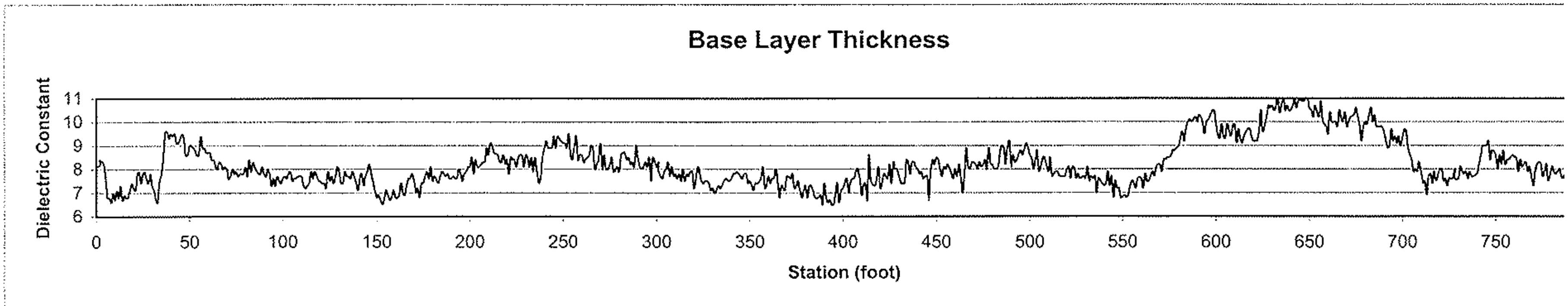


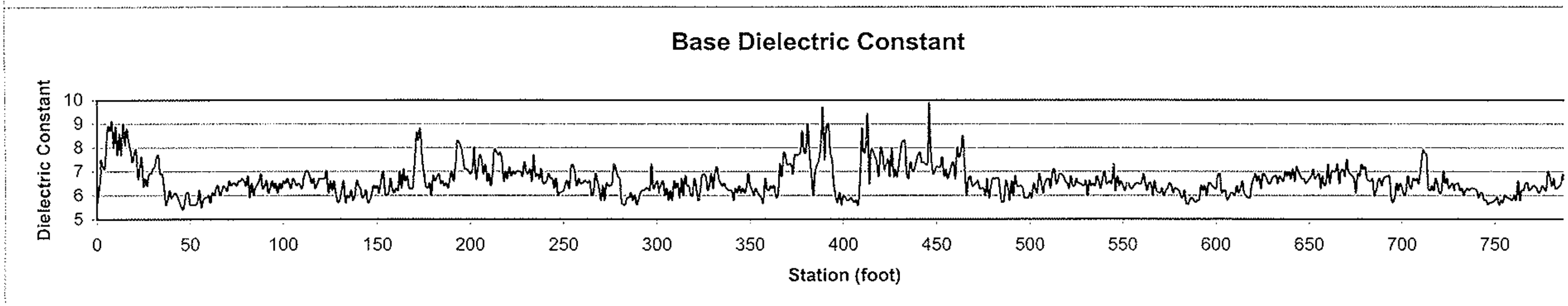


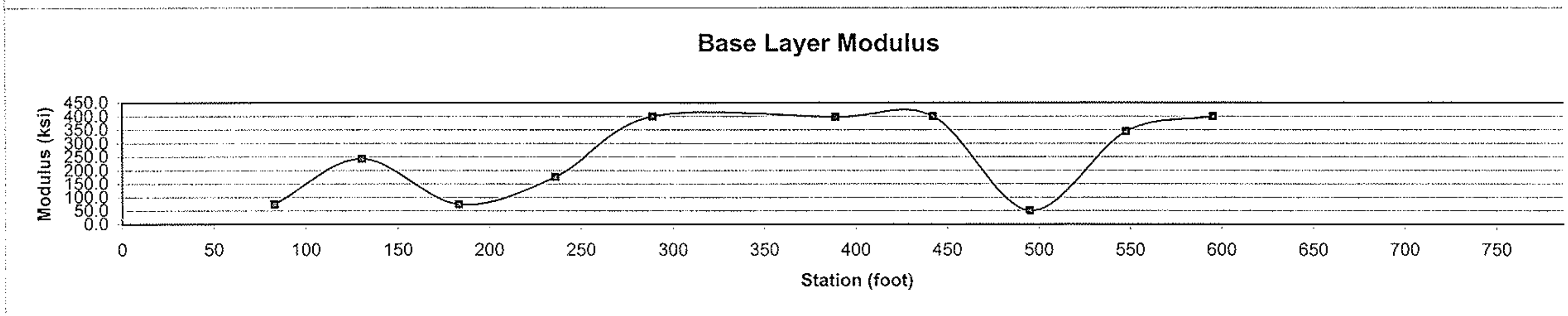
US-281 Southbound, Slow Lane, Section 480165: Crushed Concrete Aggregate Base



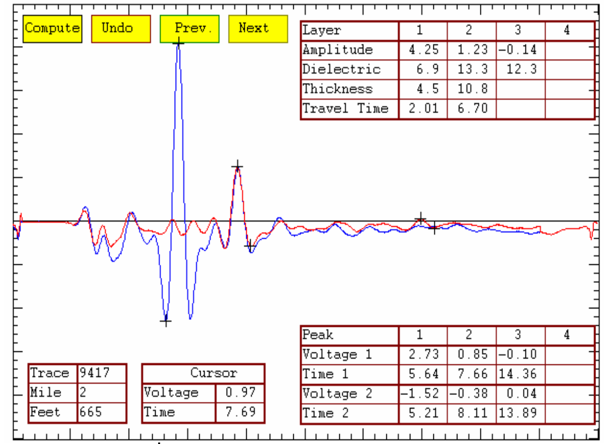
US-281 Southbound, Slow Lane, Section 480165: Crushed Concrete Aggregate Base

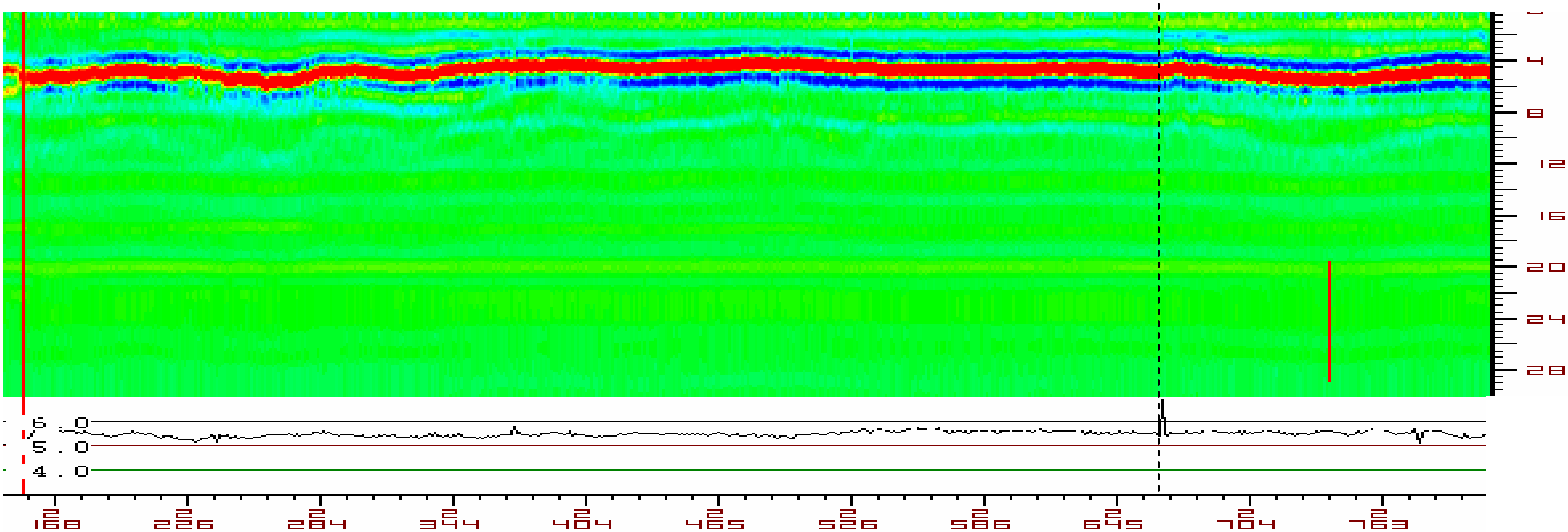




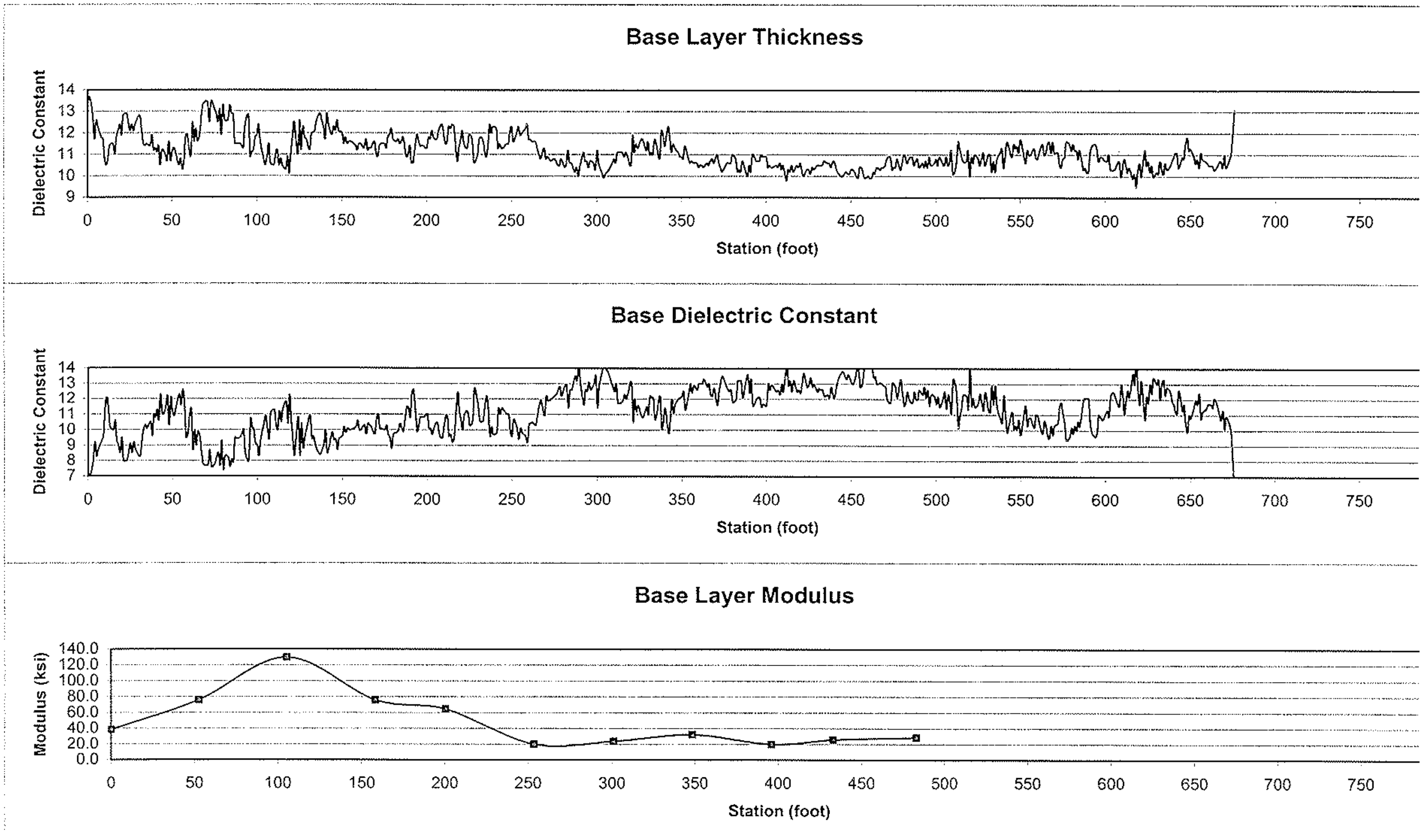


US-281 Southbound, Slow Lane, Section 480166: Caliche Base Modified with 0.5% Lime

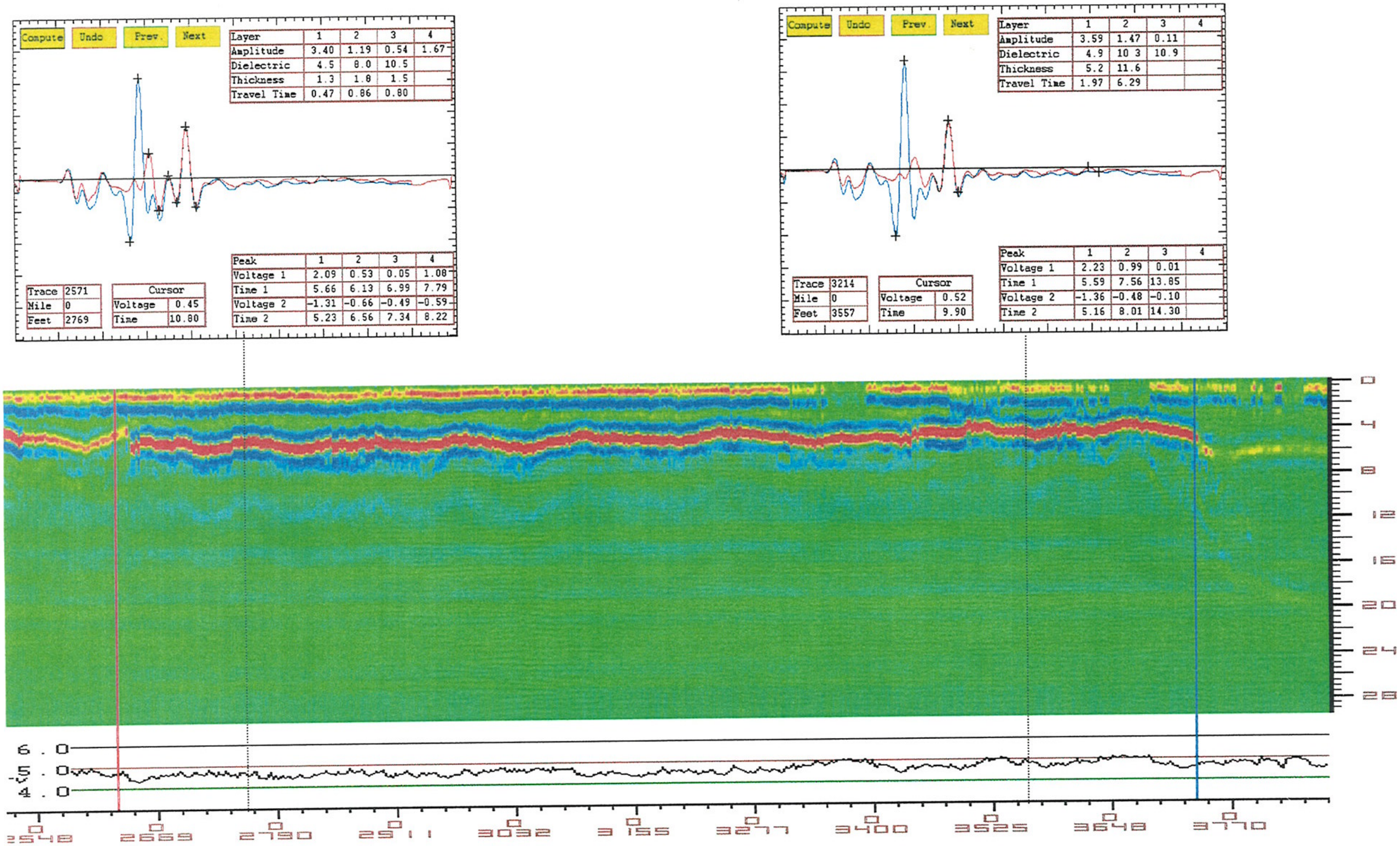




US-281 Southbound, Slow Lane, Section 480166: Caliche Base Modified with .05% Lime

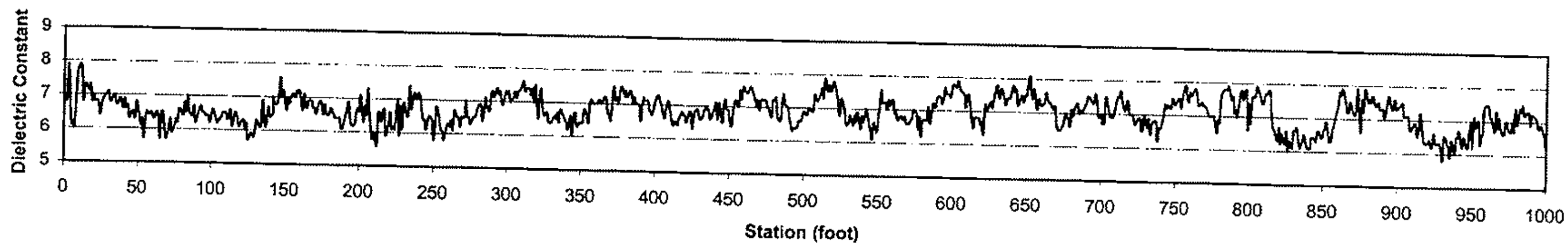


US-281 Southbound, Slow Lane, Section 480167: Caliche Base modified with ½% lime

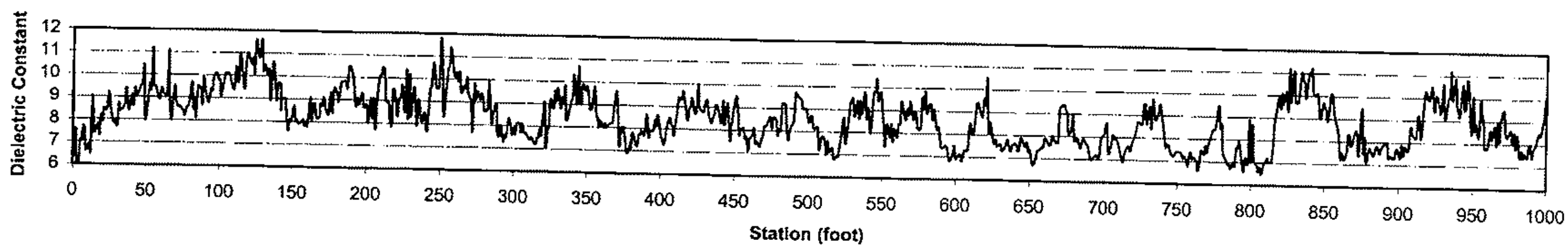


US-281 Southbound, Slow Lane, Section 480167: Caliche Base Modified with .05% Lime

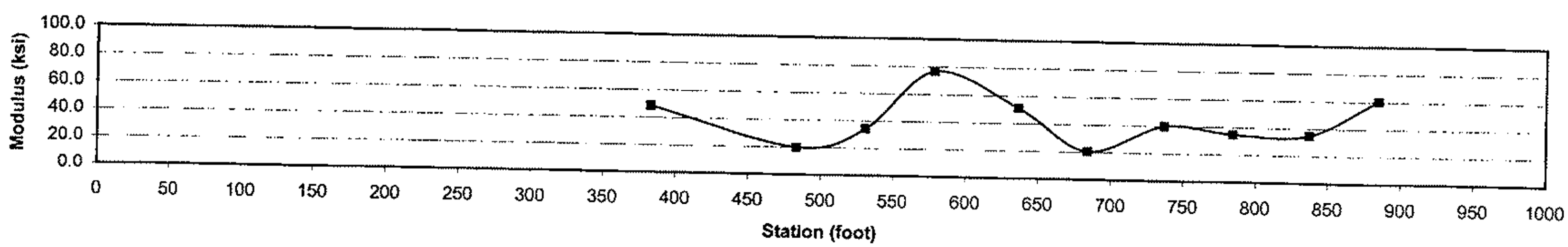
Base Layer Thickness



Base Dielectric Constant



Base Layer Modulus



APPENDIX C US-77

Summary of 2002 GPR Results

Detailed GPR Color Scheme Outputs

Detailed FWD and GPR Data Plots

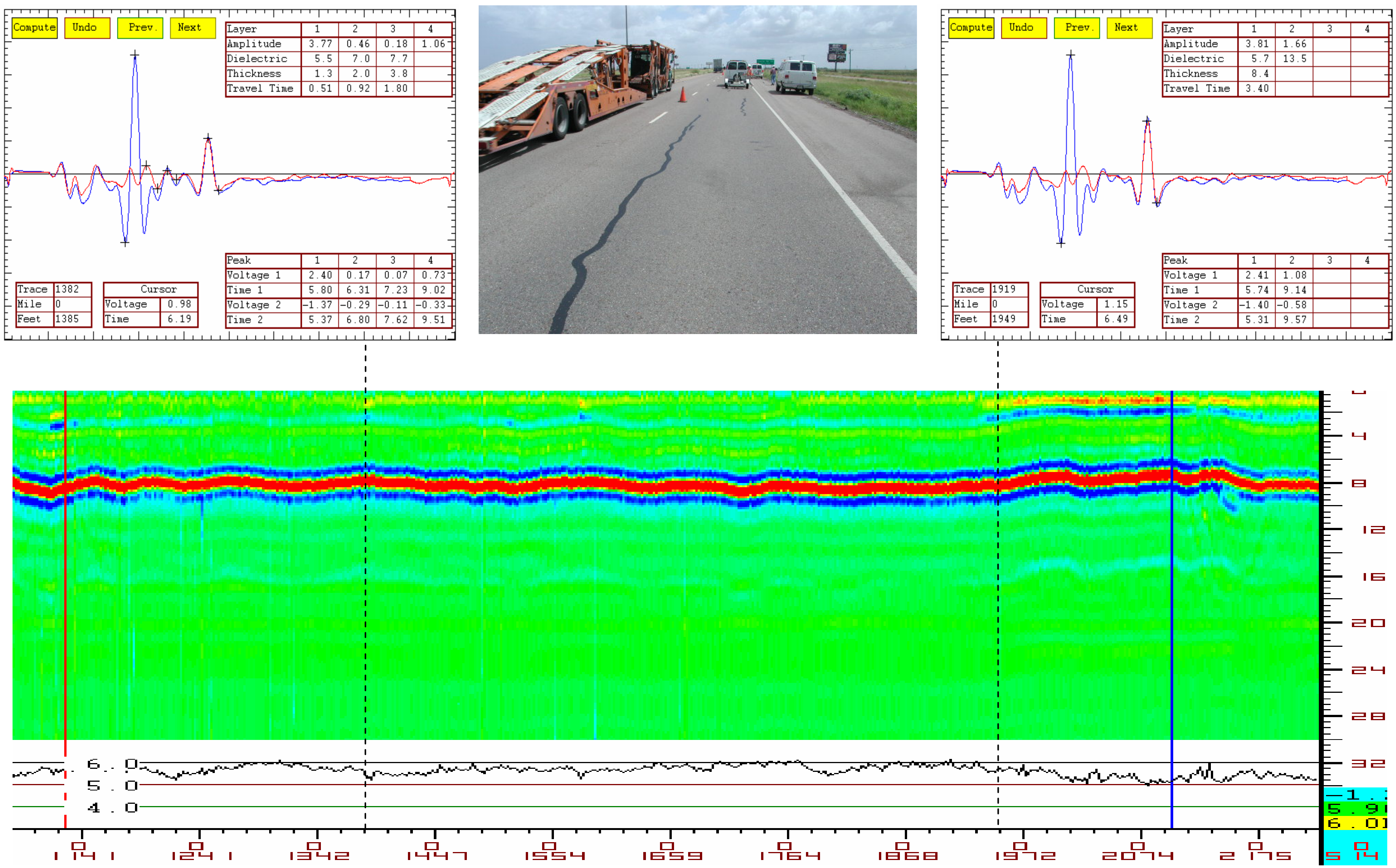
US-77: SUMMARY OF 2002 GPR RESULTS

Section	Statistic	Asphalt Surface		Base	
		Thickness (in.)	Dielectric Constant	Thickness (in.)	Dielectric Constant
No.4 Northbound CAL+2% lime	Average	8.7	5.2	18 fixed	11.0
	CoV	0.04	0.06		0.11
	Minimum	7.8	4.5		6.9
	Maximum	9.7	6.0		15.1
No.4 Southbound CAL+2%lime	Average	9.6	4.8	18 fixed	9.6
	CoV	0.07	0.08		0.12
	Minimum	7.8	4.0		7.2
	Maximum	11.1	5.9		14.9
No.1 Northbound CAL+4% cement	Average	8.4	5.7	18 fixed	11.7
	CoV	0.03	0.04		0.09
	Minimum	7.6	4.9		8.0
	Maximum	9.1	6.1		14.9
No.1 Southbound CAL+4% cement	Average	8.8	5.2	18 fixed	12.0
	CoV	0.07	0.09		0.13
	Minimum	7.6	4.5		8.8
	Maximum	10.1	6.4		17.9
No.2 Northbound Yucatan LS	Average	8.6	5.5	18 fixed	8.5
	CoV	0.04	0.04		0.07
	Minimum	7.7	4.9		7.1
	Maximum	9.4	6.2		11.1
No.2 Southbound Yucatan LS	Average	9.2	5.1	18 fixed	8.6
	CoV	0.04	0.08		0.16
	Minimum	8.1	4.2		5.6
	Maximum	10.2	6.2		13.8
No.3 Northbound LRA	Average	8.3	5.5	18 fixed	7.8
	CoV	0.03	0.04		0.07
	Minimum	7.6	5.0		6.5
	Maximum	9.0	6.1		10.3
No.3 Southbound LRA	Average	8.9	5.3	18 fixed	8.0
	CoV	0.05	0.06		0.09
	Minimum	7.9	4.6		6.5
	Maximum	10.0	5.9		13.2

US-77: SUMMARY OF 2002 GPR RESULTS (CONTINUE)

Section	Statistic	Asphalt Surface Thickness (in.)	Asphalt Surface Dielectric Constant	Base Thickness (in.)	Base Dielectric Constant
No.4 Northbound CAL+2% lime	Average	9.4	4.8	18 fixed	9.6
	CoV	0.03	0.05		0.08
	Minimum	8.4	4.3		6.7
	Maximum	10.2	5.7		14.3
No.4 Southbound CAL+2%lime	Average	10.1	4.7	18 fixed	9.2
	CoV	0.04	0.03		0.07
	Minimum	9.1	4.4		7.6
	Maximum	11.3	5.2		11.8
No.1 Northbound CAL+4% cement	Average	9.2	4.8	18 fixed	10.1
	CoV	0.04	0.04		0.07
	Minimum	8.1	4.3		8.3
	Maximum	10.0	5.5		13.2
No.1 Southbound CAL+4% cement	Average	9.7	4.6	18 fixed	9.9
	CoV	0.03	0.05		0.08
	Minimum	8.5	4.3		7.6
	Maximum	10.4	5.7		13.0
No.2 Northbound Yucatan LS	Average	9.4	4.8	18 fixed	8.0
	CoV	0.04	0.05		0.09
	Minimum	8.4	4.3		6.5
	Maximum	10.6	5.7		12.1
No.2 Southbound Yucatan LS	Average	9.8	4.7	18 fixed	7.4
	CoV	0.04	0.06		0.08
	Minimum	8.9	4.2		6.0
	Maximum	11.6	5.8		9.9
No.3 Northbound LRA No.4	Average	9.4	4.8	18 fixed	6.9
	CoV	0.04	0.04		0.06
	Minimum	8.5	4.3		6.1
	Maximum	10.9	5.8		9.4
No.3 Southbound LRA	Average	9.8	4.6	18 fixed	7.1
	CoV	0.03	0.04		0.09
	Minimum	8.4	4.3		5.9
	Maximum	10.4	5.5		11.8

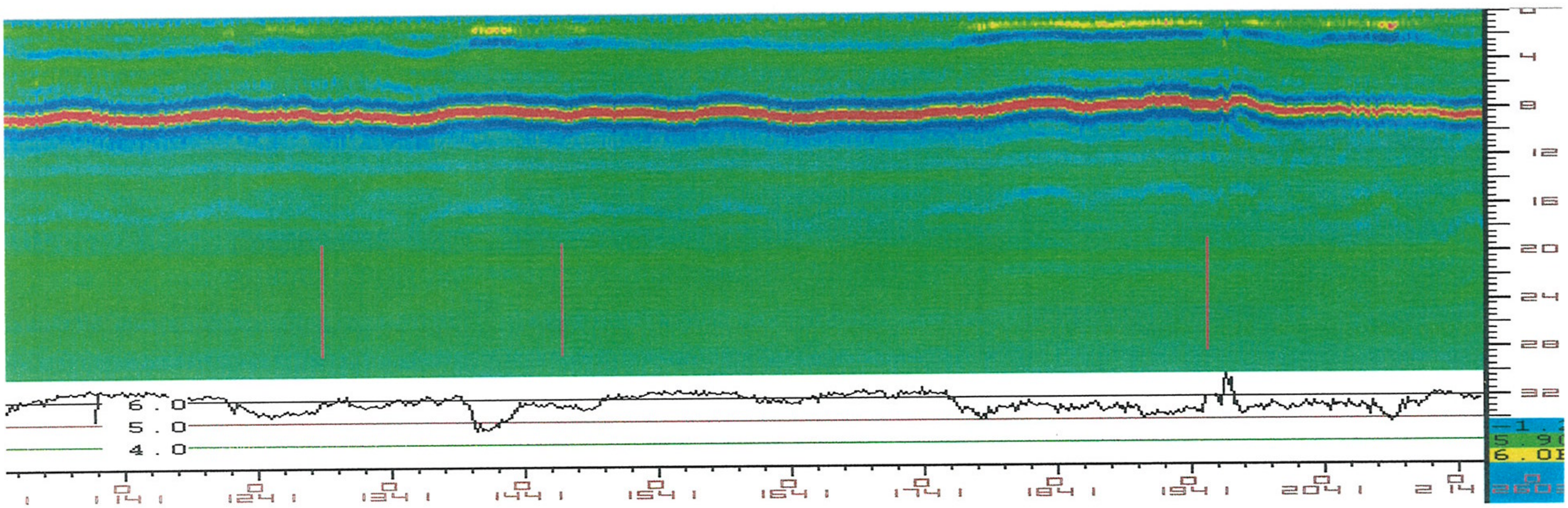
US-77 Northbound, Slow Lane, Section # 1: Caliche Base stabilized with 4% cement



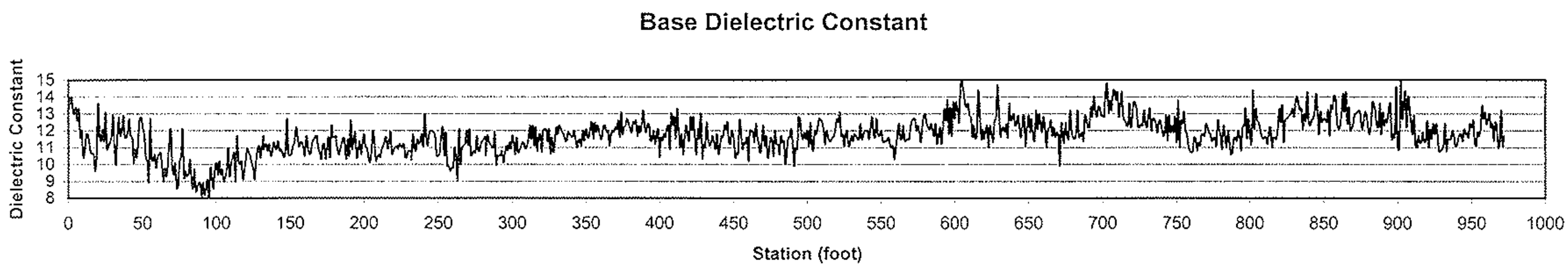
US-77 Northbound, Slow Lane, Sections #1: Maneuver over defects

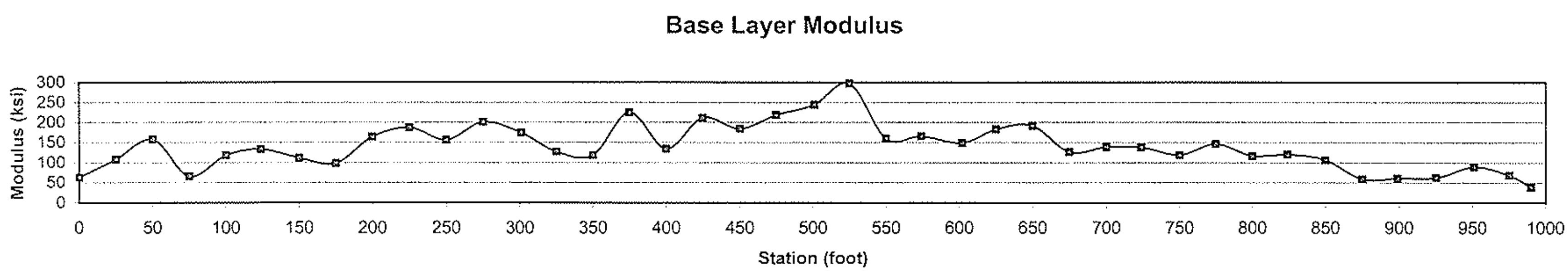
1. Typical sealed cracks in middle of lane

2. Longitudinal crack in left wheel path



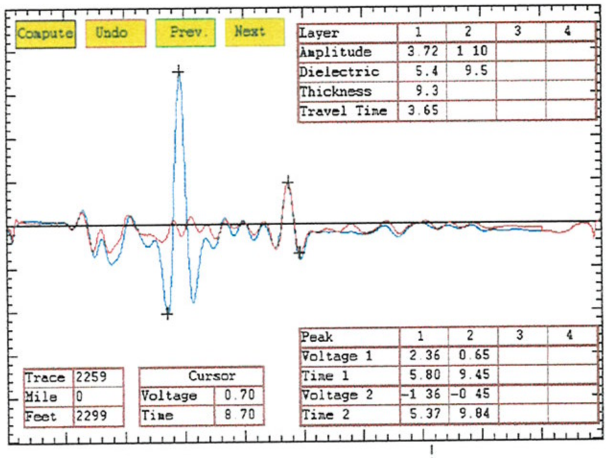
US-77 Northbound, Slow Lane, Section # 1: Caliche Base stabilized with 4% cement

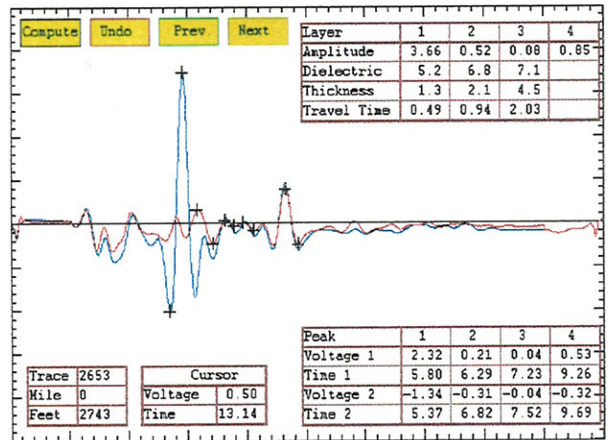


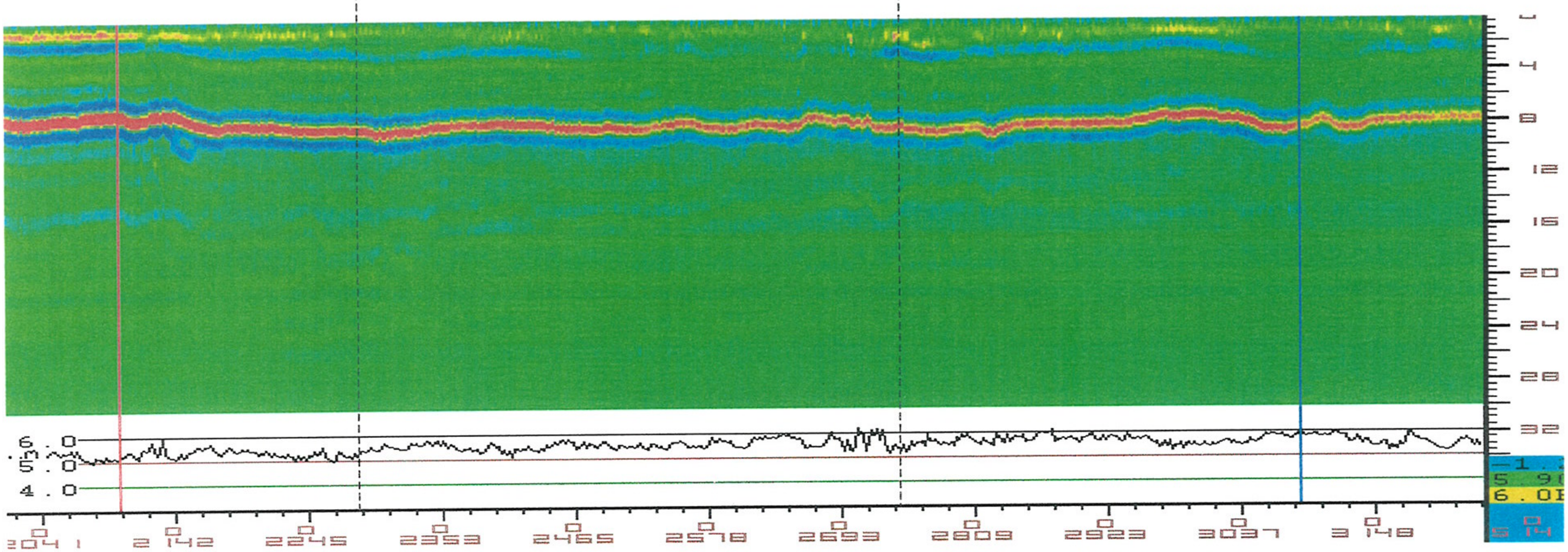


Note: Base layer thickness 18 inches fixed

US-77 Northbound, Slow Lane, Section # 2: Yucatan Limestone Base







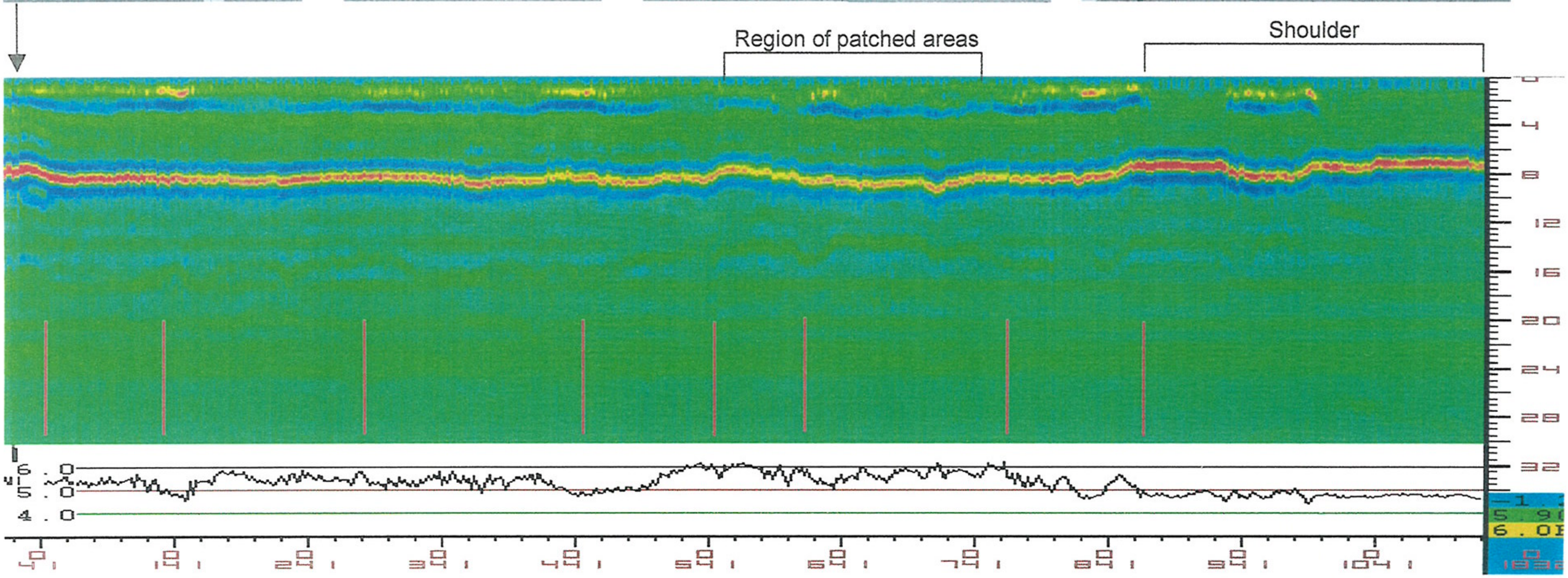
US-77 Northbound, Slow Lane, Section # 2: Maneuver over defects

1. Patched failure

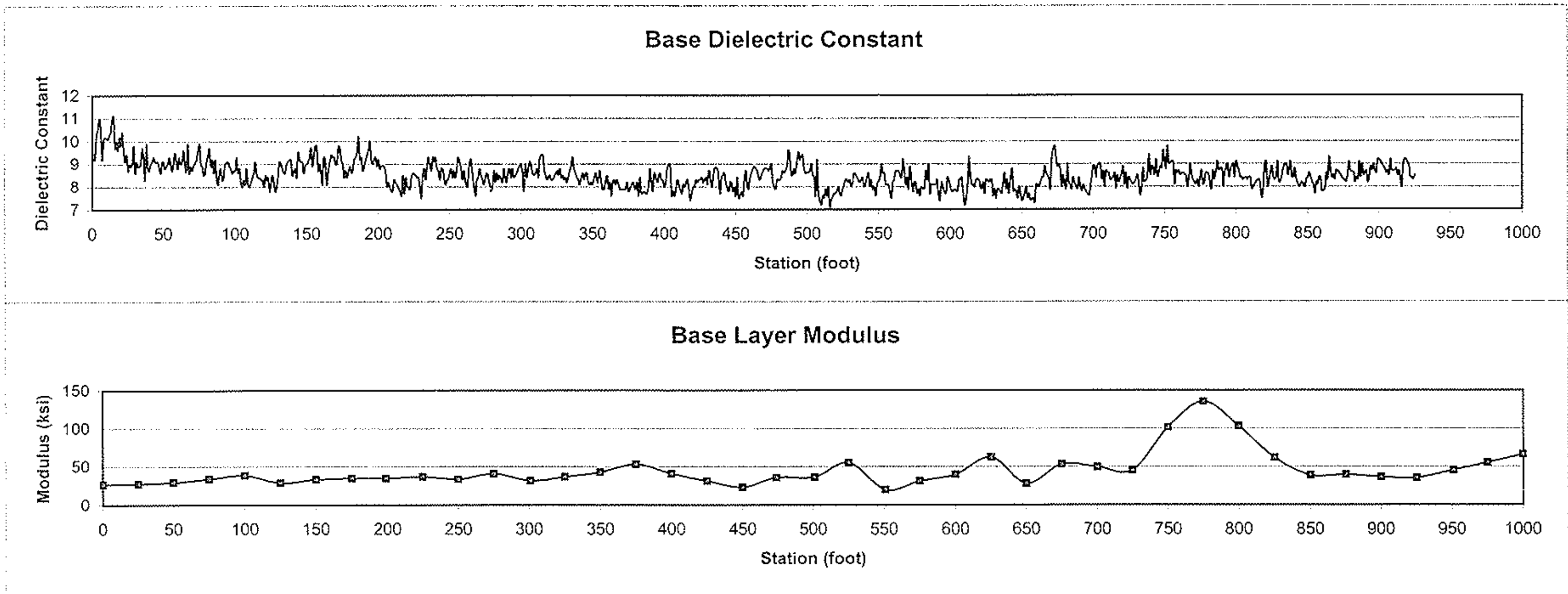
2. Typical sealed cracks

3. Patches

4. Sealed cracks on shoulder

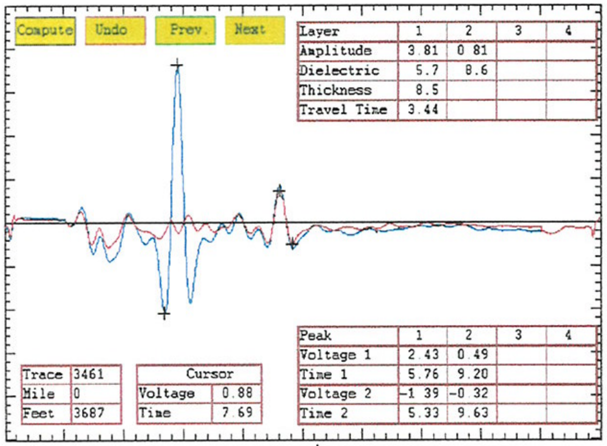


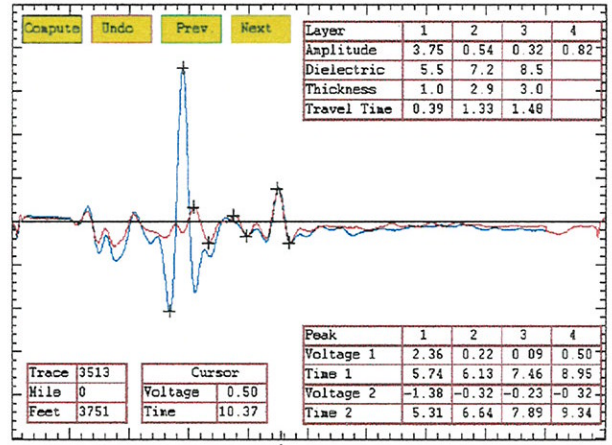
US-77 Northbound, Slow Lane, Section # 2: Yucatan Limestone Base

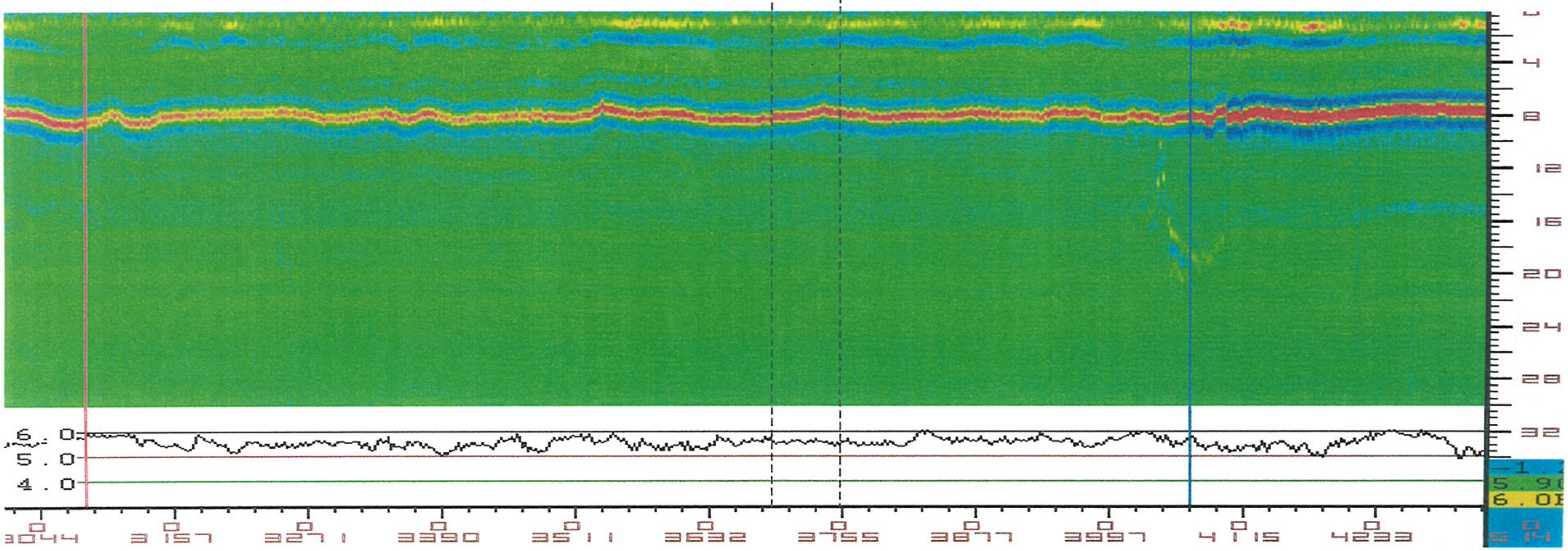


Note: Base layer thickness 18 inches fixed

US-77 Northbound, Slow Lane, Section # 3: Lime Rock Asphalt Base



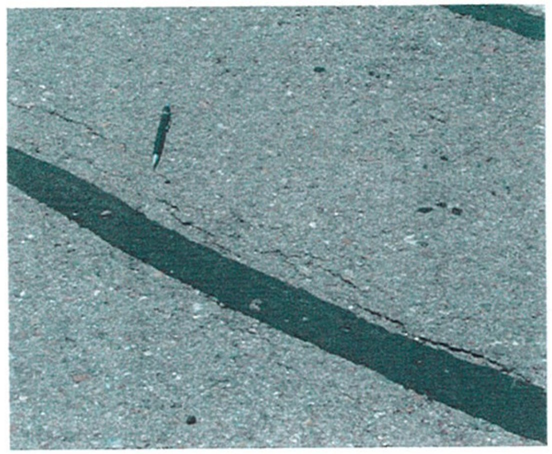




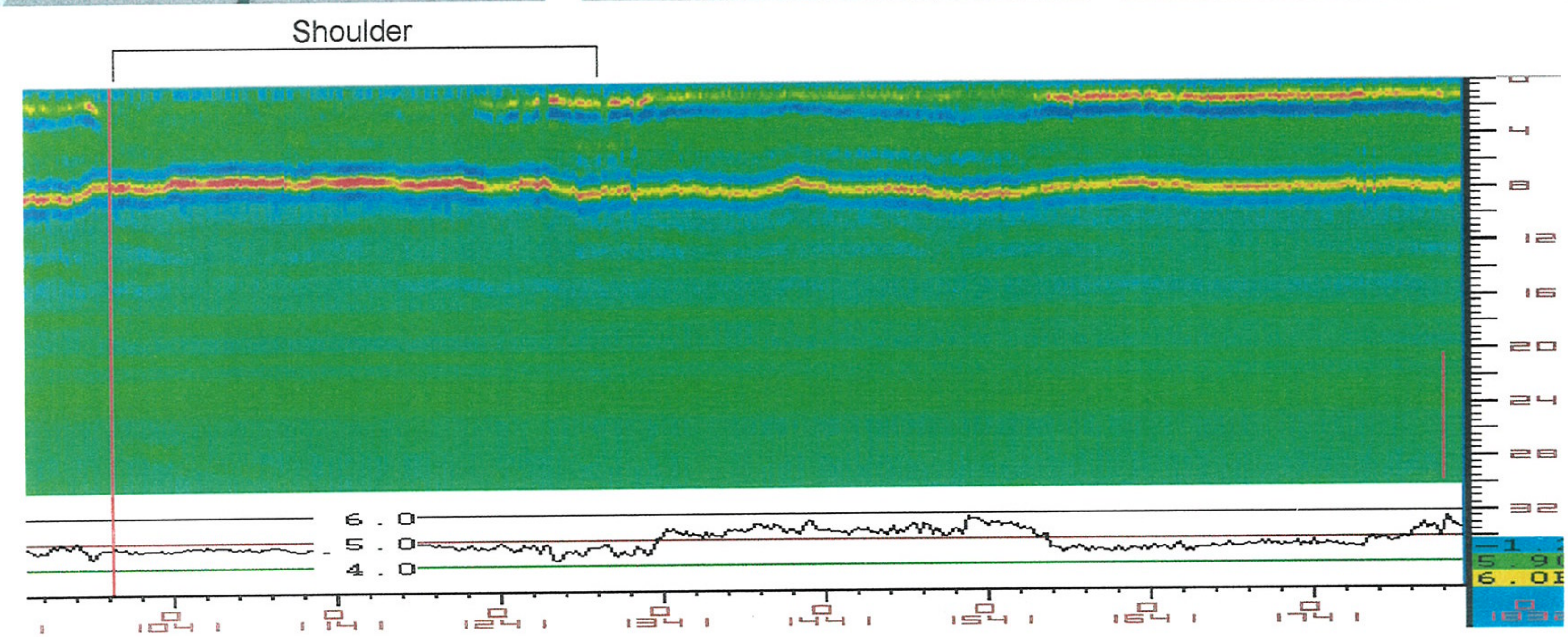
US-77 Northbound, Slow Lane, Section # 3: Maneuver over defects

1. Sealed cracks on shoulder

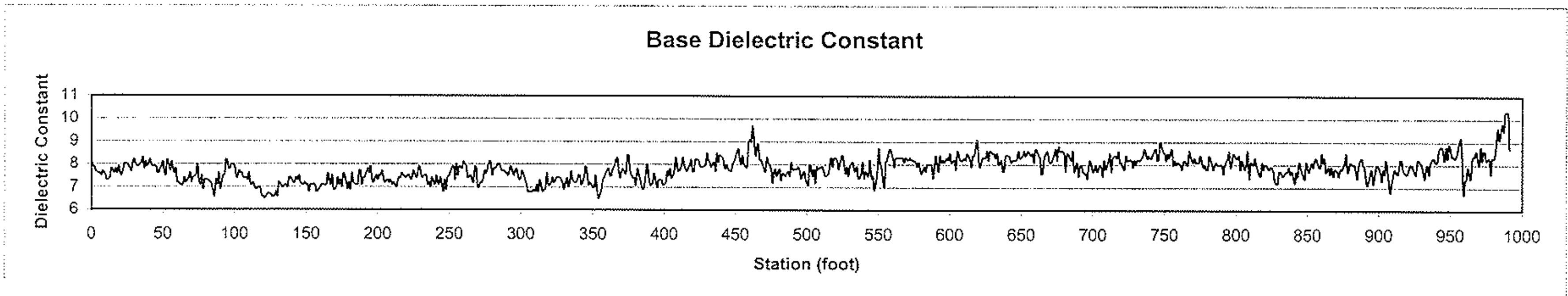
2. Crack formation in area of sealed cracks

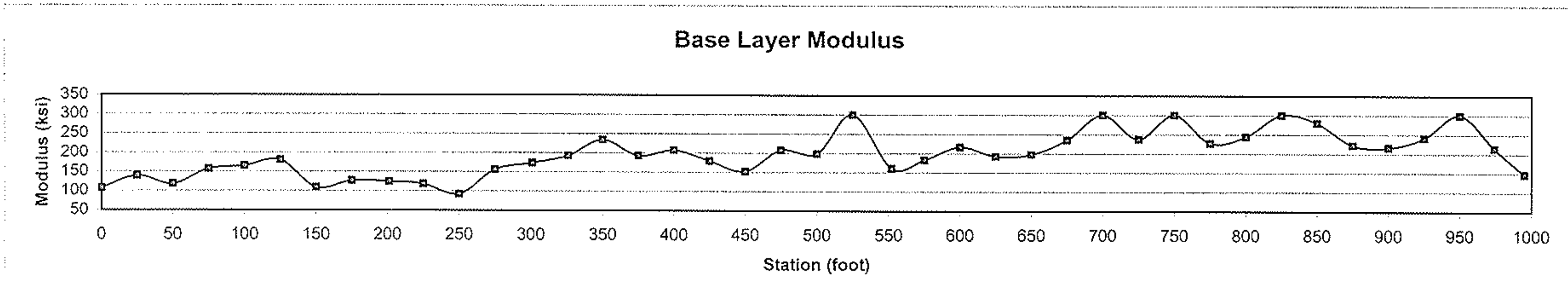


3. Typical unsealed cracks on shoulders



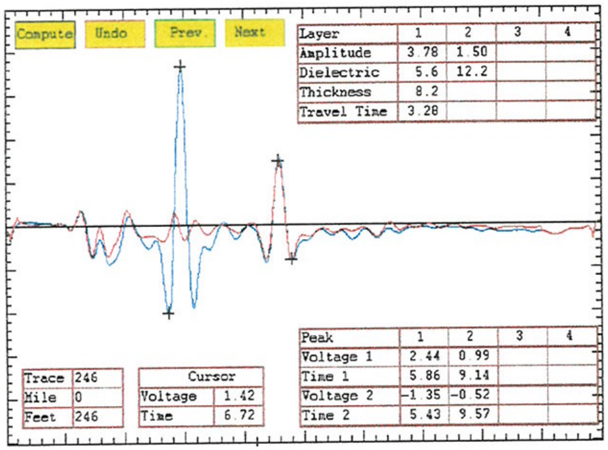
US-77 Northbound, Slow Lane, Section # 3: Lime Rock Asphalt Base

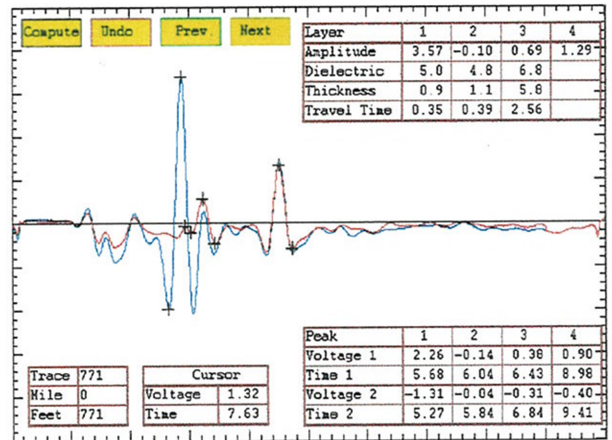


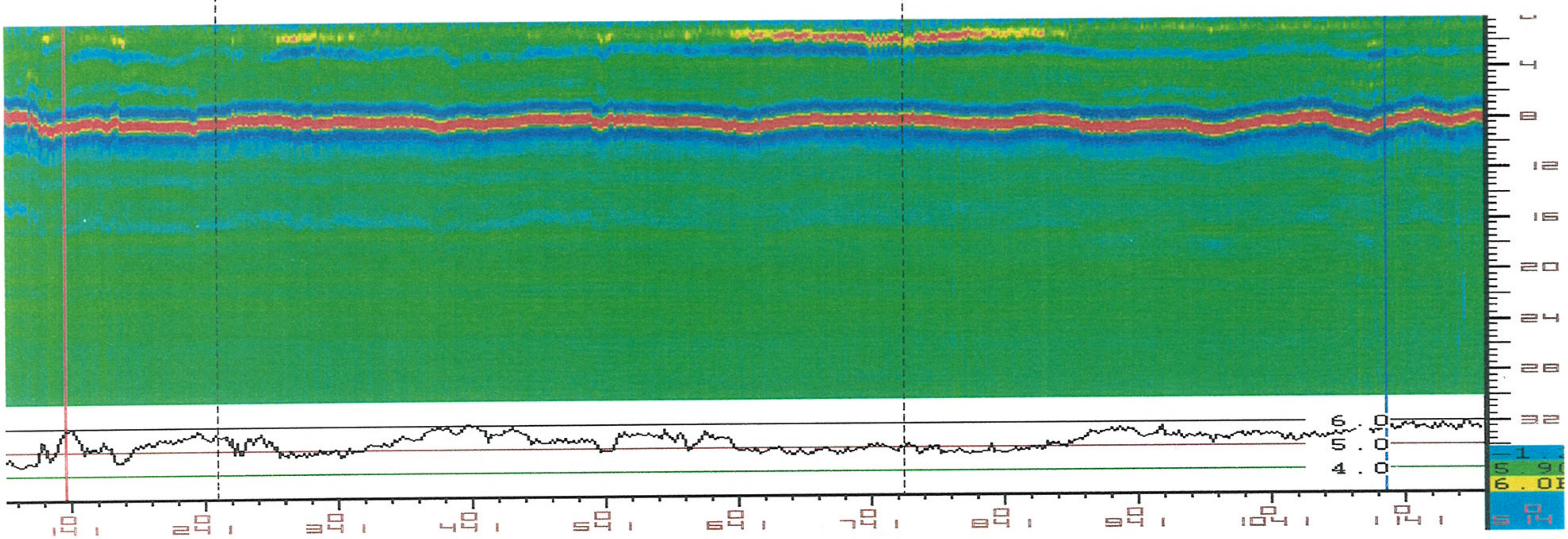


Note: Base layer thickness 18 inches fixed

US-77 Northbound, Slow Lane, Section # 4: Caliche Base stabilized with 2% lime



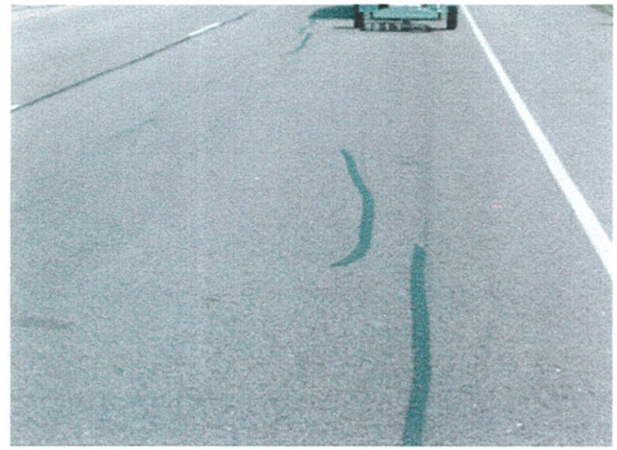


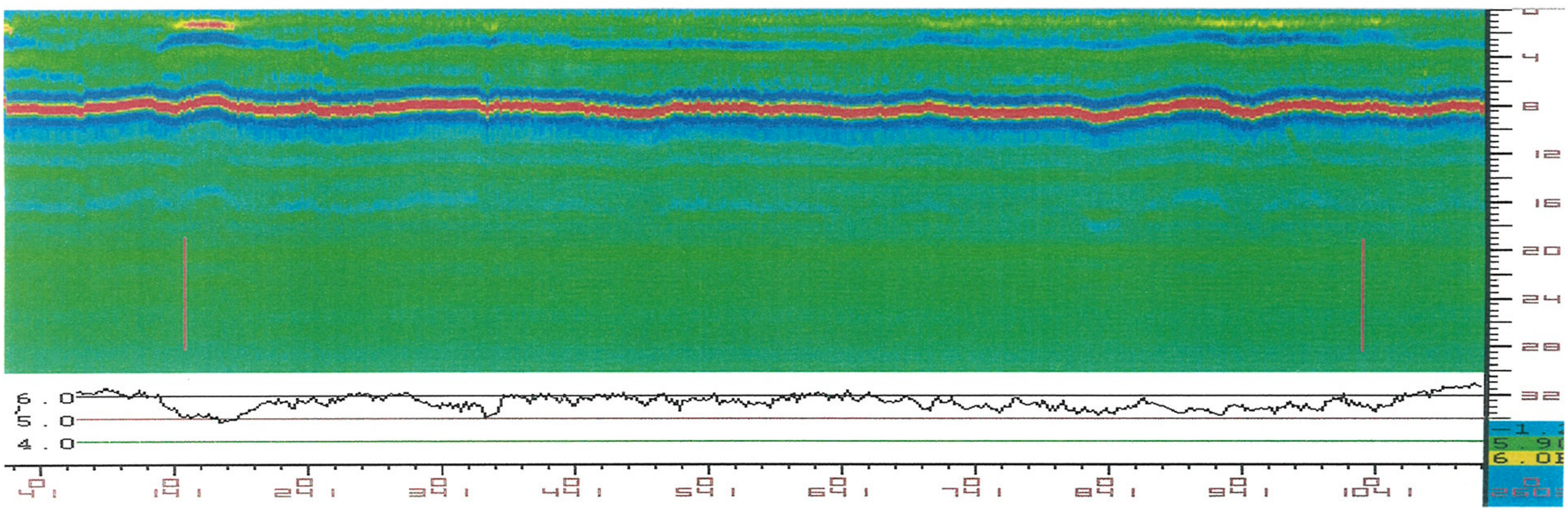


US-77 Northbound, Slow Lane, Section #4: Maneuver over defects

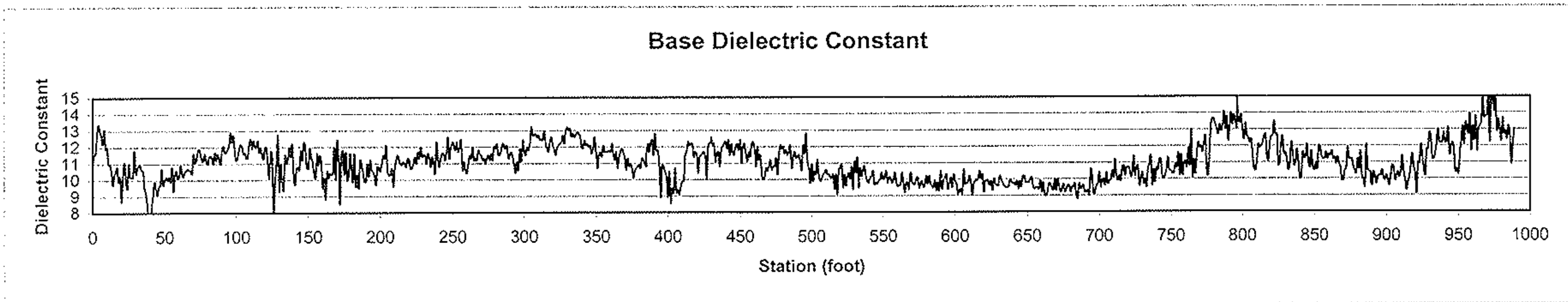
1. Typical sealed longitudinal cracks

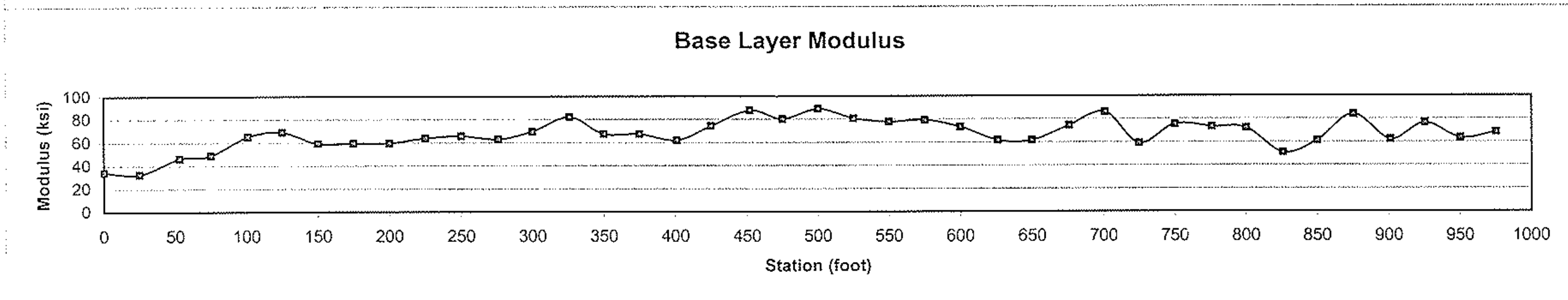
2. Longitudinal cracks appears to be on construction joint





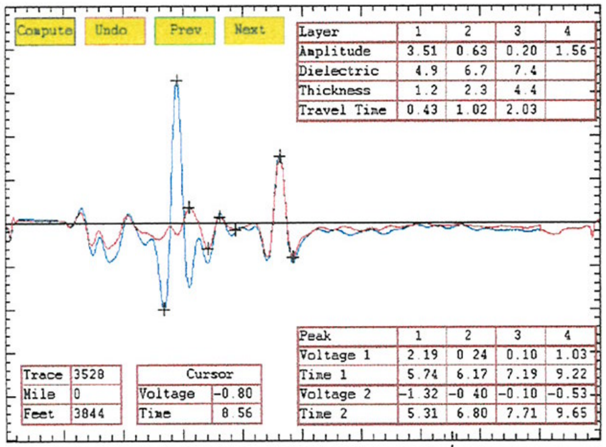
US-77 Northbound, Slow Lane, Section # 4: Caliche Base stabilized with 2% lime



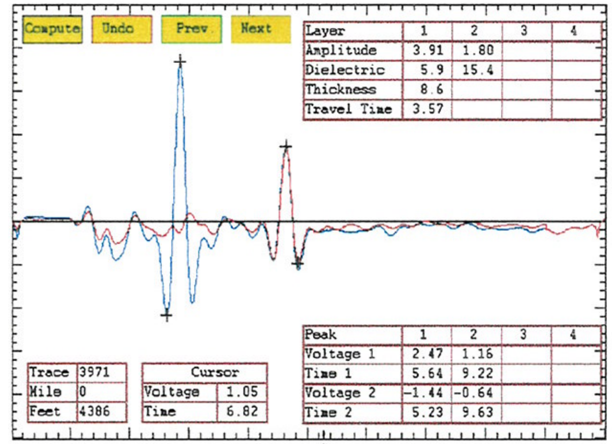


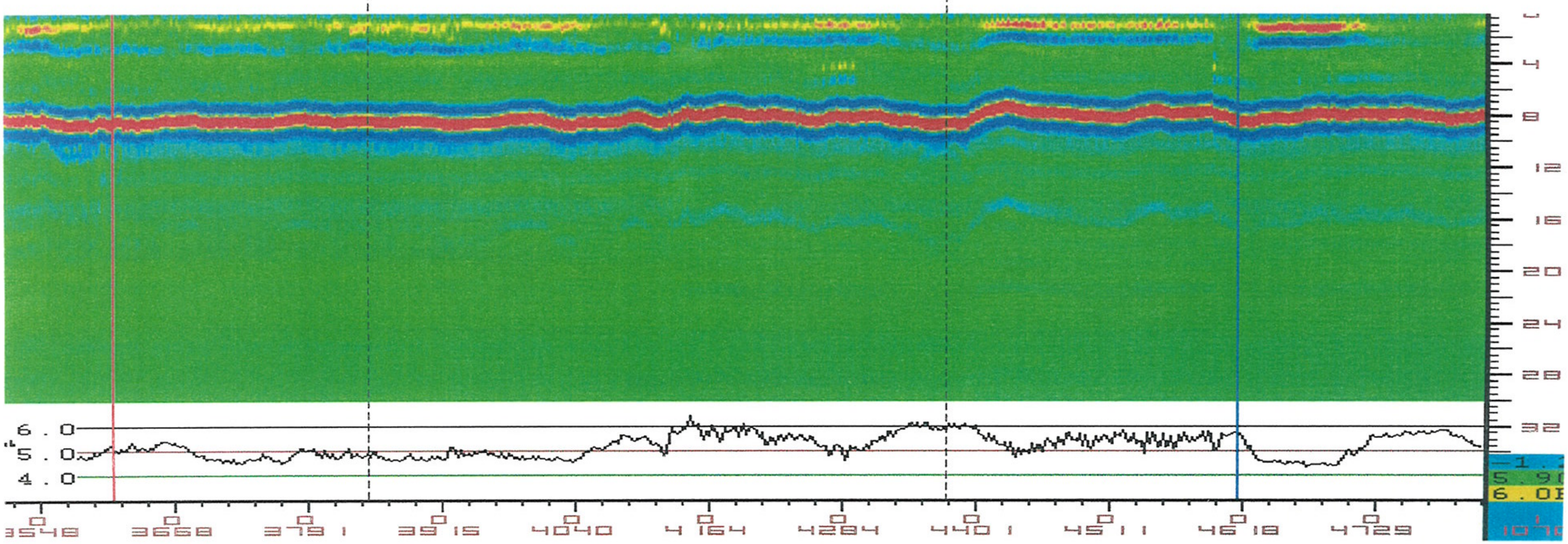
Note: Base layer thickness 18 inches fixed

US-77 Southbound, Slow Lane, Section # 1: Caliche Base stabilized with 4% cement

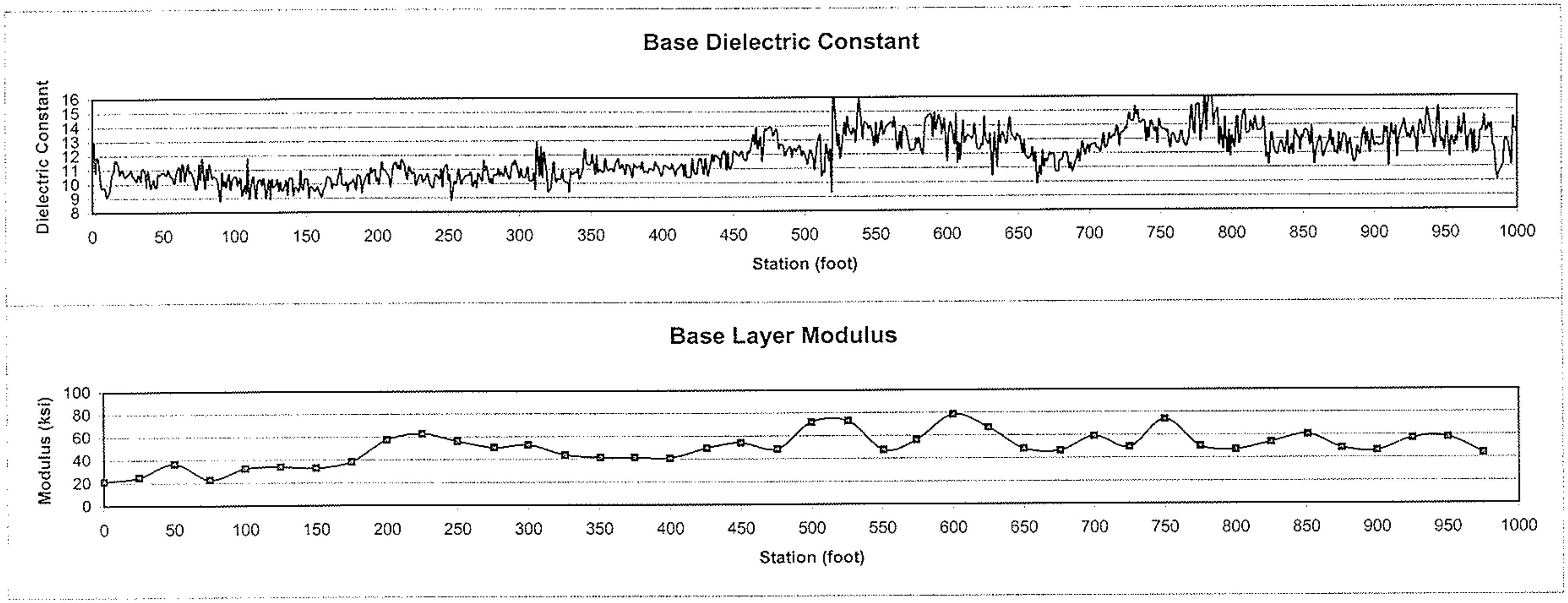


Segregation on patched area. Patch 25 X 12 feet.





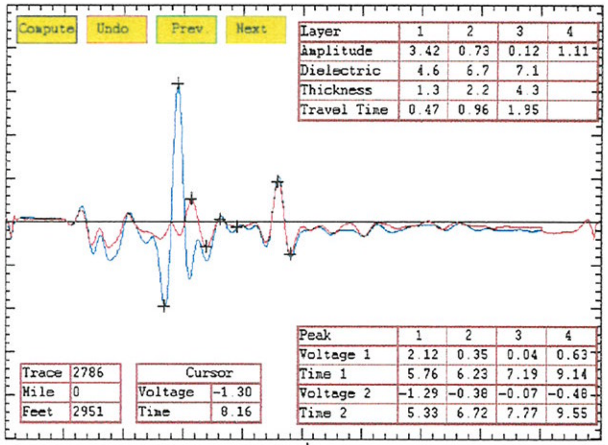
US-77 Southbound, Slow Lane, Section # 1: Caliche Base stabilized with 4% cement

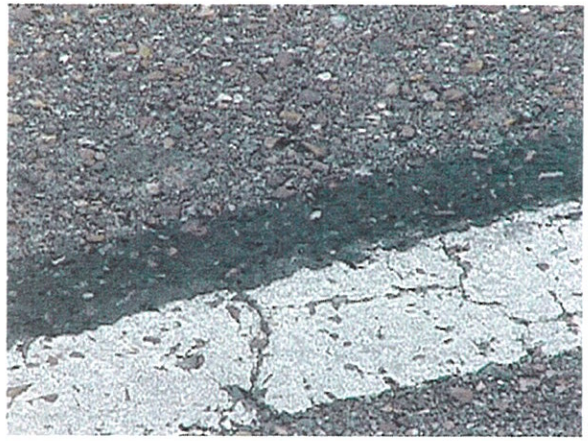


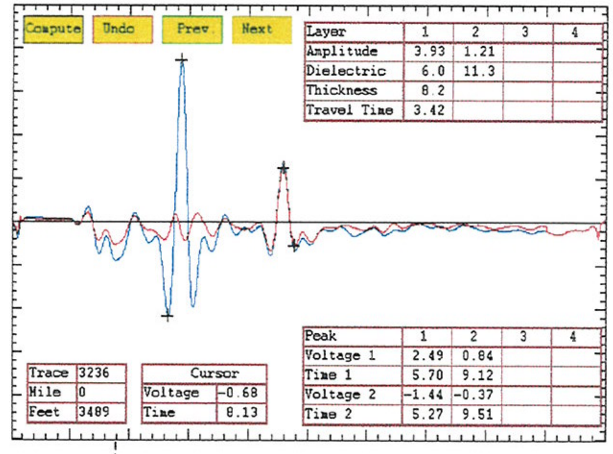
Note: Base layer thickness 18 inches fixed

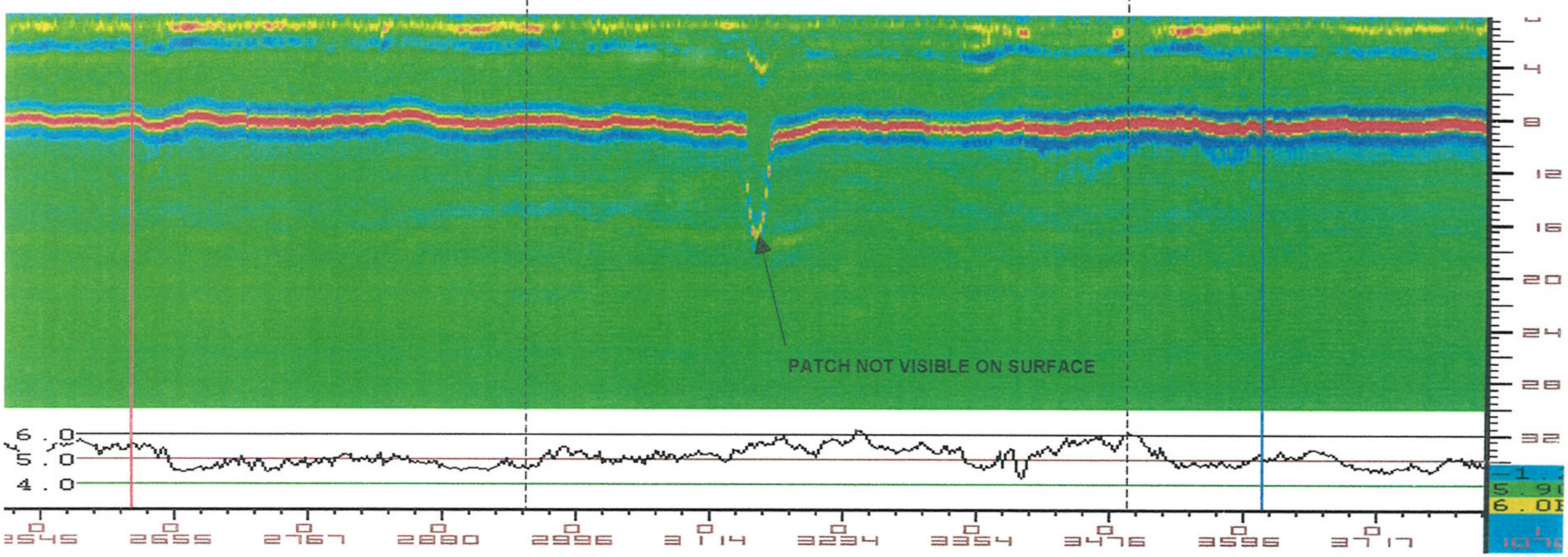
US-77 Southbound, Slow Lane, Section # 2: Yucatan Limestone Base

longitudinal crack (100 feet) on on white line

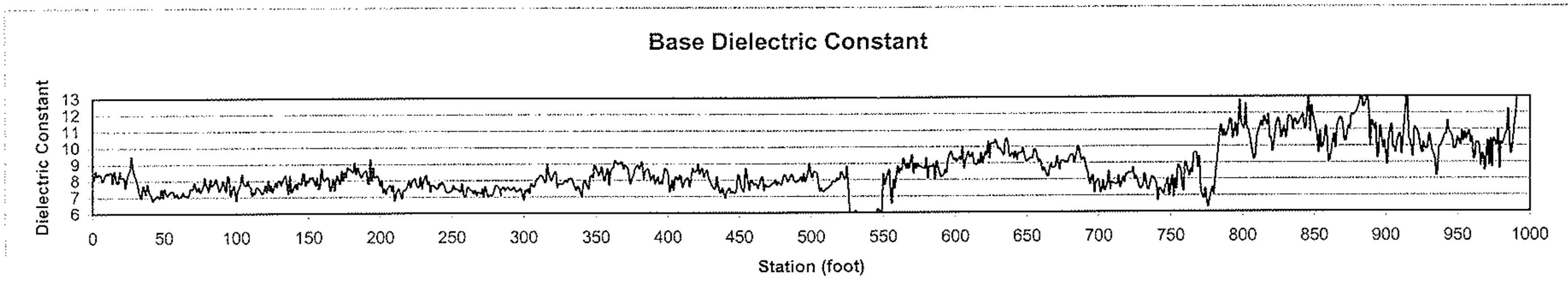


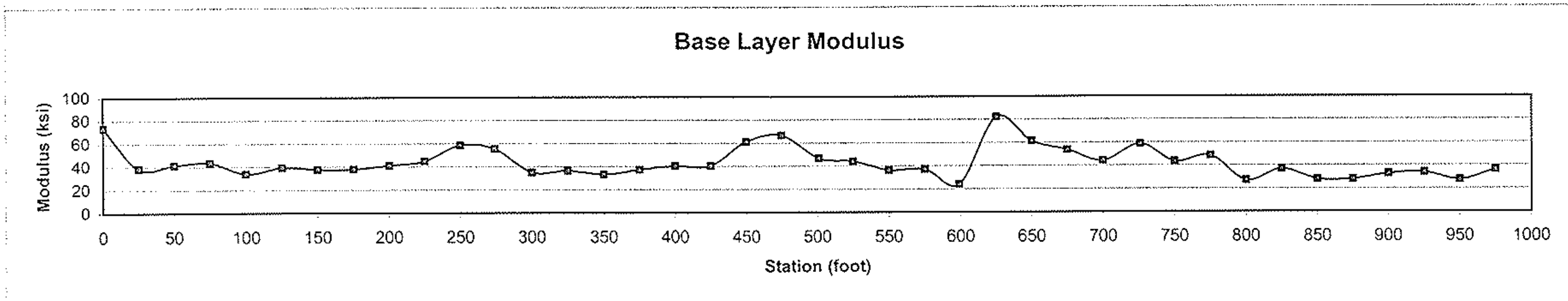






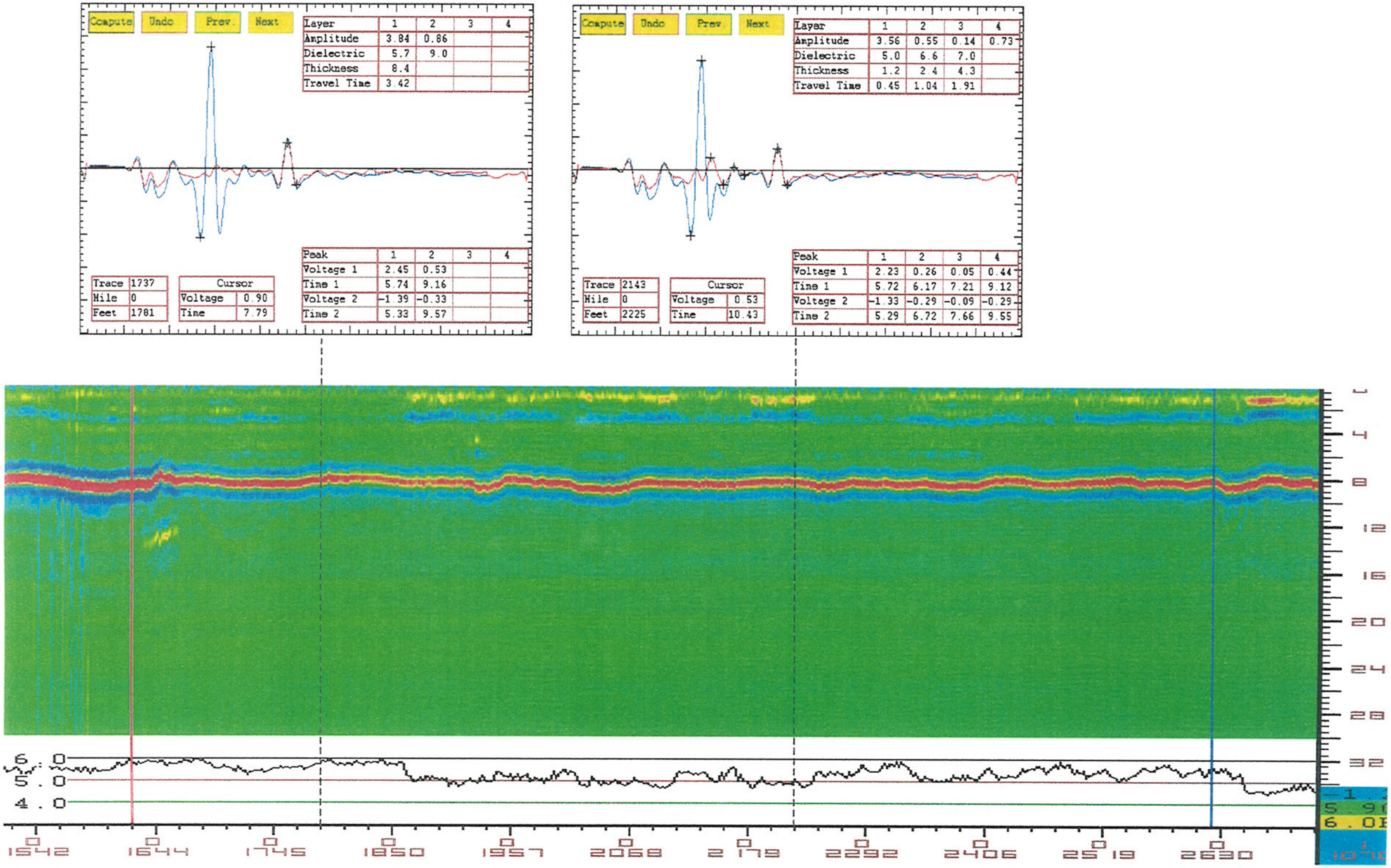
US-77 Southbound, Slow Lane, Section # 2: Yucatan Limestone Base





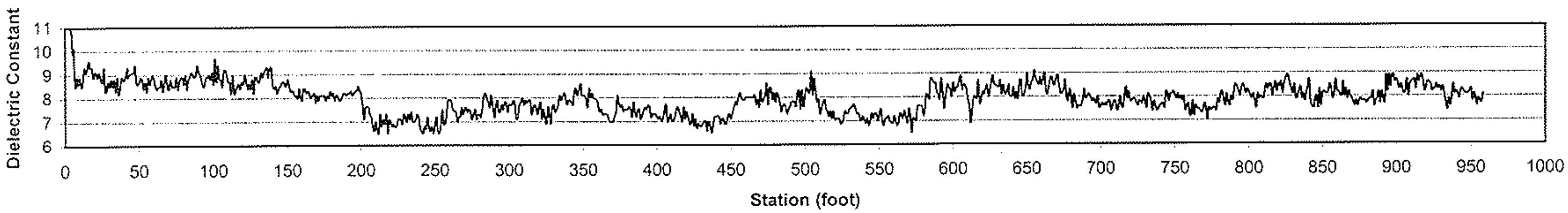
Note: Base layer thickness 18 inches fixed

US-77 Southbound, Slow Lane, Section # 3: Lime Rock Asphalt Base

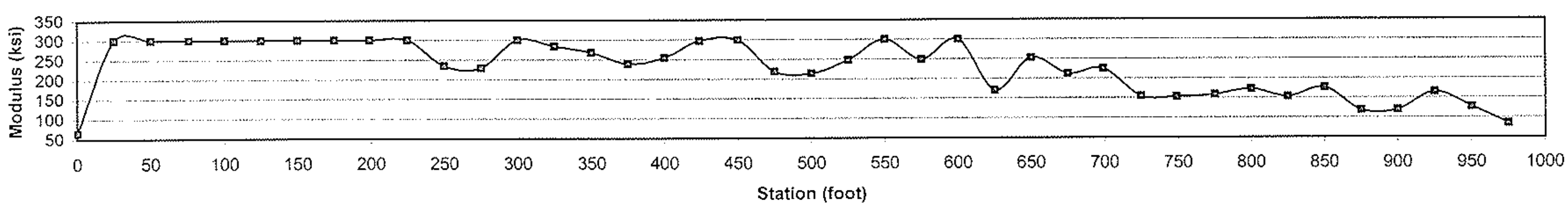


US-77 Southbound, Slow Lane, Section # 3: Lime Rock Asphalt Base

Base Dielectric Constant



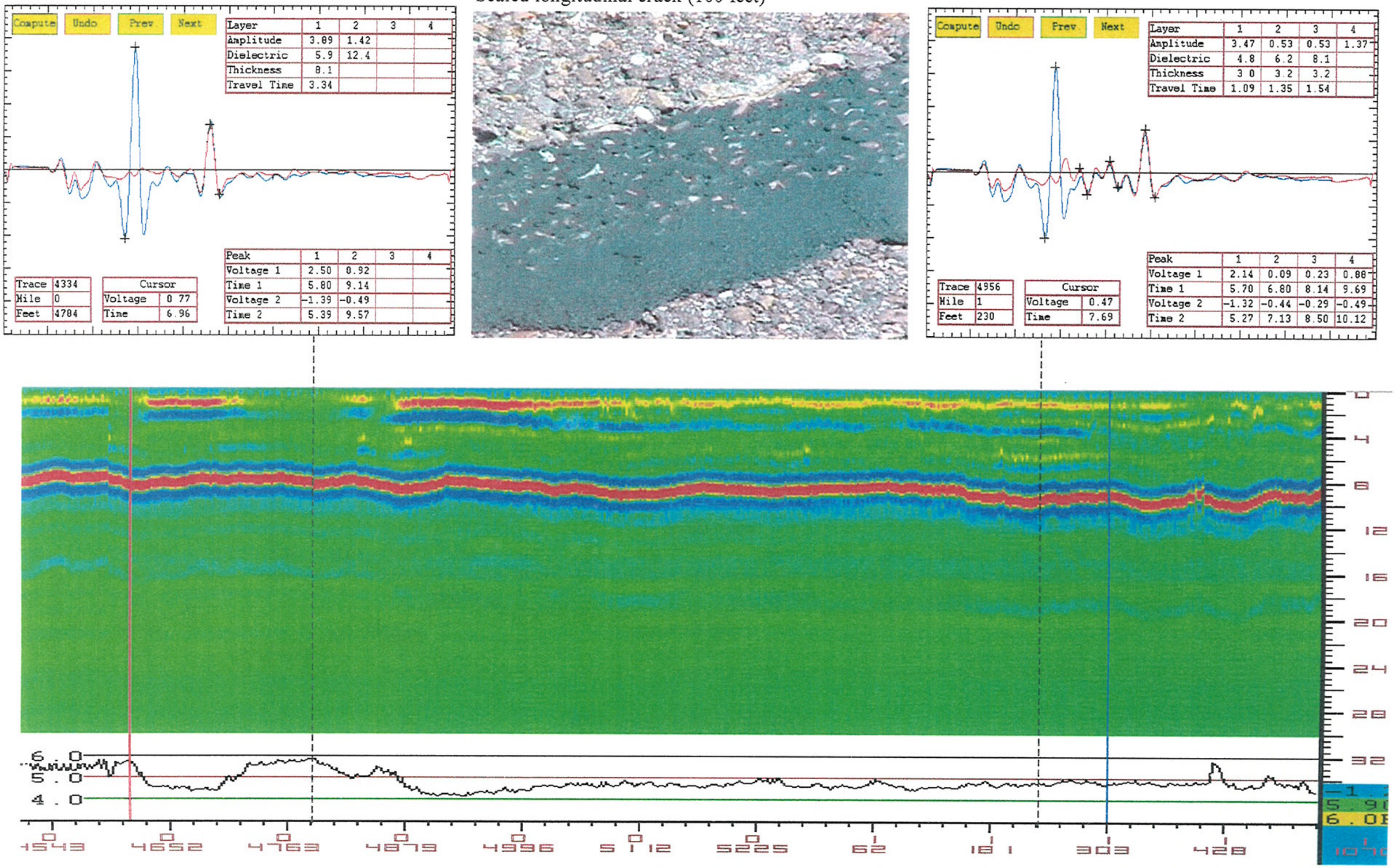
Base Layer Modulus



Note: Base layer thickness 18 inches fixed

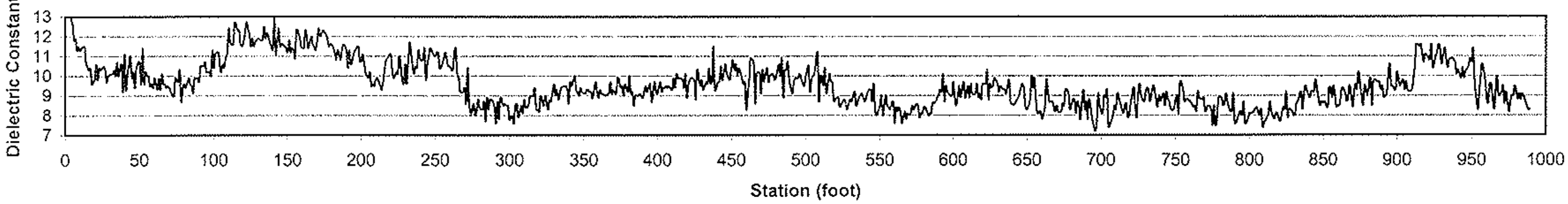
US-77 Southbound, Slow Lane, Section # 4: Caliche Base stabilized with 2% lime

Sealed longitudinal crack (100 feet)

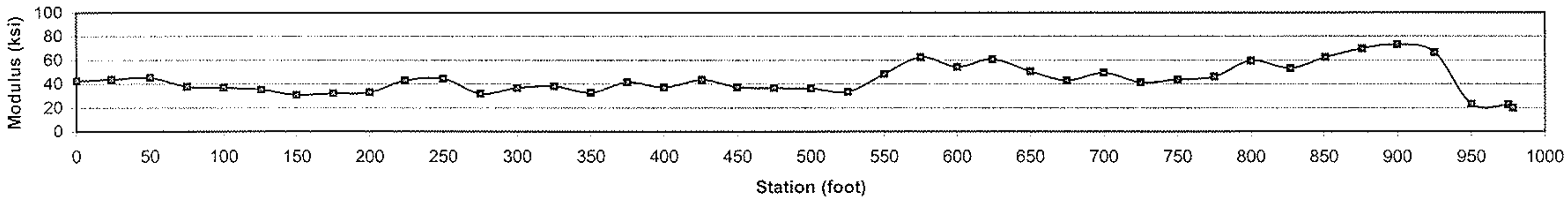


US-77 Southbound, Slow Lane, Section # 4: Caliche Base stabilized with 2% lime

Base Dielectric Constant



Base Layer Modulus



Note: Base layer thickness 18 inches fixed

APPENDIX D FM-1810

Summary of 2002 GPR Results

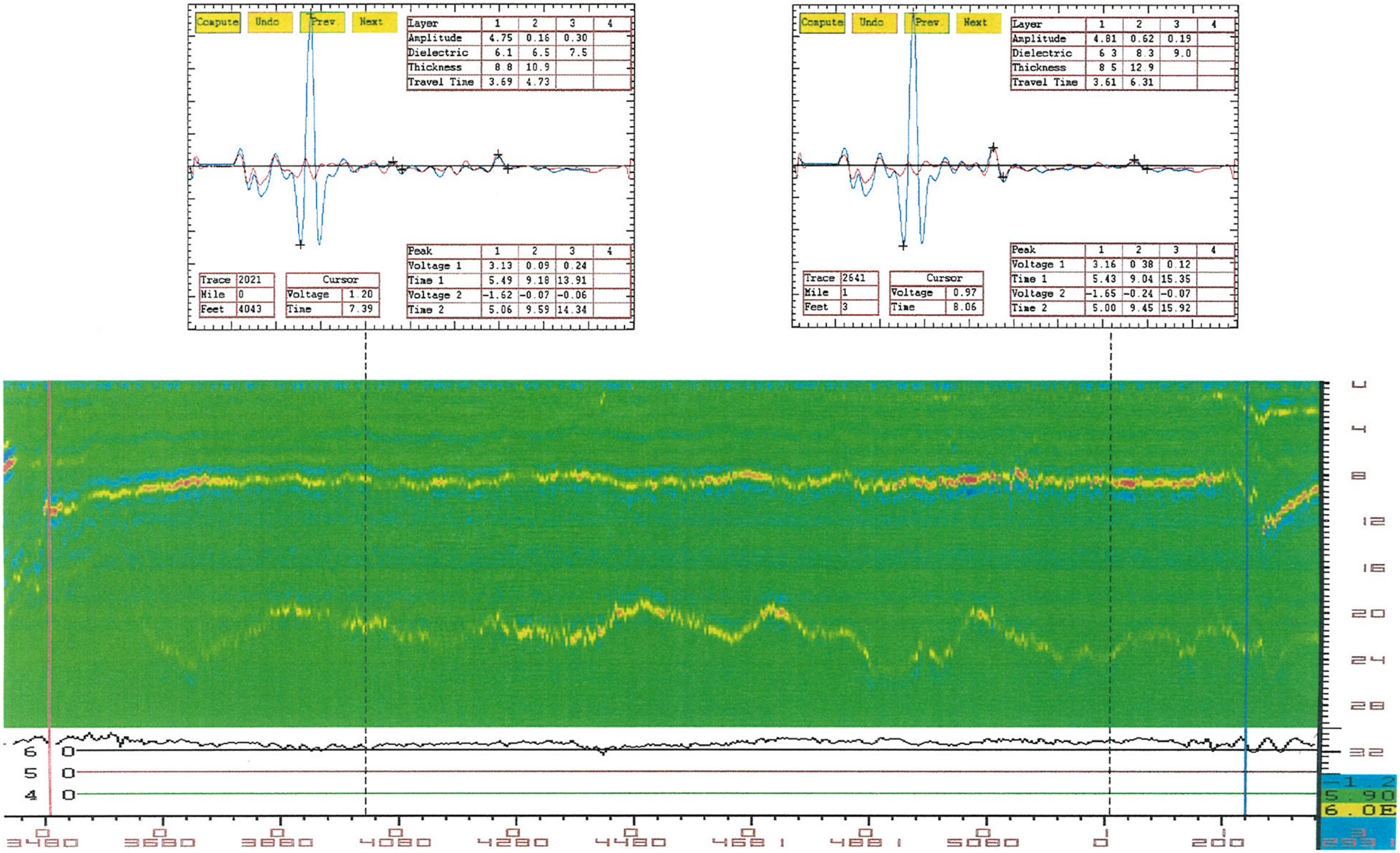
Detailed GPR Color Scheme Outputs

Detailed FWD and GPR Data Plots

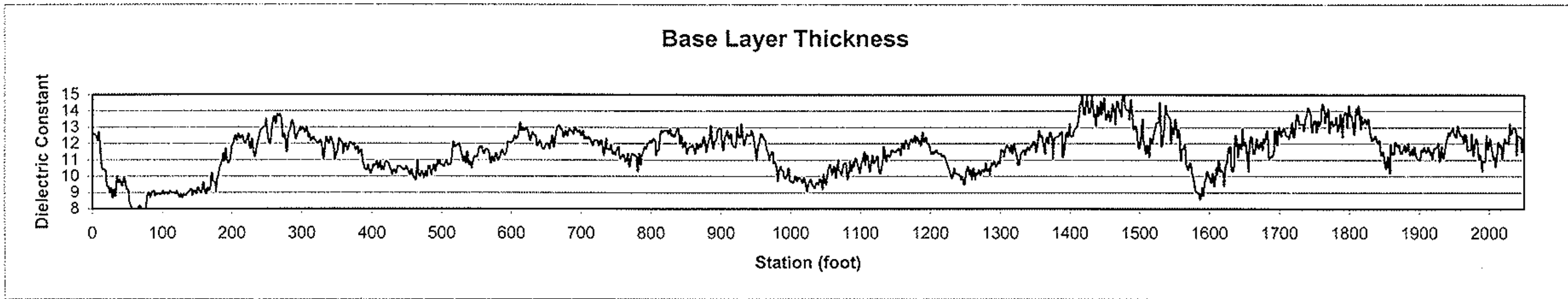
FM-1810: SUMMARY OF 2002 GPR RESULTS

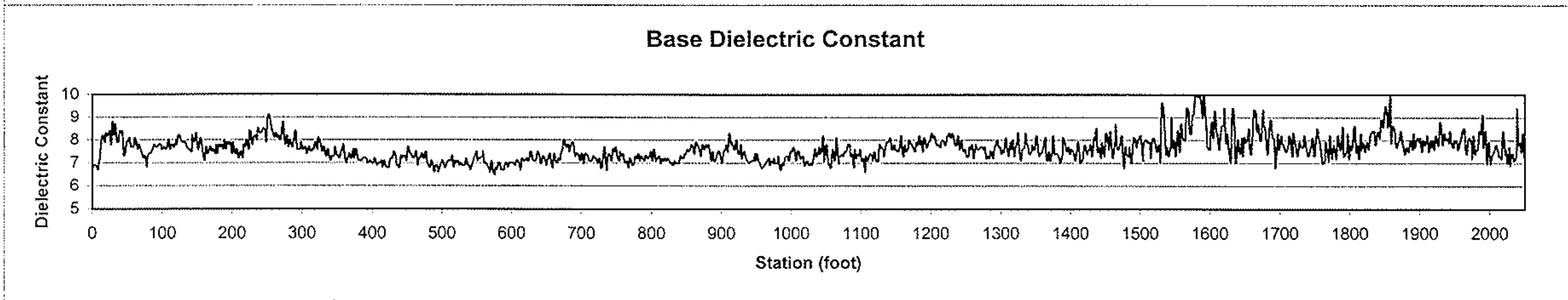
Section	Statistic	HMA		Base		Subgrade
		Thickness (in.)	Dielectric Constant	Thickness (in.)	Dielectric Constant	Dielectric Constant
No.1 Eastbound Large Stone	Average	8.5	6.3	11.6	7.6	8.4
	CoV	0.06	0.02	0.12	0.07	0.08
	Minimum	7.5	5.7	7.7	6.5	6.9
	Maximum	11.1	6.8	15.4	10.5	12.4
No.1 Westbound Large Stone	Average	8.3	6.8	12.2	8.8	9.1
	CoV	0.03	0.03	0.05	0.05	0.06
	Minimum	7.5	6.0	10.2	7.2	7.4
	Maximum	9.2	7.4	14.1	10.1	10.6
No.2 Eastbound Regular	Average	8.6	6.5	11.7	8.3	8.6
	CoV	0.04	0.04	0.06	0.06	0.06
	Minimum	7.7	5.8	9.9	7.0	7.2
	Maximum	10.1	7.4	13.7	10.2	10.7
No.2 Westbound Regular	Average	8.6	6.3	12.2	7.6	8.3
	CoV	0.11	0.03	0.15	0.06	0.07
	Minimum	6.8	5.7	4.1	7.0	6.6
	Maximum	14.6	7.2	16.9	9.3	10.1

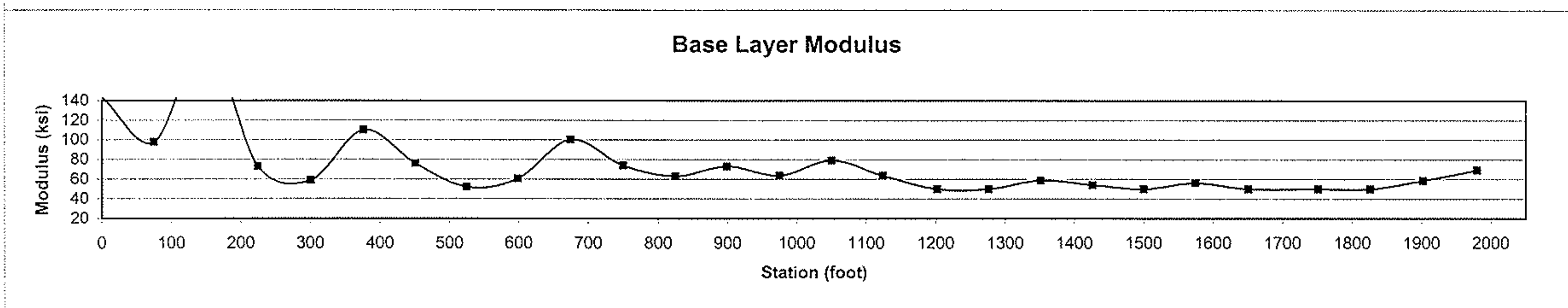
FM-1810 Eastbound Station 1+100 to 1+900: Large Stone Aggregate Base



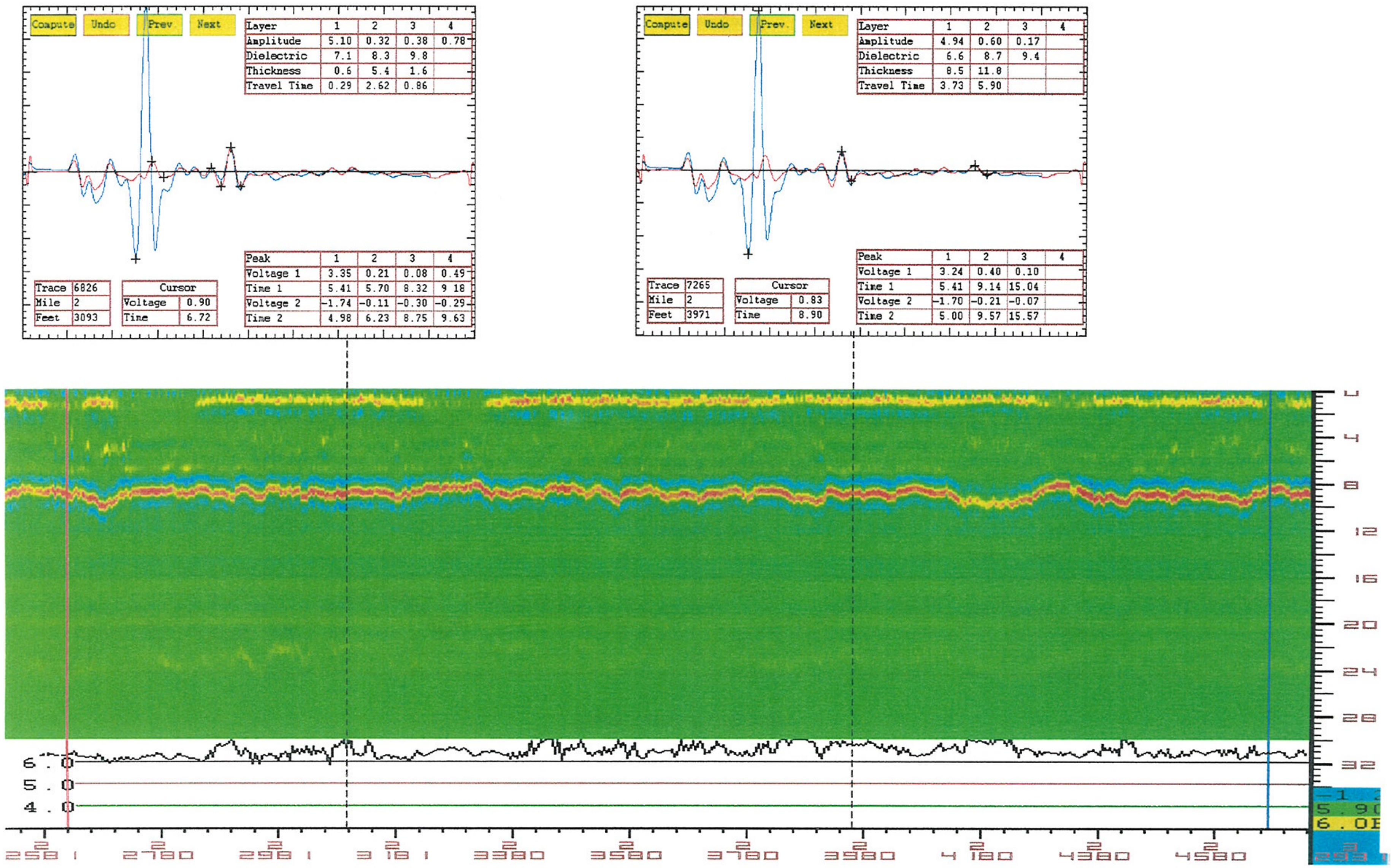
FM-1810 Eastbound Station 1+100 to 1+900: Large Stone Aggregate Base



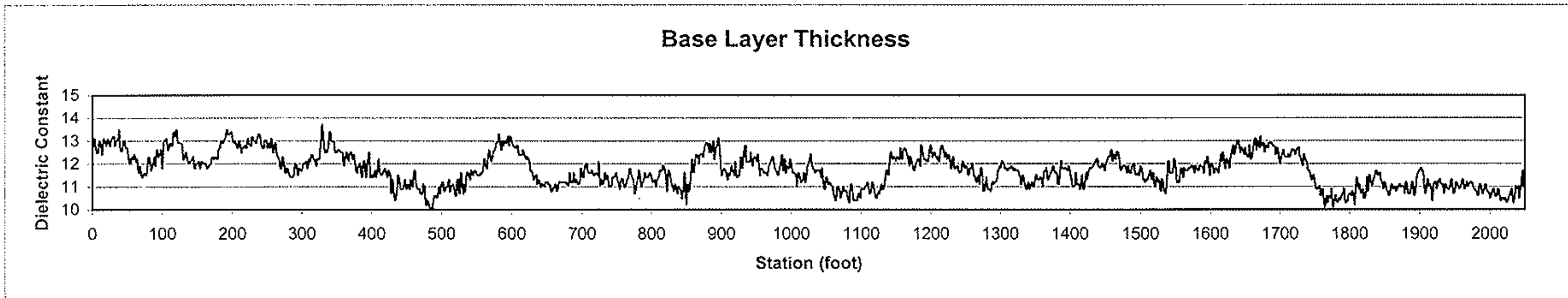


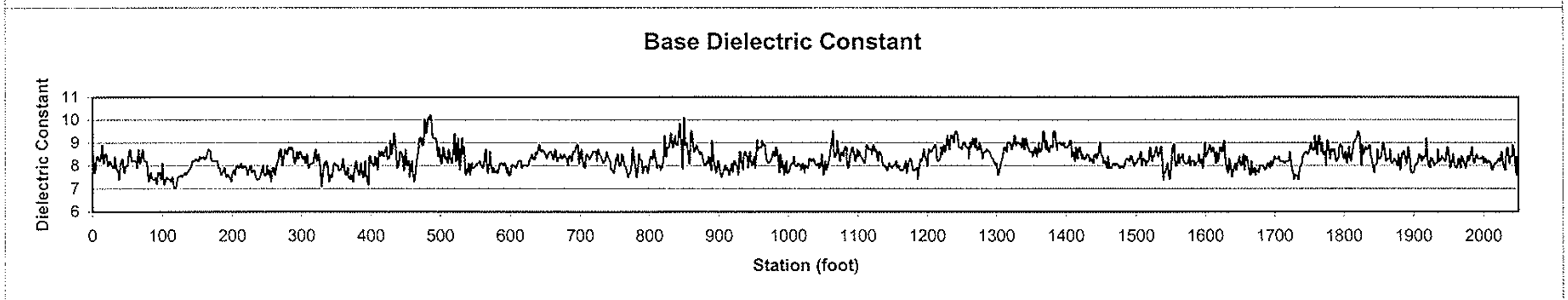


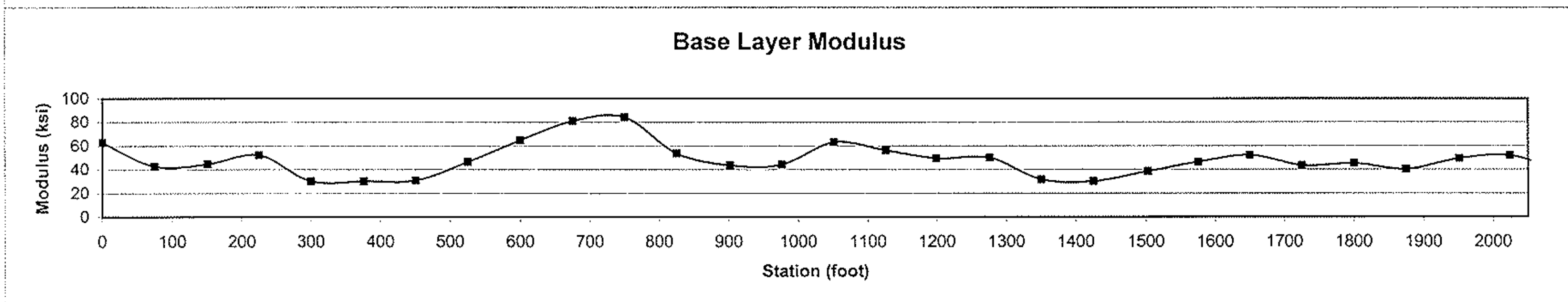
FM-1810 Eastbound Station 4+000 to 4+800 : Regular Gradation Base



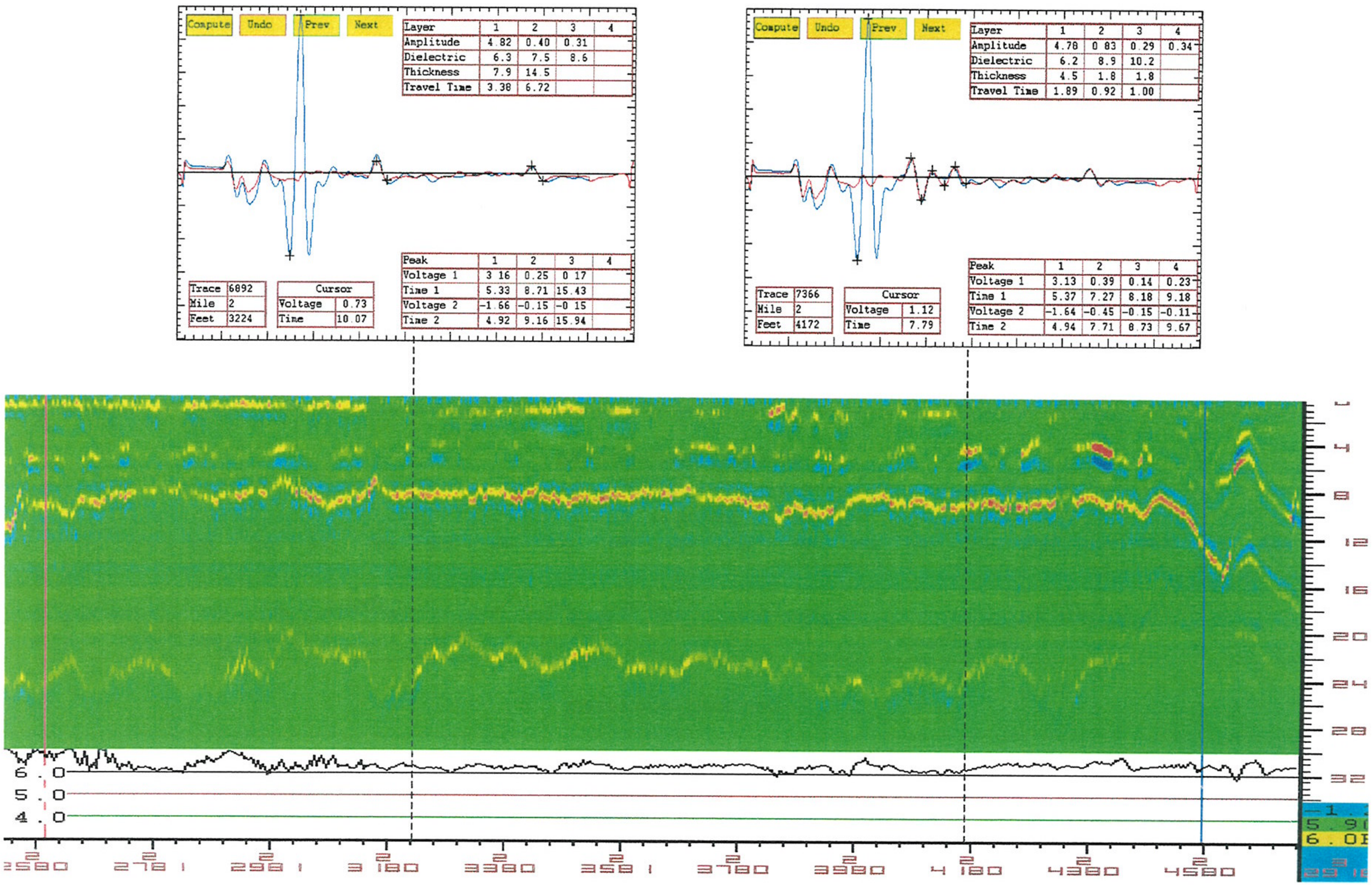
FM-1810 Eastbound Station 4+000 to 4+800 : Regular Gradation Base







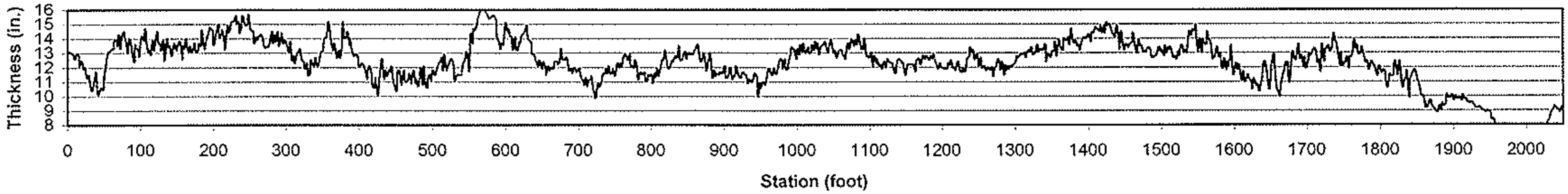
FM-1810 Westbound Station 1+900 to 1+000: Large Stone Base



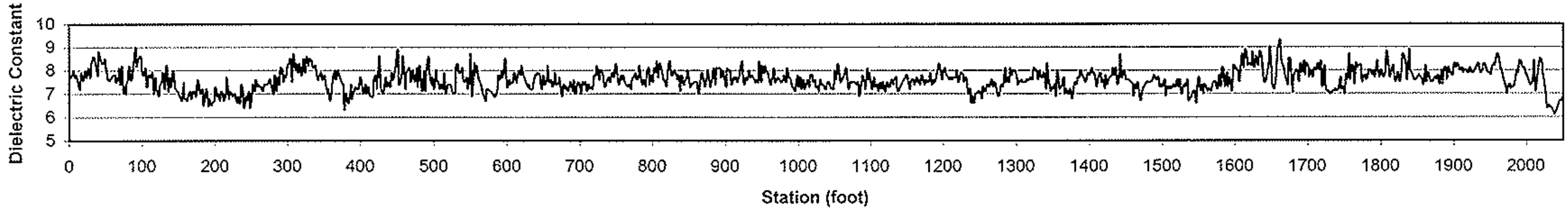
139

FM-1810 Westbound Station 1+900 to 1+000: Large Stone Base

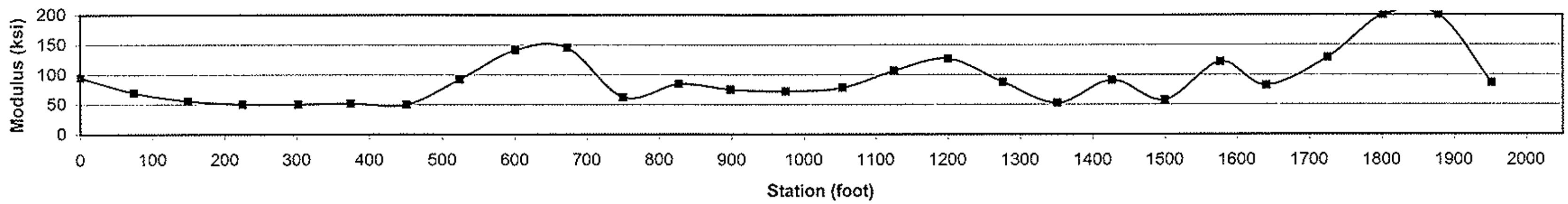
Base Layer Thickness



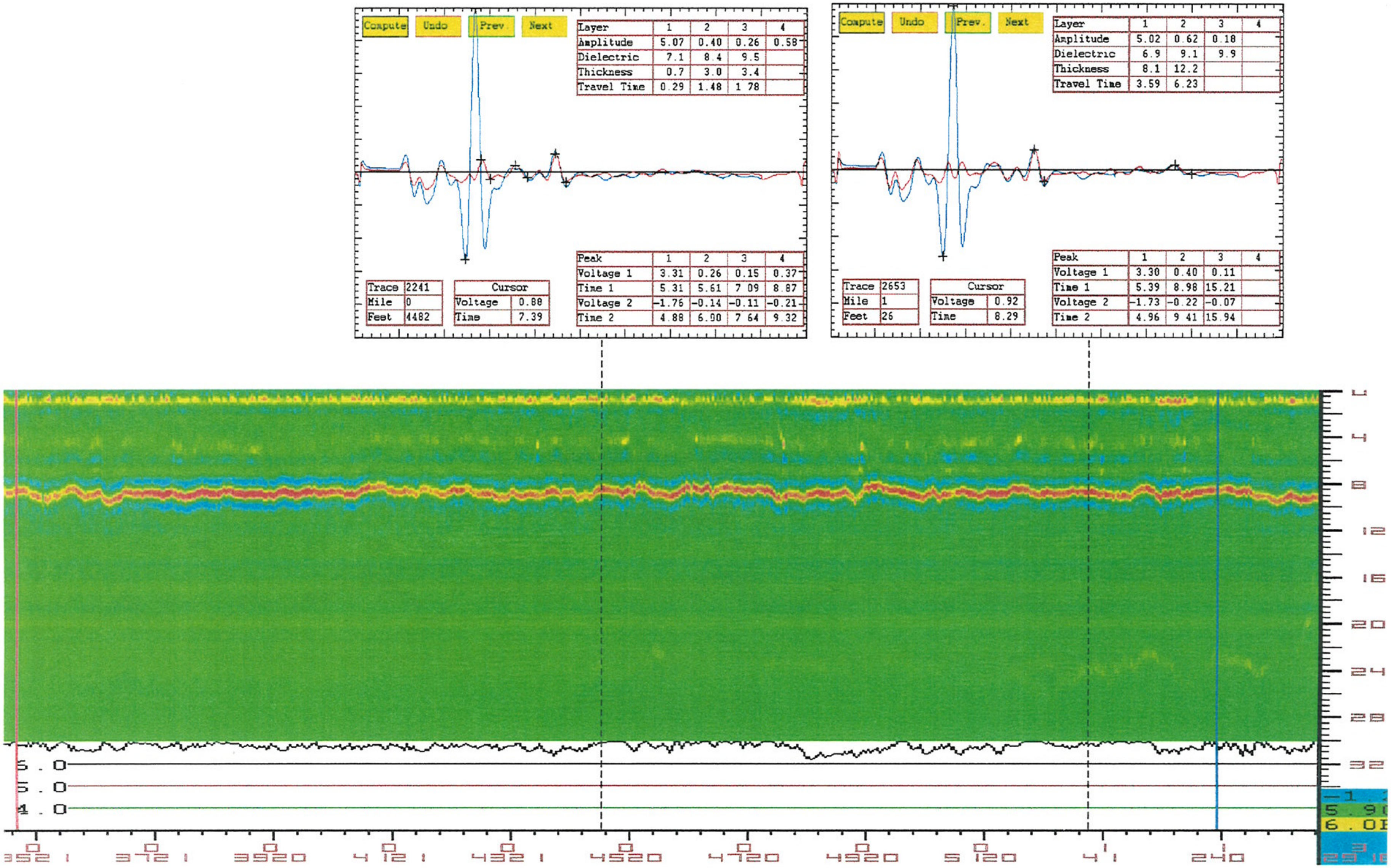
Base Dielectric Constant



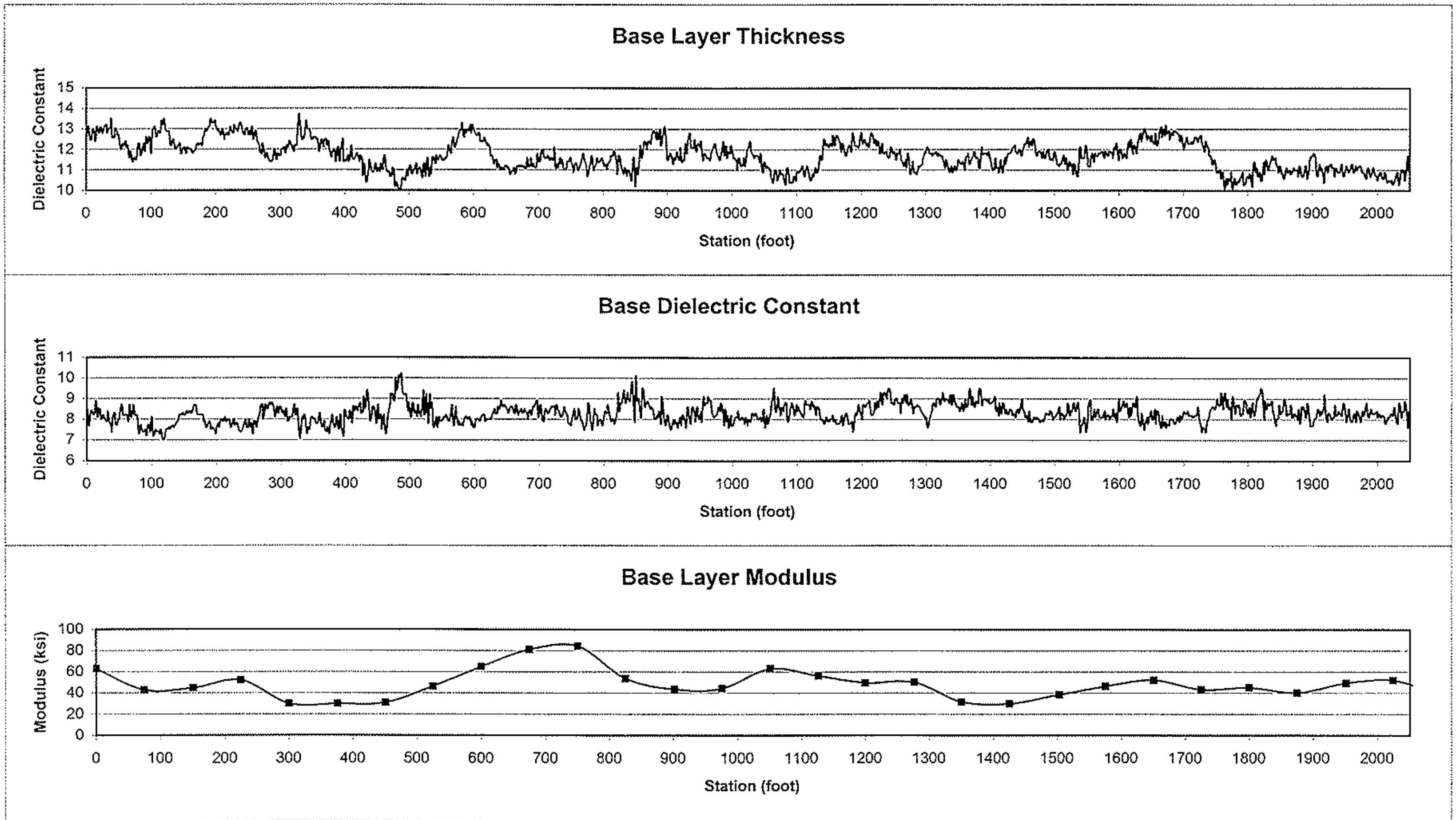
Base Layer Modulus



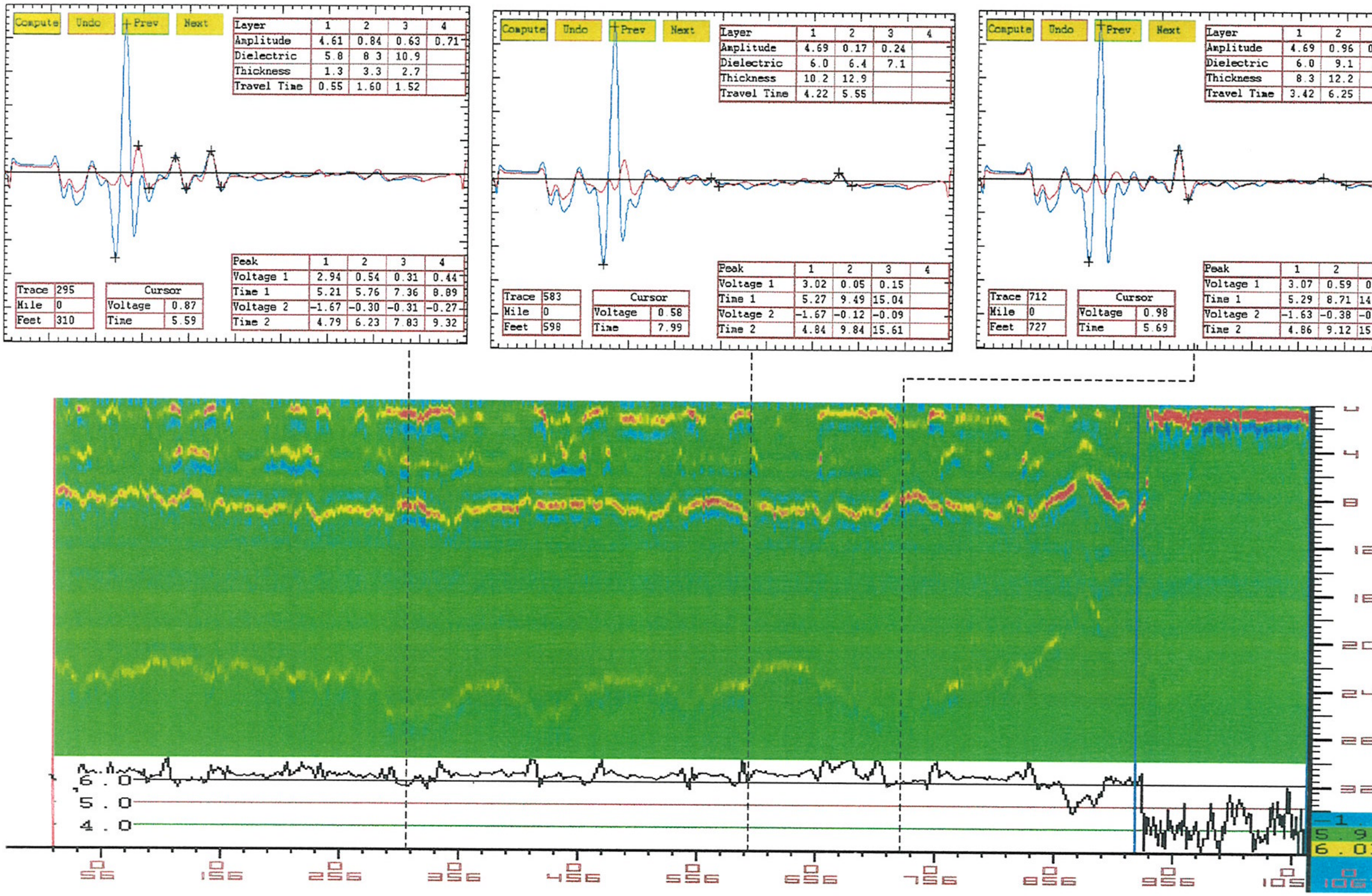
FM-1810 Westbound Station 4+800 to 4+000: Regular Gradation Base



FM-1810 Eastbound Station 4+000 to 4+800 : Regular Gradation Base



FM-1810 Westbound: Maneuver over cracks: Large Stone Base



FM-1810 Westbound: Maneuver over cracks: Large Stone Base

1. Typical mid-lane crack

2. Surface crack

3. Extended Crack

4. Crack and segregation