SURFACE PAVEMENT SOLUTIONS FOR POOR SUBGRADE CONDITIONS

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FINAL REPORT

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APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
in	Inches	25.4	millimeters	mm
ft	Feet	0.305	meters	m
yd	Yards	0.914	meters	m
mi	Miles	1.61	kilometers	km

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
AREA				
in^2	Squareinches	645.2	square millimeters	mm^2
ft^2	Squarefeet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	Acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	Gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
MASS				
oz	Ounces	28.35	grams	g
lb	Pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
TEMPERATURE (exact degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
FORCE and PRESSURE or STRESS				
lbf	Poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

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16. Abstract

The western and northern parts of Districts 4 and 6 in South Florida have shallow layers of organic and plastic soils under existing roads. These roads often exhibit large amount of cracking and distortion in a short period of time. Traditional repairs are often not practical due to high costs and extended construction time, as they require complete reconstruction of the roads with surcharge or removal of the unsuitable soils materials. The use of geosynthetic reinforcements at the base/subgrade interface are not "surface" solutions, as they require removal of the pavement and base. Therefore, the primary motivation behind the current study was to identify rehabilitation strategies which could be readily applied to the asphalt surface layers, and to develop design methodology based on some site-specific conditions so that the identified techniques could be used elsewhere if similar soil conditions are encountered. For that purpose, a 6-mile long Pilot Test Site was selected along the alignment of SR 15 / US 98 in northwest Palm Beach county, where severe pavement distresses were observed due to the existence of thick organic layers, and a reconstruction project was scheduled to begin in 2007. Following tasks were accomplished in this study: (i) Piezocone Penetration Tests (CPTu) with Porewater Dissipation Experiments were conducted at nine locations for rapid on-site evaluation of the strength and compressibility of Florida organic soils and peats; (ii) A concurrent laboratory consolidation and secondary compression test program were conducted to validate the field test results; (iii) The site-specific soil conditions (moisture content, organic content, thickness and unit weight of the organic layer) were expressed by an Organic Factor (F_{org}), which was correlated with pertinent soil properties, mechanistic pavement response parameters, and probable pavement performance; (iv) PaveTrac MT-1, GlasGrid 8501, and PetroGrid 4582 were recommended as promising asphalt reinforcing products to be incorporated in experimental pavement sections proposed at 2 different locations with different Organic Factors (F_{org}) in order to develop direct correlations between F_{org} and pavement performance, and also to facilitate relative comparisons of the candidate solutions; and (v) A conceptual design framework was developed linking laboratory/field investigations, site-specific parameters, mechanistic analysis, and transfer functions for the prediction of pavement life, so that an appropriate strategy can be adopted in future reconstruction projects.

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Organic soil, asphalt, cracking, rutting, secondary compression, geosynthetics, test sections	No Restrictions

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EXECUTIVE SUMMARY

The western and northern parts of FDOT Districts 4 and 6 in southeast Florida contains problematic organic silts and Peats at relatively shallow depths under existing roads, which cause recurring pavement distresses in the form of cracking, rutting, and long-term secondary settlement. Traditional repair methods such as complete reconstruction, surcharging or removal of unsuitable materials, often become impractical due to high costs and prolonged construction time associated with these strategies. Therefore, the primary motivation behind the current study was two-fold: (i) to identify rehabilitation strategies which could be readily applied to the asphalt surface layers, and (ii) to develop design methodology based on some site-specific conditions so that the identified techniques could be used elsewhere if similar soil conditions are encountered. For that purpose, a 6-mile long Pilot Test Site was selected along the alignment of SR 15 / US 98 in northwest Palm Beach County, where severe pavement distresses were observed due to the existence of thick organic layers, and a reconstruction project was scheduled to begin in 2007.

To achieve these study objectives, a coordinated research approach was undertaken leading to the following specific accomplishments:

(1) A comprehensive literature search and a State-of-the-Practice survey were conducted, and three promising asphalt reinforcement products were identified for use in the pilot field test sections along SR 15 / US 98. These products are (i) PaveTrac MT-1 steel reinforcing mesh manufactured by Bekaert Corporation; (ii) GlasGrid 8501 manufactured by Saint-Gobain Technical Fabrics; and (iii) PetroGrid 4582 manufactured by Propex Fabrics.

(2) Piezocone Penetration Tests (CPTu) with Porewater Dissipation Experiments were conducted at nine field locations for rapid on-site evaluation of the strength and compressibility characteristics of Florida organic soils and peats. A concurrent laboratory consolidation test program (including secondary compression tests) was conducted to validate the field test results.

(3) The site-specific soil conditions (moisture content, organic content, thickness and unit weight of the organic layer) were expressed by an Organic Factor (F_{org}), which was correlated with pertinent soil properties, mechanistic pavement response parameters, and probable pavement performance.

(4) Three experimental pavement sections, each 500-feet long and incorporating one of the candidate reinforcing products, were designed at 2 different locations with different Organic Factors (F_{org}) in order to develop direct correlations between F_{org} and pavement performance, and also to facilitate relative comparisons of the candidate solutions. The sections are repeated in both Northbound and Southbound lanes for incorporating directional variations (if any) in traffic weight and volume.

Each location also includes 4 control sections containing no reinforcements, and two sections containing Asphalt Rubber Membrane Interlayer (ARMI).

(5) A conceptual design framework was developed linking laboratory/field investigations, site-specific parameters, mechanistic analysis, and transfer functions for the prediction of pavement performance (life), so that an appropriate strategy can be adopted in future reconstruction projects.

As part of the laboratory experimental program, the Time-Stress-Compressibility relationships were investigated over a stress range (σ_v'/σ_p') of 0.3 to 1.15 encompassing both the recompression and compression zones. It was found that the (C_α / C_c) ratio at any stress level has constant values ranging from 0.03 to 0.05, which are consistent with the values reported in the literature for similar soils. The in-situ Coefficient of Consolidation (C_v), and the Compression Index (C_c) were predicted from the CPTu data, both of which showed reasonable correlation with the laboratory derived values. These findings are significant because they imply that the C_c values predicted from CPTu data may also be used to estimate C_α (since C_α / C_c is constant), and hence the long-term secondary settlement, thus avoiding lengthy and expensive laboratory testing protocols. Considering the inherent difficulty in sampling and laboratory testing of undisturbed soft organic soils, Piezocone Penetration Tests showed promise as an efficient tool for relatively rapid in-situ characterization of subsoil strength, modulus, and compressibility, all of which may be used for forensic interpretations of pavement failures, mechanistic analysis and validation of pavement performance models.

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CHAPTER 1

Introduction

1.1 INTRODUCTION AND PROBLEM STATEMENT

There are approximately 2.2 million miles of paved roads in the US, with more than 90% flexible or asphalt pavements. From a mechanistic standpoint, the load bearing capacity of a flexible pavement is dependent on the load distribution characteristics of a layered system, which consists of the pavement structure resting on multiple foundation layers including the subgrade. Often the pavement engineers have to deal with soft and highly compressible natural subgrade soils with questionable properties, leading to pre-mature failures in terms of cracking, rutting, and distortional settlement, thus resulting in frequent and costly rehabilitation projects. The western and northern parts of FDOT Districts 4 and 6 in southeast Florida is one such region with problematic organic silts and Peats at relatively shallow depth under existing roads. Traditional repair methods such as complete reconstruction, surcharging or removal of unsuitable materials, often become impractical due to high costs and prolonged construction time associated with these strategies.

It is well documented that certain geosynthetic products have been successfully used in the recent years as base and subgrade reinforcements for improving the performance of pavements constructed over soft foundations. However, the design of such pavements requires placement of the geosynthetics either at the embankment/base or the base/subgrade interface, and accordingly needs a major reconstruction effort in terms of time and budget. Therefore, the primary motivation behind the current study was to identify rehabilitation strategies which could be applied to the surface or near-surface layers only (without disturbing the underlying layers), and develop design methodology based on some site-specific conditions so that the identified techniques could be used elsewhere if similar soil conditions are encountered. For that purpose, a 6-mile long Pilot Test Site (PTS) was selected along the alignment of SR 15 / US 98 in northwest Palm Beach county, where severe pavement distresses were observed due to the existence of thick organic layers, and a reconstruction project was scheduled to begin in 2007. This created a need-driven opportunity not only to explore, develop, and propose alternative surface solution strategies based on laboratory and field investigations, but also to construct and monitor performance of experimental test sections incorporating new materials / techniques for the validation of proposed methodologies.

1.2 BACKGROUND

1.2.1 Organic Soils

In the AASHTO Classification system, organic materials are classified as type A-8 soils. Florida Department of Transportation (FDOT) defines organic material as any soil which has an average organic content of greater than five percent, or any soil for which an individual organic content test result exceeds seven percent. Organic soils generally have low strength and high compressibility compared to other soils. It is known that several northern States in the US such as Wisconsin, Minnesota, Illinois, Ohio, Michigan, New York, and the New England region have organic soils. A significant amount of organic deposits are also found in the southeastern coastal states of Florida and Georgia. Organic soils are found throughout the coastal lowlands of the Florida peninsula, which includes the Everglades known as the largest single body of organic soils in the world encompassing 2 million acres (Thomas, 1965). Organic deposits have accumulated in this region due to its close proximity to the ocean, which generates heavy rainfall and high relative humidity. The deposits occur in drowned valleys and lagoons, formed by the gradual emergence of the Coastal Plain, and by wave action on flat, imperfectly drained areas farther inland (McVay and Nugyen, 2004).

The organic-rich soil analyzed during this investigation is mainly comprised of decomposed vegetation native to South Florida hundreds of years ago. These organic-rich soils are typically referred to as muck, which is defined as a soil that is primarily comprised of humus from drained swampland. When muck contains more than 75 percent organics it is referred to as peat. Shelby tube samples collected from the test site (described later) revealed that soils at shallow depth contains dark brown organic sandy silt (organic content 20% – 40%), which is underlain by pre-dominantly dark, fibrous organic soils resembling peat (organic content 70% - 92%). The moisture contents in the organic layers range between 160% and 650%, with initial void ratios varying from 5.25 to 11.67. As mentioned earlier, muck and peat are characterized by high compressibility and very low strength, and accordingly regarded as poor foundation materials.

1.2.2 <u>Selection of Pilot Field Test Site</u>

In order to conduct this integrated laboratory and field investigation, a pilot test site was needed. After reviewing the probable dates for upcoming FDOT construction schedule in 2006 and 2007, the research team (in conjunction with FDOT personnel) identified the

SR-15 / US 98 reconstruction project site as a suitable location for the pilot test site in this investigation. The project site spans a distance of approximately six miles along SR-15 starting at Canal Point (MP 19.674) and ending at Palm Beach / Martin County line (MP 26.519). Properties along the east side of SR 15 / US 98 roadway include a mixture of residential lots, commercial properties, retail stores, churches and farmland. The FEC railroad, levee and Lake Okeechobee exist to the west of the roadway alignment. Palm Beach County is located within Southeast Florida and has relatively low topography (i.e. near sea level), warm temperatures (average annual temperature of 75 degrees Fahrenheit) and abundant rainfall (average annual rainfall of 62 inches). Palm Beach County generally has coastal ridges composed of sandy soils towards the eastern portion of the county, while areas towards the western portion have soils that are generally much more organic. Figures 1.1 shows the location of the selected pilot test site within Palm Beach County, Florida.

1.2.3 Geology of Project Location

As mentioned earlier, Palm Beach County is part of the larger Florida Platform, which is a low-lying landform along with the Atlantic Coastal Plain and the Gulf Coastal Plain. The surface consists of the Pamlico terrace, which contains sediments that were formed by the advancing and retreating seas during the Pleistocene epoch, over the shelly deposits of the Plio-Plestocene epochs during the Tertiary/Quaternary periods. Generally, the Pamlico terrace ranges from about 8 to 25 feet in thickness, while the shelly deposits are about 25 to 150 feet thick. Figure 1.2 shows a portion of the geologic map of Florida (Scott 2001).

Figure 1.1: Location of the Project Site in the North West Palm Beach County

1.2.4 USDA SCS Soil Survey

The US Department of Agriculture Soil Conservation Service (USDA SCS) has published numerous soil surveys which describes the shallow soils of areas across the United States. The USDA SCS in cooperation with the University of Florida Institute of Food and Agricultural Sciences, Agricultural Experiment Stations, and Soil Science Department published in 1978 the Soil Survey of Palm Beach County Area, Florida based on field data gathered during 1973 to 1974. Review of this survey indicates that the surficial soils of the site are mapped as "Torry muck" and "Adamsville sand, organic subsoil variant".

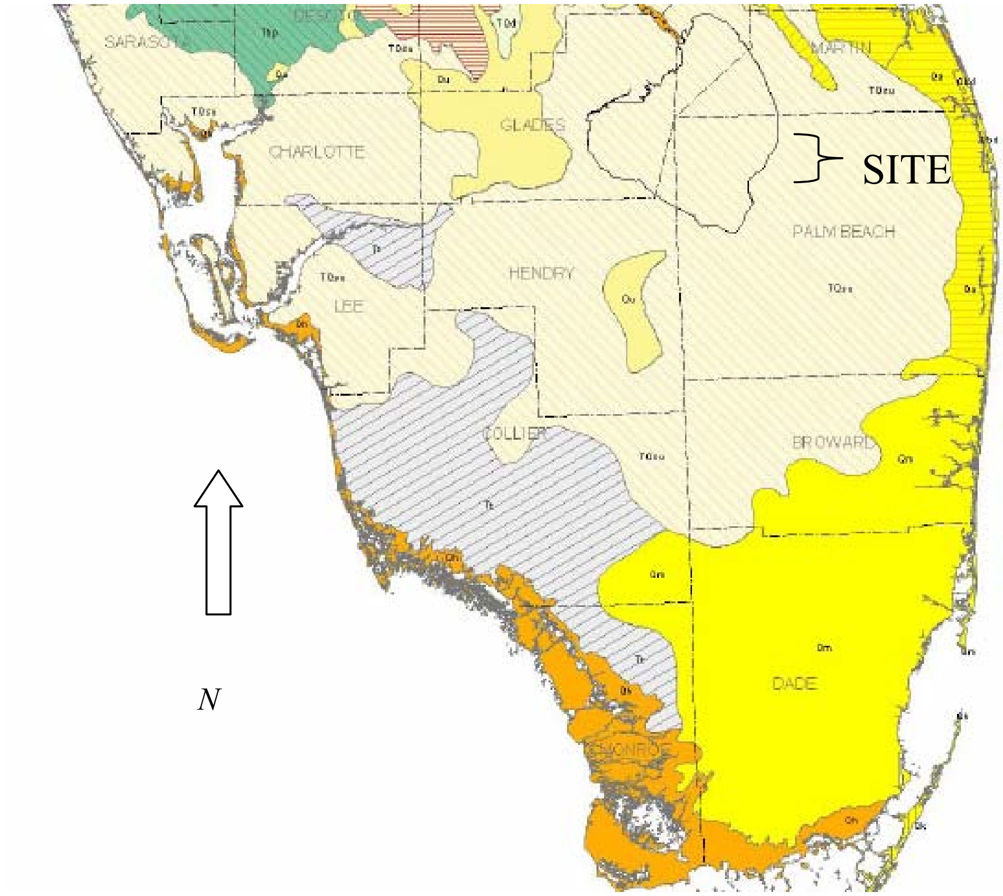


Figure 1.2: Portion of Geologic Map of the State of Florida (from Scott 2001)

"Torry muck" is described by the SCS to consist of 1.3 to 2.0 meters (51 to 80 inches) or more of organic soils over hard limestone. This soil generally forms in marshes where the plant decay matter becomes mixed with the underlying mineral materials. The SCS describes the organic soils as black or dark reddish brown, sapric material with about 2 to 50 percent fiber and 10 to 70 percent mineral content, is sticky to very sticky and is slightly to strongly acidic. "Adamsville sand, organic subsoil variant" is described by the SCS to consist of sandy fill materials that were placed over natural organic soils during development of the area. The sandy fill was about 0.6 to 0.9 meters (2 to 3 feet) in thickness. The natural soils are composed of sapric material with less than 5 percent fiber and about 40 to 70 percent mineral material. Adamsville soils are similar to Torry muck except for the sandy soils above the organic soils. The sandy soils were the result of fill placement during urban development of the region.

1.2.5 Visual Survey of Proposed Site

Preliminary Geotechnical site characterization studies were previously performed in 2004 by GEOSOL, Inc., which included Standard Penetration Test (SPT) from 93 test borings. In-situ moisture content and organic content data were available from 50 test locations. The study identified approximately 1.93 km (1.2 mile) section along the roadway (between MP 23.7 to MP 24.9) as severely distressed from visual observations. On July 4, 2005 the project team visited the site along the 9.65 km (6 mile) segment of SR 15 / US 98, visually identified the more severely distressed locations, and compared and verified these observations with those reported in the GEOSOL Report. Visual observations indicated numerous cracking and significant rutting and raveling along the roadway. The worst cracking appeared just north of the Sand Cut Canal Bridge (between Mile Post 23.700 to Mile Post 24.900) and generally consisted of longitudinal and transverse cracking several feet long that had single and branch cracking as well as areas of alligator cracking. Using the FDOT Flexible Pavement Condition Survey Handbook dated April 2003, the ratings of the cracking are mostly Class II with some Class IB and III (the cracking ranges from slight, moderate and most severe for Classes IB, II and III, respectively). The raveling was moderate to severe. Rutting was also moderate to severe (undulations) in this area. Figures 1.3 and 1.4 show these conditions along the roadway as observed during the site visit.

Figure 1.3: Severe Raveling along SR 15

Figure 1.4: Class III Cracking and Severe Raveling along SR 15

1.3 RESEARCH OBJECTIVES

The broad objectives of this research endeavor were two-fold: (1) to identify and evaluate effective products and/or techniques which could be readily applied to the pavement surface course for extending the life of the roadway built over organic layers; and (2) to develop methodologies and guidelines for design implementation. To achieve these objectives, a coordinated approach, divided into Tasks I through VI as shown in Figure 1.5, was proposed. The objectives of the current investigation were to accomplish Tasks I through V, with the construction and performance monitoring of field test sections and the integrated cost-benefit analysis to be performed as part of the second phase of the project in the future. Accordingly, the specific objectives of this study were as follows:

1. To conduct a comprehensive literature search and a State-of-the-Practice survey in order to identify promising techniques and products;
2. To conduct field Piezocone Penetration Tests for evaluating the strength and compressibility of Florida organic soils, with concurrent laboratory consolidation tests (including secondary compression) to validate/calibrate the field test results;
3. To correlate site-specific soil conditions (thickness, moisture, and organic content) with pertinent soil properties, mechanistic pavement response parameters, and probable pavement performance;
4. To design the lengths, cross sections, and the locations of experimental pavements incorporating selected products and techniques; and
5. To develop methodologies and guidelines for implementation of the design techniques in future pavement projects where similar organic soils are encountered.

PHASE I: IDENTIFICATION OF PROMISING TECHNIQUES

Literature search and State-of-the-Practice survey to identify candidate products and techniques

PHASE II: FIELD AND LABORATORY GEOTECHNICAL TESTING

Field Piezocone Penetration Tests and laboratory consolidation tests for evaluating strength and compressibility (including secondary compression) behavior of Florida organic soils and peat

PHASE III: DEVELOPING SITE-SPECIFIC CRITERIA

Relating site-specific conditions such as the thickness of the organic layer, moisture content, and organic content at a particular location with the pertinent soil properties, mechanistic pavement response parameters, and probable pavement performance

PHASE IV: DESIGN OF FIELD TEST SECTIONS

Selection of field test pavement sites based on site-specific criteria, and designing the length and cross section of the experimental pavements incorporating promising Surface Pavement Solutions (SPS)

PHASE V: IMPLEMENTATION GUIDELINES

Develop conceptual framework for correlating pavement performance (life) with site-specific conditions. Coordinated testing/monitoring plans for the field test sections are essential to develop and calibrate the performance models. Recommend methodology for implementation into future roadway projects encountering organic layers.

PHASE VI: COST-EFFECTIVENESS STUDY

Perform life-cycle cost analysis to compare different SPS techniques, using an estimate of expected life-span and anticipated maintenance costs for each candidate strategies.

Figure 1.5: Summary of Research Approach and Methodology

1.4 SCOPE AND SIGNIFICANCE OF GEOTECHNICAL INVESTIGATIONS

The pavement structure of SR 15 / US 98 consists of nearly 305 mm of asphalt, 305 mm of limerock base, and 915 mm of silty sand subgrade, underlain by organic silts and peat with thicknesses ranging from 3 to 5.5 meters (details discussed later). Generally, the primary consolidation process in organic layers is quite rapid, followed by significant secondary compression stages under sustained effective overburden pressure due to the dead weight of the pavement structure. Although the passage of traffic may initiate short pulses of primary consolidation processes in the organic layer, the major component of the deformation in the organic layer is due to the long-term continuing secondary and tertiary compression phases under the constant pavement dead weight leading to premature distress and failures.

The foregoing discussion suggests that one of the root causes of the premature pavement failure is the soft compressible nature of the underlying organic soils. Since complete removal of the organic layer is expensive and impractical, any design strategy aiming at mitigating such distress levels requires a thorough understanding of the strength and compression behavior of the organic silts and peats. Moreover, it is well-known that routine sampling and transportation of "undisturbed" specimens of soft organic soils is rather difficult, and subsequent laboratory testing especially the evaluation of secondary compression behavior is expensive and time-consuming. Therefore, there is an identified need for developing reliable and efficient tools for on-site estimation of soft soils properties, which can also be used for forensic interpretations of pavement failures, mechanistic analysis, and validation of pavement performance models. The current study evaluated the use of Piezocone Penetration Tests (CPTu) as a versatile tool for subsurface investigations relevant to pavement geotechnics. Accordingly, eleven different severely distressed locations were chosen along the alignment of SR 15 / US 98 for conducting the Piezocone Penetration Tests, and collecting "undisturbed" Shelby tube samples at various depths for subsequent laboratory testings which included comprehensive evaluation of secondary compression behavior. These experimental programs are discussed in detail later in the report.

CHAPTER 2

Literature Review

2.1 INTRODUCTION

Historically pavement engineers all over the world have faced challenging situations arising out of soft clayey subgrades, expansive clays, and organic soils and peats. Numerous chemical and mechanical stabilization, and ground modification techniques have been developed and implemented during the last several decades to encounter problems associated with difficult soil conditions, and this trend will continue into the foreseeable future. It is beyond the scope of this report to describe all these techniques; however, excellent summaries of two decades of such developments are available in two Geotechnical Special Publications (GSP) by the American Society of Civil Engineers (ASCE, 1987; ASCE 1997). In addition, the State-of-the-practice in the stabilization of soil and base for pavement applications has been elaborately documented in a 2-volume publication by the Federal Highway Administration (FHWA, 1992a; 1992b). Most recently, a systematic protocol for the laboratory and field evaluation of chemically stabilized soils has been published in a State-of-the-Practice Report (TRB Circular E-C086) by the Transportation Research Board (Petry and Sobhan, 2005).

A comprehensive literature search was conducted relevant to the current study focusing on the design, construction, and maintenance of highway pavements built over soft and compressible subgrade soils. The findings of this investigation are summarized under the following categories: (1) Promising rehabilitation strategies, techniques, and products; (2) Previous FDOT Studies; (3) Case histories outside Florida; and (4) Pertinent in-situ tests and laboratory experiments

2.2 PROMISING TECHNIQUES AND PRODUCTS

The identified strategies which hold promise as viable solutions could be divided into two broad categories: (i) "Surface" solutions, which are applied directly to the asphalt surface layer without any modification of the underlying soft organic layer; and (ii) "Deep" solutions, which are primarily applied / injected from the surface for achieving deep soil stabilization, and thus directly modifying the soft organic layers. These methods are briefly discussed below.

2.2.1 "Surface" Solutions

In the recent years, several new-generation reinforcing products and composites have been used to directly reinforce the asphalt concrete overlays in an effort to primarily mitigate reflection cracks originating from the existing cracks and joints of the underlying pavement. These include Asphalt Rubber Membrane Interlayer (ARMI), Geogrids, Fiberglass Grid, Petrogrid, Base-isolating Interlayer / Glass Fiber Reinforcement, and Interlayer Stress Absorbing Composite (ISAC). Techniques other than reinforcement include Resin-modified Asphalts and Ultra-Thin Whitetopping (UTW). Although a traditional method of accelerating the consolidation and secondary compression of organic layers by means of surcharging is an effective "surface" solution, it is generally used in a new construction or complete reconstruction scenario, and therefore not considered as a candidate technique for the current study.

2.2.2 "Deep" Solutions

Highway agencies often use deep ground modification techniques with cementitious or other chemical stabilizers for treating soft problematic soils. Some of these techniques include (but are not limited to) Deep Mixing, Soil Injection, Vibro-Concrete Columns, Lime Columns, Grouted Minipiles, and grouting (ASCE, 1997). These ground improvement techniques are mostly employed during new constructions or complete reconstruction of existing roadways.

In a typical ***Soil Injection*** Method, a lime-fly ash slurry may be injected deep into the ground with specialized nozzles (3/4" diameter with 3/16" holes around it) at high pressure (200 psi) by drilling holes from the pavement surface following a specific grid pattern. For example, the injection would start at the very bottom of a highway embankment (where poor soils are encountered), and the nozzle would be gradually raised as the slurry is slowly released into the soft foundation layers. Eventually, the pozzolanic reactions in the cementitious materials would stabilize the soils for enhanced strength and durability.

Deep Mixing is an in-situ soil mixing technology that mixes existing soils with cementitious materials using mixing shafts consisting of auger cutting heads, discontinuous auger flights, and mixing paddles. **Cement Deep Mixing** (CDM) method using cement grout stabilization agent was developed in 1975 for full scale application (ASCE, 1997).

Micropiles / Minipiles are, generically, small-diameter, bored, grouted-in-place piles incorporating steel reinforcements (ASCE, 1997). These are usually designed to transfer structural loads to more competent or stable strata. The diameter can vary between 3 to 10 inches (75 to 250 mm), length up to 98 ft. (30 m), and load carrying capacity up to 225 kips (1000 KN). Micropiles are frequently used as in-situ reinforcements such as embankments, slopes, and landslide stabilization, soil strengthening and protection, settlement reduction, and structural stability (ASCE 1997), which are all in the context of the present study.

In the I-10 Jacksonville, Florida project, Hayward Baker Company installed **Vibro-Concrete Columns (VCC)** to stabilize the foundation of bridge approach embankment on over the San Pablo Creek in order to reduce settlement of the roadway underlain by organic soils and peats. A total of 163 15-inch diameter VCCs were constructed to a depth of 20 ft. through the organic layers into the bearing stratum. In addition, a total of 141 42-inch **Vibro Replacement (VR) Stone Columns** provided a smooth transition between the VCCs and the untreated embankment. The combination of these vibro systems not only saved time and money, but also avoided the costly complete excavation and removal of the organic / peat layers (Hayward Baker, 2004).

The "Deep" solutions mentioned above, although very effective, require extensive mix-design criteria to be developed for site-specific soil conditions prior to actual design and construction. In other words, the properties of the stabilizing agents, properties of the soil to be stabilized, and finally the properties of the stabilized soils must be studied (generally through laboratory experiments) for developing optimum field mix-design criteria. Obviously, this process is costly and time consuming. For example, the Indiana Department of Transportation (InDOT) in conjunction with Purdue University is developing such criteria for Deep Cement Mixing and stabilization of highly organic soils having organic content of 48-60%, and natural moisture contents of 240 - 289% (Santagata, 2006). It is interesting to note that the secondary compression behavior of cement-modified Indiana soils were being characterized in terms of the C_α / C_c parameters similar to the secondary compression studies conducted as part of this current investigation (described later). At the time of writing this report, the InDOT study still ongoing.

Despite the fact that deep mixing is considered a promising solution, the current study focused on developing near-surface strategies, which primarily involves reinforcement or some modification of the asphalt concrete surface layers for mitigating crack propagation, and thus extending the life of the pavement. These strategies also deviate from the traditional geosynthetic solutions where the reinforcing element is placed at the interfaces of the various foundation layers, and have been proven to be quite effective especially when used in conjunction with surcharge loading of the organic layers. However, the traditional approach requires time consuming and costly major reconstruction of the pavement foundation layers.

2.3 PREVIOUS FDOT PROJECTS

Several experimental projects previously conducted by the Florida Department of Transportation are noteworthy in this context. These are as follows:

2.3.1 <u>Geosynthetic Reinforcement Evaluation I</u>

Two experimental roadway sections each 1000 ft. long were constructed in 1994 along the East and West bound traffic lanes of SR 80 in Palm Beach County (FDOT Project No. 93110-3543). The West bound section served as the control, while the East bound section was reinforced with a geogrid mat placed in the base layer. Performance was evaluated in terms of Deflection, Ride, Rutting, and Cracking through a long-term monitoring program (1994-2003). Most recently acquired data (2003) indicate that in a performance comparison with the control segment, the reinforced section developed significantly lower (about 42%) cumulative rutting. However, there were minor to no improvements in Ride quality, and virtually no change in the resistance to FWD deflections due to geogrid inclusion. Moreover, the reinforced section developed significantly higher Cracking (about 92%) compared to the control sections. These results suggest that the usual practice of placing conventional geogrids within the base/embankment has been shown to produce improved performance, and the current study was conducted to look at alternative surface solutions.

2.3.2 Geosynthetic Reinforcement Evaluation II

A more comprehensive study undertaken in 1999 included five test sections along the Northbound and Southbound lanes of SR 15 in Palm Beach County, with 4 sections containing a combination of geotextile and geogrid reinforcements incorporated in the base and subgrade, and one section serving as a control section with no reinforcement (FDOT Project No. 93130-3508). Performance was evaluated in terms of deflection, ride, rutting, and cracking from 1999 to 2005. Most recent data acquired in 2005 indicate that reinforced sections had lower FWD deflections and rutting compared to the control section in the NB lanes; however results from the SB lanes were mixed. Ride was essentially unchanged in both directions. Cracking measured only on the SB lanes showed that the Fornit/Geotextile combinations suffered higher cracking than the control section. These observations were consistent with those of the earlier study described in the previous section.

2.3.3 Asphalt Rubber Membrane Interlayer (ARMI)

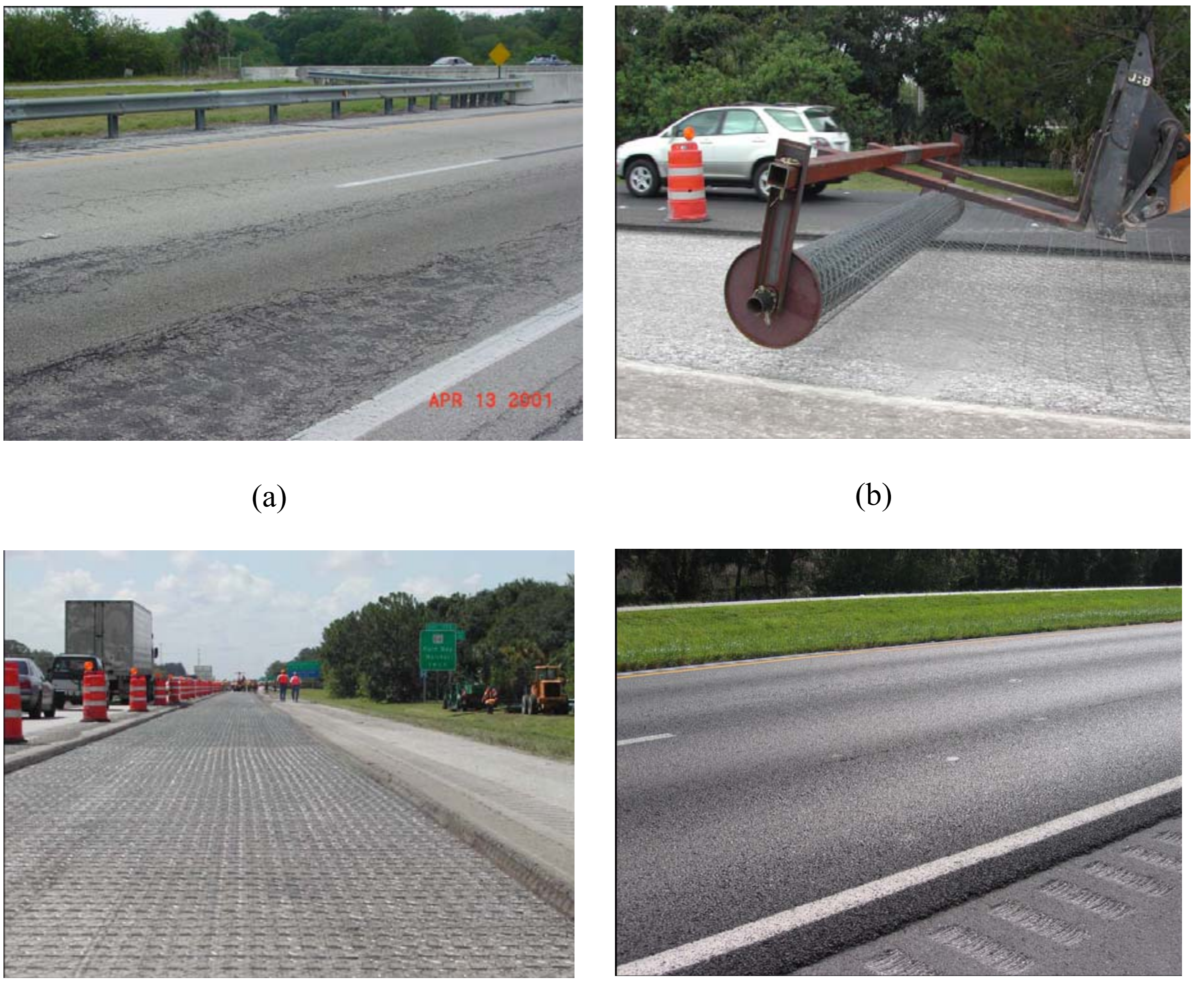
Five test sections were constructed in 1998 along the eastbound lane of SR 2 in Baker County to evaluate the relative long-term field performance of Asphalt Rubber Membrane Interlayer (FDOT Project No. 27020-3509). Most recent data acquired in 2004 suggest that various modifications of ARMI reinforcement design contributed to moderate improvements in rutting and cracking performance, and some apparent reductions in FWD deflections compared to control sections.

2.3.4 Hollywood Boulevard GlasGrid Project

GlasGrid 8501 is a Knitted glass fiber strand grid forming a pavement reinforcing mesh currently manufactured by Saint-Gobain Technical Fabrics. High stiffness of glass fiber grids has strong potential for arresting crack propagation in asphalt surface layer. Bayex 8501 GlasGrid manufactured by Bayex, a division of Bay Mills, was used for the rehabilitation of SR 820 (Hollywood Blvd.) in South Florida. Personal communications with FDOT Pavement Evaluation Engineers in 2006 revealed that there were a number of construction related issues in the project, and that the effects of GlasGrid reinforcement could not be determined conclusively at that time. Cores taken from in-service pavements showed that in many cases the grid was not at the desired locations. Recent evaluations showed that the cores separated easily at the GlasGgrid layer, and that the GlasGrid could be torn or pulled apart very easily with some signs of deterioration. The Final report for this project was still under preparation at the time of writing this report.

2.3.5 I-95 Rehabilitation with PaveTrac in Melbourne, FL

PaveTrac MT-1 is a steel reinforcing mesh manufactured by Bekaert Corporation, Netherlands. PaveTrac is a woven wire mesh with a unique transverse alternately torsioned flat reinforcing bar which provides optimum confinement, anchorage and interlock within the asphalt overlay. In June 2003, southbound I-95bridge approach to Tillman Canal Overpass near Melbourne, Florida was rehabilitated with PaveTrac MT-1. Existing asphalt pavement was heavily distressed due to wet organic subgrade. Original design specified total reconstruction with 10 inches of combined asphalt base and surface course. PaveTrac design specified 4 inches of milling, followed by the placement of mechanically fastened PaveTrac MT-1, and 4 inches of asphalt overlay in two lifts. Visual surveys conducted after 15 months showed minimal to no sign of distress in the reinforced sections. Figure 2.1 shows various stages of this project (ACF Environmental, 2004).



(a) (b)

(c) (d)

Figure 2.1: FDOT I-95 Rehabilitation Project near Melbourne, Florida. (a) Structurally Failed Asphalt Pavement (2001); (b) Laydown of PaveTrac MT-1 (2003); (c) Installed PaveTrac MT-1 over Milled Surface; and (d) Pavement in Good Condition 15 Month after Construction (2004); Courtesy: ACF Environmental

2.3.5 Evaluation of Ultra-Thin Whitetopping using HVS

In a recently completed FDOT project (Tia, 2003), researchers developed the guidelines for recommended tests, measurements and instrumentation for evaluating the performance of UTW rehabilitation process using the Heavy Vehicle Simulator APT facility at the State Materials Office (SMO). Critical design factors for the implementation of UTW in Florida were identified.

2.4 CASE HISTORIES OUTSIDE FLORIDA

A number of new-generation reinforcing composites, grids and membranes are available which can be embedded at the bottom or sandwiched within the AC surface layer in order to improve crack resistance and also absorb the stresses due to displacements in the foundation layers. These include the following:

2.4.1 Fiberglass Grid

Recently, high stiffness Fiberglass Grids (E = 10,000,000 psi) were sandwiched between the AC binder course and wearing course to reduce premature cracking of the AC overlay in the rehabilitation of US Highway 190 in Louisiana, and US Highway 96 in Texas (Darling and Woolstencroft, 2000). In addition, GlasGrids have been used in field test sections in Wisconsin (Bischoff and Toepel, 2003) and Iowa (Marks, 1990) to evaluate its effectiveness in resisting reflective cracking in AC overlays placed over PCC joints and cracks. Although these applications have different focus than the current investigation, it is important to point out that the performance of reinforced test sections were reported as marginal to unsatisfactory.

2.4.2 Base-Isolating Interlayer and Glass-Fiber Reinforcement

Consists of high ductility, yet rut resistant interlayer mixture and high stiffness Fiberglass grids as a composite system (Kim and Buttlar, 2002).

2.4.3 Interlayer Stress Absorbing Composite (ISAC)

Interlayer Stress Absorbing Composite was used in 6 test sections in Illinois. It consists of a 3-layer composite system with a low stiffness geotextile at the bottom, a viscoelastic membrane layer at the core, and a high stiffness geotextile for the upper layer; effectively isolates the AC overlay from the movements in the underlying layers (Dempsey, 2001).

2.4.4 PetroGrid

PetroGrid is a composite geosynthetic material made of petromat, a nonwoven needle punched polypropylene paving fabric, and a structural grid composed of epoxy resin coated glass fiber. In a laboratory study, asphalt overlay specimens placed over simulated pavement joints, and reinforced with PetroGrid showed greater resistance to fatigue fracture compared to control specimens (Sobhan et al. 2004; Sobhan and Tandon, 2003).

2.4.5 Resin Modified Pavement

It is a composite paving material consisting of thin 2-inch open graded HMA whose internal voids (30%) are filled with latex rubber modified Portland cement grout. Pavement demonstration project conducted along US 72 in Mississippi (Battey, 2002)

2.4.6 Ultra-Thin Whitetopping

Several field trials were conducted along US 72 in Mississippi (Battey, 2002).

2.4.7 PaveTrac MT-1 in Netherlands

First large scale PaveTrac application was installed over organic subgrade in 1983 in a suburb of the Hague, Netherlands (ACF Environmental, 2004). The pavement was exhibiting longitudinal cracks typical of a soft, saturated subgrade which is undergoing a shear failure. A total of 4 inches of asphalt was placed over the mesh. Reinforced pavement showed no structural cracking when investigated two years later in 1985. On the other hand, the unreinforced control sections, also repaved in 1983, showed extensive structural (longitudinal) cracking in 1985. It was reported that no structural cracking was observed in the reinforced sections after 19 years of service. In 1994, the pavement was resurfaced to treat environmental surface aging. Figure 2.2 shows the condition of the roadway 2 years after rehabilitation.

(a) (b)

Figure 2.2: Conditions of the Roadway in 1985: (a) Control Section; (b) Reinforced Section with PaveTrac (Courtesy: ACF Environmental)

2.4.8 Bluegrass Parkway Rehabilitation in Kentucky

Kentucky Transportation Cabinet recently initiated a 3-year project (Graves, 2005) involving the reinforcement of Asphalt surface with PaveTrac MT-1 and GlasGrid on test sections along Bluegrass Parkway in Kentucky (*Research in Progress* Database; Grant No. FRT 147: "Evaluation of Asphalt Pavement Reinforcement, PaveTrac and GlasGrid," 2005-2008). The performance of the reinforced and control sections will be monitored for 3 years. This investigation is significant because it serves almost as a parallel investigation to the current study in the sense that both the PaveTrac MT-1 and GlasGrid 8501 were recommended for inclusion into the experimental sections scheduled for construction in 2007 (discussed later in this report).

2.5 FIELD AND LABORATORY EXPERIMENTS

2.5.1 Soil Settlement

From a geotechnical perspective, settlement of foundation layers plays an intricate role in the overall performance and longevity of the pavement structure. Settlement is composed of three components: immediate or distortion settlement, consolidation or primary settlement, and secondary settlement or compression (Holtz and Kovacs, 1981). Thus, total settlement, s_t, is defined as:

$$s_t = s_i + s_c + s_s \qquad (2.1)$$

Where:

s_i = Immediate, or distortion settlement,

s_c = Consolidation or primary (time-dependent) settlement, and

s_s = Secondary compression (time-dependent) settlement.

Immediate settlement differs from the other two settlements because it is not time dependent. Immediate settlement occurs instantaneously under vertical compressive loads and is attributed to gases contained within the soil skeleton dissipating. Typical organic soils contain *5% - 10%* gas, which contribute to the immediate compression or rebound if a load is totally removed (Landva and LaRochelle, 1982). For all intensive purposes, immediate strain will be negated, leaving primary settlement and secondary settlement as the main components of total settlement.

2.5.2 Consolidation Settlement

The fundamental theory that allows the quantification of primary settlement is consolidation conceived by Karl Terzaghi (1921, 1923, and 1924). Terzaghi's consolidation theory mathematically represents a one-dimensional volume change in a porous soil due to a flow of water out of the voids of the soil matrix. The soil matrix is comprised of solid particles arranged in a skeleton and voids, which are referred to as pores once the soil matrix is fully saturated. Once the porous soil is subjected to a compressive vertical force the skeleton of solid particles begins to deform in the same direction as the applied vertical force. This deformation is the result of increased pressure in the pore-water, known as excess pore-water pressure u, diffusing out the pores of the porous soil, thus generating increased stresses on individual soil particles. This is the process of consolidation and it takes place throughout the primary settlement phase. Terzaghi's Theory of Consolidation (1921, 1923, and 1924) is based on the following main assumptions: (1) Compression and flow are one-dimensional; (2) Soil is saturated and homogeneous; (3) Soil solids and water are incompressible; (4) Darcy's law is valid; and (5) Coefficients of compressibility and permeability are constant within the range of applied stress. Equation 2.2 represents one-dimensional consolidation based on the abovementioned assumptions.

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \tag{2.2}$$

where:

c_v = $\dfrac{k(1+e_0)}{a_v \gamma_w}$ = Coefficient of consolidation,

k = Hydraulic conductivity,

e_0 = Initial void ratio,

a_v = Coefficient of compressibility,

γ_w = Unit weight of water,

z = Time-independent space coordinate (initial coordinate), and

t = Time.

To analytically solve for the pore-water pressure in Equation 2.2, the coefficient of consolidation, c_v, must be obtained. The coefficient c_v, is called the coefficient of consolidation because it contains the material properties that govern the consolidation process (Holtz and Kovacs, 1981), and it is obtained by either Casagrande's (1938) logarithm of time approach or Taylor's (1948) square root of time approach.

Equation 2.2 is a form of the diffusion equation from mathematical physics, where the diffusion constant for the soil is c_v, which depends greatly on the load increment ratio (LIR), and whether the pre-consolidation stress has been exceeded or not (Leonards and Girault, 1961). It is assumed to be constant in order to make the equation linear and easily solvable. After initial boundary conditions are specified, a closed-form solution is obtained through separation of variables (Holtz and Kovacs, 1981). Once the aforementioned conditions are specified, the pore-water pressure, *u*, is defined by Equation 2.3 below:

$$u = \Delta\sigma \sum_{N=0}^{\infty} \left[\frac{2}{M} Sin\left(M \frac{Z}{H} \right) e^{-M^2 T_v} \right] \tag{2.3}$$

where:

$\Delta\sigma$ = Stress Increment,

M = $(2N+1)\frac{\pi}{2}$,

T_v = $\frac{C_v t}{H^2}$ = Dimensionless time factor,

Z = $\frac{Z}{H}$ = Dimensionless geometry parameter, and

H = The longest drainage distance.

The analytical solution obtained through use of Equation 2.3 reasonably agrees with laboratory obtained results. Nonetheless, neither laboratory obtained results nor those obtained through use of Equation 2.3 agree with observed settlements in the field for some clays and organic soils (McVay and Nugyen 2004). The variation between predicted values using Terzaghi's theory and those observed in the field exist due to the limitations within his theory. Terzaghi's consolidation theory maintains that permeability is constant throughout the consolidation process. Although this assumption can never be fully met, it permits a reasonably good prediction of settlement, if the strain is in some sense small. In the case of some clays and organic soils, the strain is quite large, and at some point permeability begins to decrease with increasing strain.

2.5.3 Secondary Compression

At the completion of primary settlement, essentially all the excess pore-water has dissipated and the soil is said to begin the secondary compression phase. As described by Holtz and Kovacs (1981), secondary compression is a continuation of the volume change that started during primary consolidation, only it occurs at a much slower rate. Secondary compression is different from primary consolidation in that it takes place under a constant effective stress.

Raymond and Wahls (1976) and Mesri and Godlewski (1977) defined an index that quantifies the magnitude of secondary settlement as C_α, known as the secondary compression index.

$$C_\alpha = \frac{\Delta e}{\Delta \log t} \tag{2.4}$$

Where:

Δe = Change in void ratio between t_2 and t_1 along the *e vs log t* curve, and

Δt = Time between t_2 and t_1.

Likewise, Ladd et al. (1971) defined an additional measurement of secondary compression known as the rate of secondary compression, $C_{\alpha\varepsilon}$, defined by Equation 2.5.

$$C_{\alpha\varepsilon} = \frac{\Delta \varepsilon}{\Delta \log t} \tag{2.5}$$

Where:

$\Delta\varepsilon$ = The change in strain between t_2 and t_1 along the *ε vs log t* curve, and

Δt = Time between t_2 and t_1.

The secondary compression index, C_α, and the rate of secondary compression, $C_{\alpha\varepsilon}$, can be determined from the slope of the straight line portion of the dial reading versus log time curve which occurs after primary consolidation is complete. Holtz and Kovacs (1981) state that the determination of C_α and $C_{\alpha\varepsilon}$ should be evaluated over the course of one log cycle of time after the transition phase.

To allow the abovementioned indices of secondary compression to be defined and used to calculate secondary settlement, the following assumptions must hold true. As described by Holtz and Kovacs (1981), these assumptions are based on the work of Ladd (1971) and others and summarized by Raymond and Wahls (1976), are as follows.

- C_α is independent of time (at least during the time of interest);
- C_α is independent of the thickness of the soil layer;
- C_α is independent of LIR, as long as some primary consolidation occurs; and
- The ratio C_α / C_c is approximately constant for normally consolidated clays over the normal range of engineering stresses.

Secondary compression plays a critical role in soils that contain a significant amount of organic material. For organic soils that exhibit a pre-consolidation pressure, the values of C_α are small at effective stresses less than the pre-consolidation pressure, increase rapidly as the pre-consolidation pressure is approached, maximize in the range of σ_p' *to* $2\sigma_p'$, and then either decrease with effective stress or first decrease and then remain fairly constant, where σ_p is the pre-consolidation pressure (Mesri and Godlewski, 1977). Mesri et al. (1997) found peat deposits to be more susceptible to secondary compression due to high natural moisture contents and high void ratios. Mesri et al. (1997) also reported that the duration of primary consolidation is relatively short due to the high initial permeability of peat deposits. The initial high permeability of peat is typically *100 – 1,000* times the initial permeability of soft clays and silts, and the initial coefficient of consolidation of peat, c_v is *10 – 100* times larger (Mesri et al., 1997). Although the duration of primary settlement is relatively short, it does not indicate that the magnitude of primary settlement is relatively small. It is quite the opposite. Peats not only suffer from significant secondary compression, but also suffer from relatively large amounts of primary settlement. In many cases, primary consolidation is nearly completed by the end of construction, and the settlement over the entire design life of the structure occurs as secondary compression (MacFarlane 1965).

2.5.3.1 Middleton Peat Study

Mesri et al (1997) reported a comprehensive laboratory investigation on the secondary compression behavior of light brown fibrous peat obtained from test pits excavated in Middleton, Wisconsin in 1992. Table 2.1 provides the range of properties that were determined for the Middleton peat (Mesri 1997).

Table 2.1: Properties of Middleton Peat (Mesri et al. 1997)

Total Unit Weight	57.9 – 64.9 lb/ft^3
Natural Water Content	623 – 846%
In-situ Void Ratio	10.1 – 14.2
Initial Degree of Saturation	89 – 100%
Organic Content	90 – 95%
Specific Gravity of Solids	1.53 – 1.65

Based upon pore pressure measurements in the laboratory on Middleton peat at $\sigma_v' \leq \sigma_p'$, where σ_v' is vertical effective stress and σ_p' is the pre-consolidation pressure, the End-of-Primary (EOP) was reached in less than 1 minute, i.e., $t_p \leq 1\ min$, where t_p is the duration of time in which primary settlement occurs (Fox and Edil 1996). But, as σ_v' increased to pressures beyond σ_p', the t_p increased as well. For values of $\sigma_v' \approx 2\sigma_p'$, the t_p ranged from $35 - 40$ minutes (Mesri et al., 1997). The largest values of t_p obtained by Mesri et al. (1997) corresponded to the largest σ_v', i.e., when $\sigma_v' \approx (4\sigma_p' - 8\sigma_p')$ and $\sigma'_v \geq 8\ \sigma_p'$, t_p increased to $35 - 150$ minutes and to $150 - 500$ minutes respectively.

2.5.3.2 The C_α / C_c Concept

While exploring the compressive behavior of organic soils, the C_α / C_c law of compressibility must be examined, where C_α is the Secondary Compression Index and C_c is the Compression Index. Mesri and Godlewski (1977) postulated that for any given soil, there is a unique relationship between $C_\alpha = \Delta e / \Delta log\ t$ and $C_c = \Delta e / \Delta log\ \sigma'$, that holds true at all combinations of time, effective stress, and void ratio as shown in Figure 2.3.

At any given effective stress, the value of C_α from the first log cycle of secondary compression and the corresponding C_c value computed from the EOP e vs $log\ \sigma_v'$ are used to define the relationship between C_α and C_c. It is specifically noted that C_c denotes the slope of e vs $log\ \sigma_v'$ curve throughout the recompression and compression ranges. Table 2.2 illustrates a wide range of C_α / C_c values compiled from worldwide data available on peats.

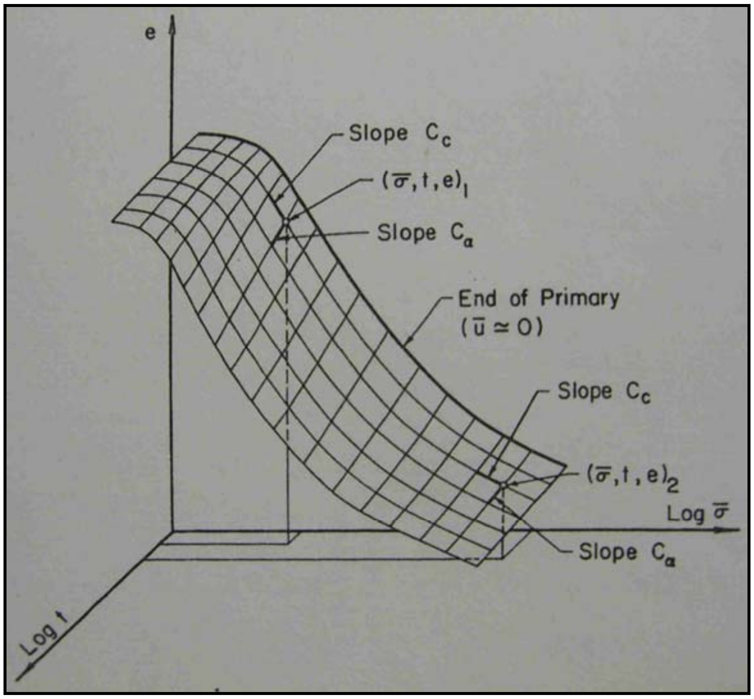


Figure 2.3: Void Ratio-Time-Pressure Relationship During Secondary Compression (Mesri and Godlewski, 1997)

Table 2.2: Values of Natural Water Content, w_0, Initial Vertical Coefficient of Permeability, k_{v0}, and C_α / C_c for Peat Deposits (Mesri et al., 1997)

Organic Soil (1)	w % (2)	k_v m/s (3)	C_α/C_c (4)	Reference (5)
Fibrous peat	850	4×10^{-6}	0.06 - 0.10	Hanrahan (1954)
Peat	520	--	0.061 - 0.078	Lewis (1956)
Amorphous and fibrous peat	500-1,500	10^{-7} - 10^{-6}	0.035 - 0.083	Lea and Brawner (1963)
Canadian muskeg	200-600	10^{-5}	0.09 - 0.10	Adams (1965)
Amorphous and fibrous peat	705	--	0.073 - 0.091	Keene and Zawodnaik (1968)
Peat	400-750	10^{-5}	0.075 - 0.085	Weber (1969)
Fibrous peat	605-1,290	10^{-6}	0.052 - 0.072	Samson and LaRochell (1972)
Fibrous peat	613-886	10^{-6} - 10^{-5}	0.06 - 0.085	Berry and Vickers (1975)
Amorphous to fibrous peat	600	10^{-6}	0.042 - 0.083	Dhowain and Edil (1981)
Fibrous peat	660-1,590	5×10^{-6} – 5×10^{-5}	0.06	Lefebvre et al. (1984)
Dutch peat	370	--	0.06	Den Haan (1994)
Fibrous peat	610-850	6×10^{-8} - 10^{-7}	0.052	Mesri et al. (1997)

Mesri et al. (1997) concluded that the most detailed measurements and existing data suggest a range of $C_\alpha / C_c = 0.06 \pm 0.01$ for peat deposits and $C_\alpha / C_c = 0.05 \pm 0.01$ for organic clays and silts. Table 2.3 depicts the tentative ASTM standard for the grouping of organic soils, including muck and peat.

Table 2.3: Grouping of Organic Materials (Tentative ASTM Standard)

Ash Content (%)	Material Description
0 - 25	Peat
50 - 75	Muck
75 - 100	Organic Silt or Clay

2.5.3.3 Florida Organic Soils Study

McVay and Nugyen (2004), from the University of Florida, in conjunction with FDOT, recently conducted a comprehensive laboratory and field investigation involving organic soils collected from three locations in Florida: (i) Sander's Creek Bridge and embankment over State Road 20 in the Florida panhandle; (ii) Toll 417/Seminole Expressway in Sanford (central Florida); and (iii) State Road 15/700 in Palm Beach County (about 10 miles south of the pilot test site selected in the current study). The study included laboratory consolidation and permeability testing, prediction of embankment settlement with time, and comparison of predicted values with field settlement measurements. Table 2.4 provides the range of pertinent properties of Florida organic soils as reported by McVay and Nugyen (2004). In the context of the present study, a summary of their research findings is provided in the following sections.

Table 2.4: Properties of Florida Organic Soils

Natural Water Content	25 – 582%
In-situ Void Ratio	0.73 – 8.68
Organic Content	4.8 – 91.5%
Specific Gravity of Solids	1.1 – 2.55

Laboratory investigation reported by McVay and Nugyen (2004) included 21 one-dimensional oedometer tests on samples grouped into 3 different categories based on organic content (OC): (i) Low (0%<OC<25%); (ii) Medium (25%<OC<50%); and (iii) High (OC>50%). All of the tests were conducted in accordance with the Multiple Stage Loading (MSL) procedure with a Load Increment Ratio (LIR) of one. The Oedometer tests were designed to duplicate field stress conditions, i.e., consolidation under in-situ pressure, consolidation under surcharge pressure, then rebound and creep under final pressure.

Results obtained from the consolidation tests suggest that the Low OC generated the typical S-shaped curve *(e vs. log t)*, while the Medium and High OCs showed significant creep effects. Defining the time at which consolidation moves from Primary to Secondary, and from Secondary to Tertiary is quite difficult. As demonstrated by the laboratory results, the Medium OC and High OC soils displayed no well-defined End-of-Primary (EOP) consolidation. Likewise, the C_α remained somewhat constant throughout the entire time period of readings.

The following are general observations that were made about the Florida organic soils:

- Soils with OC between *25%* and *50%* exhibit Secondary and Tertiary Creep, but upon unloading will undergo rebound with no downward creep;
- Soils with OC greater than *50%* will experience Secondary and Tertiary creep under loading, and will exhibit some downward creep after surcharge removal;
- Due to organic soil's high permeability, excess pore water dissipation takes place rather rapidly resulting in quick Primary consolidation. In the laboratory it was

demonstrated that some consolidation tests dissipated excess pore water pressure within *10* minutes;

- Long-term settlements are mostly from creep, which can contribute up to *50%* of the total settlement;
- Tertiary creep was observed in all soil samples with OCs greater than *25%* under loading conditions;
- Tertiary creep occurred after approximately one week of sustained load in the laboratory; and
- Tertiary creep was not only demonstrated in the laboratory, but was also present in the field during the site-monitoring program.

2.5.4 Louisiana Clay Study

Abu-Farsakh (2004) and Abu-Farsakh and Nazzal (2005) reported Piezocone Penetration test (CPTu) conducted on clays at seven different sites in Louisiana. A concurrent laboratory testing program was also undertaken. Data from the CPTu testing, such as cone tip resistance, sleeve friction, pore pressures and dissipation tests were correlated to such properties as Tangent Constrained Modulus, Overconsolidation Ratio and vertical Coefficient of Consolidation.

Since sampling of soft soils is difficult, and subsequent laboratory testing is costly and time consuming, the Piezocone Penetration Test (PCPT) is an attractive alternative in terms of both time and cost savings. By conducting a pore water dissipation test at various depths of the unsuitable layer, the c_v parameter can be estimated in-situ. In this procedure, the cone penetration is stopped at a desired depth, and the dissipation of excess pore water pressure with elapsed time is recorded. The excess pore pressure, u_e is defined as the difference between the penetration pore pressure (u) and the static equilibrium pore pressure (u_0). Several empirical, semi-empirical, analytical, and finite element based interpretation methods have been developed by researchers since the late 1980s for the estimation of the horizontal coefficient of consolidation (c_h) from the piezocone dissipation tests (Lunne et al, 1997); an excellent summary is also provided by Abu-Farsakh (2004). The vertical coefficient of consolidation c_v can then be calculated from c_h using relationships suggested by Levadoux and Baligh (1986). These procedures have been successfully implemented in six roadway test sites in Louisiana (Abu-Farsakh, 2004). However, use of these techniques in case of organic soils and peat is not well documented. The current study attempted to establish the use of Piezocone Penetration tests as a versatile subgrade characterization tool for organic rich soils in Florida.

CHAPTER 3

Preliminary Investigations

3.1 GEOTECHNICAL STUDY BY GEOSOL, INC.

A preliminary geotechnical site characterization was previously conducted by GEOSOL, Inc. in 2004, along the SR15 / US98 roadway from Canal Point up to the Palm Beach / Martin County line (GEOSOL, 2004). Brief description and important findings of this project are presented below.

3.1.1 GEOSOL Field Exploratory Program

Geosol, Inc. (GEOSOL) prepared a preliminary report of roadway soils survey for FDOT dated June 2, 2004 in connection with the SR 15/US 98 Roadway Improvements from North of the Palm Beach Canal Bridge to the Palm Beach/Martin County Line in Palm Beach County, Florida. Station numbers were previously set for the alignment and ranged from approximately 1+00 near the Palm Beach Canal Bridge, or Mile Post 19.674, to about 360+00 at the Palm Beach/Martin County Line, or Mile Post 26.519.

For their study, GEOSOL drilled ninety-three (93) Standard Penetration Test (SPT) borings (numbered B-1 to B-93) to a depth of 20 feet (6.1 m) using a Foremost-Mobile Model B-53 drill rig mounted on a truck. GEOSOL's borings were drilled in general accordance with ASTM D 1586, "Standard Practice for Penetration Test and Split-Barrel Sampling of Soils." The borings were spaced at 500-foot (152 m) intervals and staggered left and right of the roadway except for a 1.2-mile stretch north of the Sand Cut Canal Bridge (Stations 225+00 to 259+00) where the boring spacing was at 200 feet (61 m). GEOSOL noted the worst areas of distressed pavement from Stations 149+00 to 175+00, 225+00 to 259+00 and 304+00 to 312+00. These areas included SPT Borings B-30 to B-43, B-53 to B-70 and B-79 to B-83. In addition, six (6) Shelby tube samples were obtained in accordance with ASTM D 1587 at the locations of Borings B-35, B-42, B-57, B-66, B-79 and B-83, which corresponded to the worst areas of pavement distress.

3.1.2 GEOSOL Generalized Subsurface Conditions

The GEOSOL study found the site to be underlain in descending order by 7 to 20 inches (178 to 508 mm) of asphalt pavement, 12 to 18 inches (305 to 457 mm) of pavement base materials, 4 to 13 feet (1.2 to 3.9 m) of sand and gravel fill (AASHTO Soil Classification A-1-b, A-2-4 and A-3), and 4 to 18 feet (1.2 to 5.5 m) of organic silt and peat (AASHTO Soil Classification A-8). Below the organic soils are sandy silt and/or sand with shells (AASHTO Soil Classification A-2-4 and A-4) and sandy limestone to the maximum depth of exploration of 20 feet (6.1 m). The average thickness of organic layer was about 10 feet (3 m). The depth to the top of the organic soils range from 2 to 13 feet (0.61 to 4 m) below the ground surface, with an average depth of 6 feet (1.8 m) to the top of the organic soils. The depth to the bottom of the organic soils range from 11.5 to 18 feet (3.5 to 5.5 m) below the ground surface, with an average depth of 16 feet (4.9 m) to the bottom of the organic soils. Groundwater was found between 6.5 to 9.2 feet (1.98 to 2.8 m) below the ground surface, with an average depth of 8.3 feet (2.5 m).

3.1.3 GEOSOL Laboratory Program

GEOSOL performed laboratory testing on selected samples obtained from the SPT borings and Shelby tubes. Samples for the laboratory testing included organic soils, sand and silt. The tests consisted of eighty-seven (87) moisture content tests, sixteen (16) grain-size analyses plus an additional fifty-seven (57) fines content tests, fifty-one (51) organic content tests, twelve (12) Atterberg Limits Testing, six (6) Consolidation Testing, six (6) Unconsolidated-Undrained (UU) Triaxial Compression Testing and six (6) environmental corrosion tests (pH, resistivity, sulfates content and chlorides content).

3.1.4 GEOSOL Laboratory Results

Results of the laboratory testing and organic soil properties from Shelby tube samples are summarized in Table 3.1 and 3.2, respectively.

Table 3.1: Summary of GEOSOL Laboratory Test Results

Property	Sand and gravel fill (A-1-b, A-2-4, A-3)	**Organic soils (A-8)**	Sand or silt (A-2-4, A-4)
Moisture Content (%)	3.4 – 46.8	**24.8 – 592.5**	35.5 – 238.1
Organic Content (%)	4 – 6.9	**7.4 – 100**	Not tested
Fines Content (%)	1.7 – 31.5	**4.7 – 53.2**	21.6 – 62.7
Atterberg Limits Plasticity Index	Not tested	**Not tested**	Non-plastic

Table 3.2: Summary of GEOSOL Shelby Tube Soil Properties (1 lb/ft^3 = 0.157 KN/m^3)

Total Unit Weight	67.2 – 74.9 lb/ft^3
Moisture Content (%)	80.7 – 458.4
Fines Content (%)	47.3 – 99.6
Specific Gravity of Solids	1.65 – 2.20

Results of the Unconsolidated-Undrained (UU) Triaxial Compression Testing of the organic soils from the Shelby tube samples are summarized in Table 3.3.

Table 3.3: Summary of GEOSOL UU Triaxial Test Results (1 lb/ft^3 = 0.157 KN/m^3; 1 lb/ft^2 = 0.047 KPa)

Dry Unit Weight	11.1 – 28.0 lb/ft^3
Moisture Content (%)	178.1 – 458.4
Saturation (%)	92 – 100
Fines Content (%)	61.9 – 99.6
Compressive Strength	389 – 1454 lb/ft^2

3.2 IDENTIFYING FIELD TEST LOCATIONS

A 3-step approach was then undertaken to identify the exact number and locations for the Piezocone Penetration Tests (CPTu) and retrieval of Shelby tube samples. The 3 steps are as follows:

3.2.1 Site Visit and Visual Identification

On July 4, 2005 the project team visited the site along the 11.0 kilometer segment of SR 15 / US 98, visually identified the more severely distressed locations, compared and verified these observations with those reported in the GEOSOL Final Project Report, and tentatively selected the locations for the upcoming in-situ geotechnical characterizations.

3.2.2 Analysis of Site-specific Data

The subsurface investigation report by GEOSOL contained data on organic content (OC) and moisture content (MC) from 50 test boreholes. These data were further analyzed and compiled to group or classify the test locations in terms of organic contents, and also to explore any possible correlations between the observed distress levels and organic content and/or thickness of the organic layer in a particular location. These results are summarized in Figures 3.1 through 3.4. Figures 3.1 and 3.2 show the organic content, the organic layer thickness and the observed distress levels at all 50 locations. The locations colored yellow signify "severely" distressed areas, while the blue color signifies "moderately" distressed areas, as interpreted from the GEOSOL Inc. report. It is found that the organic content varied from about 10% to nearly 100%. The average depth to the muck layer was about 1.83 meters. The average muck layer thickness is about 3.05 meters, with the maximum value as high as 5.49 meters. The sites were grouped according to the organic contents (OC) and classified as follows: (i) Low (0%<OC<25%); (ii) Medium (25%<OC<50%); and (iii) High (OC>50%). The average values of these attributes are presented in Table 3.4. It is found that, in general, the "severely distressed" areas have higher average organic contents, moisture contents, and total thickness of organic layers, compared to "moderately distressed" regions.

Figure 3.3 and Figure 3.4 show the variations of moisture content and void ratios of the sites, respectively, with organic content (OC). As expected, both void ratio and moisture content are found to increase with increasing organic content (OC). It is found that the moisture content could reach as high as 600% and void ratio could exceed 9.0 in some locations. A preliminary analysis of data is presented in Table 3.4 and shows the "severely" distressed areas had higher average muck layer thicknesses, and higher moisture and organic contents compared to "moderately" distressed pavements.

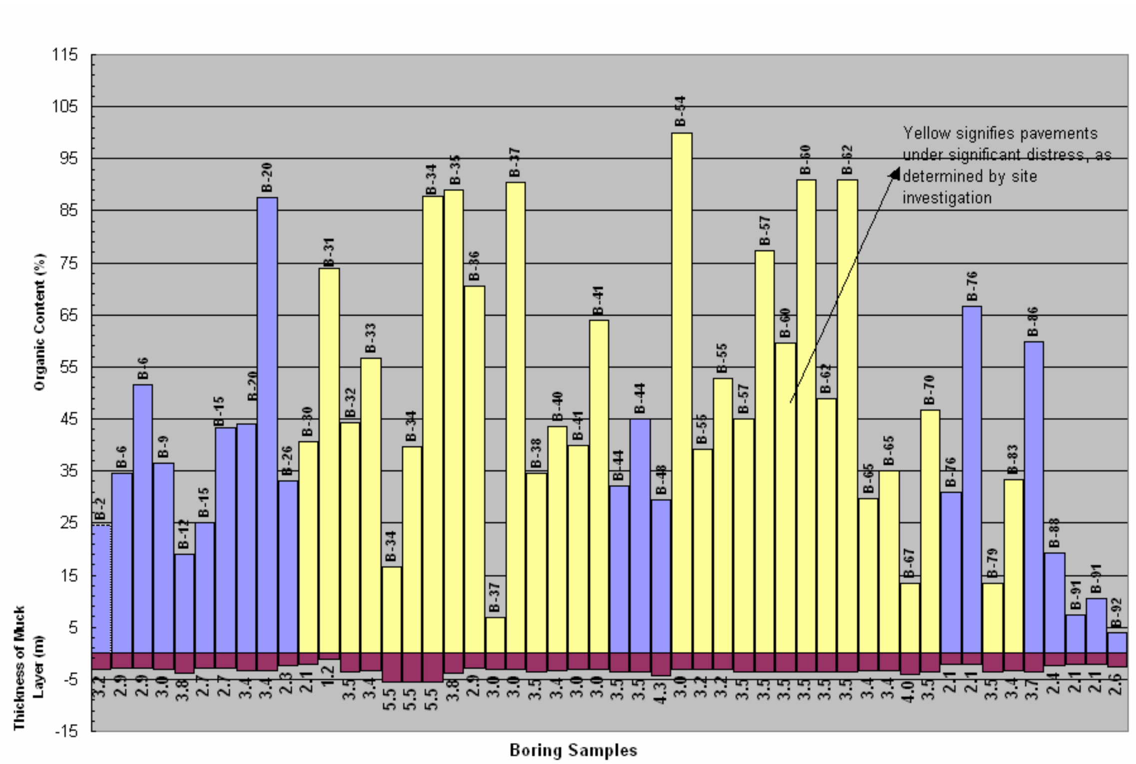


Figure 3.1: Organic Content and Organic Layer Thickness

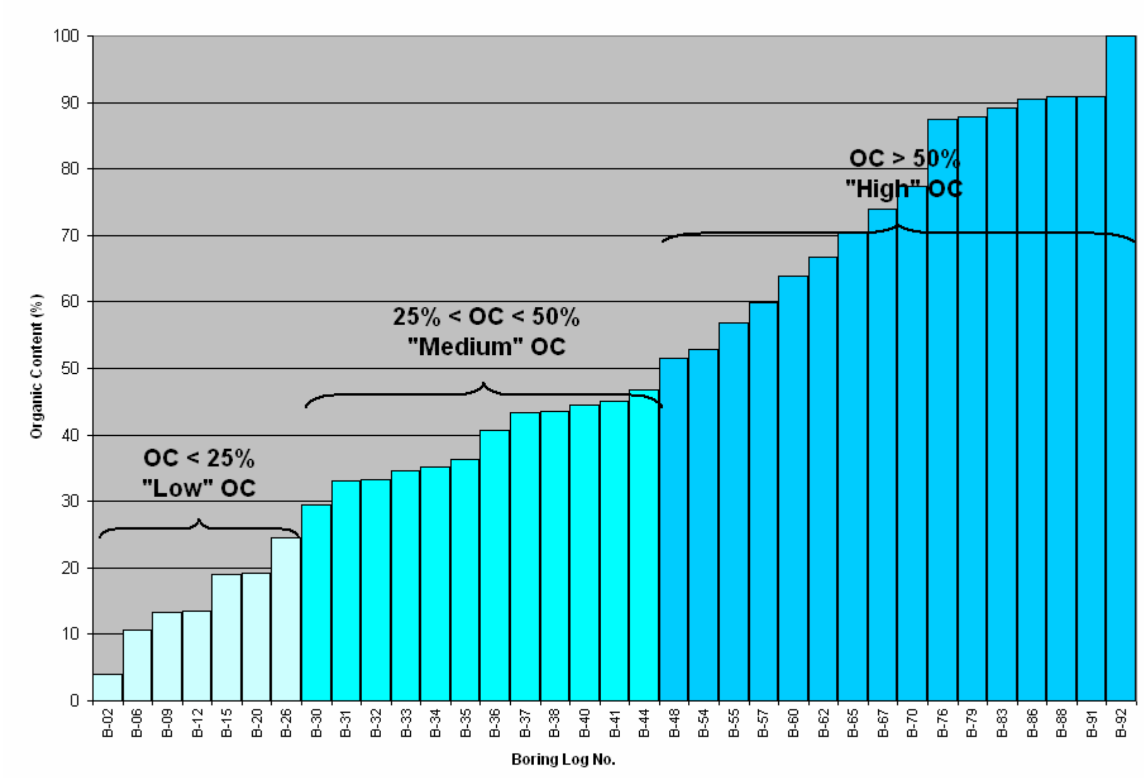


Figure 3.2: Classification of Sites According to Organic Content

Table 3.4: Characteristics of “Severely” and “Moderately” Distressed Pavements

Attributes	Moderately Distressed Pavement	Severely Distressed Pavement
Depth to Muck Layer (m)	1.83	1.83
Thickness of Muck Layer (m)	2.90	3.51
Avg. Moisture Content (%)	160	220
Avg. Organic Content (%)	36	53

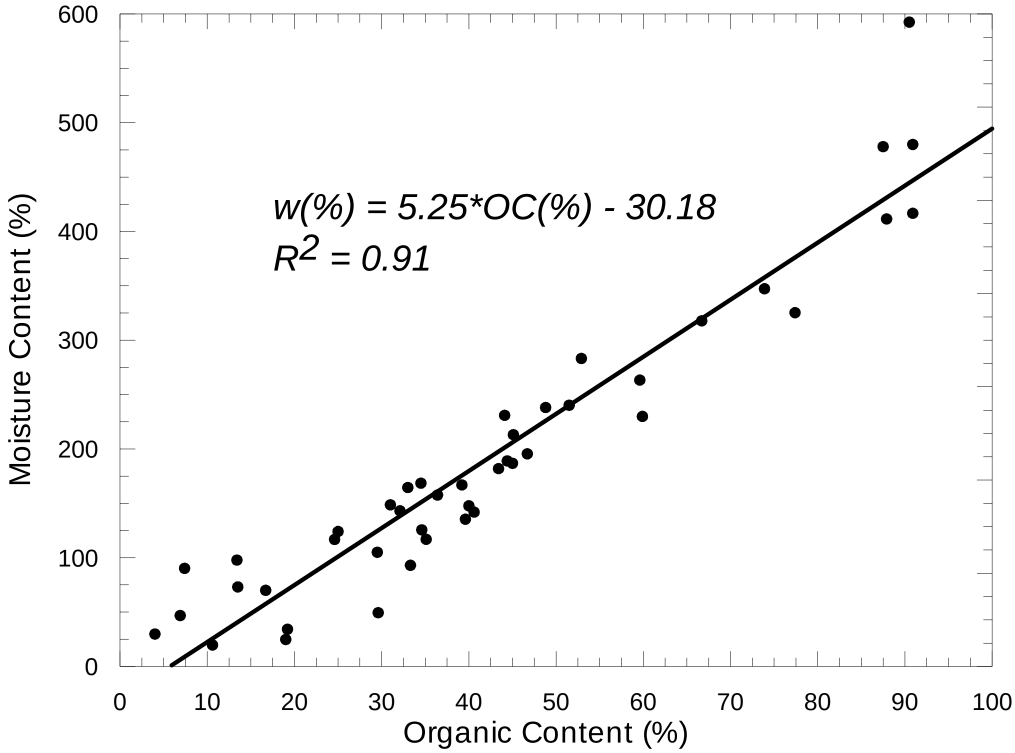


Figure 3.3: Variations of Moisture Content with Organic Content

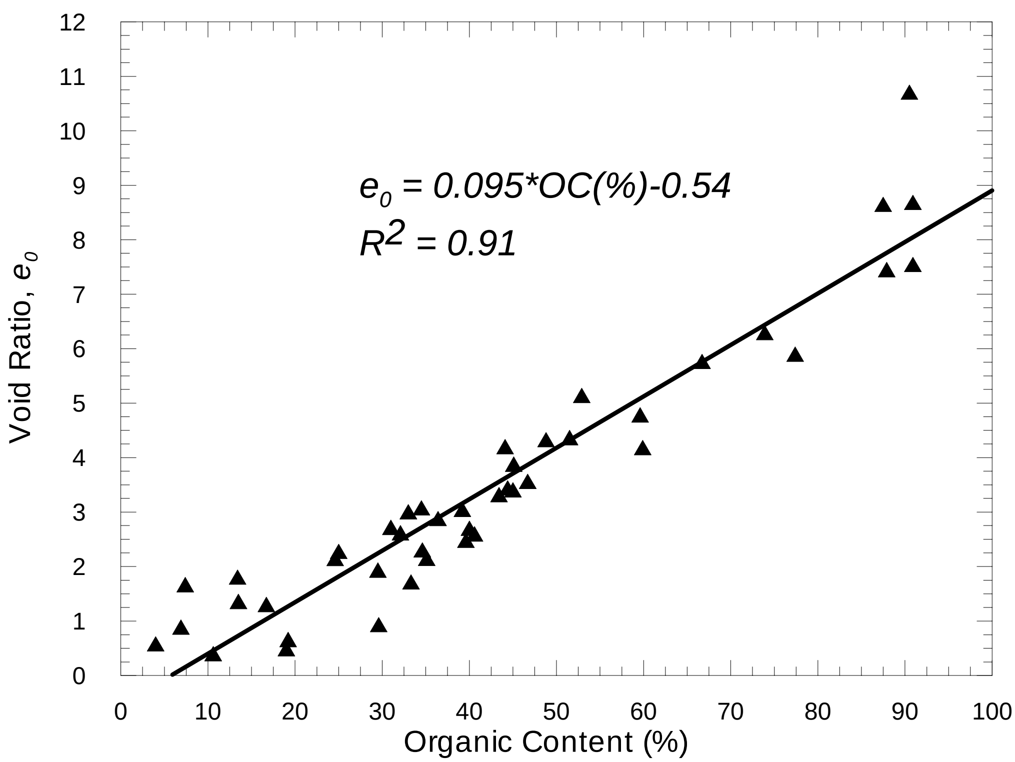


Figure 3.4: Variations of Void Ratio with Organic Content

3.2.3 Identifying CPTu and Shelby Tube Sampling Locations

During the preliminary site investigation preformed by GEOSOL, 93 SPT borings were completed. Along with the SPT borings, six (6) Shelby tubes were extracted from the observed areas of significant pavement distress. Although this provided an in-situ profiling of the underlying soil layers, and some material properties, it did not explicitly identify critical areas for further investigation. The available data from these investigations were used in conjunction with theoretical soil mechanics principles to determine 12 critical locations for Piezocone Penetration Tests and soil samplings.

The objective of this effort was to develop a criterion to identify critical locations based on organic content (OC), moisture content (MC), and muck layer thickness. Phase relationships were used to calculate a *Theoretical Organic Weight* (W_o), which represents the weight of the pure organic content within the muck at any given boring location. Hereafter, this parameter is termed the *Organic Factor*, F_{org}, which is assumed to be strongly related to the secondary compression characteristics of the organic soils and peats.

Considering a column of muck layer with total unit weight, γ_T, thickness, h_m, and a unit cross sectional area, the total wet weight, W_T of the column is given by (Figure 3.5):

$$W_T = h_m * 1 * \gamma_T = W_s + W_o + W_w \tag{3.1}$$

Where, W_s, W_o, and W_w represent the weight of the inorganic soil solids, weight of the pure organics, and the weight of pore water, respectively.

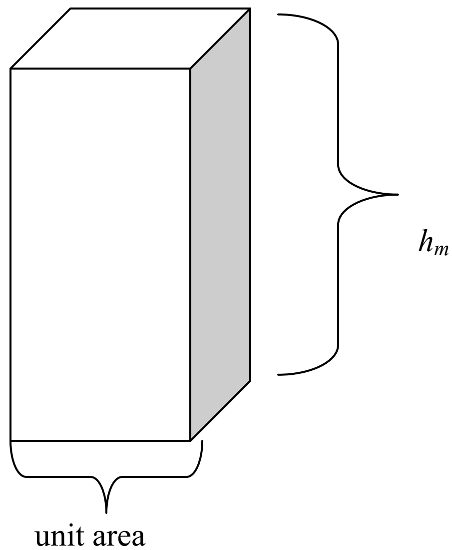


Figure 3.5: Theoretical Column of Organic Soil of Unit Cross Sectional Area

The Ash Content, *AC*, Organic Content, *OC*, and the Moisture Content, *MC* are given by:

$$AC(\%) = \frac{W_s}{W_s + W_o} x100 \tag{3.2}$$

$$OC(\%) = 100 - AC \tag{3.3}$$

$$MC(\%) = \frac{W_w}{W_s + W_o} x100 \tag{3.4}$$

Combining equations 3.1 through 3.4, the Organic Factor, F_{org} can be expressed as:

$$F_{org} = W_o = \frac{h_m \gamma_T * OC}{(1 + MC)} \tag{3.5}$$

The preliminary data from GEOSOL was used to calculate F_{org} for all 50 boreholes as shown in Figure 3.6, from which 12 borehole locations were selected for Piezocone Penetration Tests (CPTu) and Shelby tube sampling. These locations represented a wide distribution of F_{org}, which in turn incorporated *site-specific* conditions such as the total thickness of the muck layer, the organic content, the moisture content, and the unit weight. One of the goals of this project is to develop site-specific criteria, which are related to the difference in observed distress levels at various locations requiring different types of rehabilitation strategies. Table 3.5 presents the 12 locations along SR 15 / US 98 which were initially selected for the in-situ geotechnical characterizations. It is noted that location B-62 – STA 242 + 94.80 was discarded due to time constraints.

Table 3.5: Selected Locations for CPTu and Shelby Tube Sampling

BORING No.	MILE POST (MP)	STATION (ft)	ORGANIC CONTENT (%)	MOISTURE CONTENT (%)	DEPTH TO MUCK LAYER (m)	THICKNESS OF MUCK LAYERR (m)
B-33	22.600	155 + 00.75	56.8	466.2	1.83	3.35
B-34	22.645	156 + 98.16	87.9	411.5	0.61	5.49
B-35	22.676	158 + 97.76	89.1	88.3	0.91	3.81
B-37	22.752	162 + 98.37	90.5	592.5	1.83	3.05
B-41	22.903	170 + 98.01	63.9	42.3	1.83	3.05
B-54	23.981	227 + 02.48	100.0	189.2	2.13	3.05
B-57	24.078	233 + 01.07	77.4	325.3	1.52	3.51
B-60	24.191	238 + 96.90	90.9	480.0	1.83	3.51
B-62	24.267	242 + 94.80	90.9	416.7	1.83	3.51
B-65	24.381	249 + 02.02	35.1	116.9	1.83	3.35
B-67	24.456	252 + 98.94	13.5	73.1	0.91	3.96
B-70	24.570	258 + 97.75	46.7	195.5	1.83	3.51

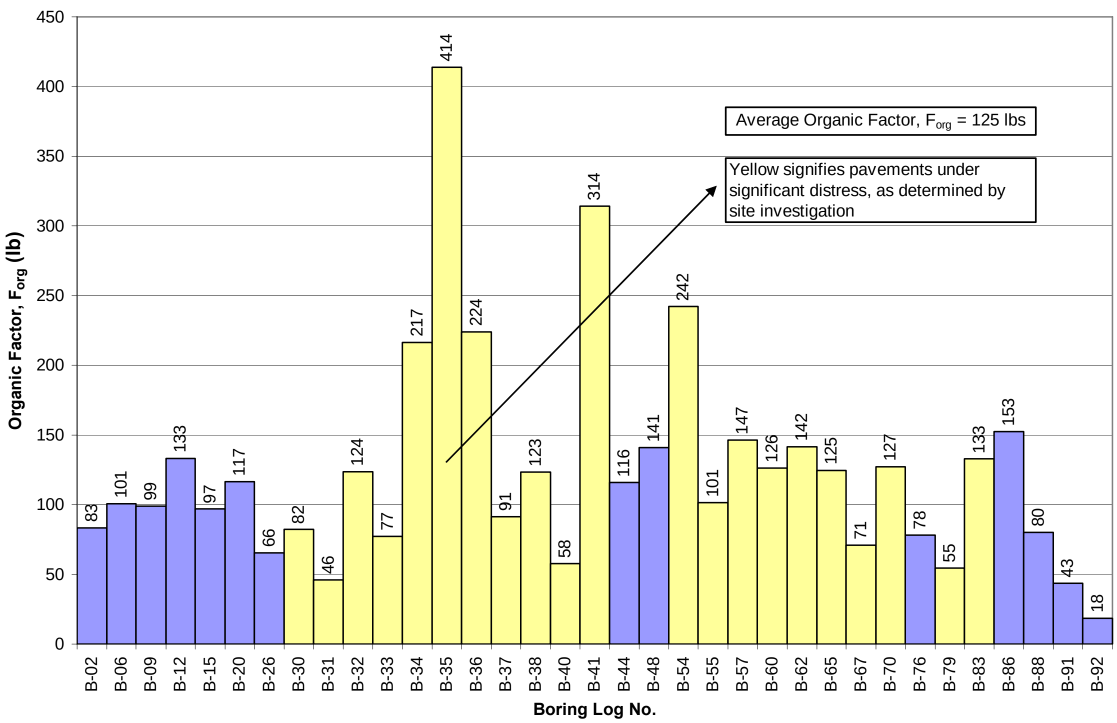


Figure 3.6: Organic Factor at Various Borehole Locations

3.3 SOIL CHARACTERIZATION – CURRENT STUDY

3.3.1 Soil Description

The organic-rich soil samples obtained from SR 15 / US 98 consist of two distinct types of soil. Visual inspection of the stratum directly under the sub-base course indicates black organic sandy silts with shell fragments. To the touch, it is highly plastic in nature and is easily deformed. Upon deformation it exhibits a high degree of cohesion. It is comprised of mainly sandy silts with almost no visual sign of decomposed plant material. The stratum directly below the aforementioned black organic sandy silt is brown peat ranging from dark brown to light brown in color. It is comprised of mainly decomposed vegetation with stalks or roots oriented in the vertical direction. It exhibits more compressive strength than the above stratum and is not easily deformed. Upon deformation, the soil particles tend to crumble showing less cohesion than the above stratum.

3.3.2 Organic Content

Organic content (OC) of the Shelby tube samples was measured approximately in accordance with ASTM D2974-00 procedure. The soils could be divided into twp pre-dominant groups: (i) the first group is defined as silty organics (silty muck), containing 25% to 75% organic material, and obtained from depths of 1.83 to 2.74 meters. They ranged in color from faded black to dark brown; (ii) the second group, defined as peat (often fibrous), contained more than 75% organic material, had light brown color, and was obtained from depths below 2.74 meters. Table 3.6 shows results from 11 sampling locations, and considered more precise than the average GEOSOL data (Table 3.5).

Table 3.6: Organic Content of Soil Samples

SITE	STATION	SAMPLE ID #	Shelby Tube Depth (m)	VOID RATIO, e_0	NATURAL MOISTURE CONTENT, w(%)	ORGANIC CONTENT, OC(%)
1	155 + 00	ST155(1)	1.83 - 2.44	3.91	205.1	60.2
	155 + 00	ST155(2)	2.44 - 3.05	6.13	367.9	61.0
2	157 + 00	ST157(1)	1.83 - 2.44	5.58	327.9	31.8
	157 + 00	ST157(2)	2.44 - 3.05	7.44	382.2	76.0
3	159 + 00	ST159(1)	1.83 - 2.44	3.19	176.3	39.3
	159 + 00	ST159(2)	2.44 - 3.05	5.67	302.2	61.4
4	163 + 00	ST163(1)	2.13 - 2.74	3.41	186.6	21.3
	163 + 00	ST163(2)	3.66 - 4.27	9.82	508.3	92.3
5	171 + 00	ST171(1)	2.13 - 2.74	-	162.1	29.5
	171 + 00	ST171(2)	3.66 - 4.27	10.74	582.3	92.3
6	227 + 00	ST227(1)	2.13 - 2.74	3.96	218.3	34.0
	227 + 00	ST227(2)	3.66 - 4.27	10.62	549.1	26.6
7	233 + 00	ST233(1)	2.13 - 2.74	4.04	195.8	31.9
	233 + 00	ST233(2)	3.66 - 4.27	7.40	370.4	90.6
8	239 + 00	ST239(1)	2.13 - 2.74	3.29	196.9	28.7
	239 + 00	ST239(2)	3.66 - 4.27	12.20	568.0	73.6
9	249 + 00	ST249(1)	2.13 - 2.74	3.63	201.2	37.5
	249 + 00	ST249(2)	3.66 - 4.27	9.56	464.7	84.0
10	253 + 00	ST253(1)	2.13 - 2.74	3.19	190.6	35.8
	253 + 00	ST253(2)	3.66 - 4.27	13.93	652.2	91.5
11	259 + 00	ST259(1)	2.13 - 2.74	4.03	195.4	32.6
	259 + 00	ST259(2)	3.66 - 4.27	9.47	453.2	84.2

It was found that the top stratum, classified as silty muck had a mean organic content of *35%,* while the bottom stratum, classified as fibrous peat had a mean organic content of *76%.* Figure 3.7(a) displays a site profile of organic content arranged in ascending order relative to station identifications from the southern most point northward. Figure 3.7(b) displays the organic content grouped into Low, Medium and High subgroups.

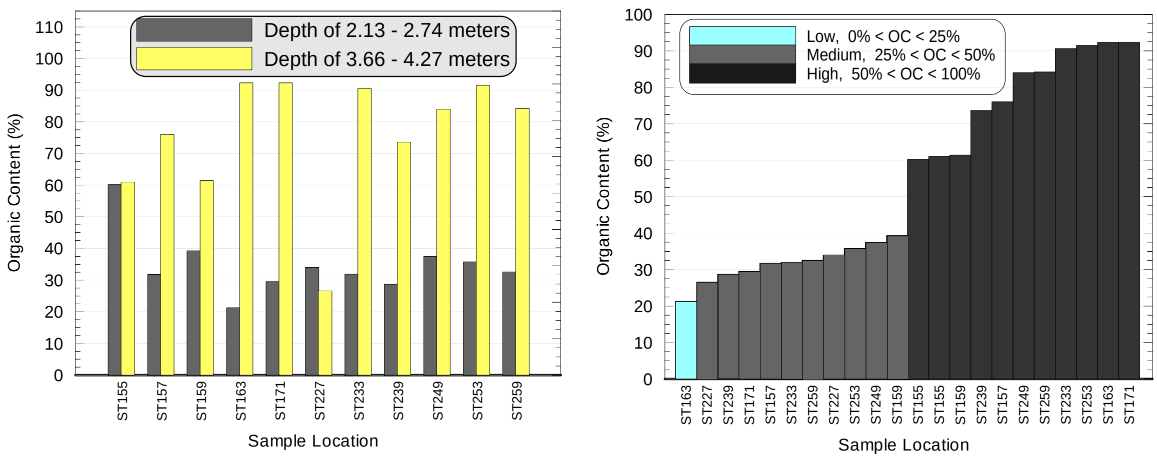


Figure 3.7: Organic Content by (a) Sample Location and (b) Subgroups

3.3.3 <u>Moisture Content</u>

Moisture content was determined on 22 samples. The top stratum, typically classified as silty muck has a mean moisture content of *205%,* while the bottom stratum, classified as fibrous peat has a mean moisture content of *473%.*

3.3.4 <u>Void Ratio</u>

The void ratio in organic soils is generally higher than soils containing a large constituent of mineral material. Extreme ranges in void ratio for amorphous to granular peat have been reported from 2 to 25, with 5 to 15 being the norm (Hanrahan, 1954). The void ratio of the top stratum, typically classified as silty muck, ranged from *3.19 to 5.58* with a mean void ratio of *3.82*. The void ratio of the bottom stratum, classified as fibrous peat, ranged from *5.67 to 13.93* with a mean void ratio of *9.36*.

3.3.5 Specific Gravity

Typical values for specific gravity of soil vary from *1.1 to 2.7*. Peats and organic-soils alike have specific gravities significantly less than mineral-rich soils, and are generally in the range *1.4 to 1.8* (Wyld, 1956). Limited number of tests conducted in this study confirmed these ranges reported in the literature.

3.3.6 Unit Weight

The unit weight at each sampling location was determined from undisturbed Shelby tube specimens at two different depths. MacFarlane (1969) reported that natural unit weights have been observed to range from $3.92\ kN/m^3$ for a moss peat to $11.76\ kN/m^3$ for an amorphous-granular peat. In the current study, the unit weight of the top stratum, classified as silty muck, ranged from $10.97\ kN/m^3$ to $11.66\ kN/m^3$ with a mean unit weight of $11.37\ kN/m^3$. For the second stratum, classified as fibrous peat, the unit weight ranged from $9.70\ kN/m^3$ to $10.98\ kN/m^3$ with a mean unit weight of $10.39\ kN/m^3$.

3.3.7 Atterberg Limits

In a related study, McVay and Nugyen (2004) reported that the Atterberg Limits are difficult to obtain on peat and organic-rich soils of Florida. This is due to the amount of fibrous material contained in the soil matrix. They state that liquid and plastic limits can be determined on highly decomposed and amorphous-granular peat, but the use of this information is questionable. In general, the benefit of obtaining Atterberg Limits on organic-rich soil is neither beneficial nor recommended. Despite these recommendations, liquid limit (LL) and plastic limit (PL) tests were performed on 22 specimens, one from each depth at each site. However, these values should be interpreted / used with lot of caution due to inherent uncertainties in obtaining repeatable results. Only the mean values are reported in Table 3.7.

Table 3.7: Results of Atterberg Limit Tests

Soil Classification	Liquid Limit			Plastic Limit			Plasticity Index
	Min	Max	Mean	Min	Max	Mean	Mean
Muck	182	274	228	92	171	123	104
Peat	334	632	492	217	489	372	120

3.3.8 <u>Undrained Shear Strength</u>

Estimate of undrained shear strength of the undisturbed samples were obtained through unconfined compression tests. The undrained shear strength of the top stratum, classified as silty muck, ranged from *17 kPa to 23 kPa* with a mean undrained shear strength of *20 kPa*. For the peat stratum, this range was from*29 kPa to 40 kPa* with a mean undrained shear strength of *33 kPa*. The presence of fibers contributes to the increase in shear strength of the peat layers.

3.3.9 Vane Shear

A pocket Torvane shear tester, Hogentogler S4951, was employed to estimate the shear strength of undisturbed Shelby tube specimens. The Torvane shear tester is equipped with a vane-disk with blades on one side, which are pressed into the soil with a constant pressure while turning the spring-loaded knob. After the test is completed, the index mark on the knob indicates the maximum shear value. Each specimen was tested using the above procedure three times with the mean of the three values recorded as the in-situ shear strength. Figure 3.8 shows the relationship between undrained shear strength determined by unconfined compression tests relative to the torvane shear tests.

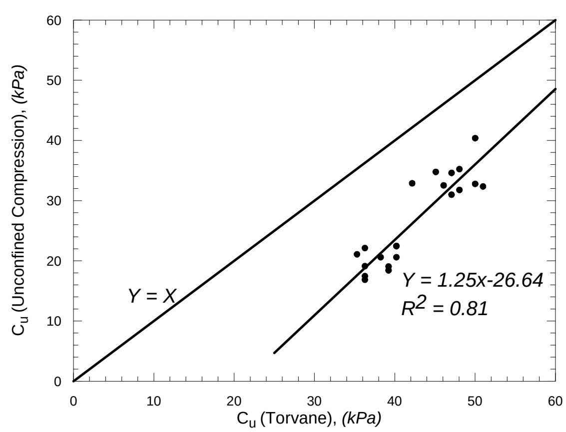


Figure 3.8: Comparison of Shear Strength: Unconfined Compression vs. Torvane Tests

3.4 DISCUSSIONS AND SUMMARY OF FINDINGS

The previous GEOSOL, Inc. data were analyzed, and a site-specific parameter called the *Organic Factor,* F_{org} was defined in terms of moisture content, organic content, unit weight, and the height of the organic layer. This parameter was used as a basis for the selection of exact locations where Piezocone Penetration Tests (CPTu) and Shelby tube samplings would be conducted. Later in the study, the *Organic Factor* (F_{org}), which represents the site-specific condition, was correlated with in-situ materials properties, and mechanistic response parameters, which in turn can be linked to probable pavement performance if suitable transfer functions are available.

Through experimentation and visual inspection it is evident that SR 15 / US 98 in northwest Palm Beach County suffers from soft subgrade conditions. The laboratory investigation into the characterization of the subgrade finds the existence of two distinct layers of organic-rich soil. The top organic stratum, classified as silty muck, typically occurs between *1.52 to 2.74* meters below grade. It displays relatively poor characteristics associated with void ratio (ranging from *3.19 to 5.58*), moisture content (ranging from *162% to 328%*), organic content (ranging from *21% to 60%*) and shear strength via unconfined compression tests (ranging from *17 kPa to 23 kPa*). The bottom organic stratum, classified as fibrous peat, typically occurs between *2.74 to 4.57* meters below grade. The fibrous peat stratum suffers from extreme values of void ratio (ranging from *5.67 to 13.93*), moisture content (ranging from *302% to 652%*) and organic content (ranging from *61% to 92%*), all in excess of the above layer. The only material property that the fibrous peat stratum exceeds the silty muck stratum in is shear strength (ranging from *31 kPa to 40 kPa*).

CHAPTER 4

Field Exploration Program

4.1 INTRODUCTION

Field exploration program included Piezocone Penetration Tests (CPTu), Porewater Dissipation Experiments, and Shelby tube sampling for subsequent laboratory experiments. The methodology of the field testing program is described in the following sections.

4.2 TEST LOCATION LAYOUT

Originally the scope included twelve CPTu test sites at about 200 to 600-foot intervals along two separate areas of the roadway which showed some to the most distress. All CPTu locations were previously marked in the field on August 31, 2005 by FDOT personnel and are situated in the center of the northbound travel lane. The locations were labeled from CPTu-1 to CPTu-12. Station numbers were previously set for the alignment and ranged from approximately 1+00 near the Palm Beach Canal Bridge to about 360+00 at the Palm Beach/Martin County Line. Maintenance of traffic (MOT) was performed by DBI, Inc. at the time of the field explorations (September 19 through 22, 2005).

4.3 CPTu TEST PROCEDURE

All CPTu tests were conducted in accordance with the Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils (ASTM

D 5778 – 95 [Reapproved 2000]). The equipment used for the CPTu testing consisted of Hogentogler hardware and software enclosed within a 20-ton heavy-duty Hogentogler rig. Data was collected in the piezocone and transmitted through the cables within the rods to the onboard computer. The piezocone was a 10-ton digital 4-channel subtraction cone. The rods were in 3.28-foot sections. Computer software consisted of the program CPT Sound 1.6 operating within the Windows 98 operating system. The following is a summary of the procedures for each test location.

1) Auger through the asphalt, base course and subgrade at the planned sounding location using the Mobile B-31 drilling rig and 6-inch diameter hollow stem augers.
2) Line up the Hogentogler drill rig over pre-augered borehole using a plumb bob attached to a string and hanging through the opening in the floor of the drill rig.
3) Prepare the piezocone by filling it with glycerin. Saturate the filters.
4) Attach the prepared piezocone to the initial rod section.
5) Place a sleeve over the piezocone to protect it during the initial stage of the sounding.
6) Lower the piezocone and rod through the floor opening and using the hydraulic ramming system to clamp to the rod.
7) Level the tip to be even with the top of the pavement surface.
8) Lower the piezocone at a rate of 0.78 inches per second and start recording data.
9) Stop collecting data at every 3.28 feet to attach an additional rod. Once the rod is attached, continue recording data.

10) Once a depth of 11.5 feet is reached, stop the downward movement of the piezocone and start recording data for the pore pressure dissipation test (may also, in addition, move up slightly to release the pressure).
11) Upon completion of the dissipation test, repeat Steps 8 and 9 until significant resistance is met in the cone to end the CPTu sounding.
12) Remove the piezocone and rods from the borehole and thoroughly clean the equipment.

Figures 4.1 shows photographs of the equipment and procedures used for the CPTu soundings and dissipation tests at CPTu-1 location. Concurrently at each CPTu site, about 40 to 50 feet south in a separate borehole(s), two Shelby tube samples were obtained in accordance with ASTM D 1587 methodology using a Central Mine Equipment Model 75 (CME 75) drill rig equipped with a hydraulically operated piston sampler (Acker Gregory Undisturbed Sampler). The Shelby tubes were sealed using melted wax and transported to the laboratory for subsequent laboratory testing. Upon completion of the tests, the groundwater level was measured in the borehole. The boreholes were then backfilled with the soil cuttings and surfaced with asphalt cold patch.


CME 75 drilling rig used for Shelby tube sampling

Mobile B-31 drilling rig used for the first 5-feet preliminary boring before CPT

CPTu-1 location – preliminary auger boring shown in background


Preliminary auger boring at CPTu-1 location


Augering first 5 feet for Shelby tube


Hogentogler CPT device inside the rig

Figure 4.1: Various Phases of the Field Exploration at CPTu-1 Location

CPTu in progress at Site 1 location

Attaching piezocone to rod

Removing silty muck from auger

Attaching Shelby tube to sampler

Pushing the Shelby tube for sampling

Backfilling the borehole / asphalt patching

Figure 4.1: (Continued..)

4.4 SITE-SPECIFIC TEST DESCRIPTIONS

4.4.1 Site 1

CPTu-1 is approximately located at Station 155+00 of the roadway alignment. Figure 4.2 shows the distressed condition at that Site in September 2005.

Figure 4.2: Cracking and Raveling of Roadway Shown Near CPTu-1 Location

Prior to conducting the CPTu-1 sounding, the upper 2 feet of the borehole was augered using the Mobile rig. For CPTu-1, CPTu sounding data was collected from depths of 0 to 11.5 feet below the pavement surface. At the depth of 11.5 feet, the dissipation test was conducted for about 22 minutes. Following completion of the dissipation test, the CPTu sounding continued until practical refusal was met at a depth of 16.4 feet. A steel rod was used to measure the asphalt and base course thicknesses of 14 and 16 inches, respectively. Using a water level indicator, the groundwater table was measured at a depth of 7.1 feet. The borehole was backfilled with the soil cuttings and surfaced with asphalt cold patch.

4.4.2 Site 2

CPTu-2 is approximately located at Station 157+00 of the roadway alignment. Figure 4.3 shows the distressed condition at CPTu-2 location in September 2005.

Figure 4.3: Cracking and Raveling of Roadway Shown Near CPTu-2 Location

Prior to conducting the CPTu-2 sounding, the upper 5 feet of the borehole was augered using the Mobile rig. For CPTu-2, CPTu sounding data was collected from depths of 0 to 11.5 feet below the pavement surface. At the depth of 11.5 feet, the dissipation test was conducted for about 22 minutes. Following completion of the dissipation test, the CPTu sounding continued until practical refusal was met at a depth of 20.2 feet. The borehole was backfilled with the soil cuttings and surfaced with asphalt cold patch.

4.4.3 Site 3

CPTu-3 is approximately located at Station 159+00 of the roadway alignment. Figure 4.4 shows the distressed condition at Site 3 as taken in September 2005.

Figure 4.4: Cracking and Raveling of Roadway Shown Near CPTu-3 Location

Prior to conducting the CPTu-3 sounding, the upper 5 feet of the borehole was augered using the Mobile rig. For CPTu-3, CPTu sounding data was collected from depths of 0 to 11.5 feet below the pavement surface. At the depth of 11.5 feet, the dissipation test was conducted for about 33 minutes. Following completion of the dissipation test, the CPTu sounding continued until practical refusal was met at a depth of 20.5 feet. The borehole was backfilled with the soil cuttings and surfaced with asphalt cold patch.

4.4.4 Site 4

CPTu-4 is approximately located at Station 163+00 of the roadway alignment. Figure 4.5 shows the distressed conditions of Site 4 in September 2005.

Figure 4.5: Cracking and Raveling of Roadway Shown Near CPTu-4 Location

Prior to conducting the CPTu-4 sounding, the upper 5 feet of the borehole was augered using the Mobile rig. For CPTu-4, CPTu sounding data was collected from depths of 0 to 11.5 feet below the pavement surface. At the depth of 11.5 feet, the dissipation test was conducted for about 30 minutes. Following completion of the dissipation test, the CPTu sounding continued until practical refusal was met at a depth of 19.2 feet. A steel rod was used to measure the asphalt and base course thicknesses of 12.5 and 10.5 inches, respectively. Using a water level indicator, the groundwater table was measured at a depth of 7.2 feet. The borehole was backfilled with the soil cuttings and surfaced with asphalt cold patch.

4.4.5 Site 5

CPTu-5 is approximately located at Station 171+00 of the roadway alignment. Figure 4.6 shows severe cracking near Site 5 in September 2005.

Figure 4.6: Class III Cracking Near Site 5

Prior to conducting the CPTu-5 sounding, the upper 5 feet of the borehole was augered using the Mobile rig. For CPTu-5, sounding data was collected from depths of 0 to 11.5 feet below the pavement surface. At the depth of 11.5 feet, the dissipation test was conducted for about 80 minutes. Following completion of the dissipation test, the CPTu sounding continued until practical refusal was met at a depth of 15.7 feet. Using a water level indicator, the groundwater table was measured at a depth of 7.4 feet. The borehole was backfilled with the soil cuttings and surfaced with asphalt cold patch.

4.4.6 Site 6

CPTu-6 is approximately located at Station 227+00 of the roadway alignment. Figure 4.7 shows the distressed condition at CPTu-6 in September 2005.

Figure 4.7: Cracking of Roadway Shown near CPTu-6 Location

Prior to conducting the CPTu-6 sounding, the upper 5 feet of the borehole was augered using the Mobile rig. For CPTu-6, CPT sounding data was collected from depths of 0 to 11.5 feet below the pavement surface. At the depth of 11.5 feet, the dissipation test was conducted for about 32 minutes. Following completion of the dissipation test, the CPTu sounding continued until practical refusal was met at a depth of 18 feet. Using a water level indicator, the groundwater table was measured at a depth of 7.1 feet. The borehole was backfilled with the soil cuttings and surfaced with asphalt cold patch.

4.4.7 Site 7

CPTu-7 is approximately located at Station 233+00 of the roadway alignment. Figure 4.8 shows the distressed condition at CPTu-7 in September 2005.

Figure 4.8: Cracking and Raveling of Roadway Shown Near CPTu-7 Location

Prior to conducting the CPTu-7 sounding, the upper 5 feet of the borehole was augered using the Mobile rig. For CPTu-7, CPTu sounding data was collected from depths of 0 to 11.5 feet below the pavement surface. At the depth of 11.5 feet, the dissipation test was conducted for about 45 minutes. Following completion of the dissipation test, the CPTu sounding continued until practical refusal was met at a depth of 16.6 feet. A steel rod was used to measure the asphalt and base course thicknesses of 10 and 8 inches, respectively. Using a water level indicator, the groundwater table was measured at a depth of 5.2 feet. The borehole was backfilled with the soil cuttings and surfaced with asphalt cold patch.

4.4.8 Site 8

CPTu-8 is approximately located at Station 239+00 of the roadway alignment. Figure 4.9 shows the distressed condition of the roadway in September 2005.

Figure 4.9: Cracking and Raveling of Roadway Shown Near CPTu-8 Location

Prior to conducting the CPTu-8 sounding, the upper 5 feet of the borehole was augered using the Mobile rig. For CPTu-8, CPTu sounding data was collected from depths of 0 to 11.5 feet below the pavement surface. At the depth of 11.5 feet, the dissipation test was conducted for about 13 minutes. Following completion of the dissipation test, the CPTu sounding continued until practical refusal was met at a depth of 17.6 feet. A steel rod was used to measure the asphalt and base course thicknesses of 11 and 8.5 inches, respectively. Using a water level indicator, the groundwater table was measured at a depth of 5.9 feet. The borehole was backfilled with the soil cuttings and surfaced with asphalt cold patch.

4.4.9 Site 9

CPTu-9 is approximately located at Station 243+00 of the roadway alignment. The CPTu sounding and Shelby tube sampling were not conducted at this location.

4.4.10 Site 10

CPTu-10 is approximately located at Station 249+00 of the roadway alignment. Figure 4.10 shows the distressed condition of the roadway in September 2005.

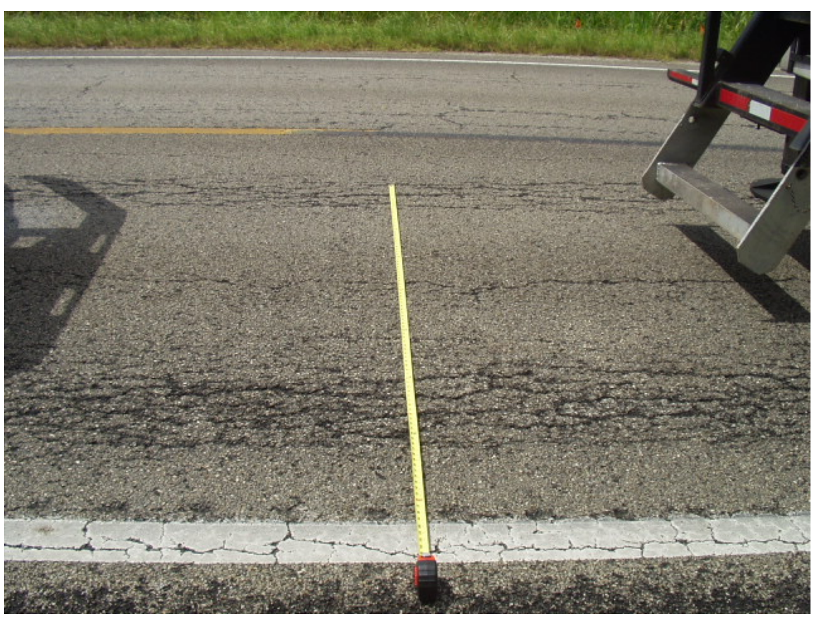


Figure 4.10: Cracking and Raveling of Roadway Shown Near CPTu-10 Location

Prior to conducting the CPTu-10 sounding, the upper 5 feet of the borehole was augered using the Mobile rig. For CPTu-10, CPTu sounding data was collected from depths of 0 to 11.5 feet below the pavement surface. At the depth of 11.5 feet, the dissipation test was conducted for about 60 minutes. Following completion of the dissipation test, the CPTu sounding continued until practical refusal was met at a depth of 17.6 feet. A steel rod was used to measure the asphalt and base course thicknesses of 12 and 11 inches, respectively. Using a water level indicator, the groundwater table was measured at a depth of 5.6 feet. The borehole was backfilled with the soil cuttings and surfaced with asphalt cold patch.

4.4.11 Site 11

CPTu-11is approximately located at Station 253+00 of the roadway alignment. Figure 4.11 shows the distressed condition of the roadway in September 2005. The CPTu sounding was not conducted at this location. Only Shelby tube samples were retrieved.

Figure 4.11: Cracking and Raveling of Roadway Shown Near Site 11 Location

4.4.12 Site 12

CPTu-12 is approximately located at Station 259+00 of the roadway alignment. Figure 4.12 shows the distressed roadway in September 2005. The CPTu sounding was not conducted at this location. Only Shelby tube samples were taken.

Figure 4.12: Cracking and Raveling of Roadway Shown Near Site 12 Location

CHAPTER 5

Laboratory 1-D Compression Testing Program

5.1 INTRODUCTION

Soils containing substantial amounts of organic material display more compressibility than soils comprised primarily of mineral constituents. This is due to their high values of moisture content and organic content, which in turn leads to high in-situ void ratios. A high void ratio constitutes a soil's "looseness," in that the soil is more porous in structure. This leads to organic soils having a high rate of permeability. Soils exhibiting high rates of permeability lead to a high flow rate of moisture being expelled from the soil matrix. When subjected to compressive forces, soils with high permeability display a significant amount of deformation in relatively short periods of time. Organic soils are generally close in proximity to the surface, which leads to relatively low pre-consolidation pressures. Thus, due to an organic soil's high permeability and low pre-consolidation pressure it suffers from excessive amounts or primary settlement.

After nearly all the moisture in the pores has dissipated from the soil matrix it is said to undergo secondary compression. Secondary compression is time-dependent and occurs at constant effective stress. Secondary compression is a continuation of the volume change that began during primary settlement and constitutes a major part of total settlement in organic soils. Secondary compression is believed to occur as a result of the soil particles being rearranged and reoriented while the organic fibers are being reduced in size. Much is still unknown about the mechanism that generates secondary compression. The extent of this investigation is to quantify and characterize secondary compression in the context of the long-term deformation and settlement of the pavement foundations leading to premature distresses and failures. Accordingly, a comprehensive laboratory testing protocol was developed and followed in the current study for evaluating the primary and secondary compression behavior of Florida organic soils and peat.

5.2 ONE-DIMENSIONAL OEDOMETER TESTS

To simulate the effects of one-dimensional in-situ compressive stress an Oedometer or Consolidometer is used. In the case of this investigation, a fixed-ring Oedometer was employed. It consists of a rigid ring to mitigate lateral deformations. The specimen under investigation is contained within the fixed ring and surrounded on top and bottom by porous stones. The porous stones enable the moisture contained within the specimen to drain as vertical compressive loads are applied. A series of tabletop consolidation apparatus, ELE EI25-0429, with a 10 to 1 lever arm was used to apply loads to the specimen in the Oedometer, as shown in Figure 5.1.

One-dimensional loading produces a vertical deformation to the specimen contained in the fixed-ring. The vertical deformation results in a volumetric change in the specimen. Prior to applying any load, the specimen is fully saturated, thus ensuring that all voids are filled with water. Once a vertical compressive load is applied, the water contained within

Figure 5.1: Oedometers and Tabletop Consolidation Devices

the pores instantaneously carries the burden of the stress. The result is an increase in the excess pore-water pressure, which is the summation of the initial hydrostatic pressure and the stress due to the applied load. This state of equilibrium only occurs for a fraction of a second and subsequently produces a flow of water out of the pores of the specimen into the surrounding Oedometer. As the excess pore-water dissipates, it enables the skeleton to support more of the burden from the applied load (total stress transitioning into effective stress). This is the process of consolidation and it is time dependent.

Unlike mineral soils, organic-rich soils dissipate excess pore-water pressure in relatively short periods of time. Organic soils generally contain high amounts of moisture and organics, which generate high void ratios. As void ratio increases, holding all else constant, permeability increases, thus causing pore-water to dissipate at relatively high rates. The quicker pore-water dissipates the larger the magnitude of primary settlement. In the case of organic-rich soils, they suffer from substantial amounts of primary settlement.

Secondary settlement begins after primary settlement and coincides with the dissipation of nearly all-excess pore-water. Secondary settlement is time-dependent and is linear when plotted versus the logarithm of time. It occurs under constant effective stress and is generally considered a creep phenomenon. As previously stated, the mechanism initiating secondary settlement is not fully understood. But in the case of organic-rich soils, secondary settlement takes place due to high initial void ratios and the relative weakness of the soil skeleton, i.e., peat grains, plates and fibers. Nearly 50 percent of the overall settlement in organic-rich soils can be attributed to secondary settlement (McVay and Nugyen, 2004).

5.3 LABORATORY TESTING PROCEDURE AND METHODOLOGY

The undisturbed samples obtained along SR 15 / US 98 in northwest Palm Beach County were contained in 7.62 cm (3.0 inch) diameter Shelby tubes. To prepare each specimen for Oedometer tests, a 5.08 cm (2.0 inch) section was cut from the bottom of each Shelby tube. The Shelby tubes were sawed perpendicular to their long axis using a fine-toothed hacksaw accompanied with a modified miterbox. To minimize vibration during sawing, the Shelby tubes were securely anchored using a set of 7.62 cm (3.0 inch) u-bolts, as depicted by Figure 5.2.

Prior to placing the specimen in the consolidation ring, the height, weight and diameter of the ring was measured and recorded. After a 5.08 cm (2.0 inch) section was obtained, the specimen was extruded using a rigid piston with a diameter the same size as the inside diameter of the Shelby tube. Once the specimen was extruded, it was promptly placed

Figure 5.2: Shelby Tube Cutting Apparatus

into a consolidation ring lubed with WD-40 and trimmed using a wire saw. The weight of the consolidation ring and specimen was weighed and recorded. The trimmings were obtained and used to determine the moisture and organic content of the specimen. The consolidation ring containing the specimen along with two porous stones and loading head were assembled in the Oedometer and placed on the tabletop consolidation apparatus. The Oedometer was filled with distilled water and remained in a static state for 30 minutes. At which time the tabletop consolidation apparatus was calibrated and adjusted to accommodate the Oedometer. Loads were then applied using a loading scheme detailed later in this chapter.

Computation of the initial void ratio is dependent upon moisture content, weight of the solids and specific gravity. Void ratio is defined as the volume of voids relative to the volume of solids. In the case of one-dimensional Oedometer tests, void ratio can be simplified as the height of voids relative to the height of solids. This holds true because the cross-sectional area remains constant. Equation 5.1 defines the height of the solids.

$$H_s = \frac{W_s}{\left(\frac{\pi}{4}D^2\right)G_s\rho_w} \tag{5.1}$$

Where:

W_s = Dry mass of soil specimen,

D = Diameter of the specimen,

G_s = Specific gravity of soil solids, and

ρ_w = Density of water.

To compute the height of the void, the height of the soil solids is subtracted from the total height of the specimen. The void ratio was determined by evaluating the ratio of void height to the soil solid's height. Throughout the consolidation process the height of the soil solids remains constant while the height of the specimen and void ratio decrease.

5.3.1 General Scheme

Two Shelby tube samples were collected from each of the 11 sites. Initial analysis of the samples collected at typical depths of *2.13 to 2.74* meters and *3.66 to 4.27* meters revealed that the first meter of the organic layer contains dark brown to black organic sandy silt resembling muck *(organic content < 75%)* with moisture contents ranging from *160% to 330%* with initial void ratios ranging from *3.19 to 5.58*, and the underlying *1.20 to 4.50* meters contain predominantly medium brown, fibrous organic soils resembling peat *(organic content > 75%)* with significantly higher moisture contents ranging from *300% to 650%* and initial void ratios ranging from *5.67 to 13.93*. Although the organic silty layer is relatively thin, its proximity to the pavement structure makes it a critical layer that contributes to the overall permanent deformation of the structure. Therefore, the laboratory tests were conducted on samples of both the organic silt, classified as muck, and the fibrous material, classified as fibrous peat, for each site (total of 22 soil types). Moreover, for each soil type, the following two series of tests were performed, bringing the total to 43 consolidation tests according to the following experimental schemes sites (SITE 5 – 2.13 to 2.74 meters was disregarded due to insufficient sample size).

5.3.1.1 Series I Test

This series includes standard consolidation tests in which the specimen is incrementally loaded past the estimated in-situ effective overburden pressure, and the effective pre-consolidation pressure, σ_p'.

5.3.1.2 Series II Test

In this series of tests, the specimen is incrementally loaded to the desired pressure, σ_v', and allowed to undergo secondary compression at constant stress for *2 - 4* weeks. For 50 percent of the sites, the constant stress was equal to the estimated in-situ pressure due to the dead weight of the overlying pavement layers. This corresponded to an applied stress level of *0.30 – 0.60*, defined by the ratio of applied pressure to the pre-consolidation pressure (σ_v'/σ_p'). Therefore, these specimens were in the recompression range, while the remaining sites were subjected to a constant stress level σ_v'/σ_p' of *1.0 – 1.15*, implying a stress state corresponding to the normally consolidated range.

CHAPTER 6

Results of Laboratory and Field Investigations

6.1 OBJECTIVE AND SIGNIFICANCE

It is assumed that the premature distresses in the form of cracking, rutting and distortional settlement along SR 15 / US 98 roadway in Palm Beach County is primarily caused by the presence of soft highly compressible organic soils and peat at relatively shallow depth. Since complete removal of the organic layer is expensive and impractical, any design strategy aiming at mitigating such distress levels requires a thorough understanding of the strength and compression behavior of the organic soils. Moreover, it is well-known that routine sampling and transportation of "undisturbed" specimens of soft organic soils is rather difficult, and subsequent laboratory testing especially the evaluation of secondary compression behavior is expensive and time-consuming. Therefore, there is a need for developing reliable and efficient tools for on-site estimation of soft soils properties, which can also be used for forensic interpretations of pavement failures, mechanistic analysis, and validation of pavement performance models. The current study evaluated the use of Piezocone Penetration Tests (CPTu) as a versatile tool for subsurface investigations relevant to pavement geotechnics. Accordingly, eleven different severely distressed locations were chosen along the alignment of SR 15 / US 98 for conducting the Piezocone Penetration Tests, and collection of "undisturbed" Shelby tube samples at various depths for subsequent laboratory testings. Specific objectives were as follows: (i) To estimate the in-situ undrained shear strength, undrained elastic modulus and Compression Index (C_c) from CPTu data; (ii) To predict the in-situ Coefficient of Consolidation (C_v) from Porewater Dissipation data using several theoretical interpretation models; (iii) To conduct laboratory consolidation tests on undisturbed soils for evaluating the primary and secondary compression behavior under simulated pavement overburden pressure; (iv) To validate CPTu predicted compression properties by comparing / correlating with the laboratory determined properties; and (v) To establish the unique C_α / C_c relationships for Florida organic soils and peat, so that the short-term CPTu data can be used to predict the Secondary Compression Index for the calculation of long-term settlement in an effort to avoid expensive and time-consuming laboratory tests in the future when dealing with similar subsurface conditions. This Chapter presents the results of the laboratory and field investigations, describes the analysis techniques and strategies, and summarizes the important findings in the context of the present study.

6.2 RESULTS OF 1-D COMPRESSION TEST PROGRAM

The results are discussed under the following three categories: (i) Primary Consolidation Behavior; (ii) Secondary Compression Behavior; and (iii) Time-Stress-Compressibility Relationships *(C_α / C_c law of compressibility concept)* applied to Florida organic soils.

6.2.1 Primary Consolidation Behavior

Standard consolidation tests were conducted on two Shelby tube samples from 11 sites (SITE 5 – 2.13 to 2.74 meters was disregarded due to insufficient sample size) to determine the Compression Index, C_c, the Pre-consolidation pressure, σ_p', the Coefficient of Consolidation, c_v, and to gain an understanding of the time required to reach the End-of-Primary (EOP) consolidation, thus depicting the onset of secondary compression. Typical compressive behaviors for all tested specimens are shown in Figure 6.1, and the calculated Compression Indices, C_c, are plotted in Figure 6.2, which is a compilation of data from the literature on the variation of C_c with natural water content (Mesri et al., 1997). It is found that the C_c values for Florida organic soils fall within the acceptable ranges of similar soils.

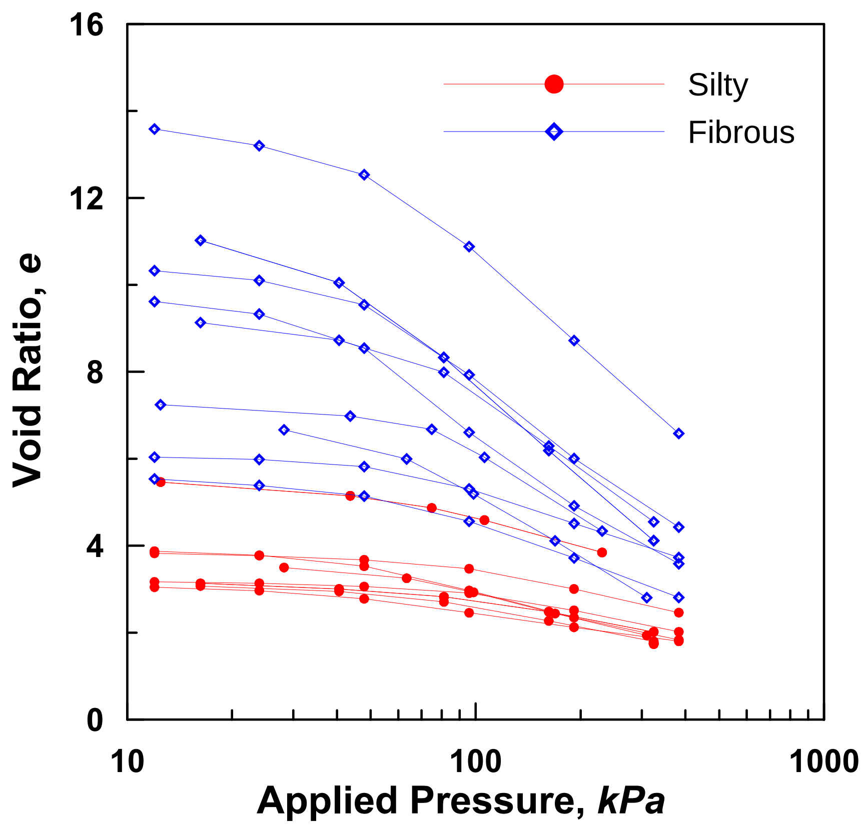


Figure 6.1: Typical Consolidation Behavior of Florida Organic Soils (1 KPa = 0.145 psi)

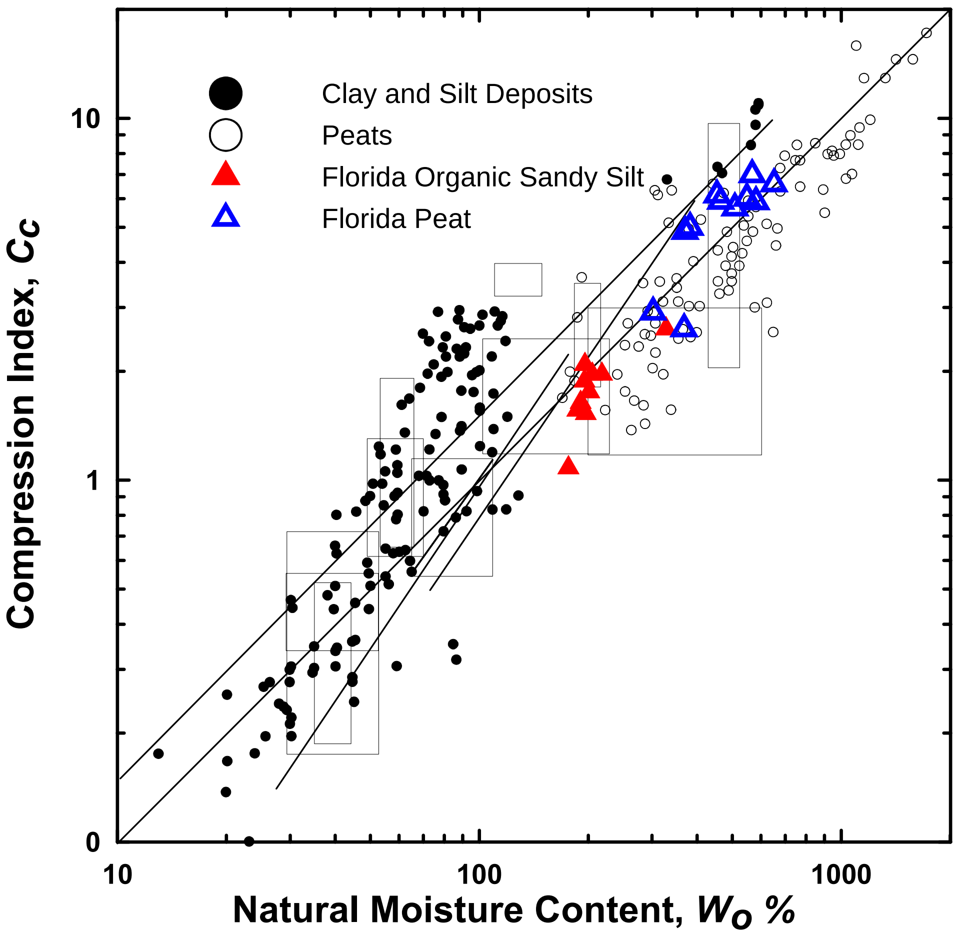


Figure 6.2: Compression Indices of Florida Soils Compared to Mesri et al., (1997) Data

The pre-consolidation pressure, σ_p', was found to vary within the range of *73-83 kPa*. In order to estimate the time at which the End-of-Primary (EOP) consolidation occurred, the standard Taylor's square root of time plots (*dial reading vs. time*$^{0.5}$) were constructed to calculate t_{90}, the time at which 90% of primary consolidation is completed. Typical plots illustrating the compression behavior with respect to time for SITE 4 ($\sigma_v'/\sigma_p' = 0.60$) and SITE 10 ($\sigma_v'/\sigma_p' = 1.0 - 1.15$) are shown in Figures 6.3 and 6.4, respectively (Appendix A depicts the compressive behavior of all investigated sites). All calculated values of time to EOP consolidation were found to be approximately 1 minute for Florida subgrades. Similar values were reported in the literature for fibrous peat and organic soils under similar stress levels (Mesri et al., 1997). In addition, the dial reading versus square root of time curves were used to determine the Coefficient of Consolidation, c_v. This rate parameter is useful in estimating the time required for completion of primary consolidation settlement in the field after the construction of the pavement structure. The results of the consolidation and secondary compression tests completed are presented in Table 6.1, and discussed in the subsequent sections. More details are available elsewhere (Riedy, 2006).

SITE 4 – STA 163 + 00
SECONDARY COMPRESSION

2.13 – 2.74 meters - Silty Muck
$\sigma'_v/\sigma'_p = 0.60$

3.66 – 4.27 meters - Fibrous Peat
$\sigma'_v/\sigma'_p = 0.60$

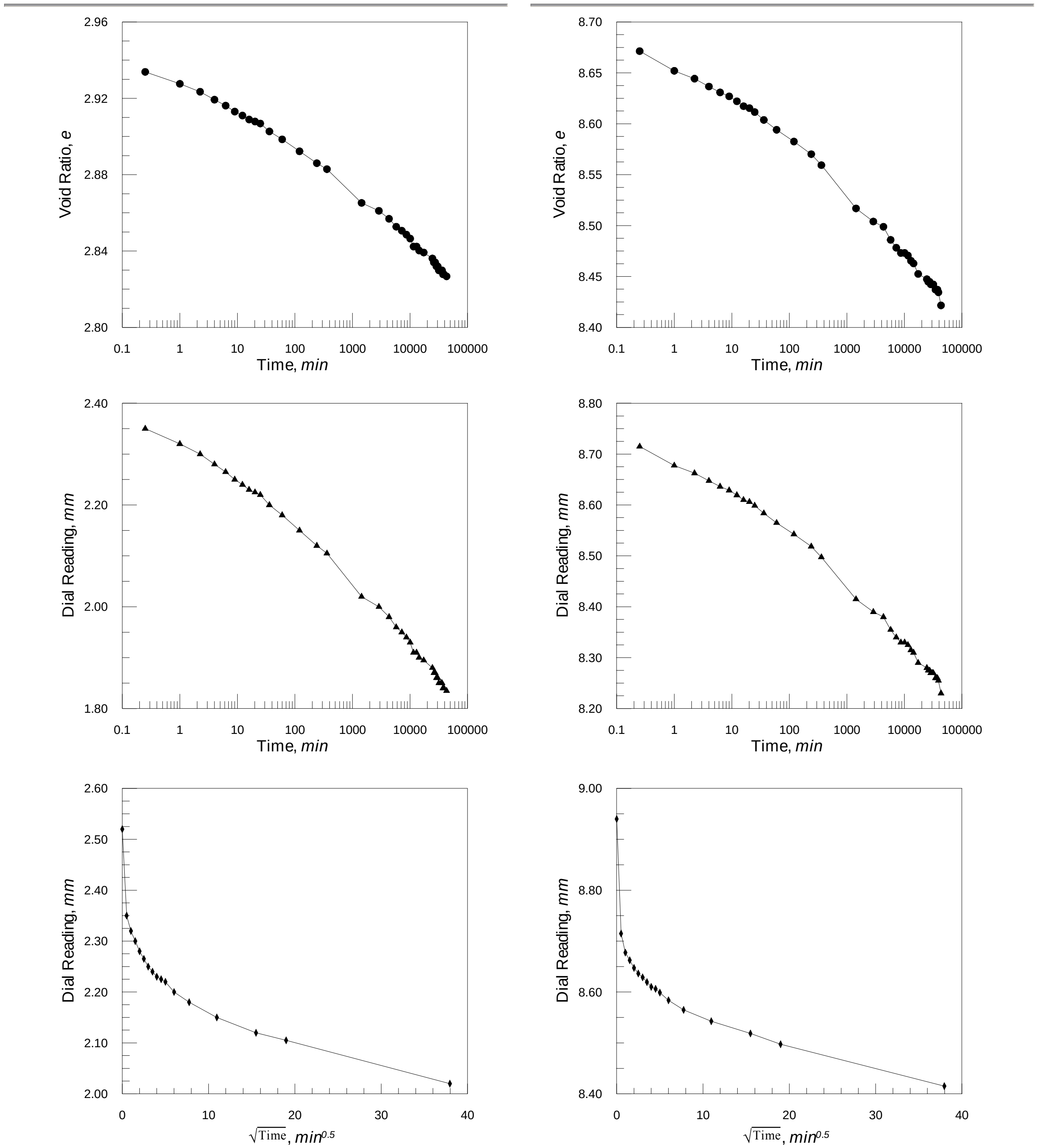


Figure 6.3: Compression Behavior of Florida Organic Soils with Respect to Time at SITE 4

SITE 10 – STA 253 + 00
SECONDARY COMPRESSION

2.13 – 2.74 meters - Silty Muck — ***σ'_v/σ'_p = 1.00***

3.66 – 4.27 meters - Fibrous Peat — ***σ'_v/σ'_p = 1.15***

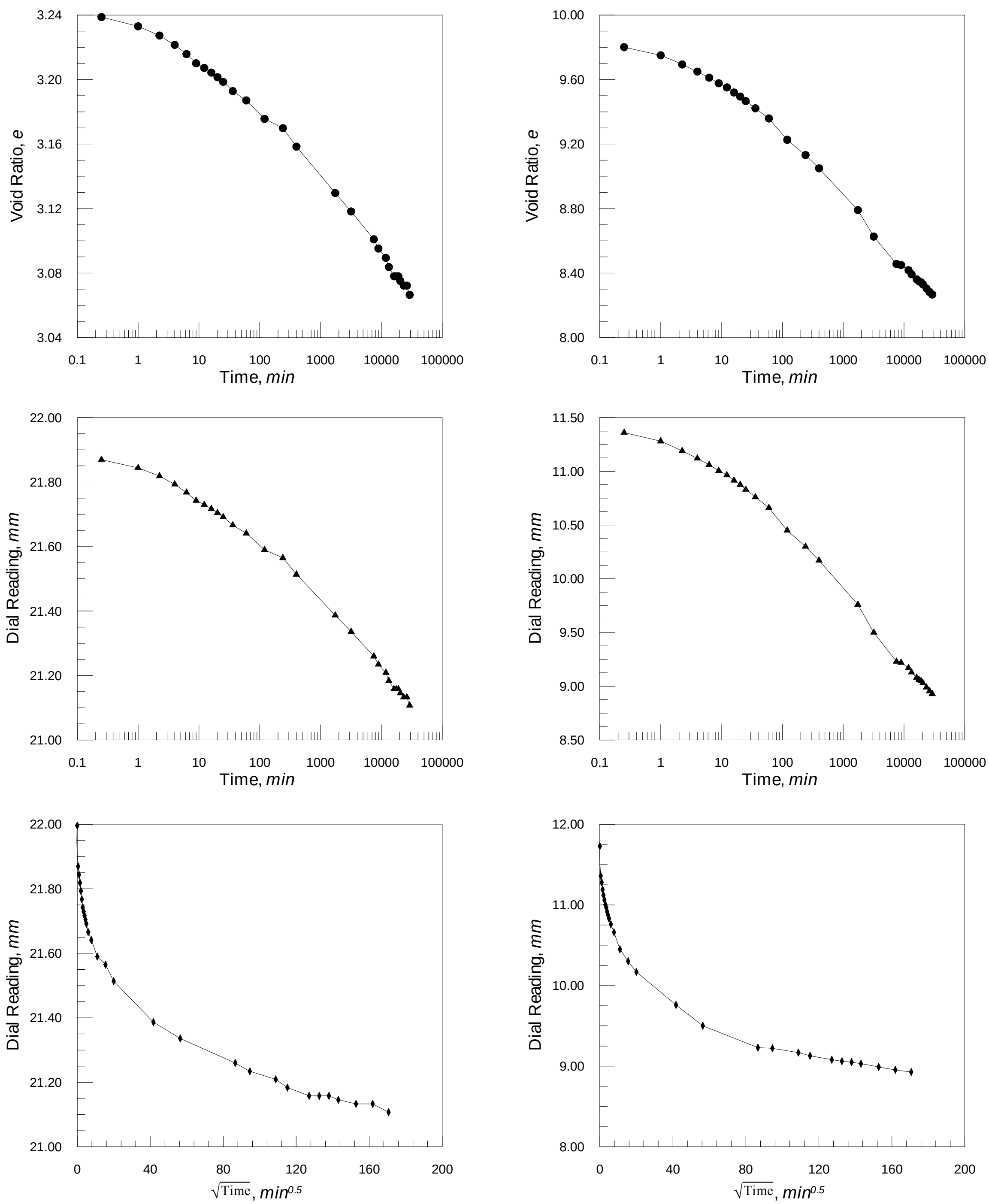


Figure 6.4: Compression Behavior of Florida Organic Soils with Respect to Time at SITE 10

Table 6.1: Results of Consolidation and Secondary Compression Tests

Site	Station	Shelby Tube Depth (m)	Soil Description (A-8)	Height of Stratum (m)	Unit Weight (kN/m^3)	w(%)	OC(%)	e_0	S_i(%)	σ'_v/σ'_p	C_c	C'_c	C_α	C_α/C'_c	c_v (cm^2/s)
1	155 + 00	1.83 - 2.44	Organic Silt	4.42	11.1	205.1	60.2	3.91	95	0.60	1.96	0.28	0.013	0.046	0.040
	155 + 00	2.44 - 3.05	Peat		10.6	367.9	61.0	6.13	91	0.30	2.62	0.36	0.004	0.011	0.018
2	157 + 00	1.83 - 2.44	Organic Silt	4.12	11.1	327.9	31.8	5.58	100	0.30	2.62	0.81	0.062	0.077	0.017
	157 + 00	2.44 - 3.05	Peat		10.1	382.2	76.0	7.44	91	0.60	5.00	0.93	0.032	0.034	0.030
3	159 + 00	1.83 - 2.44	Organic Silt	3.35	11.5	176.3	39.3	3.19	94	0.60	1.08	0.68	0.030	0.045	0.023
	159 + 00	2.44 - 3.05	Peat		10.4	302.2	61.4	5.67	90	0.60	2.91	0.77	0.023	0.029	0.023
4	163 + 00	2.13 - 2.74	Organic Silt	3.20	11.4	186.6	21.3	3.41	100	0.60	1.56	0.35	0.015	0.043	0.023
	163 + 00	3.66 - 4.27	Peat		9.8	508.3	92.3	9.82	93	0.60	5.65	0.88	0.026	0.030	0.030
5	171 + 00	2.13 - 2.74	Organic Silt	3.20	11.4	162.1	29.5	-	94	1.00	-	0.70	0.031	0.044	0.017
	171 + 00	3.66 - 4.27	Peat		10.5	582.3	92.3	10.74	88	1.15	5.88	5.16	0.143	0.028	0.023
6	227 + 00	2.13 - 2.74	Organic Silt	3.05	11.1	218.3	34.0	3.96	91	1.15	1.96	1.11	0.054	0.049	0.021
	227 + 00	3.66 - 4.27	Peat		10.2	549.1	26.6	10.62	100	1.00	6.02	4.64	0.139	0.030	0.023
7	233 + 00	2.13 - 2.74	Organic Silt	3.51	11.7	195.8	31.9	4.04	98	1.00	2.09	0.65	0.033	0.051	0.021
	233 + 00	3.66 - 4.27	Peat		10.7	370.4	90.6	7.40	92	1.00	4.87	1.76	0.063	0.036	0.026
8	239 + 00	2.13 - 2.74	Organic Silt	3.51	11.4	196.9	28.7	3.29	97	0.60	1.53	0.30	0.008	0.027	0.024
	239 + 00	3.66 - 4.27	Peat		10.6	568.0	73.6	12.20	100	0.60	6.99	1.38	0.038	0.027	0.029
9	249 + 00	2.13 - 2.74	Organic Silt	3.51	11.5	201.2	37.5	3.63	97	1.15	1.75	0.49	0.028	0.057	0.017
	249 + 00	3.66 - 4.27	Peat		10.7	464.7	84.0	9.56	100	1.00	5.90	2.87	0.108	0.038	0.018
10	253 + 00	2.13 - 2.74	Organic Silt	3.81	11.0	190.6	35.8	3.19	93	1.00	1.64	0.65	0.024	0.037	0.023
	253 + 00	3.66 - 4.27	Peat		10.3	652.2	91.5	13.93	95	1.15	6.59	7.06	0.182	0.026	0.023
11	259 + 00	2.13 - 2.74	Organic Silt	3.51	11.3	195.4	32.6	4.03	91	0.60	1.88	0.39	0.017	0.044	0.025
	259 + 00	3.66 - 4.27	Peat		11.0	453.2	84.2	9.47	84	0.60	6.14	1.65	0.057	0.035	0.021

6.2.2 Secondary Compression Behavior

It has been confirmed from the current and previous studies involving Florida organic soils and peat that the EOP consolidation is very rapid in laboratory samples. This suggests that significant settlement occurs due to secondary compression under a constant vertical effective stress, $\sigma_v^{'}$, which in actual roadways is the sustained weight of the pavement and the embankment. Since these soils are also characterized by high initial void ratios, organic contents, and natural moisture contents, there is rapid and significant primary compression as evident from high values of Compression Index, C_c.

Secondary compression tests were conducted on 22 undisturbed specimens representing all 11 sites using the incremental load ratio of one loading scheme. Each specimen was loaded to a stress level within the range of $\sigma_v^{'} / \sigma_p^{'} = (0.30 - 1.15)$ in $3 - 4$ loading intervals, with the last load remaining constant for the duration of the experiment. After the initial 24-hours of the last applied pressure, subsequent readings were then taken every 24 hours. It is observed that during the secondary phase, the variation of *e* with *log time* is approximately linear. The slope of the curve is called the Secondary Compression Index, C_α, and is defined as follows:

$$C_\alpha = \frac{\Delta e}{\log \frac{t}{t_p}} = \frac{\Delta e}{\Delta \log t} \tag{6.1}$$

where t_p is the time to End-of-Primary (EOP) consolidation, and t is any time $t > t_p$. As reported by Mesri et al. (1997), in a realistic time range of practical interest i.e. from t_p to the design life of the structure *(30 - 100 years)*, the Secondary Compression Index, C_α may remain constant with time, and may yield reasonable estimates of the secondary compression settlement. In this study, C_α was calculated during the first log cycle after the EOP consolidation (Table 6.1). This method ensured the establishment of a unique relationship between C_c and C_α while the specimen existed at a particular point on the *Void Ratio-Time-Pressure* 3-dimensional space curve.

6.2.3 Time-Stress-Compressibility Relationships (C_α / C_c Concept)

In their classic 1977 paper Mesri and Godlewski (1977) postulated that for any given soil, there is a unique relationship between $C_\alpha = \Delta e / \Delta \log t$ and $C_c = \Delta e / \Delta \log \sigma_v'$, that holds true at all combinations of time, effective stress, and void ratio as shown in Figure 6.5.

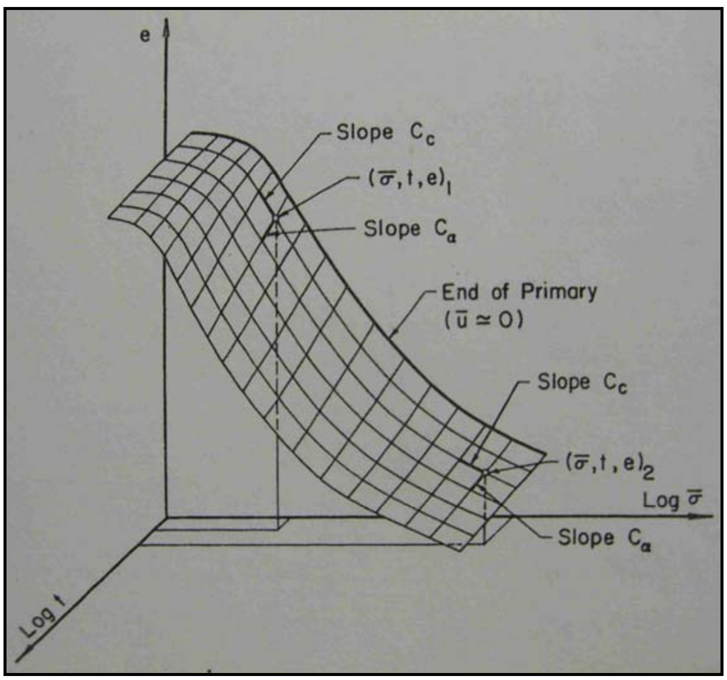


Figure 6.5: Void Ratio-Time-Pressure Relationship During Secondary Compression

At any given effective stress, the value of C_α from the first log cycle of secondary compression and the corresponding C_c value computed from the EOP *e-log* σ_v' curve are used to define the relationship between C_α and C_c. It is specifically noted that C_c denotes the slope of *e-log* σ_v' curve throughout the recompression and compression ranges.

Compilation of worldwide existing data for peat, fibrous peat, and amorphous to fibrous peat from the literature shows that the value for the C_α / C_c ratio varies within the range *0.035 – 0.10*, with most data for peat lying within the narrow range of *0.06* ± *0.01*, and for organic clays and silts within the narrow range of *0.04 – 0.06* (Mesri et al., 1997; Mesri and Godlewski, 1977).

The procedure outlined above was successfully employed for Florida organic silty muck and fibrous peat in order to develop the unique C_α / C_c relationships. As described previously, the secondary compression constant stress levels (σ_v' / σ_p') used in this study were in the range of *0.30 – 1.15*, implying both recompression and compression segments. The broad motivation for this endeavor was to develop the capabilities for determining the secondary compression index C_α, once the primary compression index C_c is estimated from laboratory or field testing such as the Piezocone Penetration Tests. Figure 6.6 shows the C_α / C_c relationships for the silty muck and the fibrous peat, and also a combined curve utilizing all available data to date. In addition, the data from the literature on peat specimens (Mesri et al., 1997) are presented for comparison purposes. It is found that the C_α / C_c ratio for the silty muck has a mean of *0.051* while the fibrous peat has a mean value of *0.028* (Figure 6.6). The combined curve incorporating all data points has a slope $C_\alpha / C_c = 0.029$ with a coefficient of determination of *0.96* (Figure 6.6). As more data become available, this relationship can be more refined. At this point, a constant value of $C_\alpha / C_c = 0.03$ is proposed for the characterization of the Florida organic soils under highway pavements along SR 15 / US 98. It is found that these values are consistent with the values reported in the literature (Table 6.2).

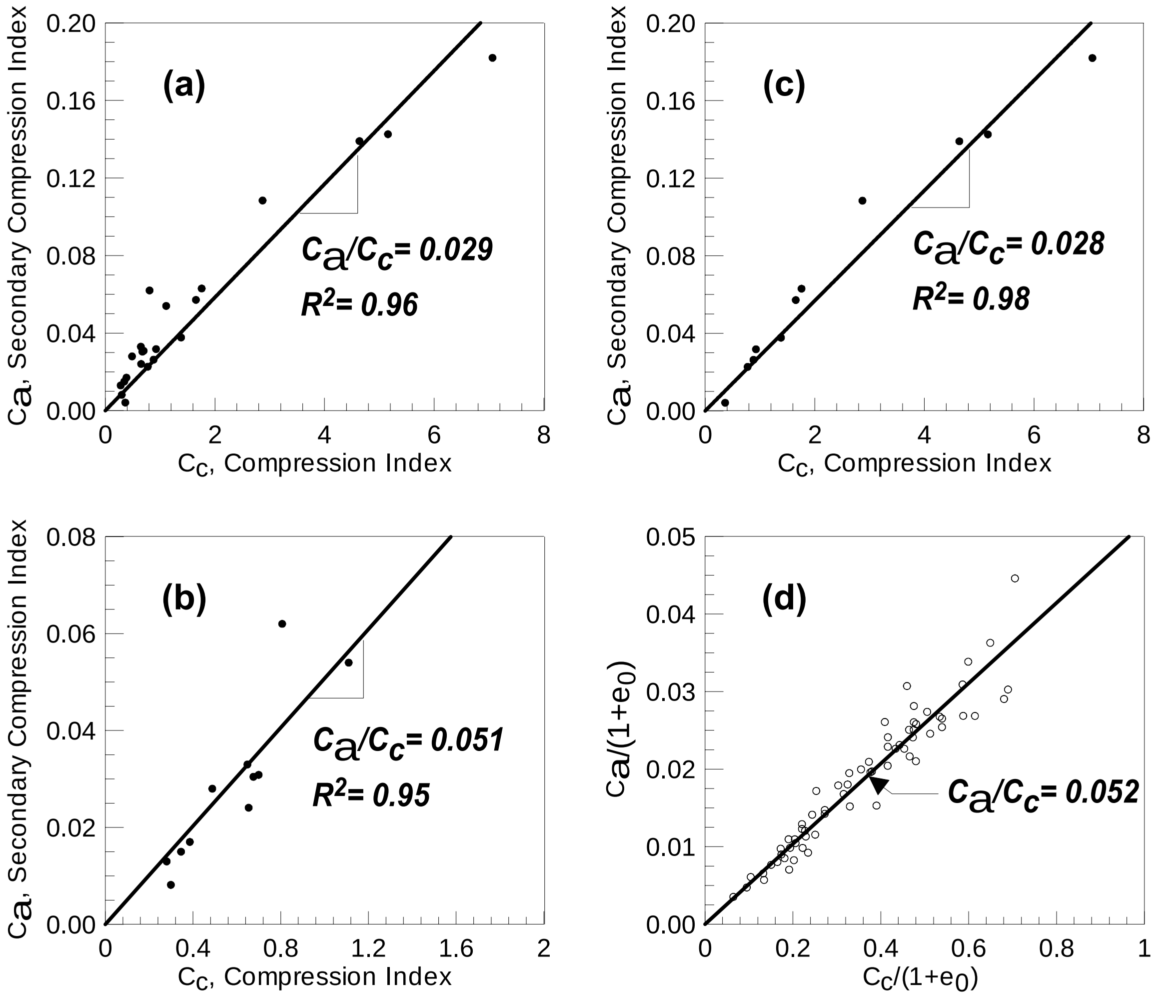


Figure 6.6: C_α / C_c Relationships – (a), (b), (c): Present Study; (d): Mesri et al., 1997

Table 6.2: Comparison of C_α / C_c Values Reported in the Literature (Mesri et al. 1997)

Organic Soil	w (%)	C_α/C_c	Reference
Fibrous peat	850	0.06 – 0.1	Hanrahan (1954)
Peat	520	0.061 – 0.078	Lewis (1956)
Amorphous and fibrous peat	500-1,500	0.035 – 0.083	Lea and Brawner (1963)
Canadian muskeg	200-600	0.09 – 0.1	Adams (1965)
Amorphous and fibrous peat	705	0.073 – 0.091	Keene and Zawodnaik (1968)
Peat	400-750	0.075 – 0.085	Weber (1969)
Fibrous peat	605-1,290	0.052 – 0.072	Samson and LaRochell (1972)
Fibrous peat	613-886	0.06 – 0.085	Berry and Vickers (1975)
Amorphous to fibrous peat	600	0.042 - 0.083	Dhowain and Edil (1980)
Fibrous peat	660-1,590	0.06	Lefebvre et al. (1984)
Dutch peat	370	0.06	Den Haan (1994)
Fibrous peat	610-850	0.052	Mesri et al. (1997)
Florida muck and peat	**160-650**	**0.028 – 0.051**	**Present study (2006)**

As mentioned earlier, for the practical design life of the structure, C_α may remain constant. One of the implications of the Void Ratio-Time-Pressure interrelationships is that the shape of the settlement curve in the secondary range can be predicted. These predictions have been reported to be consistent with laboratory and field observations (Mesri and Godlewski, 1977). Although this has huge significance for the long-term performance prediction of pavements built over muck, developing these predictive techniques is beyond the scope of the current investigations. For the time being, if C_c could be reliably estimated from a "quick" and routine in-situ test, then the secondary compression index and hence the secondary settlement can be predicted for Florida organic soils without undertaking lengthy laboratory experimental programs.

6.3 PIEZOCONE PENETRATION TEST RESULTS

6.3.1 Introduction

As mentioned previously, Piezocone Penetration Tests (CPTu) and the concurrent Porewater Dissipation Experiments were conducted for on-site estimation of strength and compressibility parameters of soft organic soils. Raw data from the CPTu soundings include the cone tip resistance (q_c) and the friction ratio (f_r). Generally, fine-grained materials have low q_c values and high f_r values (Lunne et al 1997). Organic soil is assumed to be a fine-grained material throughout the analysis. The pore pressure dissipation tests provided pore pressures over time at a given depth. The depth of 11.5 feet was chosen because it was the approximate midpoint of the organic soil layer. The equilibrium pore water pressure at the depth of 11.5 feet was calculated using the measured groundwater levels.

6.3.2 Typical Results at CPTu-1 Location

Typical data obtained from the field are presented for CPTu Site 1. Appendix B provides the field output at all other sites. At Site 1, the upper 2 feet of CPTu data were ignored due to the pre-augering of the borehole in this interval causing the very low q_c values. The q_c values from the CPTu sounding ranged between 14.89 to 301.19 tons per square foot (tsf) from depths of 2.1 to 5.1 feet. Values then ranged from 1.54 to 11.60 tsf and averaged 5.02 tsf between depths of 5.1 to 15.6 feet. Below this and up to 16.4 feet, values ranged up to 73.69 tsf. The f_r values ranged from –0.015 to 38.11 and averaged 9.49 between depths of 5.1 to 15.6 feet. Based on the provided CPTu data, the organic soil was estimated at depths of about 5 to 16 feet.

The maximum pore water pressure from the dissipation test was 3.715 pounds per square inch (psi) at 140 seconds from the start of the test. Equilibrium pore water pressure was calculated as 1.898 psi. The time for 50 percent dissipation, or pore pressure of 2.807 psi, was about 1000 seconds from the start of the test. Figures 6.7 and 6.8 show the unadjusted field output for the CPTu sounding and dissipation test, respectively at Site 1.

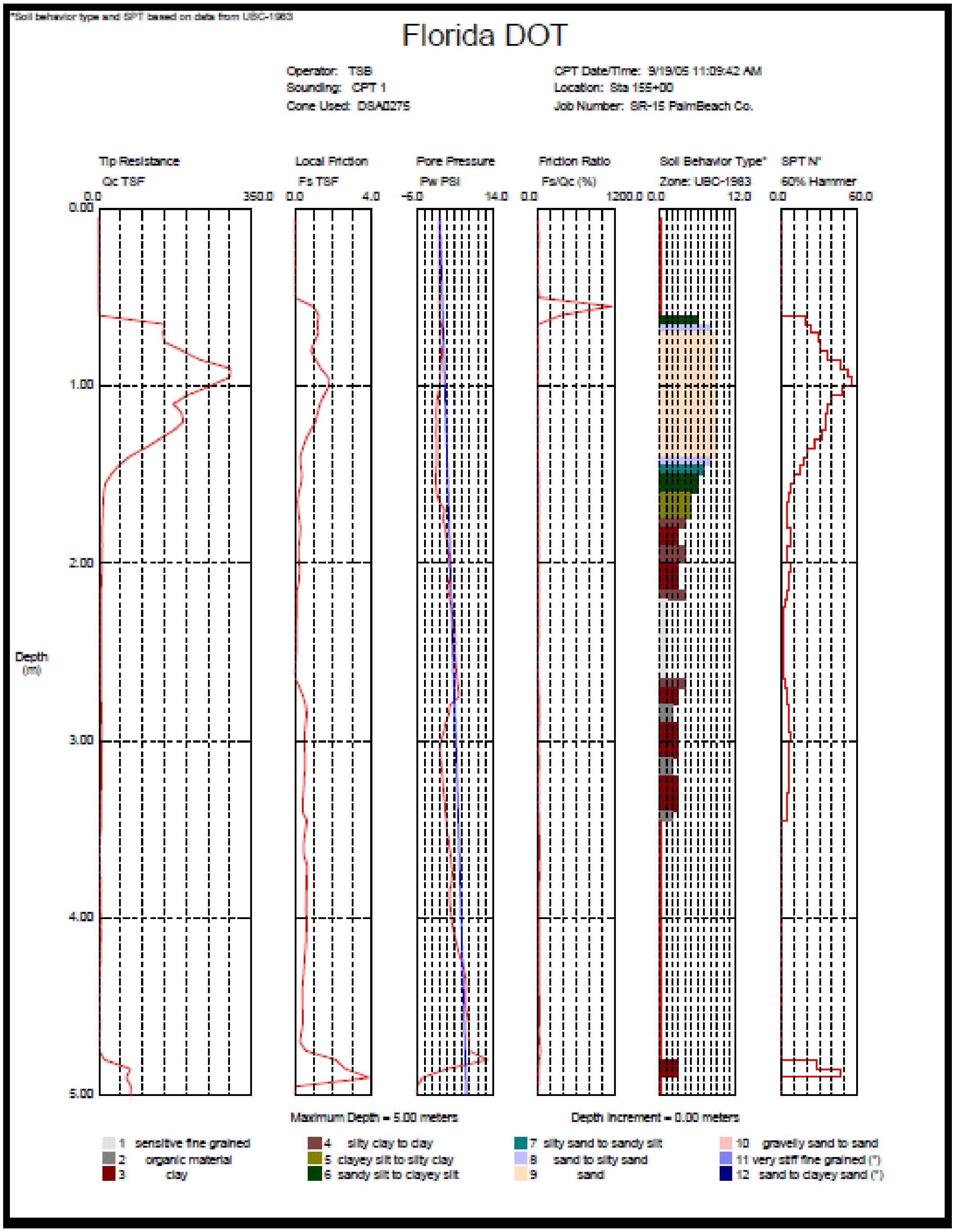


Figure 6.7: CPTu Output at Site 1

Note: 1 m = 3.28 feet

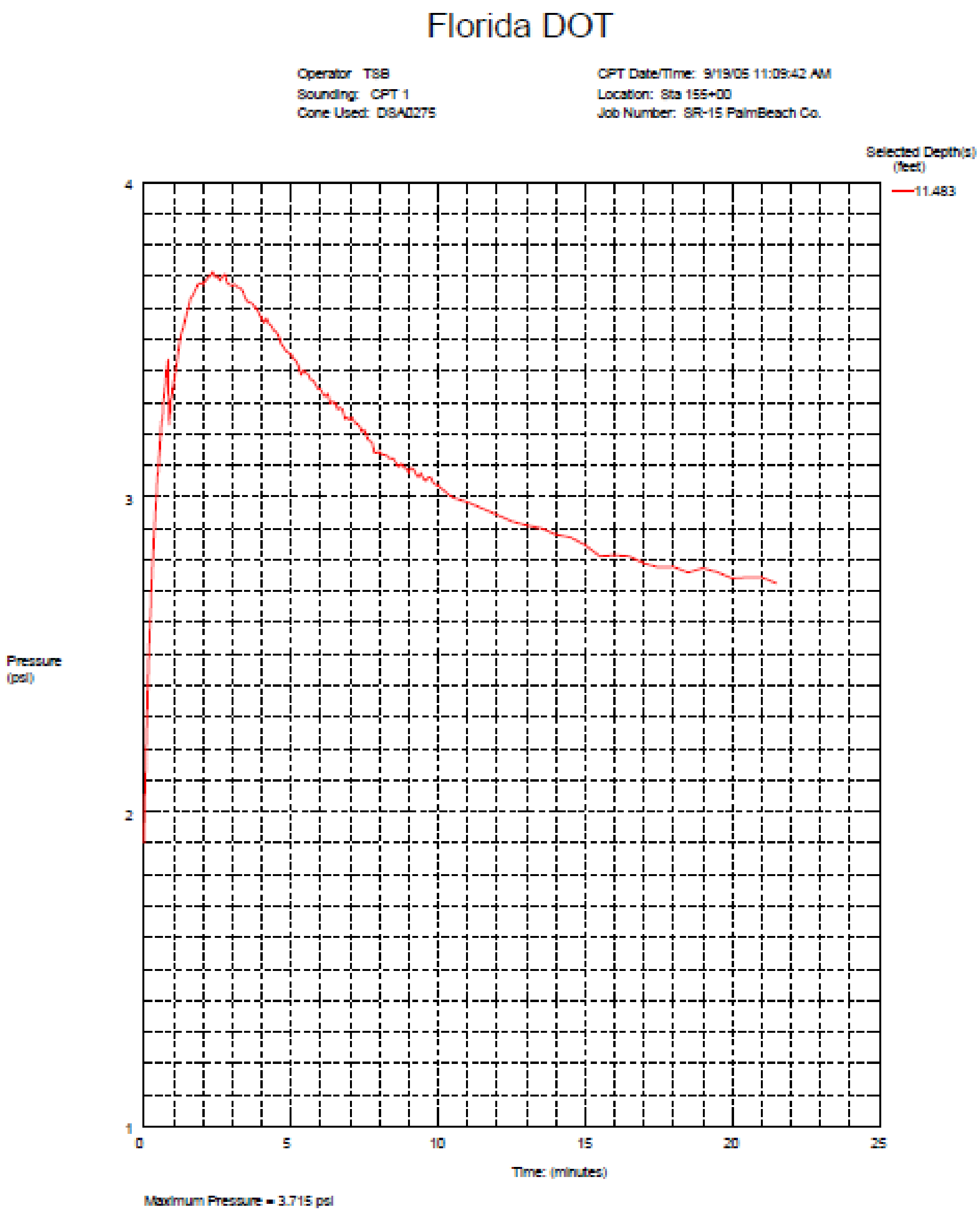


Figure 6.8: Porewater Dissipation Test Output at Site 1

6.3.3 Estimation of C_v from CPTu Data

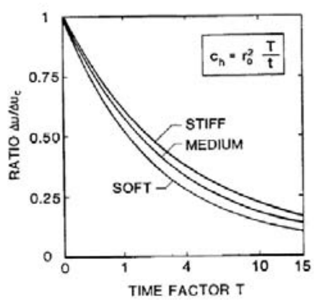
Porewater Dissipation Tests were conducted at nine out of eleven CPTu locations. Several methods have been developed to predict the horizontal coefficient of consolidation, C_h by interpreting the porewater dissipation data (Lunne et al. 1997; Abu-Farsakh et al. 2005). In the current study, three such methods, which were ranked high in the literature for their predictive capabilities (Abu-Farsakh et al. 2005), were chosen to determine the C_h , and these methods are briefly discussed in the following sections.

6.3.3.1 Method 1

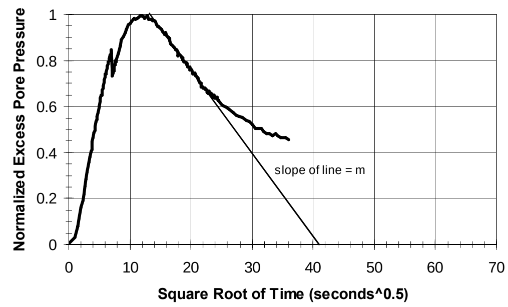
In this method, the following model was used to predict C_h (Senneset at al. 1982):

$$C_h = \frac{T_{50}}{t_{50}} r_0^{\ 2} \tag{6.2}$$

where T_{50} is the Time Factor for 50 percent dissipation, r_0 is the radius of piezocone (0.586 ft = 17.8 cm) and t_{50} is the time for 50 percent dissipation. T_{50} is obtained from the theoretical curve of normalized excess pore pressure versus the Time Factor, T for soft soils (Figure 6.9a). The real time t_{50} is obtained from a normalized excess pore water pressure (with respect to maximum excess pressure) versus the square-root-of-time curve as shown for Site 1 in Figure 6.9b. It was found that the excess pore water pressure initially increased with time, reached a maximum value, and then started a slow dissipation process. Similar behavior was also observed by other researchers (Abu-Farsakh et al. 2005). The parameter t_{50} is calculated after the initiation of the dissipation process.



(a)



(b)

Figure 6.9: Normalized Pore Pressure Dissipation Curves for Method 1

6.3.3.2 Method 2

In this method, the following model was used to determine C_h (Teh and Houlsby, 1991):

$$C_h = (T_{50}^* r_0^2 \sqrt{I_r}) / t_{50} \tag{6.3}$$

where T_{50}^* is the modified Time Factor for 50 percent dissipation (given as 0.245), and I_r is the rigidity index. I_r was determined using the equation $I_r = 170/f_r$, where f_r is the friction ratio from the CPTu tests (Bowles, 1996).

6.3.3.2 Method 3

In this method, the following equation was used to determine C_h (Teh, 1987):

$$C_h = (m / M_G)^2 \sqrt{I_r} r_0^2 \tag{6.4}$$

where M_G is the gradient of dissipation curve (given as 1.15), and m is the slope of the dissipation curve obtained from the initial linear portion (once the dissipation has started) as shown in Figure 6.9.

The three methods were employed to the dissipation curves of four CPTu Sites (Sites 1, 3, 4, and 6), for which the normalized excess pore-pressure dissipation curves demonstrated at least 50% dissipation within a practical time frame. Once C_h is known, the vertical coefficient of consolidation, C_v, was obtained from the following equation (Lunne et al. 1997):

$$C_v = C_h \frac{k_v}{k_h} \tag{6.5}$$

where k_v and k_h are the vertical and horizontal coefficient of permeability, respectively. The ratio k_h/k_v ranges from 1 to 15 for clays depending on the amount of layering (Lunne et al. 1997). For isotropic soils this ratio is 1.0, while for fibrous peat, this ratio is approximately 10.0 (Mesri et al. 1997). Since visual inspections of the current soils suggested layering, a fibrous matrix, and high degree of anisotropy, the ratio k_h/k_v was taken as 15 for calculation purposes. As an example, various model parameters to calculate C_v for Site 1 were as follows: T_{50} = 1.2; t_{50} = 1000 sec; r_0 = 17.8 cm; T^*_{50} = 0.245; I_r = 18; m = 0.034; M_G = 1.15; and average f_r = 9.49. Similar methodologies were employed for other three sites, and these details are available elsewhere (Huynh, 2006). Table 6.3 shows the computed C_v values according to the three interpretation methods. It is found that Methods 1 and 2 provide more consistent results, and have closer agreement with the C_v values from laboratory tests presented in Table 6.1 for the same Sites. Laboratory derived and field predicted C_v values are plotted in Figure 6.10 for comparison purposes. Although the C_v values predicted by Method 3 were an order of magnitude smaller than those predicted by the other 2 methods, a best-fit line was drawn through all data points in Figure 6.10 for a more conservative approximation; this relationship is expressed as follows:

$$C_{v_{FIELD}} = 1.7 * C_{v_{LAB}} \tag{6.6}$$

Equation 6.6 implies that the rate of consolidation in the field may be faster than that predicted from 1-D oedometer tests. Although these conclusions are drawn on the basis of a limited number of field tests, the current study demonstrates a methodology for rapid in-situ characterization of the compressibility (and its time-rate) behavior of organic soils and peat using the Piezocone Penetration testings.

Table 6.3: Interpretation of C_v (cm^2/sec) from Pore Pressure Dissipation Data

CPTu	Method 1	Method 2	Method 3
1	0.029	0.025	0.0013
3	0.075	0.062	0.0029
4	0.075	0.058	0.0022
6	0.067	0.069	0.0045

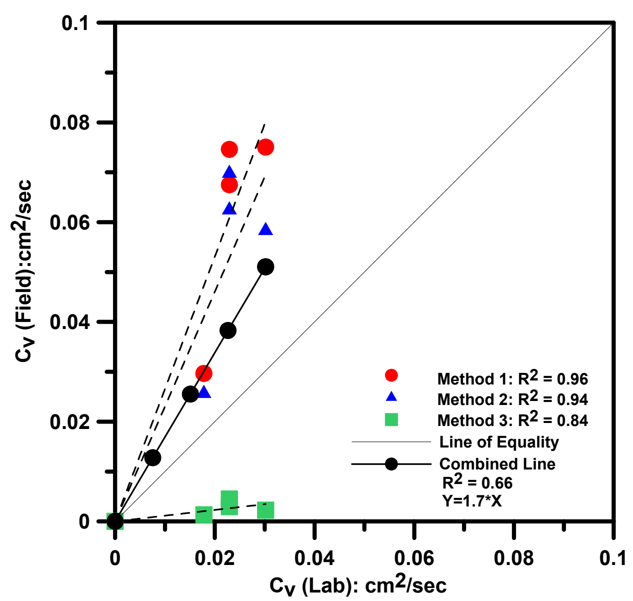


Figure 6.10: Comparison Between Field and Laboratory C_v Values

6.3.4 Strength and Modulus

Several analysis / interpretation tools available in the literature (Lunne et al. 1997) were used in this study to predict soil strength and modulus from CPTu data as described in the following sections. It is to be noted that the properties calculated by these recommended techniques are often highly dependent on the choice of appropriate model constants. The main purpose of this endeavor was to demonstrate the use of CPTu data for a rapid determination of a range in various properties (including mean values), which can be used as a first approximation for design or mechanistic analysis.

6.3.4.1 Undrained Shear Strength

The undrained shear strength, S_u can be estimated from the total cone resistance q_c using the following correlation (Lunne at al. 1997):

$$S_u = \frac{q_c}{N_{kt}} \tag{6.7}$$

where N_{kt} is the cone factor ranging between 15-20. The computed values of S_u are shown in Table 4 for N_{kt} set at both 15 and 20. The cone tip resistance q_c is taken as the average value during the penetration through the full depth of the organic layers at each site. The data show that the undrained shear strength varied between 5 KPa to about 36 KPa (with a mean value of 17 KPa) depending on the site's average cone resistance. These ranges are consistent with the values obtained by direct laboratory testing of undisturbed specimens using unconfined compression and Torvane shear tests (presented in Chapter 3).

6.3.4.2 Undrained Young's Modulus

The undrained Young's Modulus of elasticity (E_u) was derived from the undrained shear strength according to the following relationship:

$$E_u = n.S_u \tag{6.8}$$

where n is an empirical constant whose values for peat materials range from 25 to 125 depending on the stress level (Lunne et al. 1997). The computed values of E_u for n set at both 25 and 125 are presented in Table 4, in which N_{kt} values are chosen such that both the minimum and maximum values of E_u are obtained. For example, it is found that the mean value of E_u can vary between 360 KPa or 52 psi (N_{kt} = 20; n = 25) and 2400 KPa or 348 psi (N_{kt} = 15; n = 125) depending on the site's average cone resistance.

6.3.4.3 Constrained Modulus

The 1-D Constrained Modulus, M, typically measured in a laboratory oedometer test (as the inverse of the slope of the strain vs. effective stress plot), can be estimated from the cone tip resistance q_c as follows:

$$M = \alpha_m.q_c \tag{6.9}$$

where α_m is an empirical coefficient. A comprehensive array of α_m values for different soil types were reported in the literature (Sanglerat, 1972). For peat and organic soils having q_c < 0.7 MPa (≈ 7.3 tsf) and moisture content greater than 200%, the value of α_m

ranges between 0.4 to 1.0. Using these ranges, the computed values of M for different sites are presented in Table 6.4.

Table 6.4: Summary of Calculated Values for s_u, E_u, M and C_c (1 MPa = 145 psi)

CPT	s_u (kPa)		E_u (MPa)		M (kPa)		C_c silty		C_c fibrous	
	N_{kt}=15	N_{kt}=20	N_{kt}=15 n=125	N_{kt}=20 n=25	α_m=0.4	α_m=1	α_m=0.4	α_m=1	α_m=0.4	α_m=1
1	32	24	4.01	0.60	192	481	2.220	0.888	3.739	1.496
2	5	4	0.62	0.10	30	74	18.945	7.578	24.583	9.833
3	14	11	1.77	0.26	85	211	4.441	1.777	7.098	2.839
4	25	19	3.10	0.46	149	372	2.907	1.163	7.207	2.883
5	20	15	2.45	0.37	118	294	3.063	1.225	9.580	3.832
6	36	27	4.48	0.67	215	538	2.340	0.936	4.212	1.685
7	17	13	2.10	0.32	101	252	3.336	1.335	5.954	2.382
8	14	10	1.70	0.26	82	204	4.902	1.961	9.662	3.865
10	10	8	1.26	0.19	61	151	5.911	2.364	16.717	6.687

6.3.4.4 Compression Index

The constrained modulus is related to the laboratory 1-D Compression Index, C_c, as follows:

$$M = \frac{\Delta\sigma_v'}{\Delta\varepsilon} = \frac{2.3(1+e_0)\sigma_v'}{C_c} \tag{6.10}$$

where e_0 is the initial void ratio, and σ_v' is the vertical effective stress. Combining Equations 6.9 and 6.10, C_c was determined from the cone tip resistance data as shown in Table 6.4. It was found that the mean C_c value for the organic silts ranges between 2.14 and 5.34, while that for fibrous peats the mean C_c value ranges between 3.94 and 9.86.

6.3.4.5 Comparison of Field and Laboratory Compression Indices

The field predicted C_c values (Table 6.4; α_m = 1) are plotted against the laboratory derived C_c values (Table 6.1 and Figure 6.2) in Figure 6.11. The best-fit line through all data points can be expressed by equation 6.11 (R^2 = 0.87), and it implies that a reasonable correlation exists between the laboratory and CPTu predicted in-situ Compression Indices.

$$C_{c(lab)} = 1.57 C_{c(field)} \tag{6.11}$$

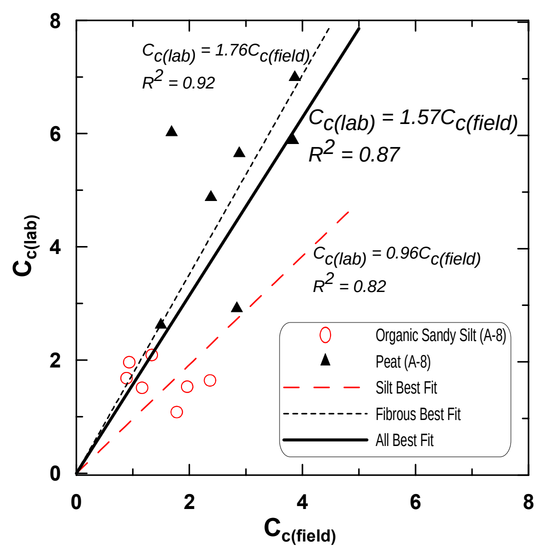


Figure 6.11: Correlation between Field and Laboratory Compression Indices

6.4 SUMMARY AND DISCUSSIONS

Presence of soft compressible organic soils and peat below the SR 15 / US 98 highway has been the major underlying cause of recurring pavement distresses in the form of cracking, rutting, and long-term secondary settlement. A coordinated laboratory and field investigation was conducted to evaluate the strength and compressibility characteristics of these problem foundation soils. A comprehensive laboratory consolidation testing program was undertaken, which also included secondary compression studies on undisturbed samples under simulated in-situ stress conditions. The Time-Stress-Compressibility relationships were investigated over a stress range (σ_v'/σ_p') of 0.3 to 1.15 encompassing both the recompression and compression zones. It was found that the (C_α / C_c) ratio at any stress level has constant values ranging from 0.03 to 0.05, which are consistent with the values reported in the literature for similar soils. The in-situ Coefficient of Consolidation, and the Compression Index were predicted from the CPTu data, both of which showed reasonable correlation with the laboratory derived values. These findings are significant because they imply that the C_c values predicted from CPTu data may also be used to estimate C_α (since C_α / C_c is constant), and hence the long-term secondary settlement, thus avoiding lengthy and expensive laboratory testing protocols. Considering the inherent difficulty in sampling and laboratory testing of undisturbed soft organic soils, Piezocone Penetration Tests show promise as an efficient tool for relatively rapid in-situ characterization of subsoil strength, modulus, and compressibility, which in turn may be used for forensic interpretations of pavement failures, mechanistic analysis, and validation of pavement performance models.

CHAPTER 7

Implementation Guidelines

7.1 INTRODUCTION

This Chapter synthesizes various Phases illustrated in Figure 1.5 describing the research approach and methodology. The primary tasks for this endeavor include:

(i) identifying some practical and promising Surface Pavement Solutions (SPS) which can be incorporated within the surface layer for extending the service life of the pavement structure (Figure 1.5: Phase I)

(ii) identifying the locations of field test sections based on geotechnical investigations and site-specific conditions (Figure 1.5: Phases II and III);

(iii) designing the field test sections that include both control segments, and sections incorporating target SPS products (Figure 1.5: Phase IV); and

(iv) recommending methodology for correlating probable pavement performance with site-specific conditions (Figure 1.5: Phases I through V)

7.2 SELECTION OF PROMISING STRATEGIES

In addition to conducting extensive literature review, and a State-of-the-Practice Survey of the DOTs (described later), the PI interviewed / surveyed the engineers from several state DOTs (including Texas, Virginia, North Carolina, and Indiana) during the 85th Annual Meeting of the Transportation Research Board (2006) to identify construction / rehabilitation strategies adopted by other states when soft subgrade soils were encountered. In general, it was found that agencies are more inclined towards complete removal of unsuitable materials or employing deep ground modification techniques (deep mixing, soil injection, grouting, etc.) using cementitious materials for treating soft problematic soils. These techniques are mostly applicable to new constructions or complete reconstruction of existing pavement structures. As mentioned previously, although some of the "deep" methods can be regarded as surface injected treatments, extensive mix-design criteria must be developed first for site-specific soil conditions to ensure effective stabilization of the soft ground. This process requires significant time and expenditure, and therefore was not recommended in the current study. Instead, the focus was on developing near-surface strategies, which primarily involves reinforcement of the asphalt surface layers with high-stiffness products for mitigating crack propagation, thus deviating from the traditional geosynthetics solutions where the reinforcing element is placed at the interfaces of the various foundation layers (thus involves reconstruction of the pavement underlying layers). Based on the specific requirements of the current project, experience and performance data gathered on several candidate solutions and their ready availability, the following reinforcing products were recommended for use in the field test sections to be constructed along SR15 / US98. Product specifications are included in Appendix B.

7.2.1 PaveTrac MT-1

This is a steel reinforcing mesh manufactured by Bekaert Corporation, Netherlands, (Figure 7.1; Appendix C) with strong potential for resisting the growth and propagation of cracks in asphalt surface layers. Case histories include portions of I-95 in Melbourne, Florida (ACF Environmental, 2004), rehabilitation of asphalt pavements on soft organic soils in Netherlands, and Bluegrass Parkway rehabilitation (ongoing) project (Graves, 2005).

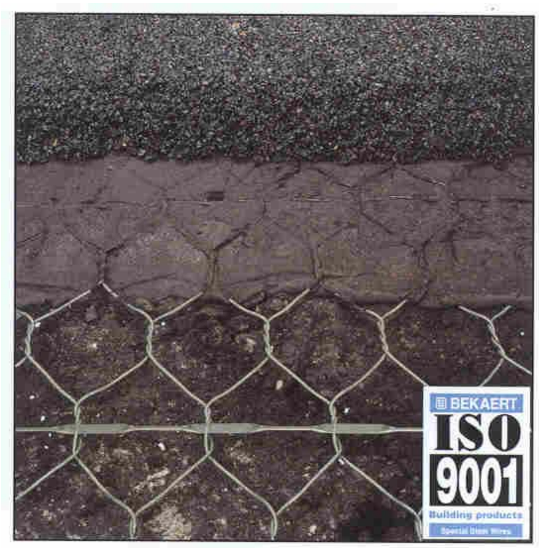


Figure 7.1: PaveTrac MT-1 Placed underneath the Asphalt Surface

7.2.2 <u>GlasGrid 8501</u>

Manufactured by Saint-Gobain Technical Fabrics, this is a knitted glass fiber strand grid forming a pavement reinforcing mesh (Figure 7.2; Appendix C). High stiffness of glass fiber grids has strong potential for arresting crack propagation in asphalt surface layer. Case histories include pavement rehabilitation project in Hollywood, Florida, ongoing Bluegrass Parkway Project in Kentucky (Graves, 2005), and in the mitigation of reflection cracking in Mississippi, Iowa, and Wisconsin (Darling and Woolstencroft, 2000; Bischoff and Toepel, 2003; Marks, 1990).

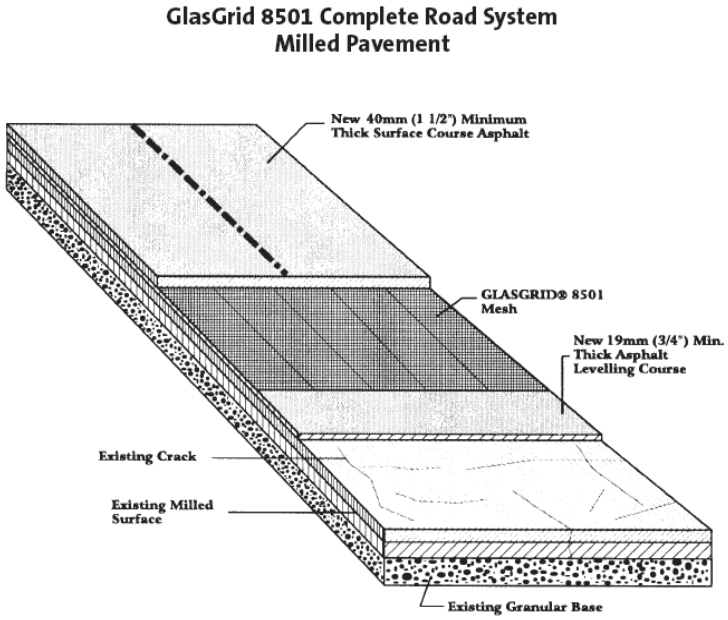


Figure 7.2: Schematic of Glasgrid 8501 placed under a new asphalt surface course

7.2.3 PetroGrid 4582

PetroGrid Style 4582 is a pavement interlayer composite made from a nonwoven, needle-punched, polyester paving fabric laminated to a heavy duty structural grid composed of glass fiber embedded in polymeric resin (Figure 7.3; Appendix C). Currently manufactured by Propex Fabrics, the PetroGrid 4582 with a combination of flexible fabric and stiff glass fiber grid accommodates movements in the underlying layers while providing crack arresting mechanisms due to grid action. Recent laboratory studies showed strong potential for resisting reflection and fatigue cracking in asphalt overlays (Sobhan and Tandon, 2003; Sobhan et al. 2004).

Figure 7.3: PetroGrid placed over a tack coat on the cracked asphalt surface

As mentioned previously, PaveTrac and GlasGrids are being used to reinforce test sections in another similar project along Bluegrass Parkway in Kentucky, and the project will be monitored for 3 years to evaluate performance of reinforced sections from 2005 to 2008 (Graves, 2005; Kentucky Transportation Cabinet)

7.3 IDENTIFYING LOCATIONS FOR FIELD TEST SECTIONS

As presented previously in Figure 3.6, an attempt was made to correlate the observed distress levels in the existing pavement with an *Organic Factor*, F_{org}, at each individual boring locations (Equation 7.1).

$$F_{org} = \frac{h_m \gamma_T * OC}{(1 + MC)} \tag{7.1}$$

F_{org} represents the weight of an organic soil column of unit cross sectional area, and incorporates the height (h_m), organic content (OC), and moisture content (MC) at that location (total unit weight γ_T has an average value of about 70 lbs/ft^3 as obtained from undisturbed specimens). F_{org} is a good indicator of the site-specific condition concerning the poor foundation soils. Cone Penetration Tests were conducted at 9 locations based on the distressed zones indicated on Figure 7.4 (repeat of Figure 3.6). Since extensive site characterization was completed in these locations, it was also logical to select future test sections within these known areas. Accordingly, 500-ft test sections were selected within the significantly distressed zones as follows: (i) Location 1: MP 22.60 to MP 23.168; Boring 33 – Boring 41; average F_{org} = 190 lbs; and (ii) Location 2: MP 23.964 to MP 24.532; Boring 54 to Boring 70; average F_{org} = 138 lbs (discussed later). These locations were selected to represent differences in F_{org} values, so that the relative performance of various SPS products can be evaluated under different site-specific conditions.

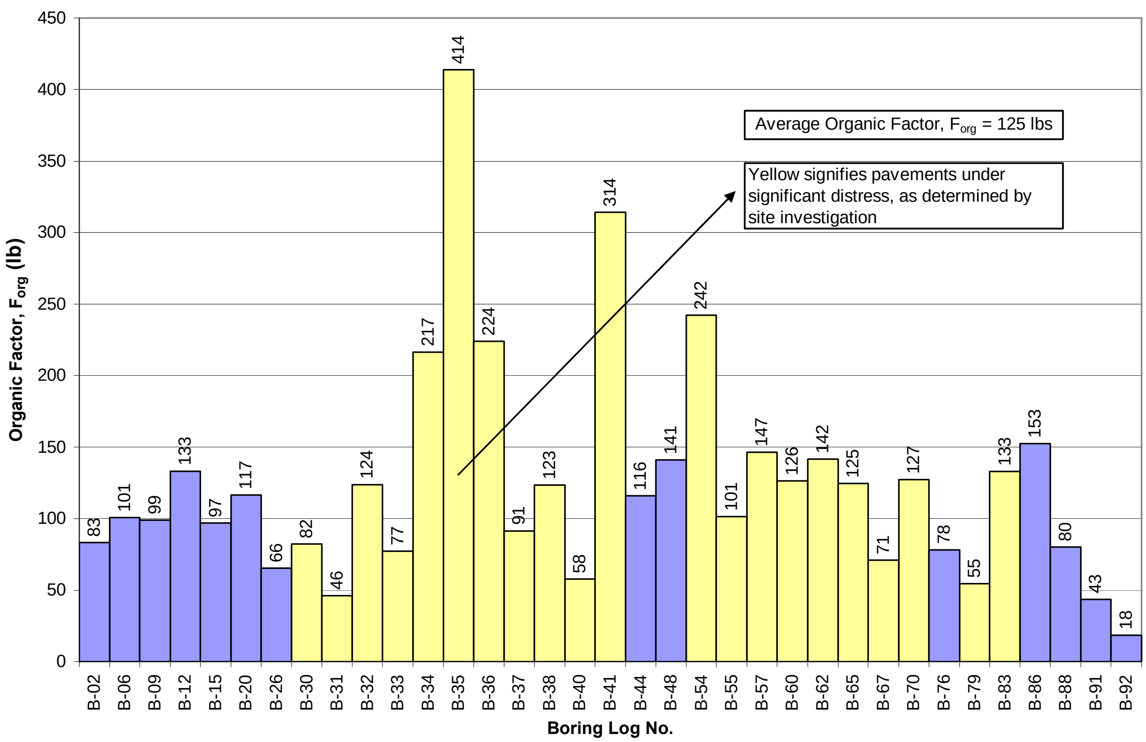


Figure 7.4: Organic Factors at Various Borehole Locations (1 lb = 0.4536 Kg)

To further understand the relationship between the site conditions (the thickness of the muck, organic content, moisture content, etc.), the Organic Factor, F_{org} is plotted against the in-situ modulus of elasticity as interpreted from cone penetration test results (Table 6.4) and presented in Figure 7.5.

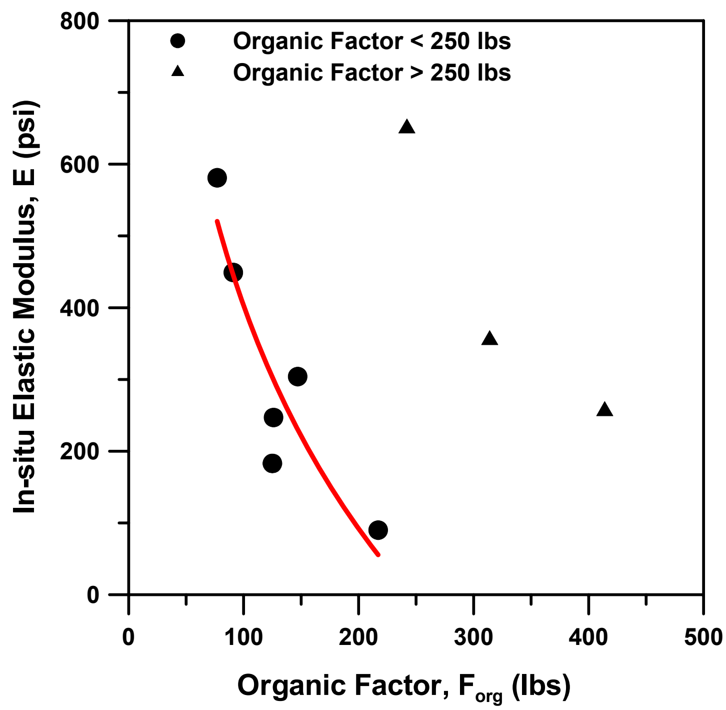


Figure 7.5: Relationship between Organic Factor and Elastic Modulus

Following conclusions may be drawn from Figure 7.5:

1. The modulus (E) of the organic soil decreases as F_{org} increases, which is expected; and
2. For most cases, F_{org} is less than about 250 lbs. These six data points are shown with circular symbols. When F_{org} > 250 lbs, the 3 data points were distinctly showing a different correlation as indicated by the triangular symbols. It is possible that in case F_{org} exceeds a critical value, the degradation of modulus occurs at a different rate.

However, in the absence of sufficient data, this difference in behavior cannot be established, and therefore no trend line was drawn through the 3 triangular symbols.

When F_{org} < 250 lbs, the best-fit relationship is expressed as follows ($R^2 = 0.83$):

$$E = -449\ln(F_{org}) + 2469 \qquad (7.2)$$

Based on Equation 7.2, the average Elastic Moduli for field test locations 1 and 2 are about 113 psi and 257 psi, respectively (implying a 56% drop in modulus due to an increase of F_{org} by 38%). These average values of moduli incorporate quite different site-specific conditions thus facilitating a relative comparison of various SPS strategies selected for this study.

7.4 DESIGN OF FIELD TEST SECTIONS

Figure 7.6 presents the two proposed locations for the test sections, as well as a layout of the reinforced and the control segments. PaveTrac, GlassGrid, Petrogrid, and ARMI will be installed in four 500-ft contiguous segments along with 2 control segments (each 500-ft long) as shown in Figure 7.6. This test section configuration is repeated along both the northbound and the southbound lanes at each test site. The proposed layout is expected to account for any variation in traffic volume <u>and/or</u> traffic loading along the northbound and southbound lanes, thus allowing a meaningful comparison of the performance of various test sections. It is to be noted that the use of ARMI is part of the original cross section design prepared by the FDOT consultant C3TS for the rehabilitation of SR15. Therefore, a total of four different reinforcing materials will be evaluated in the test sections as part of this research project.

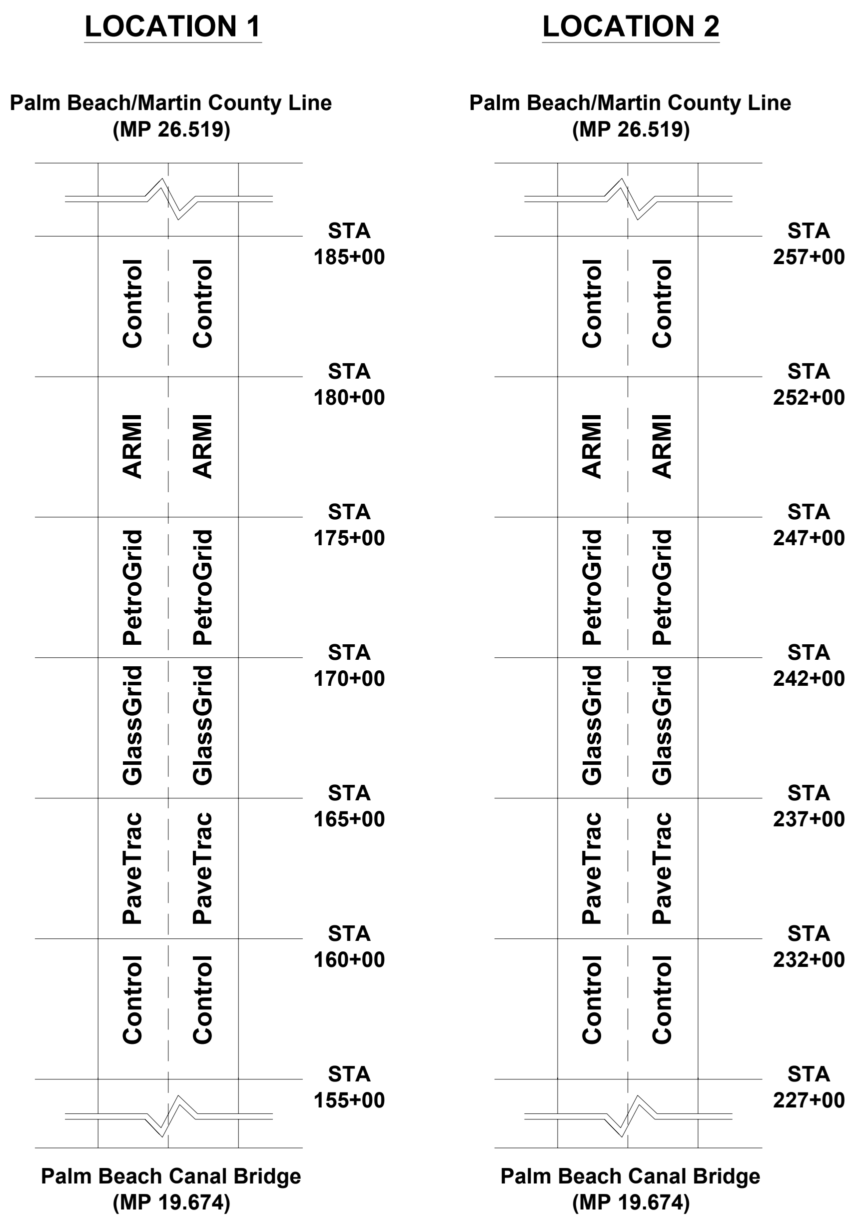


Figure 7.6: Layout of the proposed field test sections along SR15 / US98

7.4.1 <u>Design Cross Sections</u>

Following design cross sections are recommended for the test sections:

7.4.1.1 Control Section

The eight control sections will be constructed similar to the original design recommended by the FDOT consultant C3TS, with the exception that no ARMI layer will be used. In these sections, 4.5 inches of existing asphalt surface will be milled and resurfaced with a new asphalt layer.

7.4.1.2 ARMI Section

The four ARMI sections will be constructed according to the original C3TS design, which includes milling 4.5 inches of existing asphalt layer, placement of a 0.75 inch ARMI layer, and resurfacing with a new asphalt course.

7.4.1.3 GlasGrid Section

The four GlasGrid sections will consist of 4.5 inches of milled surface, placement of 0.75 inch asphalt leveling course, the GlasGrid interlayer, and a new asphalt surface layer as shown in Figure 7.7.

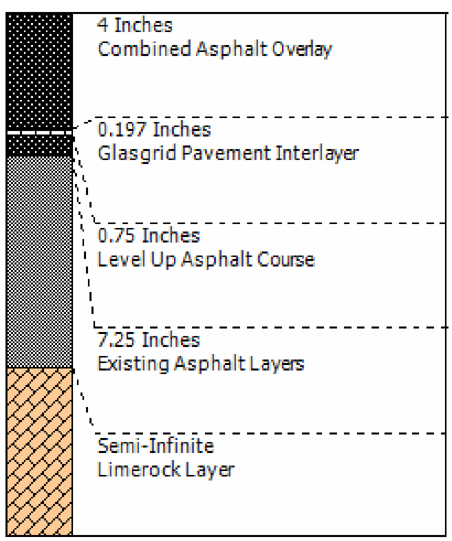


Figure 7.7: Recommended design for the GlasGrid section

7.4.1.4 PaveTrac Section

The four PaveTrac sections will consist of 4.5 inches of milled surface, PaveTrac steel mesh with mechanical fastening, and a new asphalt surface layer.

7.4.1.5 PetroGrid Section:

The four PetroGrid sections will consist of 4.5 inches of milled surface, 1 inch of asphalt leveling course, the PetroGrid layer, and a new asphalt surface course.

7.5 MECHANISTIC ANALYSIS WITH KENLAYER

A pavement simulation / analysis study was conducted using the KENLAYER (Huang, 2004) mechanistic computer program. The primary objective of this endeavor was to determine the stresses, strains, and deflections in the control test sections under simulated traffic loading. Under a mechanistic-empirical (M-E) design framework, these pavement responses can later be used to predict the future performance of the structure if suitable *Transfer Functions* are established for various pavement layers. Following are the various components of this phase of the investigation.

7.5.1 Structural Layers and Properties

According to the Roadway Soil Survey Report by Geosol, Inc. (2004), the existing test section locations in general have the following characteristics:

1. Stratum 1: Asphalt layer; approximate thickness 12 inches
2. Stratum 2: Base layer; brown fine to coarse sand with limerock and shell fragments; approximate thickness 12 inches
3. Stratum 3: Subgrade; light brown fine sand; thickness 24 – 48 inches

4. Stratum 4: Dark brown organic sandy silts and peat (A-8); average thickness 10 ft.

7.5.2 Control Section Properties

As presented in the earlier sections, the rehabilitation of the SR15 / US98 will involve milling and resurfacing of 4.5 inches of the existing pavement. Accordingly, a series of pavement mechanistic analyses was performed on a 5-layer pavement system with the following assumed properties:

(i) 4.5-inch new asphalt surface; Resilient modulus = 350,000 psi

(ii) 7.5-inch old asphalt base; resilient modulus = 300,000 psi

(iii) 12-inch limerock base; resilient modulus = 30,000 psi

(iv) 36-inch sandy subgrade; resilient modulus = 10,000 psi

(v) 120-inch organic soil and peat; elastic modulus = 100, 400, 800, 1200 psi

Properties of the asphalt, limerock base and sandy subgrade are typical values for pavement layer properties in Florida. The average muck layer thickness from 9 CPT locations came out to be about 10 feet. The elastic modulus of the muck layer was interpreted from the cone tip resistance data, and was presented in Table 6.4. The values ranged between 100 to about 700 psi (0.69 to 4.9 MPa). Since there is a lot of variability in the organic layer properties, the analysis was carried out for a modulus up to 1200 psi (8.3 MPa), which still represents a pseudo-soft subgrade.

7.5.3 Reinforced Sections

Mechanistic analysis was performed on a reinforced section in which a GlasGrid 8501 structural mesh was used on top of a 0.75-inch leveling course, and overlaid with new asphalt surface course such that the final thickness of the new structure is 4.5 inches. GlasGrid has very high modulus of elasticity in the order of 10 million psi, with a thickness of about 0.2 inches.

7.5.4 Predicted Traffic in SR 15 / US 98

Based on the available traffic data, the year 2007 and year 2020 AADT are 4811 and 5400, respectively, implying a 14-year design period. Following the procedures outlined in the FDOT Flexible Pavement Design Manual, the estimated total 18-kip Equivalent Single Axle Load (ESAL) application in 14 years is 2.11 millions.

7.5.5 KENLAYER Computer Program

The KENLAYER program solves an elastic multilayer system under a circular loaded area. The solutions are superimposed for multiple wheels (single, dual, dual-tandem, and dual-tridem), applied iteratively for nonlinear layers, and collocated at various times for viscoelastic layers. Damage analysis can be performed for multiple periods each with different set of material properties, and multiple load groups. The damage caused by fatigue cracking and permanent deformation in each period over all load groups is summed up to evaluate the design life. In this study, the multilayer system is considered to be linear elastic with constant resilient modulus for all layers. Analysis is performed for 4 different moduli of the organic layer as indicated above.

7.5.6 Failure Criteria and Transfer Functions

The two widely accepted failure criteria for flexible pavements include (i) the fatigue cracking due to repeated tensile stresses and strains at the bottom of the asphalt layer, and (ii) permanent deformation due to compressive strains on top of the foundation soil. The existing SR 15 / US 98 pavement structure is severely distressed with both permanent deformation and cracking of the asphalt layer.

KENLAYER uses the following well-known fatigue cracking criteria:

$$N_f = f_1(\varepsilon_t)^{-f_2}(E_1)^{-f_3} \tag{7.3}$$

in which N_f is the allowable number of load repetitions to prevent fatigue cracking; ε_t is the tensile strain at the bottom of the asphalt layer; E_1 is the elastic modulus of the asphalt layer; and f_1, f_2, and f_3 are constants determined from laboratory fatigue tests with f_1 modified to correlate with field performance observations. The Asphalt Institute suggested 0.0796, 3.291, and 0.854 for f_1, f_2, and f_3, respectively in their analytically based design procedures (Huang, 2004).

KENLAYER uses the following failure criteria for permanent deformation:

$$N_d = f_4(\varepsilon_c)^{-f_5} \tag{7.4}$$

in which N_d is the allowable number of load repetitions to limit permanent deformation; ε_c is the compressive strain at the top of the subgrade, and f_4 and f_5 are constants obtained from road test or field performance with values of 1.365×10^{-9} and 4.477, respectively as suggested by the Asphalt Institute (Huang, 2004).

These *Transfer Functions* (specifically the f_i constants) are not calibrated for the asphalt materials and organic soils existing in Florida roadways, and therefore only the pavement responses can be computed in the current analysis, but life prediction can be made only at the conceptual level (discussed later).

7.5.7 Results of Mechanistic Analysis

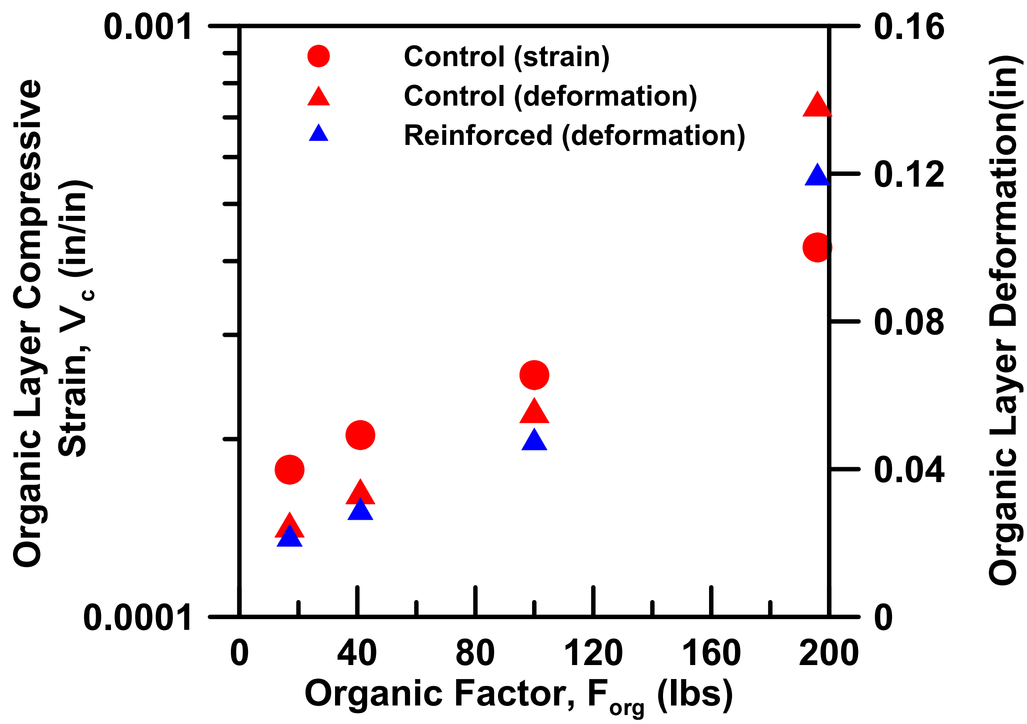
Table 7.1 summarizes the critical stresses and strains in the SR15 pavement system due to one application of a 9000 lb traffic load. Variations of these response parameters with the organic layer modulus and the Organic Factor are presented in Figure 7.8. Typical computer outputs for control and reinforced sections are provided in Appendix C. It is found from Figure 7.8 that both the compressive strain on top of the organic layer (ε_c in Equation 7.4) and the organic layer vertical deformation increases as the Organic Factor increases. The corresponding deformation in the GlasGrid reinforced section is slightly lower than the control, and this effect is more pronounced as the Organic Factor increases.

Also shown in Figure 7.8 is the variation of the tensile strain at the bottom of the asphalt layer (ε_t in Equation 7.3) with the Organic Factor. This relationship is more significant since the central focus of this study is to develop near-surface rehabilitation strategies. It is found that the tensile strain in the control section remains nearly unchanged with

F_{org}, but the reinforced section significantly reduces the tensile strain, which is the major factor in the growth and propagation of tensile cracks in the asphalt surface layer. It is also found that the tensile strain in the GlasGrid section gradually increases with the Organic Factor, and approaches the values representative of the control section at F_{org} near or above 200 lbs. Figure 7.8 may serve as an important mechanistic-empirical (M-E) design tool, since two critical response parameters have been correlated to the site-specific condition represented by the Organic Factor, F_{org}.

Table 7.1: Mechanistic Analysis and Pavement Responses

Organic Layer Modulus, psi (F_{org}: Equ. 7.2)	**Asphalt Layer Tensile Strain, ε_t (with Glasgrid)**	**Organic Layer Compressive Strain, ε_c (with Glasgrid)**	**Organic Layer Vertical Displacement, in. (with Glasgrid)**
100 (196)	-4.093E-05 (-4.977E-06)	4.221E-04 (5.37E-04)	0.139 (0.119)
400 (100)	-4.519E-05 (-1.782E-06)	2.566E-04 (2.962E-04)	0.056 (0.047)
800 (41)	-4.672E-05 (-5.952E-07)	2.031E-04 (2.238E-04)	0.034 (0.028)
1200 (17)	-4.754E-05 (-6.318E-08)	1.775E-04 (1.907E-04)	0.025 (0.021)



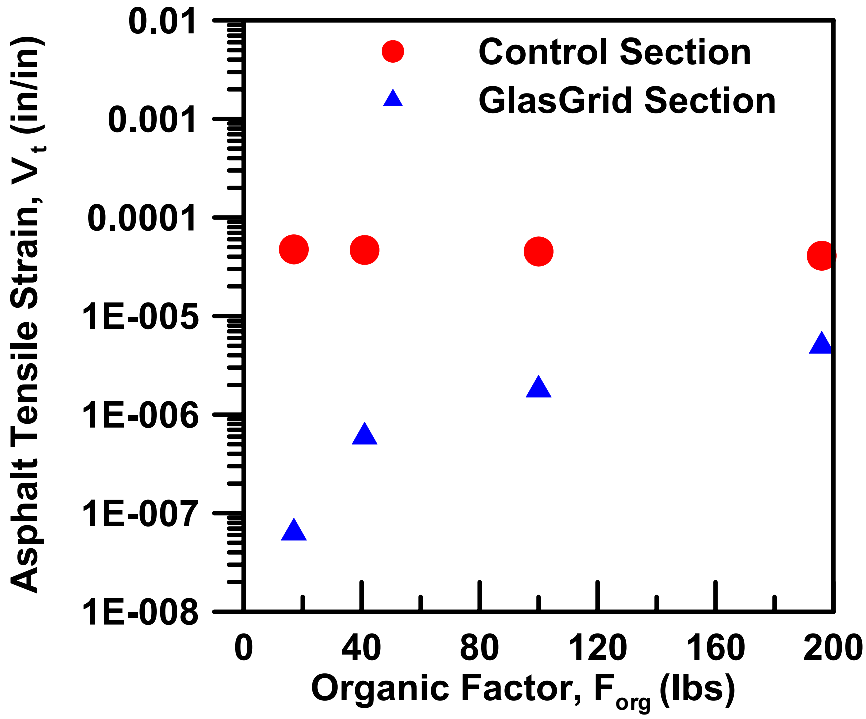


Figure 7.8: Variations of Pavement Response Parameters with Organic Factor

7.6 ORGANIC FACTOR AND PAVEMENT LIFE

It is well known that the weakest link in a Mechanistic-Empirical (M-E) design framework is the development of *Transfer Functions* which are the mathematical relationships (such as Equations 7.3 and 7.4) used to predict pavement performance based on the computed responses (stresses, strains, and deflections) obtained from a mechanistic analysis. In the absence of such Transfer Functions developed and calibrated for Florida materials and conditions, the allowable ESAL repetitions to failure of SR 15 cannot be predicted. These relationships are empirically developed from laboratory studies and/or field observations. Pavement life in years can then be estimated by dividing the allowable number of repetitions by the actual number of ESAL applications per year, which is generally known from traffic count data.

A major goal of this project is to develop some site-specific criteria which will aid in designing longer lasting pavements when dealing with organic soils and muck. Conceptually, pavement life can be predicted for various site-specific properties (Organic Factor, F_{org}) using the general procedure described above, and summarized below:

1. Develop Figure 7.5 or Equation 7.2 from direct geotechnical site investigations such as Cone Penetration Testing and/or laboratory testing;
2. Use properties determined in step 1, and available properties of other layers into a suitable mechanistic analysis package;
3. Develop and calibrate *Transfer Functions* such as equations 7.3 and 7.4 (requires laboratory testing and/or field observations);

4. Use the response parameters from step 2 into step 3 to predict allowable number of load repetitions (18-kip ESALs) to failure for each failure criteria;
5. Divide the actual ESAL/year from the traffic data by the allowable ESAL from step 4 to compute a Damage Ratio, D for each failure criteria as follows:

$$D = \frac{ESAL / Year}{ESAL(allowable)} \tag{7.5}$$

6. Identify the maximum or critical Damage Ratio (D_{crit}) by applying both failure criteria in step 5, and determine the pavement life (N) in years from Equation 7.6:

$$N = \frac{1}{D_{crit}} \tag{7.6}$$

The above steps are repeated for different site conditions, and conceptually, the N-F_{org} relationship can be presented in Figure 7.9 or Equation 7.7 as follows:

$$N = C_1 \ln(F_{org}) + C_2 \tag{7.7}$$

in which N is the pavement failure life in years, and C_1 and C_2 are model constants. Figure 7.9 shows that pavement life decreases with the increase in Organic Factor. The above methodology provides a Mechanistic-Empirical (M-E) design framework for directly linking the site-specific conditions to the probable future performance of the pavement structure. Once the field test sections are completed, and a sound and systematic monitoring / testing plan is implemented, data will be available to develop or further refine the relationships for more reliable prediction of pavement performance.

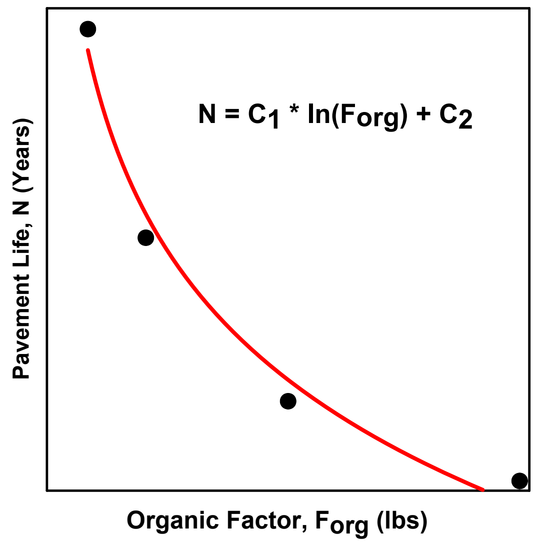


Figure 7.9: Conceptual Relationship between Organic Factor and Pavement Life

7.7 SUMMARY AND DISCUSSIONS

A desired outcome of the present study is to link the geotechnical site conditions to probable pavement performance, so that an appropriate rehabilitation strategy can be adopted. This is a multi-stage process which includes laboratory/field investigations, mechanistic analysis, determining site-specific criteria, developing transfer functions, and prediction of pavement life or performance. A conceptual framework has been developed and presented. The transfer functions can be developed through observed performance data from in-service pavements such as the experimental test sections scheduled to be constructed in 2007. Appropriate guidelines are provided for implementing these procedures in new construction or reconstruction projects.

CHAPTER 8

State-of-the-Practice Survey

8.1 BACKGROUND

In an effort to identify suitable rehabilitation strategies that can be applied to the asphalt surface layer, it was important to conduct a nationwide State-of-the-practice survey for documenting the most common and any up-and-coming techniques followed by various DOTs and highway agencies for mitigating the problems associated with soft subgrade soils. A questionnaire survey was sent out to the appropriate division of all State Department of Transportation (DOTs), and nearly 72% responded to the survey. Figure 8.1 shows this survey questionnaire with 6 focused questions, and several choices for directed answers, along with an opportunity for providing additional comments and alternative responses.

8.2 SURVEY RESULTS

The survey data was analyzed and compiled, and graphically presented in Figures 8.2 through 8.7. Each Figure is associated with the responses received for one question.

8.3 IMPORTANT FINDINGS

Following are the important findings from the survey:

1. About 25% of the respondents stated that they encounter organic or other soft subgrdes 25-50% of the time, while 10% stated that deal with soft soils more than 50% of the time.

2. 45% of the respondents suggested that they observe rutting, cracking and differential settlements on the surface layer which are caused by soft soils.
3. More than 40 % of the respondents perform complete or partial removal of the unsuitable soils, 20% employ surcharging, and nearly 15% favor deep mixing with some form of chemical stabilizers.
4. Nearly 70% of the respondents have used some geosynthetic product at the base/subgrade interface to mitigate soft subgrade problems. Nearly 10% used it to reinforce the new asphalt overlay.
5. Nearly 45% stated that geosynthetics improved performance of reinforced sections compared to control sections.
6. Nearly 38% never used Cone Penetration Tests (CPT) for soil characterization. Nearly 15% have used it for rapid soil classification and strength evaluation, and 10% for soil compressibility evaluation.

QUESTIONNAIRE SURVEY

Please "circle" one or more as appropriate:

A) How often does your state DOT encounter organic soils and/or soft subgrade conditions in pavement projects?

1. Rarely
2. 5-10% of the time
3. 10-25% of the time
4. 25-50% of the time
5. >50% of the time

B) What type of distress do you observe on the pavement surface that is directly attributable to organic soils or soft subgrade conditions?

1. None
2. Rutting in wheel path
3. Differential settlement / secondary compression
4. Alligator and/or other form of cracking
5. A combination of the above

C) If soft subgrades are encountered, what design/construction/mitigation strategy do you follow?

1. Complete or partial removal of soft soils / new fill materials
2. Surcharge loading
3. Over-asphalting
4. Deep mixing, stabilization, chemical injection, CSV columns
5. Others – please specify:

D) Have you used any geosynthetic reinforcements to mitigate soft subgrade problems? Specify type of product if applicable.

1. None / never
2. Geosynthetics at base/subgrade interface
3. Geosynthetics at pavement/base interface
4. Geosynthetics within the new asphalt layer (overlay)
5. Other locations / other type of reinforcement

E) Rate your experience with geosynthetic or other reinforcing products.

1. Not applicable / never used – please provide comments
2. In most cases, geosynthetics improved performance compared to control sections
3. In most cases, the benefits of geosysnthetics were not evident or realized
4. Reinforcing the (i) surface (asphalt) layer <u>OR</u> (ii) base/subgrade layer was more effective
5. Any comments

F) Use of Cone Penetration Test (CPT) for soil and subgrade characterization

1. Never used
2. Routinely used
3. Used for rapid soil classification and undrained strength evaluation
4. Used for compressibility evaluation of soft subgrade
5. Other

Figure 8.1: State-of-the-Practice Survey Questions

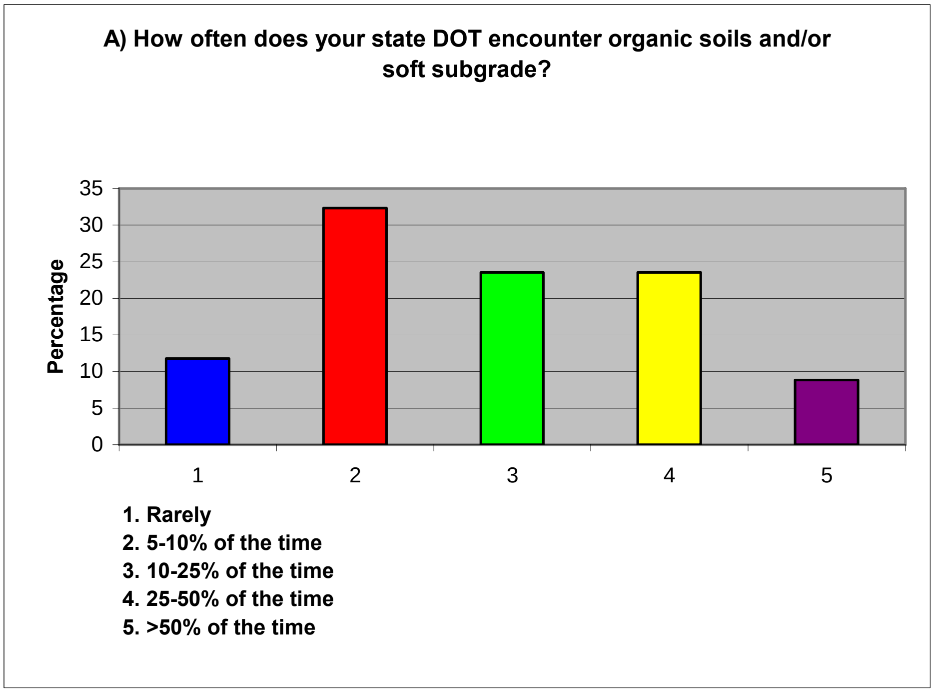


Figure 8.2: Results and Analysis of Survey Question A

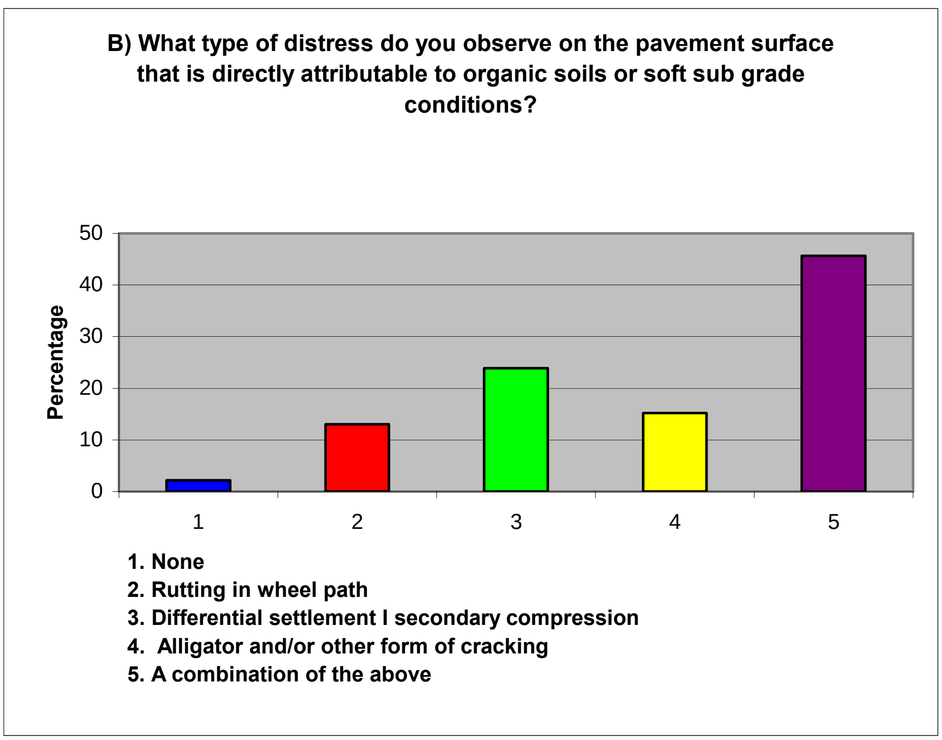


Figure 8.3: Results and Analysis of Survey Question B

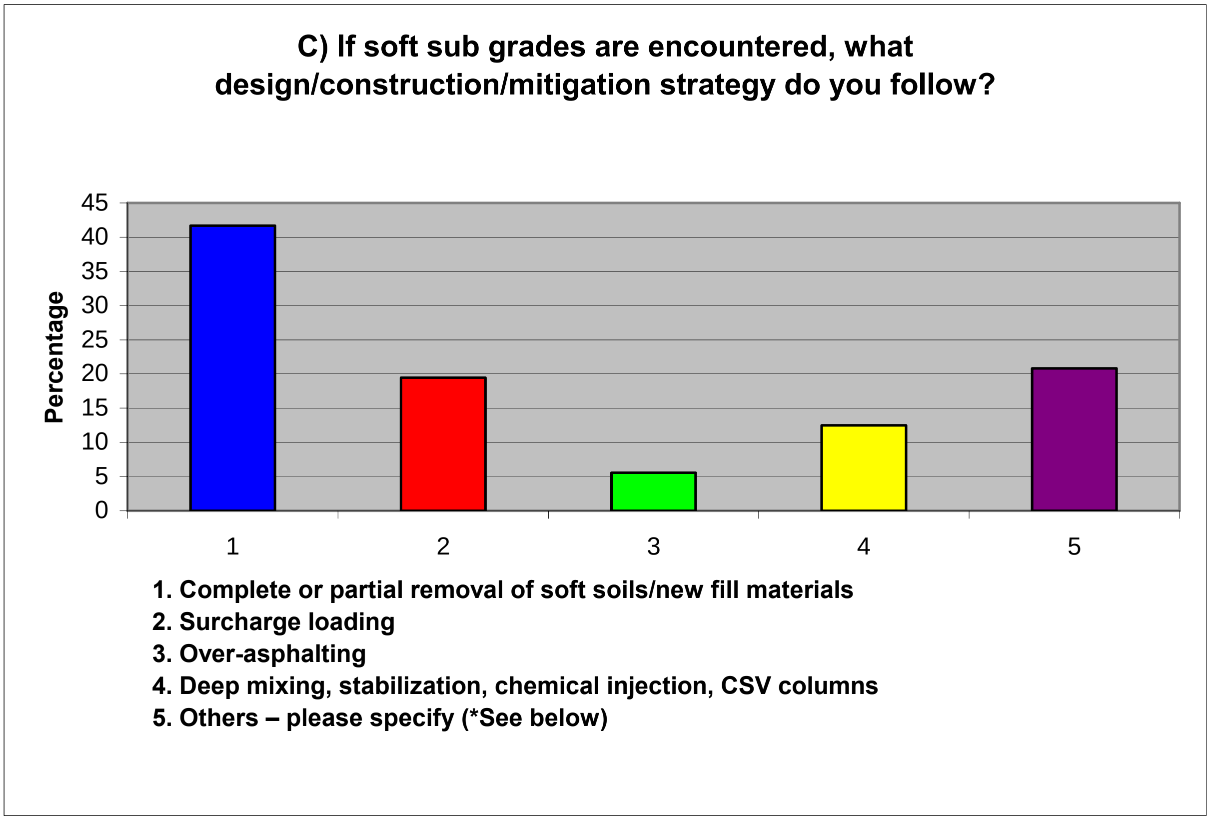


Figure 8.4: Results and Analysis of Survey Question C

<u>Other Responses / Comments</u>

*Dig out.
Better Drainage, geosynthetics
Method depends on project, location and type of construction.
Dewatering (wick drains)-1 project only
Fabric reinforcement.
Geogrids and Geosynthetics
Lightweight fill.
LWF (lightweight fill), slag, expanded shale, extruded polysterene, and cellular geosynthetic with LWF.
Geotextiles to separate new material from soft subgrade (prevent mixing).
Combination of above with fabrics 1 and stabilization with lime/cement.
Granular drains, full depth reclamation with fly ash
Lime or cement.
N/A
Geotextiles as part of the subgrade.
Geosynthetic separation/reinforcement.
Geosynthetic reinforcement.

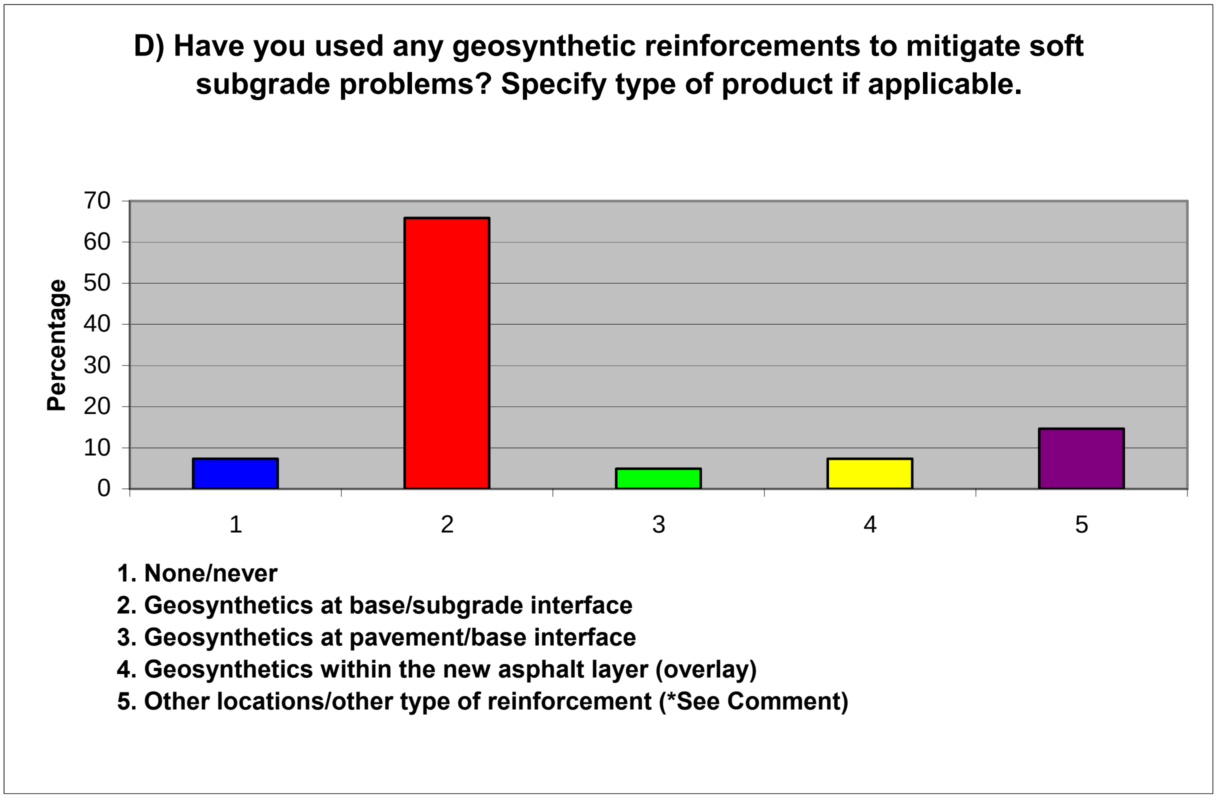


Figure 8.5: Results and Analysis of Survey Question D

Other Responses / Comments

*We have used geosynthetics at subbase/subgrade interface-doesn't work well.
Georgird in lower 1/3 of aggregate base course.
MDT typically subexcavates and replaces subgrade soils to a depth of 2 feet when soft soils are encountered. Often, a geogrid and non-woven geosynthetic are placed in the bottom of the subexcavation.
Geogrid or engineering fabric under the subgrade

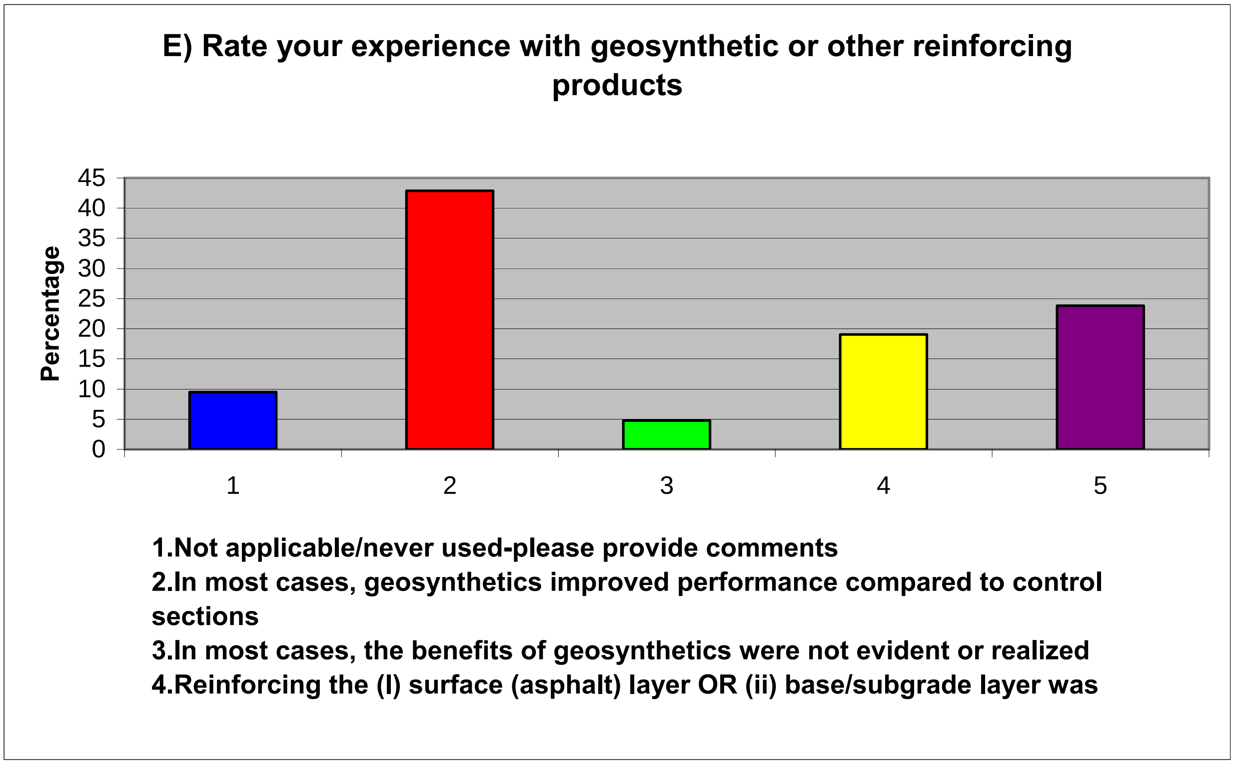


Figure 8.6: Results and Analysis of Survey Question E

Other Responses / Comments

*OK for emergency passage over soft subgrades.
We do find that we get benefit from the use of geosynthetics but we have not placed them next to control sections.
Our primary experience with geosynthtic reinforcement has been using them as SAMIs between an old PCC pavement and asphalt overlay, the results never impressed us.
Never Compared
MDT typically subexcavates and replaces subgrade soils to a depth of 2 feet when soft soils are encountered. Often, a geogrid and non-woven geosynthetic are placed in the bottom of the subexcavation. This makes constructing pavement in soft soil environments easier. The long term benefit is unknown.
Greatly depends on situation. Very useful in property stability for stable platform to work on better use of fabrics then grids.
Stability and drains have worked well and have been cost effective
Stabilization is generally more cost effective.
The geosynthetics haven not been in place long enough to evaluate. Did assist in providing construction platform.

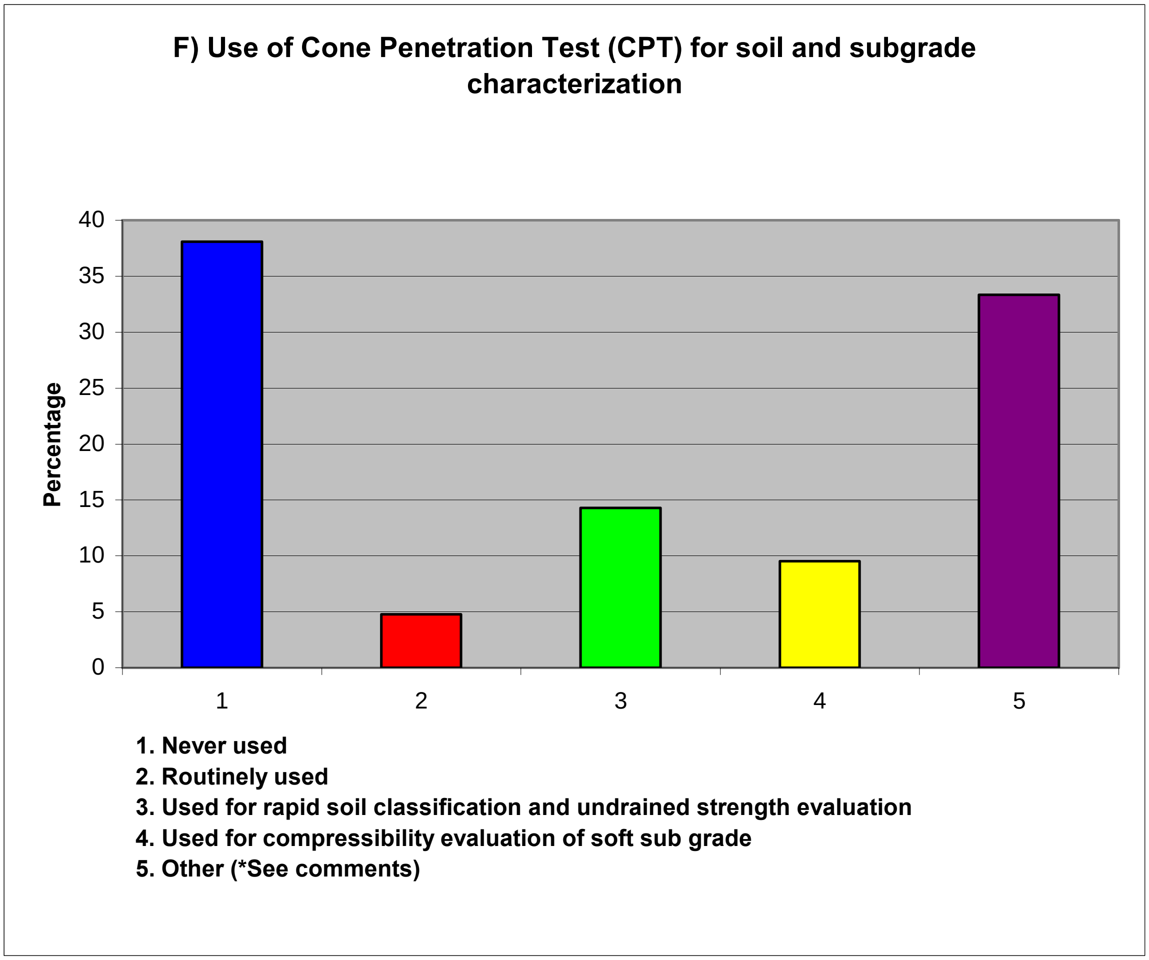


Figure 8.7: Results and Analysis of Survey Question F

<u>Other Responses / Comments</u>

*Use FWD to get resilient modulus for design soft soils typically = low modulus values.
Although it is used mostly for bridge/wall structures-rarely, if ever, for pavement projects.
Great in field.
We use FWD data for our recycling projects
Occasionally
Rarely used, not widely applicable in NYC soils.
Research tool at this time.
We have used DCP instead of CPT.
Soil characterization for geotechnical design for structures and embankments.
Limited use for subgrade evaluation-not effective in stony soils.
We occasionally use the DCP to check for PCC rubbilization suitability and as construction acceptance for chemically modified subgrade compaction.
Recently started doing dynamic cone penetrometer. Have used in place CBR on several projects.

CHAPTER 9

Summary and Conclusions

Presence of soft compressible organic soils and peat below the SR 15 / US 98 highway has been the major underlying cause of recurring pavement distresses in the form of cracking, rutting, and long-term secondary settlement. A research study was undertaken with the following broad objectives: (1) to identify and evaluate effective products and/or techniques which could be readily applied to the pavement surface course for extending the life of the roadway built over organic layers; and (2) to develop methodologies and guidelines for design implementation. A desired outcome of this research is to establish links between the geotechnical site-specific conditions and the probable pavement performance, so that an appropriate rehabilitation strategy can be adopted for future reconstruction projects in which organic subgrades are encountered. To achieve these goals, a coordinated research approach was established leading to the following accomplishments:

1. A comprehensive literature search and a State-of-the-Practice survey were conducted, and three promising asphalt reinforcement products were identified for use in the pilot field test sections along SR 15 / US 98. These products are (i) PaveTrac MT-1 steel reinforcing mesh manufactured by Bekaert Corporation; (ii) GlasGrid 8501 manufactured by Saint-Gobain Technical Fabrics; and (iii) PetroGrid Style 4582 manufactured by Propex Fabrics.

2. Piezocone Penetration Tests (CPTu) with Porewater Dissipation Experiments were conducted at nine locations for rapid on-site evaluation of the strength and compressibility characteristics of Florida organic soils and peats. A concurrent laboratory consolidation test program (including secondary compression tests) were conducted to validate the field test results.
3. The site-specific soil conditions (moisture content, organic content, thickness and unit weight of the organic layer) were expressed by an Organic Factor (F_{org}), which was correlated with pertinent soil properties, mechanistic pavement response parameters, and probable pavement performance.
4. Three experimental pavement sections, each 500-feet long and incorporating one of the candidate reinforcing products, were designed at 2 different locations with different Organic Factors (F_{org}) in order to develop direct correlations between F_{org} and pavement performance, and also to facilitate relative comparisons of the candidate solutions. The sections are repeated in both Northbound and Southbound lanes for incorporating directional variations (if any) in traffic weight and volume. Each location also includes 4 control sections containing no reinforcements, and two sections containing Asphalt Rubber Membrane Interlayer (ARMI).
5. A conceptual design framework is developed linking laboratory/field investigations, site-specific parameters, mechanistic analysis, and transfer functions for the prediction of pavement performance (life), so that an appropriate strategy can be adopted in future reconstruction projects.

As part of the laboratory experimental program, the Time-Stress-Compressibility relationships were investigated over a stress range (σ_v'/σ_p') of 0.3 to 1.15 encompassing both the recompression and compression zones. It was found that the (C_α / C_c) ratio at any stress level has constant values ranging from 0.03 to 0.05, which are consistent with the values reported in the literature for similar soils. The in-situ Coefficient of Consolidation, and the Compression Index were predicted from the CPTu data, both of which showed reasonable correlation with the laboratory derived values. These findings are significant because they imply that the C_c values predicted from CPTu data may also be used to estimate C_α (since C_α / C_c is constant), and hence the long-term secondary settlement, thus avoiding lengthy and expensive laboratory testing protocols. Considering the inherent difficulty in sampling and laboratory testing of undisturbed soft organic soils, Piezocone Penetration Tests show promise as an efficient tool for relatively rapid in-situ characterization of subsoil strength, modulus, and compressibility, which in turn may be used for forensic interpretations of pavement failures, mechanistic analysis, and validation of pavement performance models.

CHAPTER 10

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APPENDIX A

1-D COMPRESSION vs. TIME

SITE 1 – STA 155 + 00

SECONDARY COMPRESSION

1.83 – 2.44 meters - Silty Muck
$\sigma'_v/\sigma'_p = 0.60$

2.44 – 3.05 meters - Fibrous Peat
$\sigma'_v/\sigma'_p = 0.30$

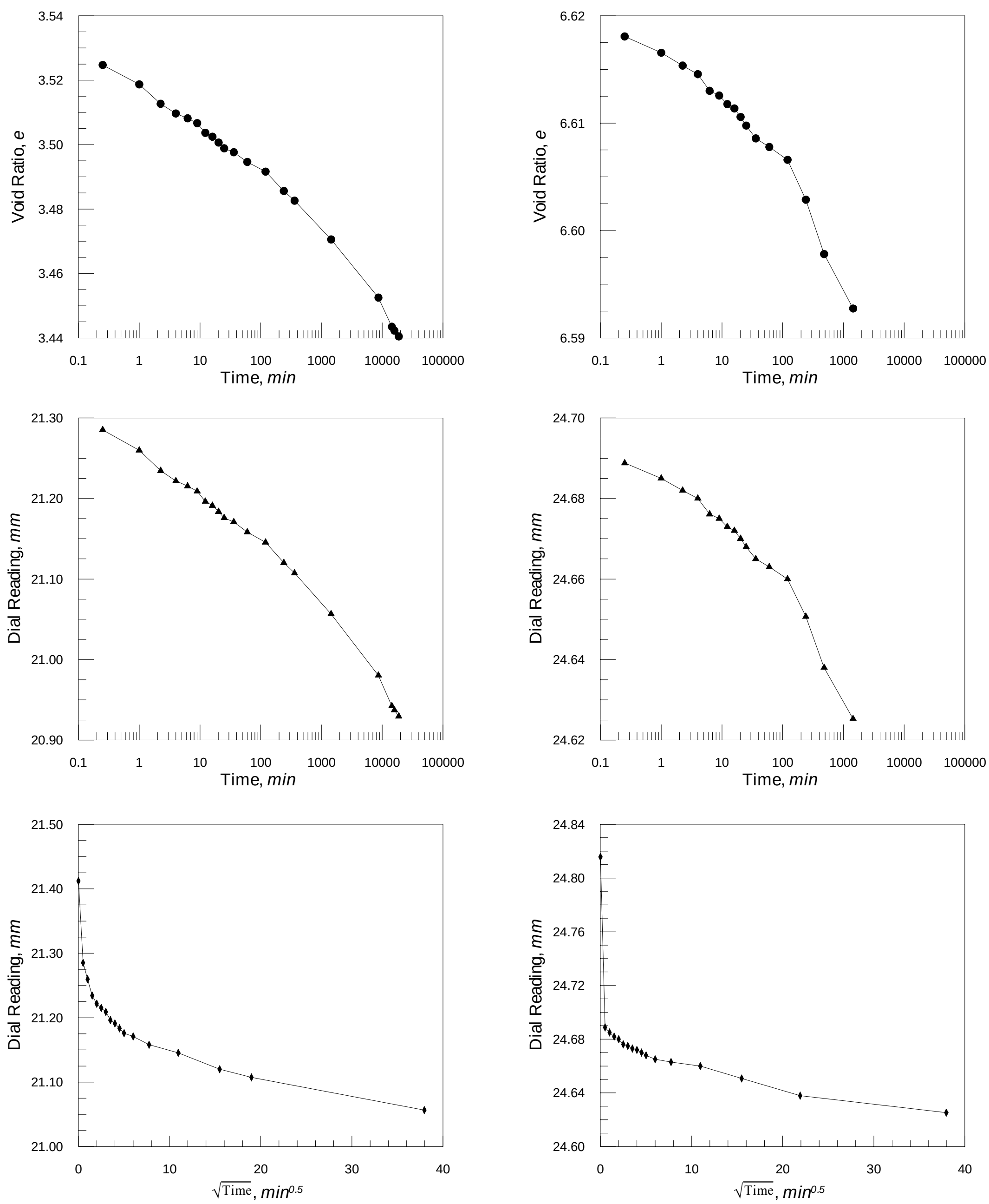


Figure A-1: Compression Behavior of Florida Organic Soils with Respect to Time at SITE 1

SITE 2 – STA 157 + 00
SECONDARY COMPRESSION

1.83 – 2.44 meters - Silty Muck
$\sigma'_v/\sigma'_p = 0.30$

2.44 – 3.05 meters - Fibrous Peat
$\sigma'_v/\sigma'_p = 0.60$

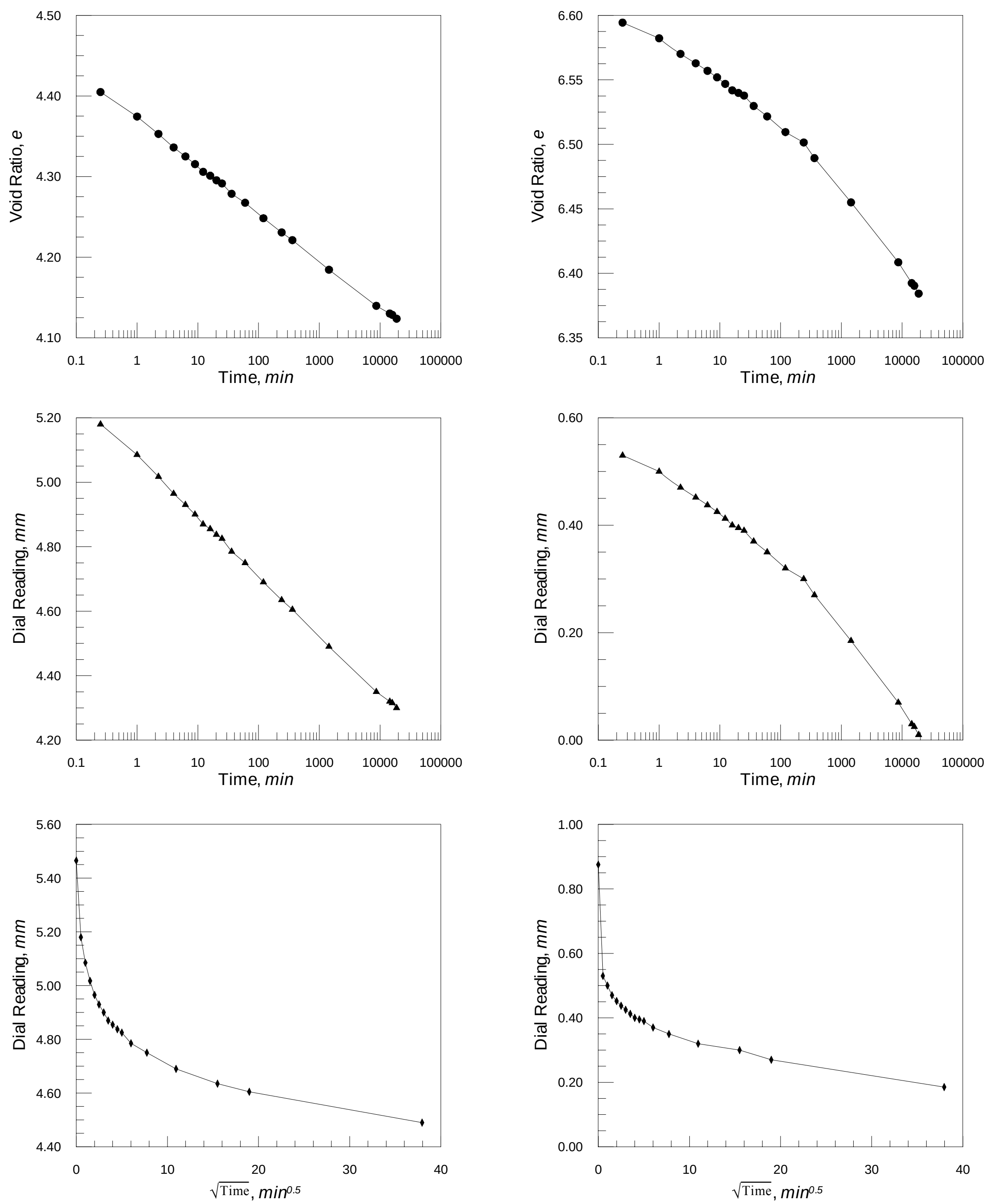


Figure A-2: Compression Behavior of Florida Organic Soils with Respect to Time at SITE 2

SITE 3 – STA 159 + 00
SECONDARY COMPRESSION

1.83 – 2.44 meters - Silty Muck
σ'_v/σ'_p = 0.60

2.44 – 3.05 meters - Fibrous Peat
σ'_v/σ'_p = 0.60

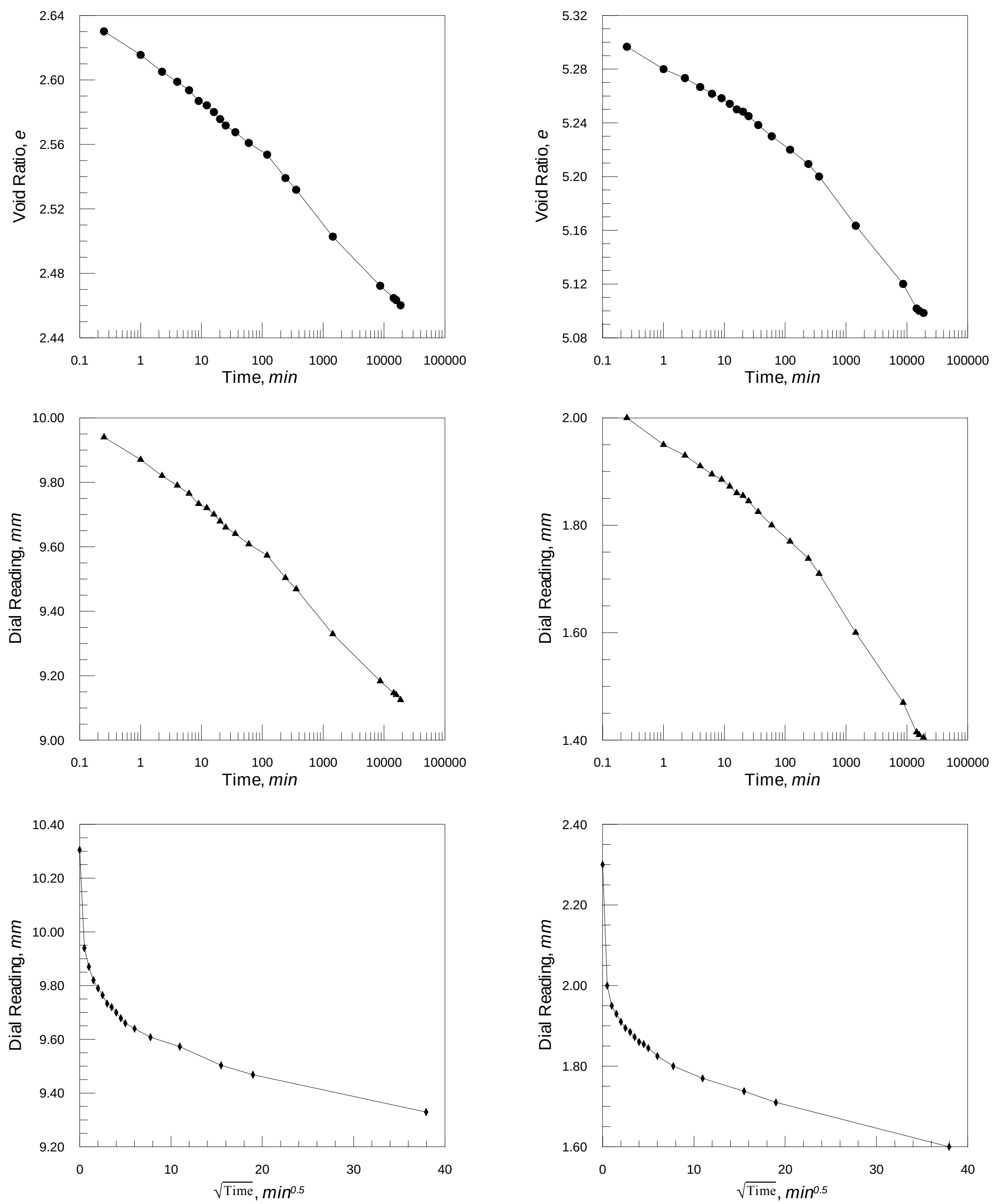


Figure A-3: Compression Behavior of Florida Organic Soils with Respect to Time at SITE 3

SITE 4 – STA 163 + 00
SECONDARY COMPRESSION

2.13 – 2.74 meters - Silty Muck
σ'_v/σ'_p = 0.60

3.66 – 4.27 meters - Fibrous Peat
σ'_v/σ'_p = 0.60

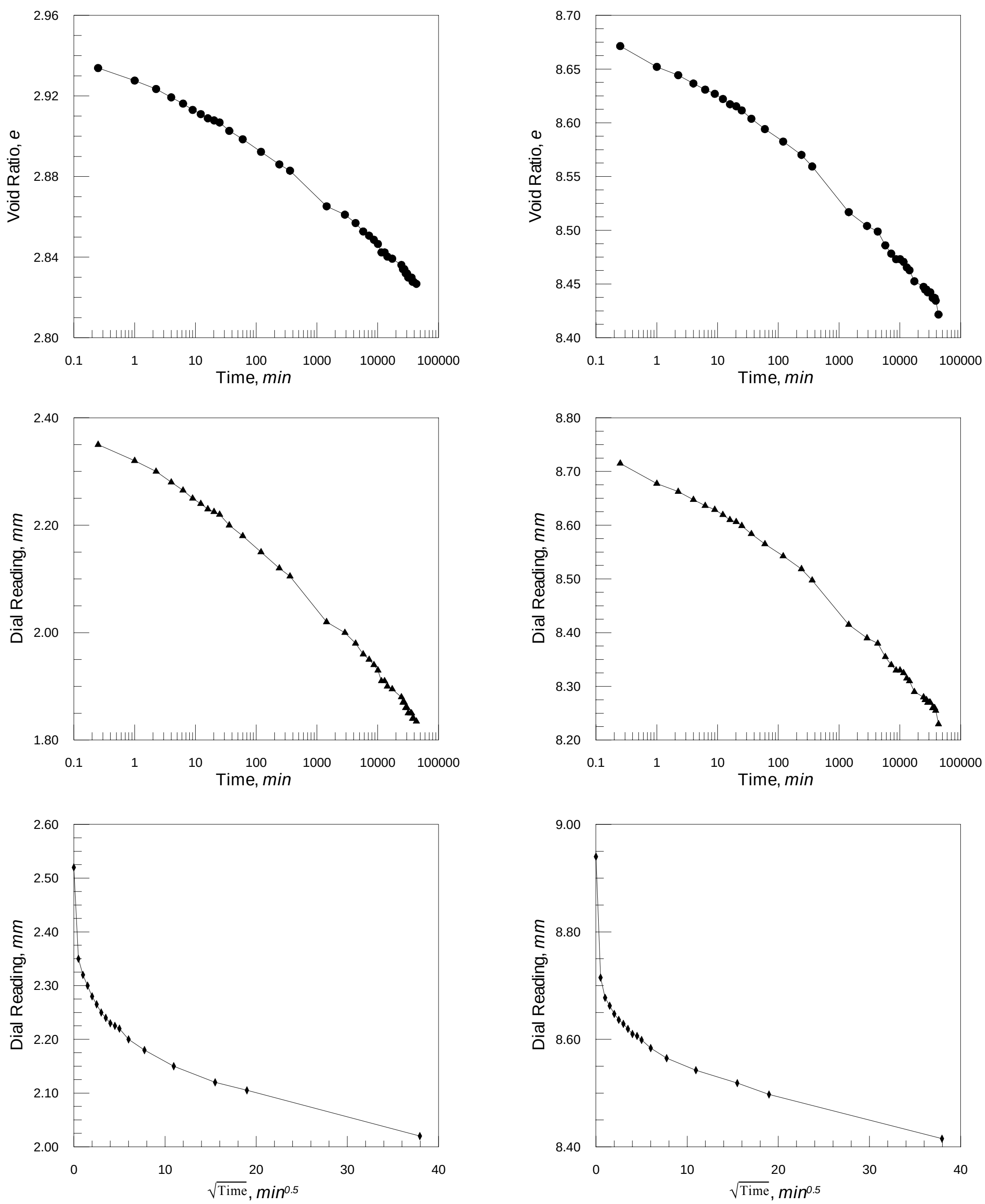


Figure A-4: Compression Behavior of Florida Organic Soils with Respect to Time at SITE 4

SITE 5 – STA 171 + 00
SECONDARY COMPRESSION

2.13 – 2.74 meters - Silty Muck
σ'_v/σ'_p = 1.00

3.66 – 4.27 meters - Fibrous Peat
σ'_v/σ'_p = 1.15

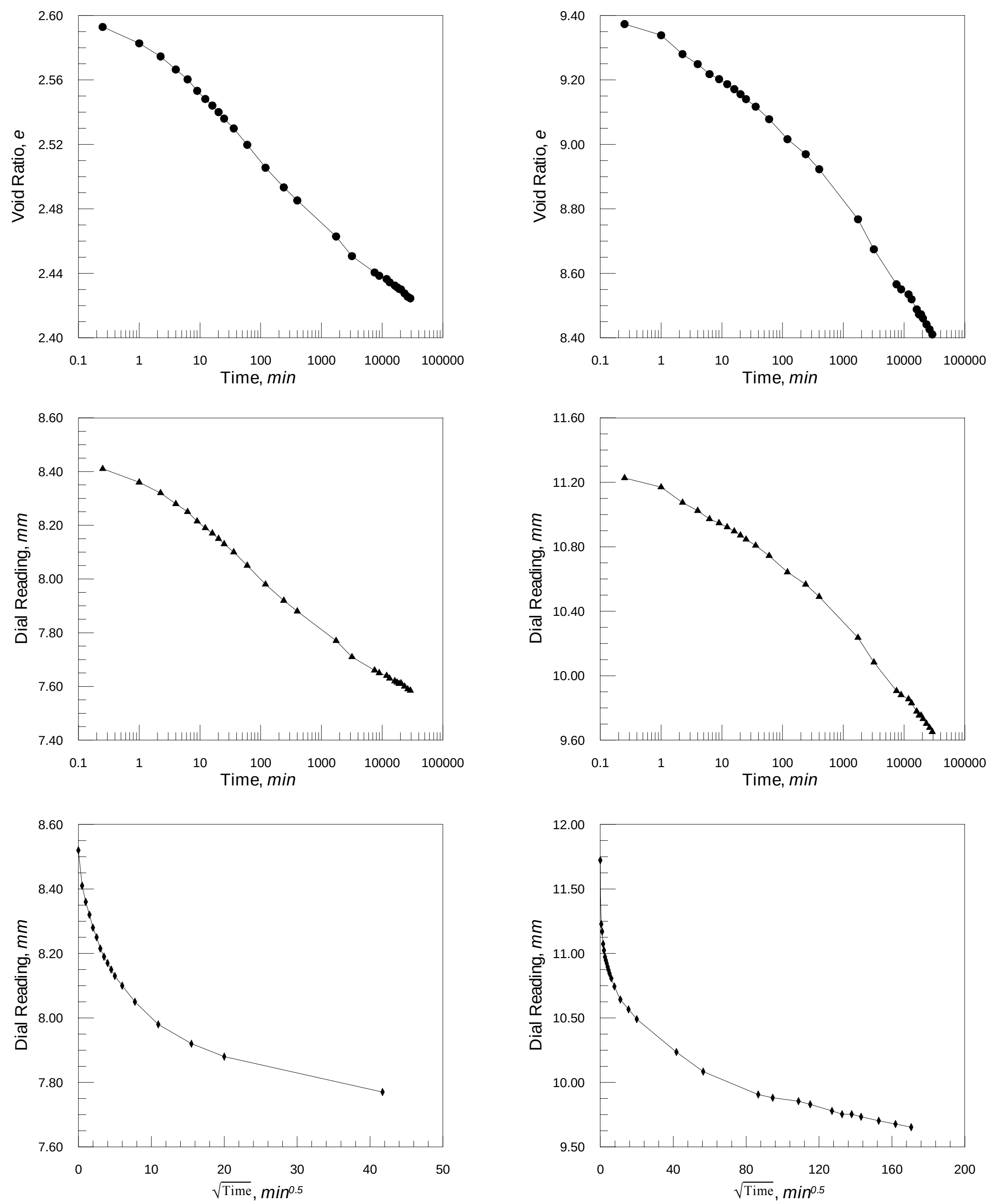


Figure A-5: Compression Behavior of Florida Organic Soils with Respect to Time at SITE 5

SITE 6 – STA 227 + 00
SECONDARY COMPRESSION

2.13 – 2.74 meters - Silty Muck
σ'_v/σ'_p = 1.15

3.66 – 4.27 meters - Fibrous Peat
σ'_v/σ'_p = 1.00

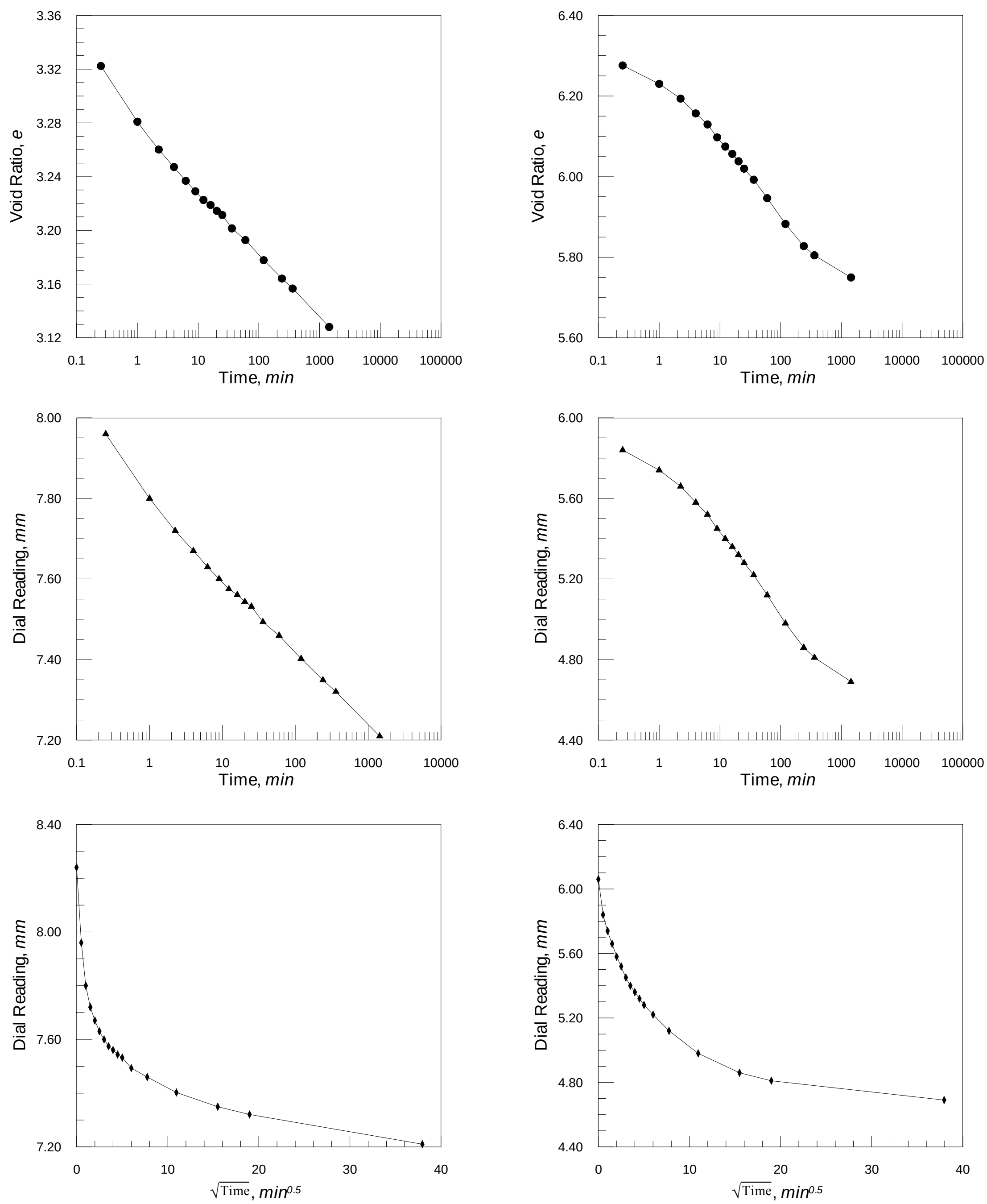


Figure A-6: Compression Behavior of Florida Organic Soils with Respect to Time at SITE 6

SITE 7 – STA 233 + 00
SECONDARY COMPRESSION

2.13 – 2.74 meters - Silty Muck
σ'_v/σ'_p = 1.00

3.66 – 4.27 meters - Fibrous Peat
σ'_v/σ'_p = 1.00

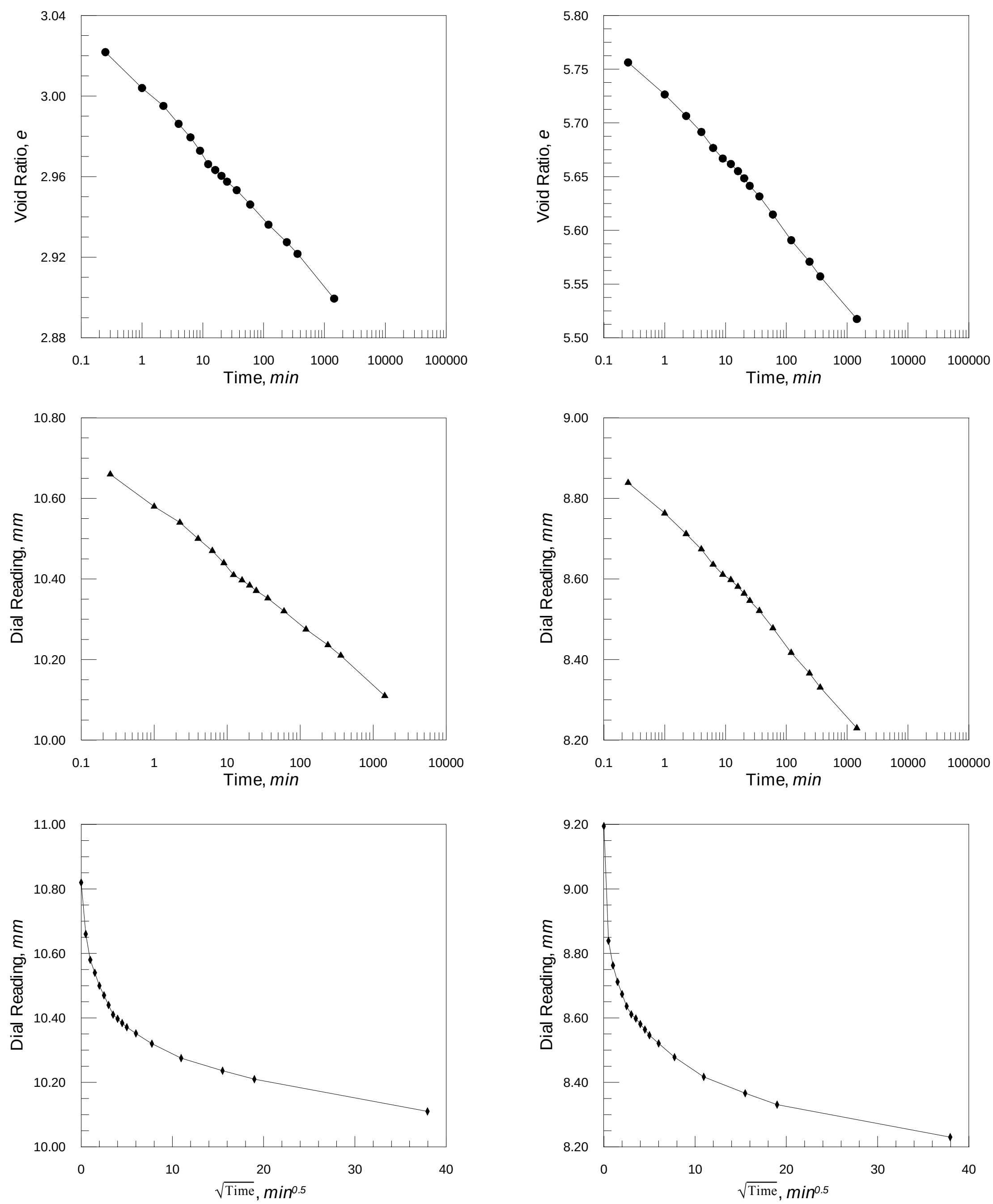


Figure A-7: Compression Behavior of Florida Organic Soils with Respect to Time at SITE 7

SITE 8 – STA 239 + 00
SECONDARY COMPRESSION

2.13 – 2.74 meters - Silty Muck
σ'_v/σ'_p = 0.60

3.66 – 4.27 meters - Fibrous Peat
σ'_v/σ'_p = 0.60

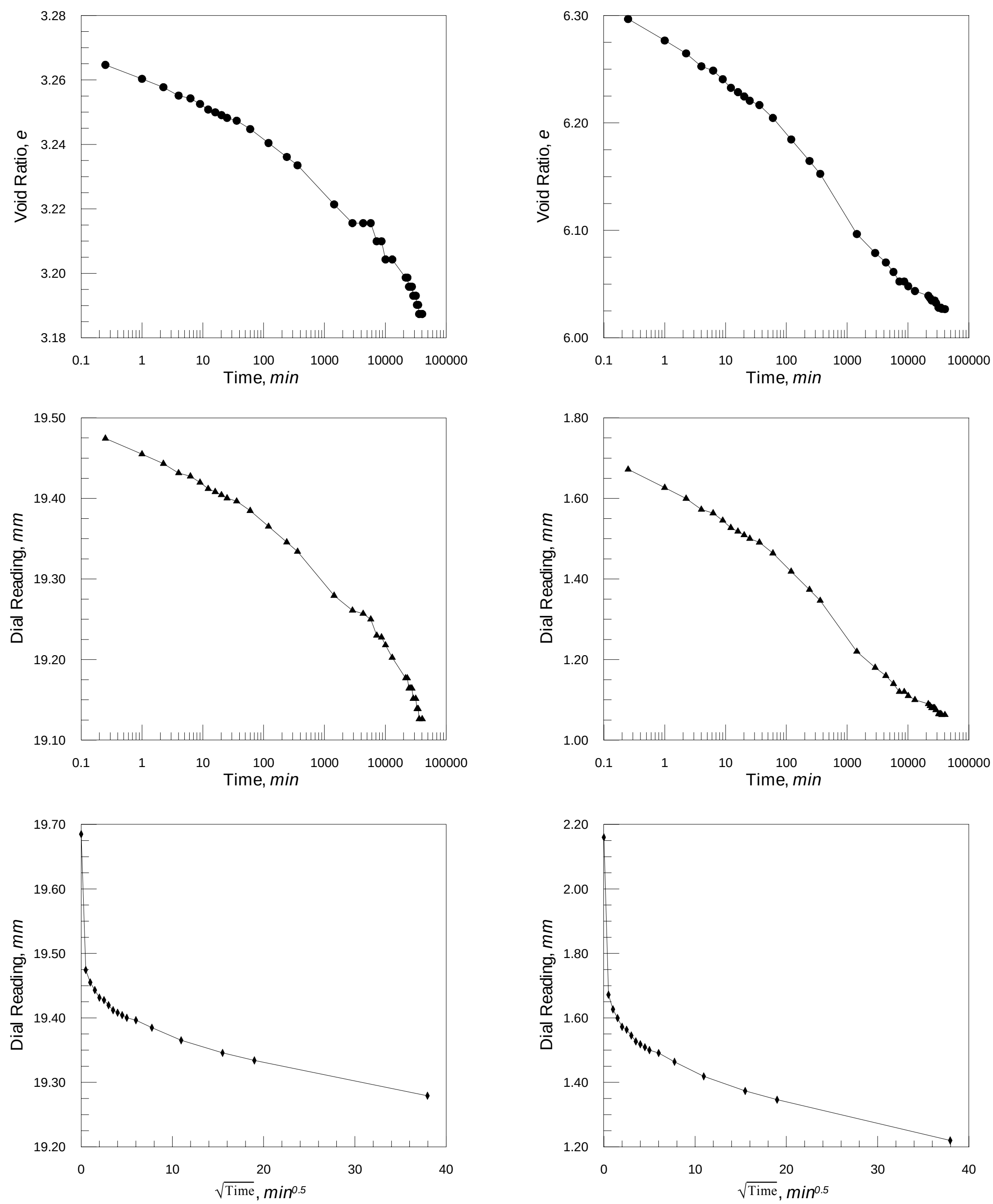


Figure A-8: Compression Behavior of Florida Organic Soils with Respect to Time at SITE 8

SITE 9 – STA 249 + 00
SECONDARY COMPRESSION

2.13 – 2.74 meters - Silty Muck
σ'_v/σ'_p = 1.15

3.66 – 4.27 meters - Fibrous Peat
σ'_v/σ'_p = 1.00

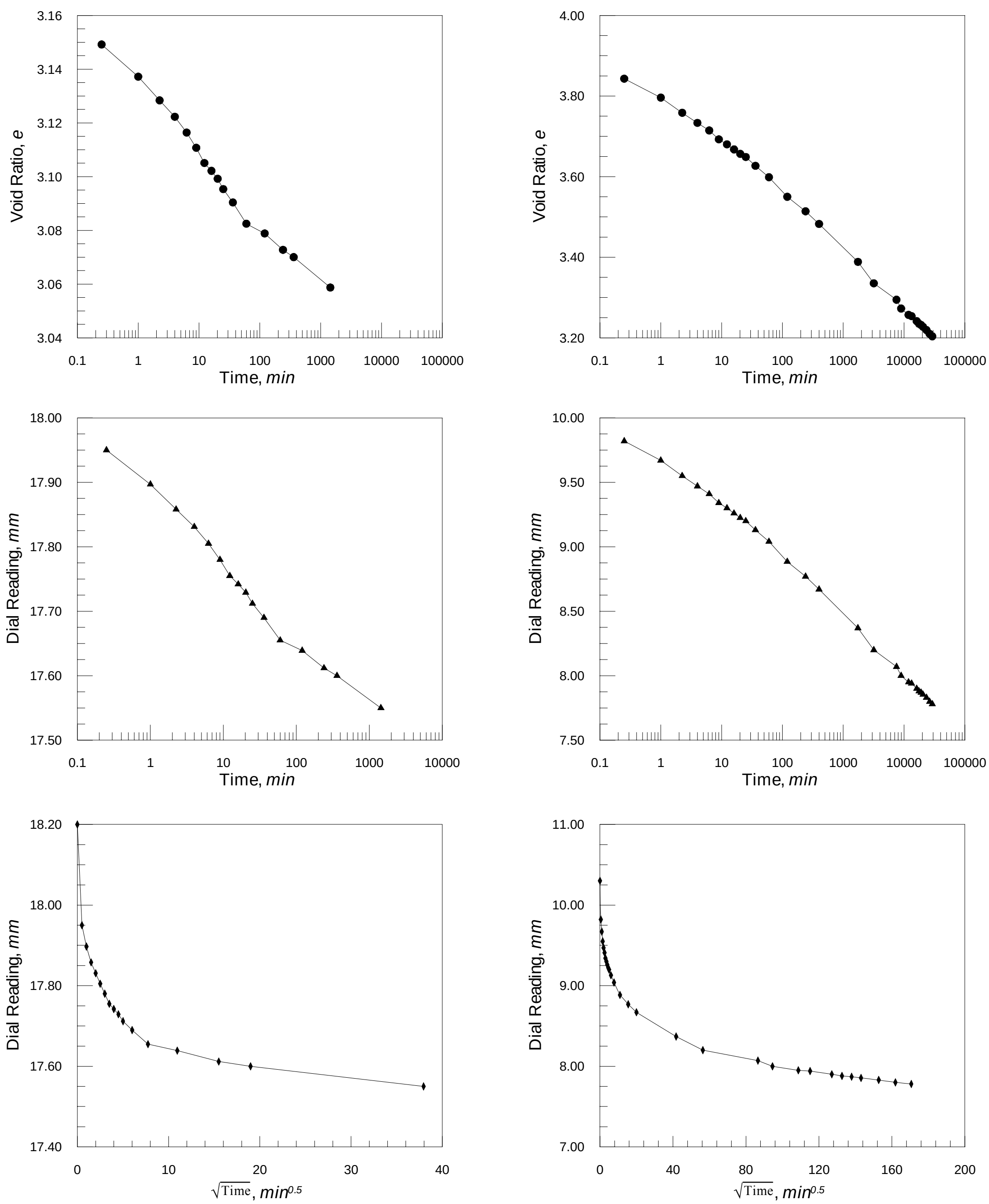


Figure A-9: Compression Behavior of Florida Organic Soils with Respect to Time at SITE 9

SITE 10 – STA 253 + 00
SECONDARY COMPRESSION

2.13 – 2.74 meters - Silty Muck
$\sigma'_v/\sigma'_p = 1.00$

3.66 – 4.27 meters - Fibrous Peat
$\sigma'_v/\sigma'_p = 1.15$

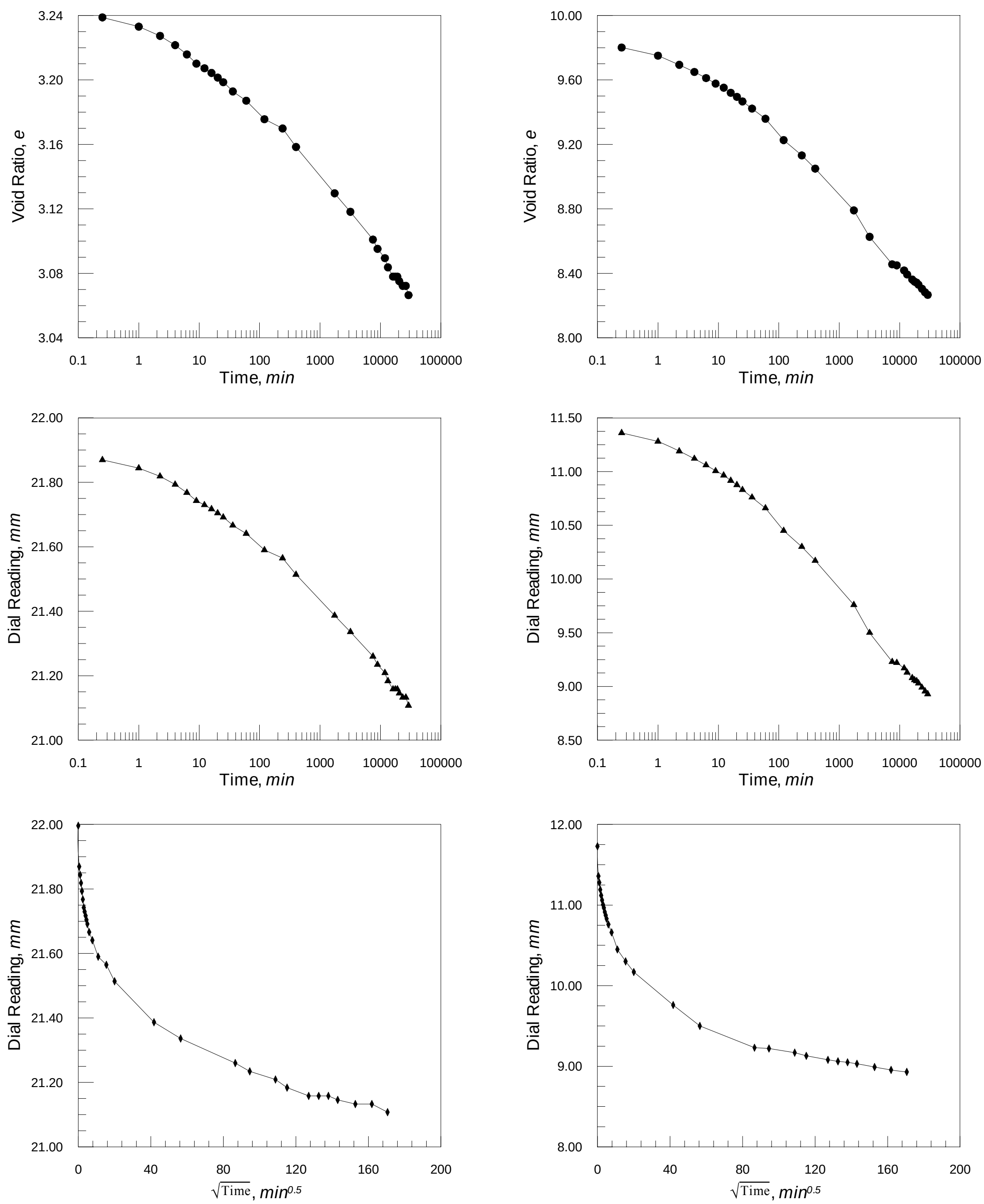


Figure A-10: Compression Behavior of Organic Soils with Respect to Time at SITE 10

SITE 11 – STA 259 + 00
SECONDARY COMPRESSION

2.13 – 2.74 meters - Silty Muck
σ'_v/σ'_p = 0.60

3.66 – 4.27 meters - Fibrous Peat
σ'_v/σ'_p = 0.60

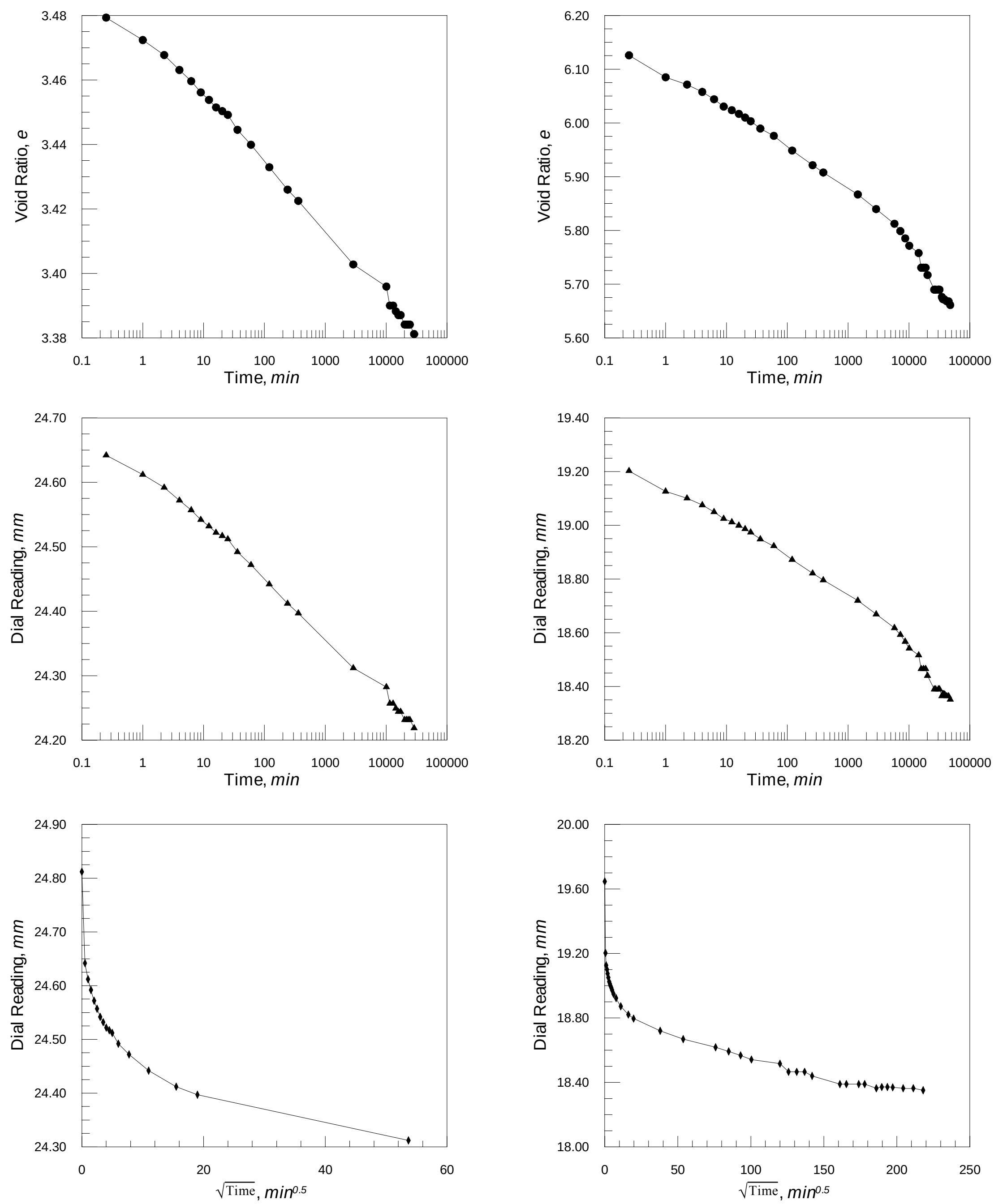
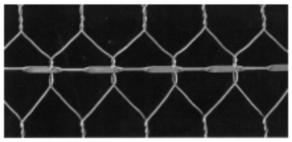


Figure A-11: Compression Behavior of Organic Soils with Respect to Time at SITE 11

APPENDIX B

Product Specifications



PRODUCT DATA SHEET **BEKAERT**

PaveTrac® 1
Asphalt reinforcement

• Description:

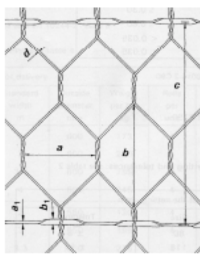
PaveTrac® is a woven hexagonal wire netting manufactured of Bezinol® coated steel wire reinforced in the transverse direction at regular intervals by flat alternately twisted reinforcing wires interwoven in the mesh.

• Applications:

PaveTrac® is used as reinforcement netting for road construction and foundations when recovering concrete or asphalt roads.

• Geometry:

a =	3.15 in	(80 mm)
b =	4.65 in	(18 mm)
c =	9.65 in	(245 mm)
d =	0.0965 in	(2.45 mm)
a_1 =	0.118 in	(3 mm)
b_1 =	0.276 in	(7 mm)



• Tensile Strength:

- on weaving wire : min. breaking load 405 lb (1,800 N)
- on flat bar : min. breaking load 2,700 lb (12,000 N)

• Coating:

Bezinol® (95% Zn – 5% Al)

- on weaving wire : minimum 125 g/m^2
- on flat bar : minimum 80 g/m^2

• Approvals:

Conforms to
ASTM A856

Quality System in American Plants

ISO 9001

Recommendations – Installation

1. General
   - ✓ Thoroughly clean the pavement.
   - ✓ Apply leveling course, if required, when holes are over 1 in deep.
   - ✓ When placing on concrete, crack and seat unstable concrete, determined by falling weight deflectometer (FWD) test.
   - ✓ Do not use slurry seal during periods of prolonged rain or in below freezing temperatures.

2. PaveTrac® Installation
   - ✓ Unroll the mesh from the top of the roll. The steel mesh should curve downward.
   - ✓ Flatten the steel mesh. A rubber-tired roller is recommended.
   - ✓ Fix the first transverse torsioned flat bar to the road surface. Use recommended nails and hook clips.
   - ✓ Overlap roll ends up to one aperture (c-distance) or 12-inch maximum.
   - ✓ Overlap roll edges a minimum of 12 inches and maximum of 30 inches. Do not overlap flat bars.
   - ✓ Fix the mesh to the road surface. A rapid set polymer modified emulsion (slurry seal) is recommended.
   - ✓ Refer to manufacturer for alternate fixing methods.
   - ✓ Apply a light tack coat before slurry sealing over concrete.

3. HMA Overlay Installation
   - ✓ Place HMA overlay. The HMA thickness over the mesh shall be in accordance with the overlay design and shall not be less than 2 inches.

Recommendations – Handling

Delivered in units with 4 rolls strapped side by side with dunnage.

Widths	Length	Weight
6.6 ft. (2 m)	164 ft. (50 m)	381 lbs (173 kg)
9.8 ft. (3 m)	164 ft. (50 m)	573 lbs (260 kg)
10.8 ft. (3.3 m)	164 ft. (50 m)	628 lbs (285 kg)
13.1 ft. (4 m)	164 ft. (50 m)	760 lbs (345 kg)

N.V. Bekaert S.A. – Bekaertstraat 2 – 8550 Zwevegem – Belgium
Tel. +32 (0) 56 / 76 69 86 Fax +32 (0) 56 / 76 79 47

Bekaert Corporation – 2000 Isaac-Shelby Drive – Shelbyville Kentucky 40065 – Phone 800-372-6940 - Fax 502-633-1561

PetroGrid®

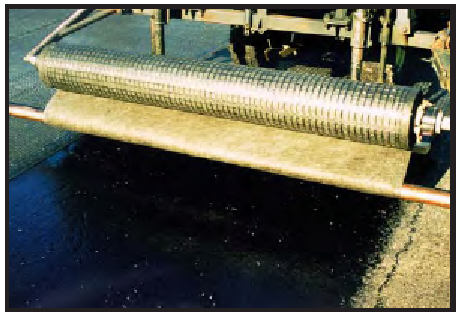
INTRODUCTION

PetroGrid Style 4582 is a pavement interlayer composite made from a nonwoven, needle-punched, polyester paving fabric laminated to a heavy duty structural grid composed of glass fiber embedded in polymeric resin. When combined with an asphalt cement tack coat, the fabric side of the composite provides the moisture barrier and stress absorption functions. The high modulus grid on the top side of the composite structurally reinforces the overlay. These combined functions of PetroGrid help extend overlay service life by minimizing reflective cracking due to crack and joint movement in the underlying pavement.

Close-up of PetroGrid installed over tack coat

BENEFITS

- Minimizes water infiltration as a pavement moisture barrier
- The overlay reinforcing grid achieves its full strength with less than 5% elongation, to minimize reflective cracking
- Pavement layer bonding enhanced by the asphalt tack coat and by interlock of the overlay with the structural grid
- The paving fabric on the composite, meets national guideline specification, AASHTO M 288
- The excellent bond to the fabric and high junction strength give the reinforcing grid dimensional stability
- Convenient width,easy to handle and install, and compatible with traditional paving fabric installation



Installation of PetroGrid on road surface

Table 1. Physical Properties - Style 4582

Property	Test Method	American Standard	Metric
Unit Weight	-	16 oz/yd^2	542 g/m^2
Tensile Strength[1]	GSI/GG1 or ASTM D 6637	560 lbs/in	100 kN/m
Ultimate Elongation	GSI/GG1 or ASTM D 6637	< 5%	
Strength at 2% Strain[1]	GSI/GG1 or ASTM D 6637	280 lbs/in	50 kN/m
Grid Junction Strength[2]	GSI/GG-2	18 lbs	80 N
Peel Strength[1]	ASTM D 413	10 lbs/ft	146 N/m
Aperture Size[3] MD/XD	-	.98/.98 in	24.9/24.9 mm
Width[4]	-	80 in	2.03 m
Length	-	60 lin yds	54.8 lin m
Nonwoven Paving Fabric[5]			
Grab Tensile[1]	ASTM D 4632	101 lbs	450 N
Elongation[1]	ASTM D 4632	>50%	>50%
Asphalt Retention	ASTM D 6140	0.2 gal/yd^2	0.91 l/m^2

Notes: 1 minimum average roll values (MARV) 2 Tested with grid adhered to fabric 3 centerline to centerline 4 Width is grid plus 2 fabric edge (148 in. wide rolls are also available). 5 meets AASHTO M-288/paving fabric specifications
MD-machine direction XD-cross-machine direction

PETROGRID® 4582

GLASS GRID COMPOSITE PAVEMENT INTERLAYER
GUIDE SPECIFICATION

DESCRIPTION - This work shall consist of furnishing and placing a paving fabric/structural grid composite as a full width or strip interlayer over existing pavement prior to placement of an asphalt concrete overlay. The composite shall be installed as indicated on the plans and contract documents.

MATERIAL REQUIREMENTS - Composite: The composite paving material shall consist of a nonwoven polyester paving fabric bonded to an epoxy coated glass fiber structural grid. The composite shall meet the physical properties of the product specified on Table 1.

Tack Coat: Uncut asphalt cement is strongly preferred for the tack coat. An asphalt emulsion may be used only if approved by the Engineer. The Contractor shall follow the special recommendations of the paving composite manufacturer when an asphalt emulsion is used. The use of cutbacks or emulsions that contain solvents shall not be permitted.

USE - The paving composite interlayer system is used as a moisture barrier within a pavement and as a stress absorbing membrane interlayer. The glass fiber structural grid interlocks with the overlay providing structural reinforcement to resist cracking in the overlay.

CONSTRUCTION AND INSTALLATION REQUIREMENTS - The paving composite interlayer system is installed in much the same way as an unreinforced paving fabric system.

Shipping and Storage: The paving composite shall be kept dry and wrapped such that it is protected from the elements during shipping and storage. If stored outdoors, the fabric shall be elevated and protected with a waterproof cover.

Weather Limitations: The air and pavement temperatures shall be at least 50°F (10°C) and rising for placement of asphalt cement and shall be at least 60°F (16°C) and rising for placement of asphalt emulsion. Neither asphalt tack coat nor paving composite shall be placed when weather conditions are not suitable, in the opinion of the Engineer.

Surface Preparation: The pavement surface shall be dry and cleaned of all dirt and oil to the satisfaction of the Engineer. Cracks wider than 1/8 inch (3 mm) wide shall be cleaned and filled with suitable bituminous material approved by the Engineer. Potholes and locally failed and cracked pavement sections shall be repaired as directed by the Engineer. If milling or existing rough pavement exists, then a leveling course shall be placed prior to installation of the paving composite interlayer system.

Tack Coat Application: The tack coat shall be applied using a calibrated distributor spray bar. Hand spraying, squeegee and brush application may be used only in locations where the distributor truck cannot reach. The tack coat shall be uniformly applied at a rate sufficient to saturate the fabric and to bond the fabric to the existing pavement surface. The tack coat application rate shall be typically 0.25 gallons per square yard (1.14 liters per square meter), with a range of 0.23 to 0.27 gallons per square yard (1.05 to 1.2 liters per square meter) as required by the roadway surface and environmental conditions. If the engineer approves the use of emulsions, the application rate must be increased the appropriate amount to offset the water content of the emulsion. The temperature of the tack coat shall be sufficiently high to permit a uniform spray pattern. Asphalt cements shall be sprayed at temperatures between 290°F (143°C) and 325°F (163°C). For asphalt emulsions, the distributor tank temperatures shall be maintained between 130°F (55°C) and 160°F (71°C). The target width of the tack coat application shall be equal to the paving fabric width plus 6 inches (152 mm). The tack coat shall be applied only as far in advance of paving composite installation as is appropriate to ensure a tacky surface at the time of paving composite placement. Traffic shall not be allowed on the tack coat. Excess tack coat shall be cleaned from the pavement.

Paving Composite Placement: The paving composite shall be placed onto the tack coat using mechanical or manual laydown equipment capable of providing a smooth installation with a minimum amount of wrinkling or folding. Unlike traditional paving fabric, this composite cannot be stretched or continuously placed around curves. The paving composite must be cut and realigned to place on curves. Paving fabric shall not be installed in areas where the overlay asphalt concrete tapers to a minimum compacted thickness of less than 1.5 inches (38 mm). When asphalt emulsions are used, the emulsion shall be allowed to cure properly such that essentially no water moisture remains prior to placing the paving composite. Composite wrinkles severe enough to cause folds shall be slit and laid flat. Brooming and/or rubber-tire rolling will be required to maximize paving composite contact with the pavement surface. Additional hand-placed tack coat may be required at overlaps and repairs as required by the Engineer. Turning and braking of the paver and other vehicles shall be done gradually and kept to a minimum to avoid movement and damage to the paving composite. Damaged composite shall be removed and replaced with the same type of composite and a tack coat.

Joints and Overlaps: At joints, composite rolls shall overlap by 2 to 6 inches (51 to 152 mm). End joints and joints from repair of wrinkles should be made to overlap or "shingle" in the direction that the pavement overlay will be placed. Excess composite shall be cut and removed to ensure that overlaps of adjacent rolls do not exceed 6 inches (152 mm). A uniform application of tack coat shall be applied between all composite overlaps.

All areas with paving fabric placed will be paved the same day. No traffic except necessary construction traffic shall be allowed to drive on the paving composite.

Overlay Placement: Asphalt overlay construction shall closely follow composite placement. Excess tack coat that bleeds through the paving composite shall be removed by broadcasting hot mix or sand on the paving composite. Excess sand or hot mix should be removed before beginning the paving operation. In the event of rainfall on the paving composite prior to the placement of the asphalt overlay, the paving composite must be allowed to dry completely before the overlay is placed. Overlay asphalt thickness shall meet the requirements of the contract drawings and documents. The minimum compacted thickness of the first lift of overlay asphalt concrete over the interlayer composite shall not be less than 1.5 inches (38 mm).

MEASUREMENT AND PAYMENT - The paving composite interlayer will be measured by the square yard (square meter). The accepted quantities of paving composite will be paid for at the contract unit price per square yard (square meter) in place.

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GlasGrid®

Pavement Reinforcement Mesh — contract specifications

ROAD REINFORCEMENT MESH

1. The reinforcement mesh shall be a knitted, glass fiber strand grid with the following characteristics:
   a. Tensile strength as per ASTM D 6637
      8501 - 100 kN/m x 100 kN/m* (560 lb/in x 560 lb/in**) component strand strengths.
      8502 - 200 kN/m x 100 kN/m (1120 lb/in x 560 lb/in) component strand strengths.
   b. Area weight as per ASTM D 5261-92
      8501 - 370 g/m² (11 oz/yd²) 8502 - 560 g/m² (16 oz/yd²)
   c. Coated with a modified polymer coating
   d. Elongation at break less than 5% as per ASTM D 6637
   e. Melt point above 218°C (425°F)
   f. The mesh will be self-adhesive, with sufficient bond to allow normal construction traffic and paving machinery operations.
   g. Mesh opening 12.5 mm x 12.5 mm (1/2" x 1/2")
2. Prior to laying the GlasGrid® mesh, the following surface treatment shall be carried out.
   a. Perform any remedial work such as base repairs, crack sealing, pothole filling, leveling course applications, etc., that would normally occur before an asphalt course overlay, as directed by construction engineer.
      NOTE: A leveling course is always recommended.
   b. The surface temperature before laying the grid shall be between 5°C and 60°C (40°F and 140°F).
   c. The surface shall be dry and free of dirt, swept or vacuum cleaned by a mechanical device, as well as freed of oil, vegetation and other debris.
3. It is optional to spray a tack coat below or onto the GlasGrid mesh. If a tack coat is sprayed below the GlasGrid it must be completely cured prior to the installation of GlasGrid. If tack coat is sprayed on top of GlasGrid the tack must either be fully cured or aggregate chips must be placed onto the grid prior to paving. The proper time for curing depends on the type of tack coat used and the environmental conditions at the time of construction.
   NOTE: It is important to correctly select the most appropriate type of tack coat and discuss this selection with the manufacturer's representative to properly address your specific project.
4. GlasGrid mesh shall be laid out either by hand or by mechanical means under sufficient tension to eliminate ripples. Should ripples occur, these must be removed by pulling the grid tight or in extreme cases (on tight radii), by cutting and laying flat. Transverse joints must be lapped in the direction of the paver by 75-150 mm (3-6 inches). Overlap longitudinal joints 25-50 mm (1-2 inches).
5. The surface of the GlasGrid mesh shall be rolled with a rubber coated drum roller, or pneumatic tired roller, one or two passes being sufficient to activate the adhesive. Tires must be cleaned regularly with asphalt cleaning agent.
6. Construction and emergency traffic may run on GlasGrid mesh after being rolled. However, it must be ensured that damage is not caused to the grid by vehicles turning or braking etc., and that the GlasGrid mesh must be kept clean of mud, dust and other materials. Damaged sections shall be removed and patched, taking care to underlap the full roll.
7. All GlasGrid mesh placed in a day shall be covered with asphalt concrete the same day, within permissible laying temperatures to a minimum compacted thickness of 40 mm (1.5 inches).
8. GlasGrid mesh must be stored in dry covered conditions free from dust and stocked vertically to avoid misshaped rolls.
9. GlasGrid mesh must be laid and rolled over ironworks or other obstructions before cutting around the perimeter of the obstructions. Cutting is achieved by using a sharp utility knife.
10. A representative of the manufacturer must be present during installation of this material and all work must be carried out in accordance with the manufacturer's specification.

***Please note this is a typical specification, which may be changed in appropriate circumstances, to suit your needs. No change may be made in the specification without first obtaining the written approval of Saint-Gobain Technical Fabrics. Final design thickness will depend on usage.**
Please consult your technical representative if any changes are required.

INASMUCH AS SAINT-GOBAIN TECHNICAL FABRICS (SGTF) HAS NO CONTROL OVER INSTALLATION DESIGN, INSTALLATION WORKMANSHIP, ACCESSORY MATERIALS, OR CONDITIONS OF APPLICATION, SAINT-GOBAIN TECHNICAL FABRICS DOES NOT WARRANT THE PERFORMANCE OR RESULTS OF ANY INSTALLATION OR USE OF GLASGRID. THIS WARRANTY DISCLAIMER INCLUDES ALL IMPLIED WARRANTIES, STATUTORY OR OTHERWISE, INCLUDING THE WARRANTY OF MERCHANTABILITY AND OF FITNESS FOR A PARTICULAR PURPOSE. THE PURCHASER AND/OR USER SHOULD PERFORM ITS OWN TESTS TO DETERMINE THE SUITABILITY AND FITNESS OF THE PRODUCT FOR THE PARTICULAR PURPOSE DESIRED IN ANY GIVEN SITUATION.

* All metric values are nominal.
** All imperial values are approximate.

Saint-Gobain Technical Fabrics
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GLASGRID

GlasGrid is manufactured at an ISO 9002-1994 registered facility of Saint-Gobain Technical Fabrics

SAINT-GOBAIN
TECHNICAL FABRICS

APPENDIX C

Typical KENLAYER Outputs

KENLAYER: Control Section

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TITLE:  CONTROL-400 (MUCK E=400 PSI)

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA = 0, SO DAMAGE ANALYSIS WILL NOT BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) =  1
NUMBER OF LOAD GROUPS (NLG) =  1
TOLERANCE FOR INTEGRATION (DEL) -- =  0.001
NUMBER OF LAYERS (NL)------------- =  5
NUMBER OF Z COORDINATES (NZ)------ =  2
LIMIT OF INTEGRATION CYCLES (ICL)- =  80
COMPUTING CODE (NSTD)------------- =  9
SYSTEM OF UNITS (NUNIT)------------=  0

Length and displacement in in., stress and modulus in psi
unit weight in pcf, and temperature in F

THICKNESSES OF LAYERS (TH) ARE : 4.5  7.5  12  36
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.4  0.4  0.4  0.4  0.4
VERTICAL COORDINATES OF POINTS (ZC) ARE:  4.45  60.5
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE :    1  3.500E+05   2  3.000E+05
   3  3.000E+04    4  1.000E+04    5  4.000E+02

LOAD GROUP NO. 1 HAS 1 CONTACT AREA
CONTACT RADIUS (CR)--------------- =  5.35
CONTACT PRESSURE (CP)------------- =  100
RADIAL COORDINATES OF  1  POINT(S) (RC) ARE :  0

PERIOD NO.  1   LOAD GROUP NO.  1

   RADIAL     VERTICAL     VERTICAL    VERTICAL     RADIAL   TANGENTIAL    SHEAR
 COORDINATE  COORDINATE  DISPLACEMENT   STRESS      STRESS     STRESS      STRESS
                                       (STRAIN)    (STRAIN)   (STRAIN)    (STRAIN)
  0.00000     4.45000      0.06448      65.956      17.609     17.609      0.000
 (STRAIN)                             1.482E-04  -4.519E-05  -4.519E-05  .000E+0(
  0.00000    60.50000      0.05629       0.109       0.008      0.008      0.000
 (STRAIN)                             2.566E-04  -9.682E-05  -9.682E-05  .000E+0(
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Reinforced Section

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TITLE: REINFORCED-400 (MUCK E 400 PSI; GLASGRID E = 10,000,000 PSI)

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA = 0, SO DAMAGE ANALYSIS WILL NOT BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) =  1
NUMBER OF LOAD GROUPS (NLG) =  1
TOLERANCE FOR INTEGRATION (DEL) -- =  0.001
NUMBER OF LAYERS (NL)------------- =  7
NUMBER OF Z COORDINATES (NZ)------ =  2
LIMIT OF INTEGRATION CYCLES (ICL)- =  80
COMPUTING CODE (NSTD)------------- =  9
SYSTEM OF UNITS (NUNIT)------------=  0

Length and displacement in in., stress and modulus in psi
unit weight in pcf, and temperature in F

THICKNESSES OF LAYERS (TH) ARE : 3.55  0.2  0.75  7.5  12  36
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.4  0.4  0.4  0.4  0.4  0.4  0
VERTICAL COORDINATES OF POINTS (ZC) ARE:  3.5  60.5
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE :    1  3.500E+05   2  1.000E+07
   3  3.500E+05    4  3.000E+05    5  3.000E+04   6  1.000E+04   7  4.000E+02

LOAD GROUP NO. 1 HAS 1 CONTACT AREA
CONTACT RADIUS (CR)--------------- =  5.35
CONTACT PRESSURE (CP)------------- =  100
RADIAL COORDINATES OF  1  POINT(S) (RC) ARE :  0

PERIOD NO.  1 .  LOAD GROUP NO.  1
```

RADIAL COORDINATE	VERTICAL COORDINATE	VERTICAL DISPLACEMENT	VERTICAL STRESS (STRAIN)	RADIAL STRESS (STRAIN)	TANGENTIAL STRESS (STRAIN)	SHEAR STRESS (STRAIN)
0.00000	3.50000	0.05522	73.186	49.830	49.830	0.000
(STRAIN)			9.521E-05	1.782E-06	1.782E-06	.000E+0(
0.00000	60.50000	0.04718	0.118	-0.034	-0.034	0.000
(STRAIN)			2.962E-04	-8.623E-05	-8.623E-05	.000E+0(