December 2012

Geotechnical Solutions for Soil Improvement, Rapid Embankment Construction, and Stabilization of the Pavement Working Platform

Performance Assessment of Lime and Fly Ash Chemically Treated Subgrade

SHRP 2 Renewal Project R02

PROJECT REPORT

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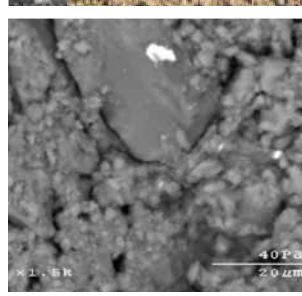

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Institute for Transportation

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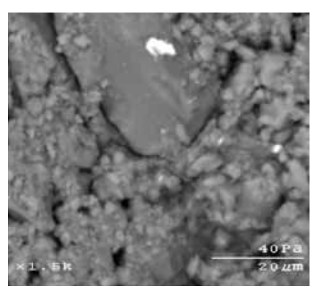

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Chemical treatment and stabilization of subgrades is a long-standing method to construct working platforms and improve the support conditions for pavement systems. Lime, cement, and fly ash are common chemical stabilization agents and are often incorporated with subgrade materials to improve volumetric stability, freeze-thaw performance, and/or subgrade stiffness.

Although laboratory test methods and design procedures are relatively well established, the long-term (5+ years) field performance characteristics of treated or stabilized subgrades is poorly documented and was the focus of this study. The main objectives of this project were as follows:
• Document engineering properties (in situ strength/stiffness) and mineralogical/micro-structural characteristics of chemical stabilized subgrades, in comparison with natural subgrades at the same sites
• Understand factors that contribute to long-term engineering behavior of stabilized subgrade

Nine test sections were selected to assess engineering properties of old stabilized subgrades in Texas, Oklahoma, and Kansas. The selection of the test sites was based on the type of subgrade, availability of old construction records, and age. Subgrades at six of these sites were stabilized with lime and the other three with fly ash. Eight of these test sites were more than 10 years old, and one test site was about 5 years old. Eight sites consisted of flexible pavement supported on base and stabilized subgrade or just stabilized subgrade, and one site consisted of concrete pavement supported on cement treated base and stabilized subgrade

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SHRP 2 R02 PROJECT:
GEOTECHNICAL SOLUTIONS FOR SOIL IMPROVEMENT, RAPID EMBANKMENT CONSTRUCTION, AND STABILIZATION OF THE PAVEMENT WORKING PLATFORM

PERFORMANCE ASSESSMENT OF LIME AND FLY ASH CHEMICALLY TREATED SUBGRADE

PROJECT REPORT

Prepared for
The Second Strategic Highway Research Program
Transportation Research Board
of
The National Academies

David White, Wenjuan Li, Pavana Vennapusa, Barry Christopher,
Jie Han, and Heath Gieselman

DECEMBER 2012

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SHRP 2 R02

GEOTECHNICAL SOLUTIONS FOR SOIL IMPROVEMENT, RAPID EMBANKMENT CONSTRUCTION, AND STABILIZATION OF THE PAVEMENT WORKING PLATFORM

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EXECUTIVE SUMMARY

Background

Chemical treatment and stabilization of subgrades is a long-standing method to construct working platforms and improve the support conditions for pavement systems. Lime, cement, and fly ash are common chemical stabilization agents and are often incorporated with subgrade materials to improve volumetric stability, freeze-thaw performance, and/or subgrade stiffness. Although laboratory test methods and design procedures are relatively well established, the long-term (5+ years) field performance characteristics of treated or stabilized subgrades is poorly documented and was the focus of this study. Typically, the improved performance characteristics of stabilized subgrade are not accounted for in pavement thickness design because of the lack of reliable long-term performance data. Stabilization of the subbase/base layers and other stabilization methods such as mechanical stabilization (e.g., geosynthetics) also warrant long-term field performance characterization, but was beyond the scope of this study.

Project Objectives

The main objectives of this project were to: (a) document engineering properties (in situ strength/stiffness) and mineralogical/micro-structural characteristics of chemical stabilized subgrades, in comparison with natural subgrades at the same sites, and (b) understand factors that contribute to long-term engineering behavior of stabilized subgrade. At each of the test sites, in situ tests were performed including: dynamic cone penetrometer (DCP), falling weight deflectometer (FWD), light weight deflectometer (LWD), and plate load tests (PLT). Samples were collected to document moisture content, soil classification, pH, and shear strength. Mineralogical/micro-structural analysis was performed using scanning electron microscopy (SEM) and energy dispersive spectrometry (EDS).

Key Findings

Nine test sections were selected to assess engineering properties of old stabilized subgrades in Texas, Oklahoma, and Kansas. The selection of the test sites was based on the type of subgrade, availability of old construction records, and age. Subgrades at six of these sites were stabilized with lime and the other three with fly ash. Eight of these test sites were more than 10 years old, and one test site was about 5 years old. Eight sites consisted of flexible pavement supported on base and stabilized subgrade or just stabilized subgrade, and one site consisted of concrete pavement supported on cement treated base and stabilized subgrade. FWD tests were conducted on the pavement surface, DCP tests were conducted in base and subgrade layers by drilling a hole through the pavement, static PLTs were conducted directly on top of the stabilized subgrade by making a 0.36 m diameter core hole in the pavement, and undisturbed Shelby tube samples were obtained for laboratory classification, shear strength, and pH, SEM, and EDS analysis.

Some significant findings from the field and laboratory testing are as follows:

- FWD testing conducted at 8 to 50 test locations at each site showed non-uniform conditions at each site with coefficient of variation of surface deflections varying from about 10 to 30%, and E_{FWD} value for stabilized subgrade from about 20 to 70%. The FWD plate deflections on top of the flexible pavements are strongly influenced by the CBR profile of the underlying stabilized subgrade layer.

- The in situ elastic modulus of chemical stabilized subgrades determined from the static PLT varied from 7 MPa to 317 MPa at the nine test sites. The MEPDG recommended typical modulus value for lime stabilized soils is 310 MPa with a range of 207 MPa to 414 MPa, and a deteriorated modulus value for lime stabilized soil is 103 MPa. Two out of the six lime stabilized subgrade sites tested showed average modulus < 103 MPa (note: MEDPG does not provide typical values for fly ash stabilized subgrades).

- Field results indicated that the elastic modulus value determined in the field is dependent on the test method used. On average, LWD and the back-calculated FWD modulus were about 0.7 times and 8.3 times the static PLT modulus, respectively. This is divergence in calculated modulus values is an important aspect to considered when selecting design values are establishing QC/QA target values.

- The ratio of LWD modulus of stabilized subgrade and natural subgrade varied from about 4 to 11. Similarly, CBR ratios between stabilized and natural subgrade ranged from about 2.2 to 7.4. Results indicated that these ratios are influenced by the thickness of the stabilized layers (lower the thickness, lower the ratio).

- The improved soil strength and stiffness due to chemical stabilization remained after many years of construction, but testing revealed that the in situ stiffness is highly non-uniform in the longitudinal direction (i.e., along the road alignment) and vertical direction (i.e., with depth).

- Scanning electron microscopy (SEM) analysis of treated subgrade samples showed that cementitious reaction products formed and remained in lime stabilized subgrade samples even after several years after construction.

- This study identified that the top of the stabilized layer is often weaker than near the center of the stabilized layer. Additional research is needed to understand why this is occurring (e.g., construction issue, environmental factors, etc.).

- Pavement performance was good at all of the test sites.

Results from this study provide new information that should be of great interest to pavement designers dealing with selection of design parameters for chemically stabilized subgrade layers. Additional research is recommended to further advance the understanding of the long-term behavior of mechanically and chemically stabilized subgrade and subbase layers.

CHAPTER 1. INTRODUCTION

1.1 INTRODUCTION

Although in existence for several decades, many geo-construction technologies face both technical and non-technical obstacles preventing broader utilization in transportation infrastructure projects. The research team for Strategic Highway Research Program 2, Project Number R02 (SHRP 2 R02) *Geotechnical Solutions for Soil Improvement, Rapid Embankment Construction, and Stabilization of the Pavement Working Platform* is investigating the state of practices of transportation project engineering, geotechnical engineering, and earthwork construction to identify and assess methods to advance the use of geoconstruction technologies. Such technologies are often underutilized in current practice, and they offer significant potential to achieve one or more of the SHRP 2 Renewal objectives, which are rapid renewal of transportation facilities, minimal disruption of traffic, and production of long-lived facilities. Project R02 encompasses a broad spectrum of materials, processes, and technologies within geotechnical engineering and geoconstruction that are applicable to one or more of the following "elements" of construction (as defined in the project scope): (I) new embankment and roadway construction over unstable soils; (II) roadway and embankment widening; and (III) stabilization of pavement working platforms.

The SHRP 2 R02 research team completed a comprehensive review of literature, a detailed assessment of several technical obstacles that interfere with more widespread use, and evaluation of mitigation strategies/action items in terms of benefit-to-cost (B/C) ratio for each of the element III technologies. Long-term performance of chemical and mechanical stabilized construction platforms and pavement foundation systems was identified as high B/C ratio. Because of performance uncertainty and absence of long-term performance data, pavement engineers are not certain that stabilized subgrade can provide sufficient and uniform support over the design life of the pavement. The structural benefit of stabilized subgrade is generally not considered in most pavement design codes (e.g., AASHTO 1998). One of the major obstacles for inclusion of the improved engineering properties of stabilized layers in pavement design was identified as lack of well-documented and accessible case histories with benefits related to long-term (5+ years) performance.

Chemical stabilization of soft soil has been used in United States since more than 60 years (Rafalko et al. 2007). The chemical additives include lime, cement, fly ash, cement kiln dust, and other nontraditional additives. Several factors influence the quality and long-term performance of stabilized subgrade, such as additive content, construction method, and environmental factors and so on. Laboratory test results of a typical mix design including soil strength and stiffness measurements are usually well documented in the short term (up to 90 days). However, long-term performance is difficult to measure and is therefore typically relied upon for the short term only. Thus, chemical stabilized subgrade is primarily considered as an approach for creating a construction platform. The long-term performance data of pavements supported on stabilized layers are desired for further development of this technology.

This report addresses two technical problems, the lack of performance data for stabilized pavement subgrades that are more than 5 years old and lack of understanding of the factors that contribute to long-term engineering behavior of stabilized subgrades supporting pavements. This research addressed these problems by conducting in situ and laboratory tests for chemical (lime or fly ash) treated subgrades. Laboratory tests include moisture content, sieve analysis, pH test, scanning electron microscope (SEM), and unconsolidated-undrained (UU) triaxial tests. In situ tests include dynamic cone penetrometer (DCP), falling weight deflectometer (FWD), light weight deflectometer (LWD), plate load test (PLT), and soil sampling. Mineralogical and microstructure analysis were performed on stabilized subgrades to better understand the nature of chemical reactions between the soil and stabilizers. Nine test sites were selected in Texas, Oklahoma, and Kansas. The selection of the test sites was based on the type of subgrade, availability of old construction records, and construction year. Eight test sites were constructed more than 10 years ago, and one test site was constructed more than 5 years ago. Stabilization of the subbase/base layers and other stabilization methods such as mechanical stabilization (e.g., geosynthetics) also warrant long-term field performance characterization, but was beyond the scope of this study.

1.2 RESEARCH GOALS AND OBJECTIVES

The primary research goal of this study was to document in situ performance characteristics of chemically treated subgrade soils. The main objectives of this research were to:

- Evaluate the in situ stiffness and associated variability of the chemically treated subgrades,
- Characterize the chemical components and microstructure of in-service stabilized subgrade soils, and
- Determine stiffness improvement ratio between the chemically treated subgrade soils and the natural subgrades.

1.3 RESEARCH BENEFIT AND SIGNIFICANCE

This research is of significance to state agencies, design engineers, material suppliers, and contractors because it provides evidence of the long-term performance characteristics of chemically treated subgrade, which is not well documented in the literature. The results are presented as case studies. The results from this study provide new information that documents the longitudinal and vertical non-uniformity of chemically treated subgrade. Based on a literature review conducted as part of the of SHRP 2 R02 project, some of the advantages and disadvantages of other stabilization technologies are also described in this report. These types of long-term performance studies are needed at many more locations and should incorporate mechanical stabilization and stabilization of subbase/and base laterals as well.

1.4 REPORT ORGANIZATION

This report is organized into six chapters. Chapter 2 is a literature review that summarizes features of test methods used in this study, and design, quality control and assurance, and case studies for chemical stabilized soil. Chapter 3 describes both field and laboratory test methods. Chapter 4 describes the test results from the nine case study sites in TX, OK, and KS. Chapter 5 summarizes the key findings from this study. Recommendations for future researcher are provided in Chapter 6. A list of key references on the topic of chemically stabilization is provided at the end of the report. The appendix contains all of the SEM images collected from the laboratory assessments of the stabilized soils, field test results, FWD analysis, and construction records if they existed for the various projects.

CHAPTER 2. BACKGROUND/LITERATURE REVIEW

In this chapter, several previous studies are reviewed for long term performance of chemical stabilized subgrades. A literature review of design methods, quality control and assurance, and in situ testing methods are also presented. The key reference list provided with this report was developed as an outcome of the SHRP 2 R02 research products and is included after Chapter 6. A few of the key references are summarized in the following.

2.1 LONG-TERM PERFORMANCE OF CHEMICAL STABILIZED SUBGRADES

Three references were identified in the literature that focuses on evaluations of long-term performance of chemical stabilized subgrades. The first study by Little et al. (1995a) describes results from on investigations of structural improvements of stabilized bases and subgrades after several years of pavement service life. A total of 30 test sites in Texas with lime stabilized subgrades were investigated. Falling weight deflectometer (FWD) test results were back calculated to determine the natural and stabilized subgrade moduli. Dynamic cone deflectometer tests verified the FWD results. At all but one site, back calculated moduli of stabilized subgrades were equal or greater than 200 MPa. Typically, a good quality of aggregate base is about 200 MPa. For 27 out of 30 test sites, back calculated FWD moduli showed that the modulus ratio between lime stabilized and natural subgrades was greater than 3. The authors stated that, if the structural benefit of stabilized subgrades needs to be considered in pavement design, a minimum modulus ratio value of 3 can be used (relative to the natural subgrade).

The second study by Hopkins et al. (2002) reported on an evaluation of the long-term performance of chemical stabilized subgrades in Kentucky. A total 20 test sections were selected and the subgrades were stabilized using lime or cement. The laboratory and field tests included grain size, index property, moisture content, specific gravity, unconfined triaxial compression test, in situ CBR, standard penetration test, and falling weight deflectometer. Some key findings are summarized as follows:

- The soil types of natural subgrades were generally modified from silts (ML) to sandy silts (SM) after treatment. The clay faction of natural subgrades was reduced.
- In situ CBR of lime stabilized subgrades were about 14 times higher than the natural subgrades.
- Moisture content of the top un-stabilized subgrade was 3 to 4 percent greater than moisture content of stabilized subgrade.
- The FWD moduli ranged from 19- 455 MPa (2,700 to 66,100 psi) for natural subgrades and 149-896 MPa (21,600 to 130,000 psi) for stabilized subgrades.
- The FWD modulus of the granular base supported by the stabilized subgrade was greater than that value of the granular base supported by un-stabilized subgrade.

The third study by Jung et al. (2008) investigated the performance of six lime kiln dust (LKD) stabilized subgrades in Indiana. These stabilized subgrades were constructed between 1996 and 2002. Comparisons of moisture content, fines content, soil type, pH value, CBR, and resilient modulus (M_R) are made between stabilized and natural subgrade.

Key findings are as follows:

- The fines content of the natural subgrades was reduced by 20 to 40% after treatment.
- Water content of the stabilized and natural subgrade was uniform at each test site.
- The natural subgrades were modified from silty or clayey soils to non-plastic silty sand for stabilized subgrades.
- pH values of the natural subgrades ranged from 7.5 to 8.0, while the pH values of stabilized subgrades ranged from 8.5 to 11.0. The high pH of stabilized subgrades verified the presence of lime in stabilized subgrades.
- The average CBR of natural subgrades increased 5 to 15 times after treatment.

The LKD stabilized subgrades performed well after 5-11 years. The authors stated that the uniformity of stabilized subgrades was questionable. Improvement of the construction quality control program was recommended to ensure that the LKD treated subgrades will provide long-term performance of the pavement systems.

2.2 DESIGN METHODS

2.2.1 Lime Stabilization of Subgrades and Bases

Determining the lime content addition rate is the primary objective of mixture design for lime stabilization. The optimum lime content is dependent on how the stabilized material will be used and the soil constituents. The design objects may involve a reduction in plasticity, construction expediency, or permanent engineering changes, which affect the strength/stiffness of the mixture and performance of the pavement. Mixture preparation, specimen preparation, curing conditions, and testing are four factors considered as part of a laboratory testing program. Special testing is required for sulfate-bearing clay to prevent deleterious sulfate-induced heave. Winterkorn and Pamukcu (1990) provide test results showing the stabilizing effect of lime on different soil types.

Because applications of lime can be broad in stabilization, several mix design methods have been developed. According to TRB (1987), these methods are:

1. California procedure (Terrel et al. 1979)
2. Eades and Grim procedure (Eades et al. 1966)
3. Illinois procedure (Terrel et al. 1979)
4. Oklahoma procedure (TRB 1987)
5. South Dakota procedure (TRB 1987)
6. Texas procedure (AASHTO T-220)
7. Thompson procedure (Thompson 1970)
8. Virginia procedure (VTM-11 Virginia Test Method for lime stabilization)

An example of one of these methods, the Texas procedure, is summarized below.
Step 1: Based on the grain size and PI data, the lime percentage is determined by using a graphical solution developed by Terrel et al. (1979).
Step 2: Optimum moisture and maximum dry density of the mixture are determined in accordance with AASHTO T-212 or Tex-113-E.

Step 3: Test specimens 6 in (15.2 cm) in diameter and 8 in. (20.3 cm) in height are compacted at optimum moisture content to maximum dry density.
Step 4: All specimens are placed in a triaxial cell and cured in the following manner:

a. Cool to room temperature.
b. Dry at temperature not exceeding 60° C (140° F) for about 6 hour until one-third of the molding moisture is remove.
c. Cool for at least 8 hr.
d. Subject specimens to water exposure via capillary action for 10 days (AASHTO T-212).

Step 5: The cured specimens are tested in unconfined compression with AASHTO T-212 section 7 and 8 or Tex-117-E.

A design process flow chart is provided in Winterkorn and Pamukcu (1990). Two design criteria are used: (1) pavement structural behavior and (2) durability. In addition, swell potential needs to be reduced to a satisfactory level for lime-modified soil.
To deal with sulfate induced problems with lime stabilized soils, the National Lime Association (2000) provides guidelines as following:

- Sulfate levels too low to be of concern: The total level of soluble sulfates is below 0.3% (3000 ppm).
- Sulfate levels of moderate risk: The total levels of soluble sulfates are between 0.3% (3000 ppm) to 0.5% (5000 ppm). During construction, water content should be at least 3% to 5% above optimum for compaction. The mellowing period may be extended to longer than 72 hours.
- Sulfate levels of moderate to high risk: The total levels of soluble sulfates are between 0.5% (5000 ppm) to 0.8% (8000 ppm). The same mix design and construction can be followed as for soil containing 0.3-0.5 % sulfate. Laboratory tests are recommended to evaluate swell potential and the required period of mellowing between mixing and compaction.
- Sulfate levels of high and unacceptable risk: The total levels of soluble sulfates are greater than 0.8% (8000 ppm). Due to high sulfate levels, treatment requires lime slurry, mixing, mellowing, curing water contents of 3%-5% above optimum for compaction, and mellowing period longer than 72 hours. The double application of lime may also be needed.

Although the benefits of improved soil properties are not considered in most current design approaches in United States, a study conducted by Qubain et al. (2000) shows that lime treated subgrade soil can be successfully incorporated into pavement design with economic benefit by increasing the strength of subgrade. Three approaches were applied in this study: (1) assessing the effective resilient modulus for the lime treated subgrade, (2) applying a conservative CBR of 15 to account for lime stabilization, and (3) treating the lime-stabilized subgrade as a subbase layer and assigning it a structural–layer coefficient. Little information is available in the literature; however, that documents the long-term performance of stabilized soils for permanent foundation materials.

2.2.2 Fly Ash Stabilization of Subgrades and Bases

Class C fly ash is recommended as a suitable stabilizer for fine-grained plastic soils such as clay, as well as coarse-grained soil. The American Coal Ash Association (ACAA 2008) provides a detailed description of fly ash stabilization. One of the distinguishing characteristics of fly is the potential for rapid set. Delayed compaction (greater than about 2 hours) can reduce in reduced strength gain and reduced compaction density (up to 1.6 KN/m^3 (10 pcf) or more). In addition to compaction delay, the moisture content of the soil fly ash mixtures significantly influences the compressive strength. To deal sulfate attack problems for stabilized materials, fly ash with the high sulfate concentrations should be avoided.

No standard test procedures currently exist for the design of material stabilized with self-cementing ash (ACAA, 2008). However, an effective procedure can be used to determine moisture-density and moisture-strength relationships of the stabilized material, based on adaptation of ASTM C593 (Fly Ash and Other Pozzolans for Use with Lime) and ASTM D 1633 (Compressive Strength of Molded Soil-Cement Cylinders). The design procedure follows:

1. Blend soil, fly ash and water to make a minimum of five test specimens. Moisture contents of the specimen should be up to 10% below to 6% above the optimum moisture content for maximum density. The specimens have a height-to-diameter ratio of 1.15.
2. Compact specimens over a wide range of moisture contents. Use specified compaction time delay (<2 hours) and 102-mm (4.0-inch)-diameter by 117-mm (4.625-inch)-high mold. Standard Proctor compaction energy or modified proctor compaction energy may be used. Alternatively, it can use specimens with 50.8 mm (2 in.) in diameter by 50.8 (2 in.) high. Advantages for using these specimens are material and time saving. Additionally, the test results obtained from those specimens are very close with using the standard Proctor specimens (O'Flaherty et al. 1963).
3. Cure test specimens for a period of 7 days at 38°C (100°F) in accordance with ASTM C593, and
4. Determine compressive strength of specimens.

Modification of the compaction procedures may be required for mix designs of granular materials stabilized with fly ash. For stabilized pavement section or other applications where a higher degree of stabilized is desired, additional laboratory tests needs to conducted assess properties of the stabilized materials required for specific design procedures. Stabilized granular material to be used for pavement base course or subbase tests can be evaluated through ASTM C593 to assess the freeze-thaw durability of the stabilized materials.

2.3 DESIGN METHODS

The MEPDG (NCHRP 2004) provides some guidance on the input characterization, which covers lean concrete, cement treated, soil cement, lime–cement–flyash, lime–flyash, and lime stabilized layers. Elastic modulus (E) of a 28-day cured specimen is considered as a design parameter for all stabilized materials, except for lime stabilized materials. Resilient modulus (M_r) is considered as a design input parameter for lime stabilized materials. For level 1 design, input values are required to be determined from laboratory testing; however, it is indicated in the design guide that there are no standard laboratory testing protocols to determine E. For level 2 design, the following correlations from unconfined compressive strength of samples obtained from field or DCP are recommended to determine the input values.

- Lean concrete (Thompson 1986) – E (psi) = 57000 $\sqrt{f_c'}$, where f_c' is compressive strength determined in accordance with AASHTO T22
- Open graded cement stabilized material – no correlations area available
- Soil-cement (Thompson 1986) – E (psi) = 1200 x q_u, where q_u is unconfined compressive strength (psi) determine in accordance with ASTM D1633
- Lime-cement-flyash (ACA 1991) – E (psi) = 500 + q_u, where q_u is unconfined compressive strength (psi) determine in accordance with ASTM C593
- Lime stabilized soils (Little 2000) – M_r (ksi) = 0.124 q_u + 9.98, where q_u is unconfined compressive strength (psi) determine in accordance with ASTM D5102

For level 3 design, typical values are provided in the design guide reportedly based on experience and historical records. A summary of the suggested typical input values for chemically stabilized materials is provided in Table 1. Also summarized in this table are the deteriorated typical E or M_r values for stabilized layers (after the material has been subjected to repeated traffic loading) for use in HMA pavement design.

Table 1. Summary of typical input values for chemically stabilized materials provided in MEPDG for level 3 design (NCHRP 2004)

Chemically stabilized material	**E or M_r Range (MPa)**	**E or M_r typical (MPa)**	**Deteriorated E or M_r Typical (MPa)****
Lean concrete	10,342 to 17,237	13,790	2,068
Cement stabilized aggregate	4,826 to 10,342	6,895	689
Open graded cement stabilized aggregate	—	5,171	345
Soil cement	345 to 6,895	3,447	172
Lime-cement-flyash	3,447 to 13,790	10,342	276
Lime stabilized soils	207* to 414*	310*	103

* For reactive soils with 25% passing No. 200 sieve and PI of at least 10
** The deteriorated typical values are suggested for HMA pavement design only

CHAPTER 3. TEST METHODS

Field and laboratory tests were conducted to investigate pavement performance, characterize soil engineering properties, and analyze soil morphology and chemical composition. Field and laboratory tests are discussed as follows:

3.1 FIELD TESTS

dynamic cone penetrometer (DCP), falling weight deflectometer (FWD), light weight deflectometer (LWD), plate load test (PLT), and boring and sampling were used in this study. The position for each tests were determined using real-time kinematic-global positioning system (RTK-GPS).

3.1.1 Real-Time Kinematic-Global Positioning System

RTK-GPS was employed to record in situ test locations with spatial coordinates (x, y and z). Precision of the system can reach approximately 10 mm horizontal and 20 mm vertical.

3.1.2 Dynamic Cone Penetrometer

Dynamic cone penetrometer (DCP) tests were performed to determine pavement profiles and subgrade strength in according with ASTM D6951-03 "*Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.*" Holes with a diameter of 38 mm (1.5 in.) were drilled into the pavement layers before testing (Figure 1). Extension rods were added to DCP to a depth of 1.5 m (59 in.). Dynamic penetration index (DPI) and California bearing ratio (CBR) of the subgrade materials were calculated using Equation (1):

$$\mathrm{CBR} = \frac{292}{(\mathrm{DCPI})^{1.12}} \tag{1}$$

The weighted average CBR values were calculated for each test location.

Figure 1. (a) Drilling 1.5 inch diameter hole prior to DCP test, (b) dynamic cone penetrometer test

3.1.3 Falling Weight Deflectometer

FWD tests were conducted on ACC and PCC surfaces with a KUAB 2M-FWD 150. The FWD equipment is shown in Figure 2. The loading plate diameter was 300 mm (12 in.). One seating drop was followed three test drops applied using impact loads of 27 KN (6000 lb), 40 KN (9000 lb), 54 KN (12000 lb), and 72 KN (16000 lb). The deflections were measured using seven deflection sensors mounted on a raise-lower bar and the impact force was measured using a load cell. The sensor distances from the center of loading plate (D_0) are summarized in Table 5. The ERI data analysis (ERIDA) method was used to estimate the subgrade layer properties presented in this report. For FWD data analysis on rigid pavements, the ERIDA software uses the AREA method, which is based on closed from solutions that assume a two layer system of PCC slab being loaded on top of an elastic subgrade with modulus (Esg) or a dense liquid modulus of subgrade reaction (k) (ERI 2009). The AREA method used in the ERIDA includes deflection basin parameters from sensors located at 0 cm (0 in.), 31 cm (12 in.), 71 cm (24 in.), and 91 cm (36 in.) from the load center.

Temperature measurements of pavement were recorded at different depths through small drilled holes before FWD testing. Kim et al. (1995) conducted a study to determine temperature corrections for deflections, and Equation (2) was presented to covert deflections (D_0) to a reference temperature as following

$$D_{68} = 10^{\alpha(68-T)} * D_T \tag{2}$$

where:

D_{68} = adjusted deflection to the reference temperature of 20 °C (68 °F)
D_T = deflection measured at temperature T (°F)
α = $3.67 \times 10^{-4} \times t^{1.4241}$ for lane center

t = thickness of AC layer (in.), and
T = the AC layer mid-depth temperature (°F) at time of FWD testing

Table 2. KUAB 2M-FWD 150 Position of seven deflection sensors

Deflection Sensors	Offsets from center of loading plate
D1	15 cm (6 in.)
D2	31 cm (12 in.)
D3	46 cm (18 in.)
D4	71 cm (24 in.)
D5	91 cm (36 in.)
D6	122 cm(48 in.)
D7	152 cm (60 in.)

Figure 2. Kuab falling weight deflectometer

3.1.4 Light Weight Deflectometer

Light weight deflectometer tests were performed in large core holes directly on the base layer, stabilized subgrade, and natural subgrade to analyze stiffness and strength. Figure 3 shows LWD testing in a core hole. The tests were conducted using a 300 mm diameter plate and a drop height of 0.5 m, following manufacturer recommendations (Zorn 2003). The average deflection was measured after three seating drops followed by three test drops. The following equation was used to calculate E_{LWD} (see Vennapusa and White 2009):

$$E = \frac{(1-v^2)\sigma_0 a}{d_0} f \qquad \textbf{(3)}$$

where:

E = elastic modulus
d_o = measured settlement,
v = Poisson's ratio (assumed as 0.4),

σ_0 = applied stress,
a = plate radius
f = shape factor. Values assumed = 2 for stabilized subgrade, = π/2 for natural subgrade, and = 8/3 for base layer.

Figure 3. Light weight deflectometer test

3.1.5 Plate Load Test

A static plate load test was performed at the subgrade surface to measure load-deformation response and determine stiffness in accordance with ASTM D 1195 "*Standard Test Method for Repetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements.*" A static load was applied on a 300 mm diameter plate. The pavement deflections were calculated using data measured by three 50 mm linear voltage displacement transducers, while the actual applied load was measured by a load cell. Equation (3) was applied to determine initial (E_{V1}) and re-load (E_{V2}) modulus, and the deformation reading was taken at 0.2 to 0.4 MPa plate contact stresses for the stabilized subgrades. Using Equation (4), the modulus of subgrade reaction was converted to an equivalent value for a 762 mm (30 in.) diameter plate (Terzaghi 1955). In general according with AASHTO T 222-81, the uncorrected modulus of soil ($k'u$) was calculated using Equation (5). Correction of K'_U values for plate bending was made using AASHTO T222-81 procedure. PLT testing is shown in Figure 4.

$$K'_U = K'_{U1}\frac{\mathrm{B+B1}}{\mathrm{2B}} \tag{4}$$

B_1=side dimension of a square plate used in load test (m)
B=width of footing (m),
K'_U=modulus of subgrade reaction (kPa/mm), and
K'_{U1}=stiffness estimated from a static plate load test (kPa/mm)

$$K'_{U1} = \frac{69.0 \text{ kPa (psi)}}{\text{average deflection}} \quad \textbf{(5)}$$

Figure 4. Plate load test in core hole.

3.1.6 Boring and Sampling

Pavement coring was used to assess the pavement foundation layers for testing and sampling. A 355 mm (14 in.) inside diameter core barrel was used (Figure 5). Shelby tubes (71 mm (3 in.) diameter) were hydraulically pushed into the subgrade materials to obtain the undisturbed samples unconsolidated-undrained triaxial compression tests (Figure 5). Bag samples were collected for the base material, natural subgrade and stabilized subgrade. The stabilized subgrade samples were collected at 50 to 76 mm (2 to 3 in.) intervals. Natural subgrades were collected from the underlying stabilized subgrade layer and in ditch areas adjacent to the test locations. All samples were sealed in plastic bags or buckets and transported to the ISU soil research lab for further laboratory tests. Figure 6 shows a lime stabilized subgrade surface and pavement core. For each core location, effort was made to minimize use of coring water to not disturb the surface of the underlying foundation layers.

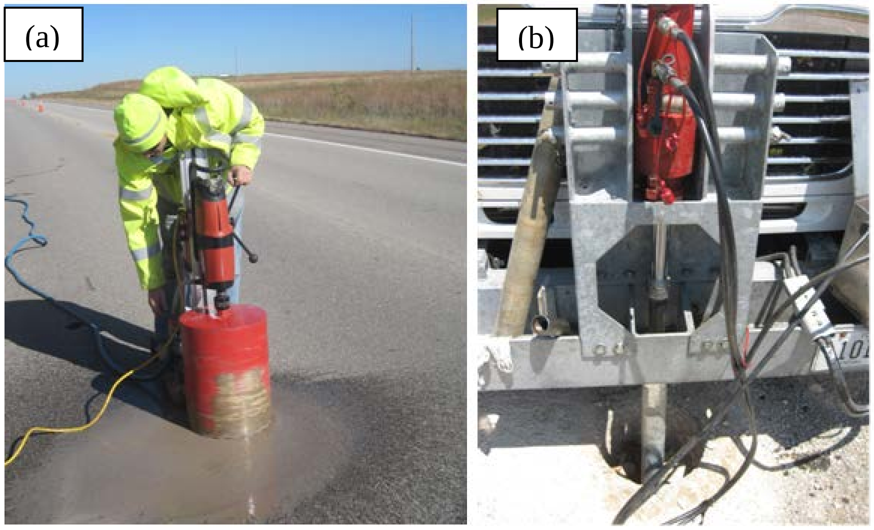


Figure 5. (a) Paving coring (b) collecting of undisturbed Shelby tube sample

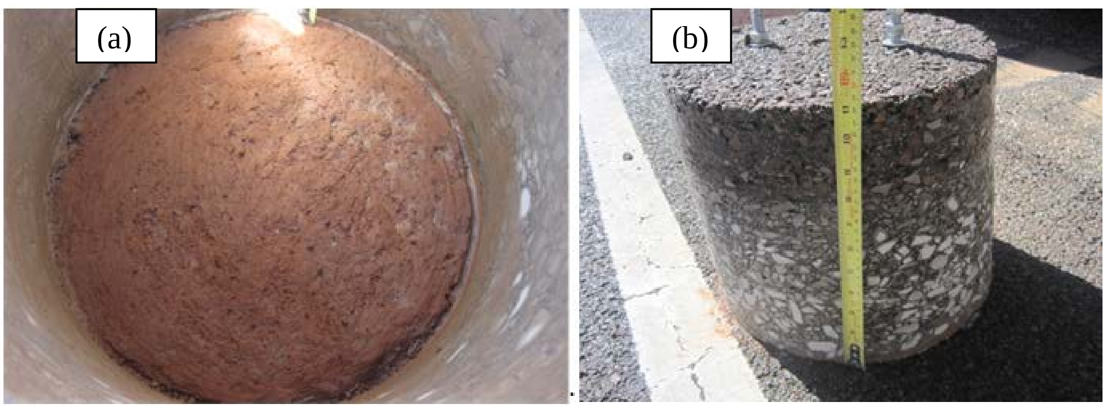


Figure 6. (a) Top stabilized subgrade (b) pavement core

3.2 LABORATORY TESTS

Laboratory tests included moisture content, gradation, index properties, pH tests, unconsolidated-undrained triaxial compression tests (UU), and scanning electron microscope (SEM) and energy-dispersive x-ray spectral analysis.

3.2.1 Moisture Test

The moisture content of soil samples was determined following ASTM D 2216-09 "*Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*." The moisture contents of Shelby tube and bag samples were measured within one week of being transported to the laboratory.

3.2.2 Particle Size Distribution Analysis and Index Properties

Bag samples of subgrade and base were tested to determine their particle size distribution in accordance with ASTM D422-63 "*Standard Test Method for Particle-Size Analysis of Soils.*" Atterberg limits tests were conducted in accordance with ASTM D4318-05 "*Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.*" The samples were prepared using the wet method and passed the No.40 sieve. The multi-point method was applied for liquid limit tests.

Soils were classified in according with the Unified Soil Classification System (USCS) and (AASHTO) classification methods.

3.2.3 pH Test

The pH measurement of stabilized and natural subgrade samples was carried out in accordance with ASTM D 4972-01 (2007) "*Standard Test Method for pH of Soils.*" Each 10 g sample was mixed with 10 ml distilled water. Three buffer solutions (pH=4.0, pH=7.0, and pH=12.0) were used for calibration of the meter (Accumet XL20) before testing. After 15 minutes of mixing, the pH of samples was measured.

3.2.4 Unconsolidated-Undrained Triaxial Compression Tests

UU tests were used to determine the undrained shear strength and were conducted in general accordance with ASTM D 2850 "*Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils.*" The tests were conducted using undisturbed Shelby tubes samples of stabilized subgrade. A confining pressure of 34.5 kPa (5 psi) was used for all tests. Figure 7 shows an extracted Shelby tube. Before extruding, all Shelby tubes were stored in a moisture room. The ratio of height to diameter of 2 (142 mm height and 71 mm diameter) was used to prepare test samples. Mass of samples was measured prior to the test and moisture contents were measured after the test to determine volumetric/gravimetric parameters.

Figure 7. Example of Shelby tube sample after extraction

3.2.5 Scanning Electron Microscope Analysis

SEM analysis was used to identify morphology features of the natural and stabilized subgrade materials. A Hitachi S2460-N variable pressure system was used with a FEI Quanta 250 FEG scanning electron microscope. Using a clean razor blade, test specimens were prepared with relatively flat surfaces for examination (Figure 8). Quantitative mineralogical analysis of the subgrade samples was conducted using energy dispersive spectrometry (EDS). Element maps provide the distribution of elements at the surface of the samples. White products common to several samples at the US 183 test site (Figure 9), were investigated using SEM.

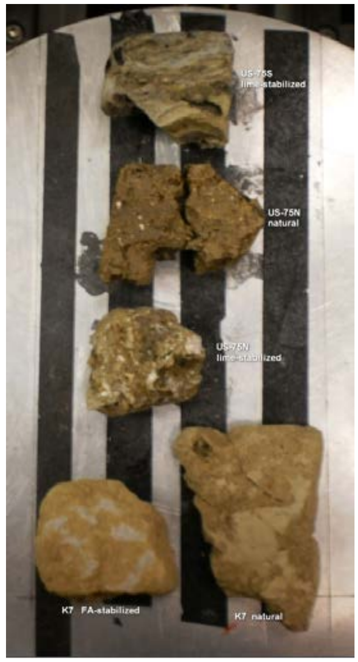


Figure 8. Prepared SEM samples from test sites in Kansas

Figure 9. White product presented in stabilized subgrade at test site of US 183

CHAPTER 4. CASE STUDIES

This chapter consists of site information, material properties, SEM analysis, pH, and in situ soil strength/stiffness measurements for each test site. The site information describes site location, pavement profile, construction history, and in situ test point locations. The material properties of soil include soil classification, index properties, and moisture content. The results of pH values of stabilized and natural subgrade are presented. SEM analysis provides information on soil constituents and chemical composition. The results of soil strength and stiffness are analyzed to evaluate the long-term performance characteristics of the stabilized subgrade. Site location, section length, layer thickness, stabilizer, and construction year at each site are summarized in Table 3.

Table 3. Summary of test site information

Road Name	Location	Section Length	Nominal Layer Thickness	Stabilizer	Cons. Year
SH 121	Fort Worth, Tarrant County, TX	370 m	(1) 75 mm AC (2) 200 mm flex base (3) 200 mm stab. subg.	lime	1995
FM 1709	Fort Worth, Tarrant County, TX	300 m	(1) 150 mm AC (2) 200 mm flex base (3) 150 mm stab. subg.	lime	1994
US 287	Mansfield, Tarrant County, TX	600 m	(1) 89 mm AC (2)280 mm flex base (3) 356 mm stab. subg.	lime	1982
US 183	Clinton, Washita County, OK	300 m	(1) 300 mm AC (2) 203 mm stab. subg.	5% lime	1999
SH 99	Seminole, Seminole County, OK	500 m	(1) 254 mm AC (2) 152 mm base (3) 203 mm stab. subg.	12-14% fly ash	1999
US 59	Clinton, Washita County, OK	500 m	(1) 254 mm AC (2) 254 mm base (3) 203 mm stab. subg.	12-14% fly ash	2000
US 75 SB	Lyndon, Osage County, KS	700 m	(1) 330 mm AC (2) 50 mm base (3) 100 mm stab. subg.	5% lime	1995
US 75 NB	Hoyt, Jackson County, KS	220 m	(1) 229 mm PCC (2) 102 cement treated base (3) 152 mm stab. subg.	lime	1995
K 7	Doniphan, Doniphan County, KS	500 m	(1) 229 mm AC (2) 300 mm stab. subg.	14-18% fly ash	2005

4.1 SH 121, TX

4.1.1 Site Description

This project was located on SH121 in Fort Worth, Tarrant County, Texas. The general location of this site is shown in Figure 10. This road is a four-lane State Highway. The road was constructed in 1982, and originally consisted of a 25 mm (1 in.) thick asphalt concrete (AC), 200 mm (8 in.) flex base, and 200 mm (8 in.) lime stabilized subgrade. A HMA overlay with a thickness of 50 mm (2 in.) was placed in 2008. The current pavement consists of a 75 mm (3 in.) thick asphalt concrete (AC), a 200 mm (8 in.) flex base, and 200 mm (8 in.) lime stabilized subgrade. The length of this test section is approximately 370 m (1214 ft). The Iowa State University (ISU) research team conducted in situ testing on August 4, 2010 with assistance and traffic control provided by Texas DOT.

The plan view of in situ test locations is shown in Figure 11. The research team preformed FWD tests on the surface of ACC pavement at intervals of about 10 to 30 m at test points 1 to 14. DCP tests were conducted at test point 4. After coring, LWD tests were performed on the top of stabilized subgrade at test points 4, 7, and 11. PLT tests were performed on the top of stabilized subgrade at test points 4 and 7. Bag samples of base and stabilized subgrade were collected at test points 4, 7, and 11.

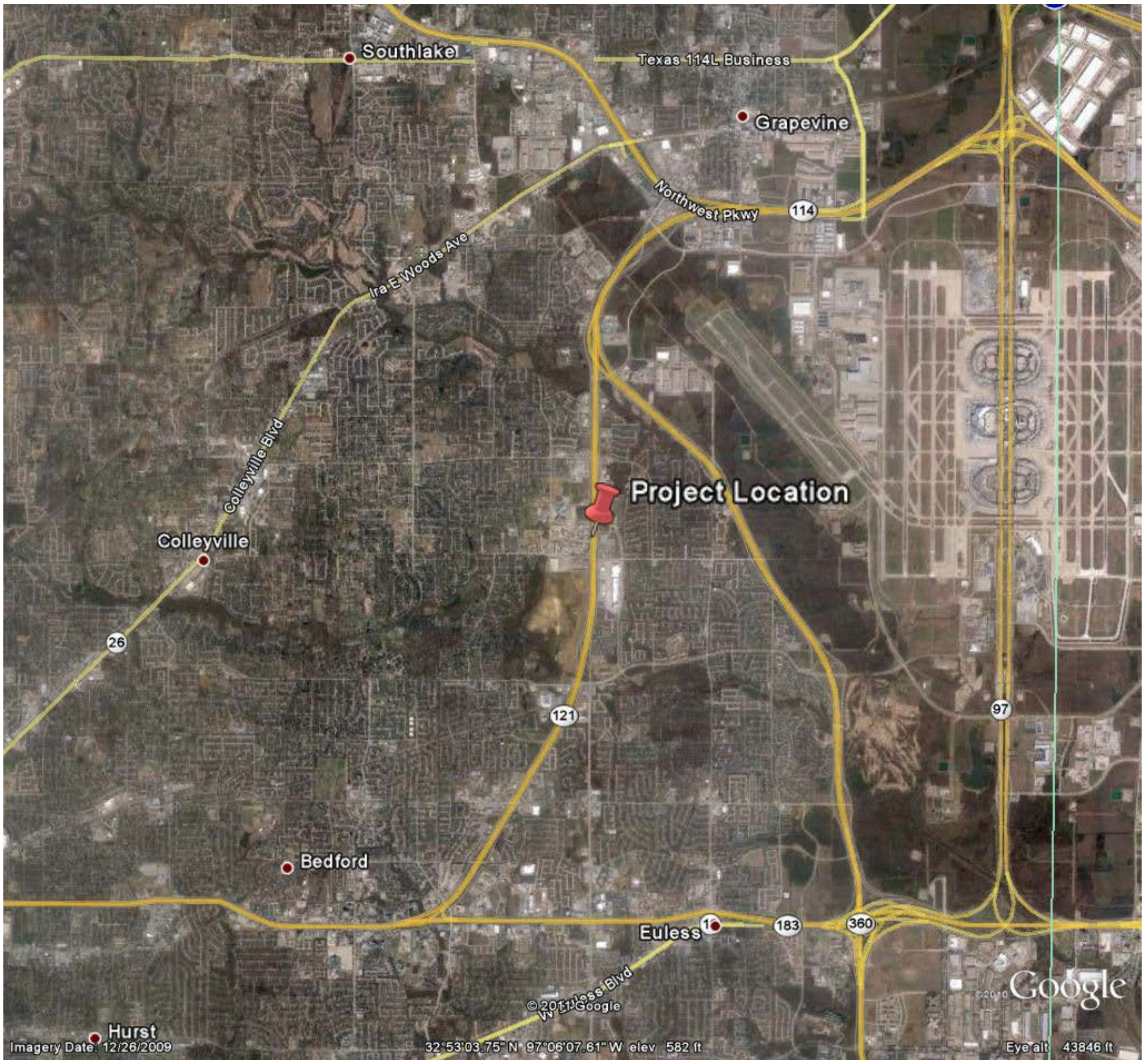


Figure 10. Project location of SH 121 Fort Worth, Texas

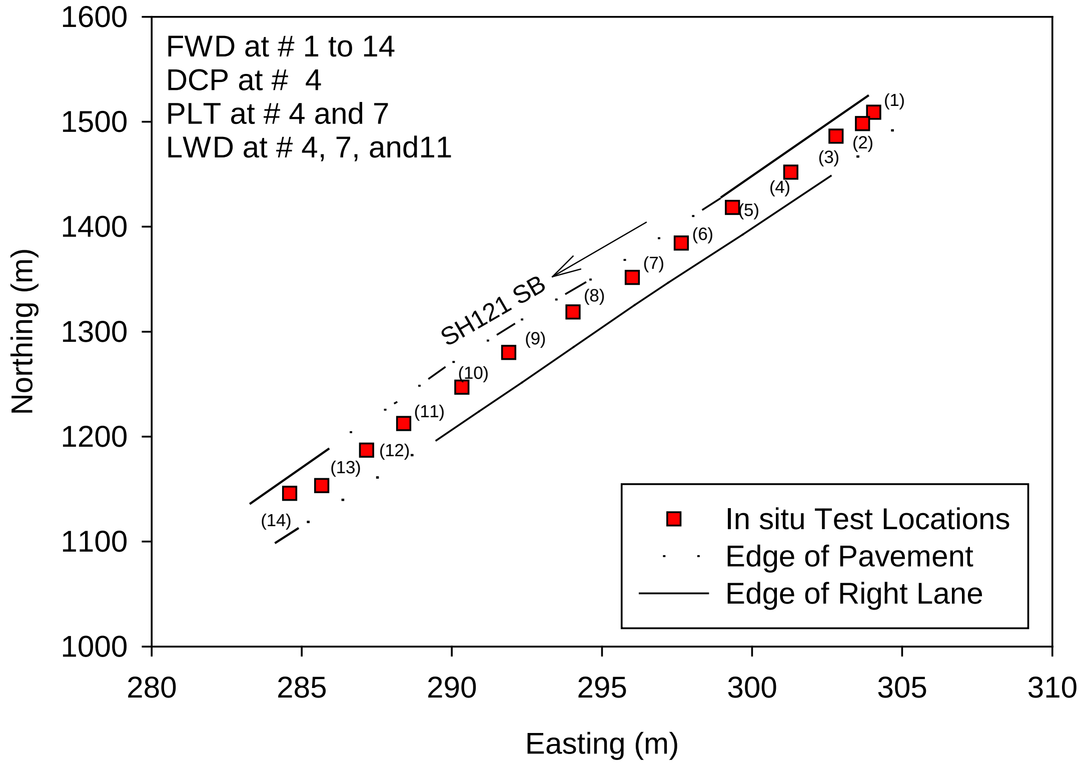


Figure 11. Test section plan layout with RTK GPS test locations SH 121 Fort Worth, Texas

Figure 12. Site overview SH 121 Fort Worth, Texas

4.1.2 Test Results and Analysis

4.1.2.1 Material Properties of Base and Subgrade

The base and stabilized subgrade samples were taken at test point 7. According to USCS and AASHTO, the flex base was classified as GM and A-1-b, and the stabilized subgrade was classified as SM and A-2-4. Table 4 presents material properties of base and subgrade. The sand content of stabilized subgrade was about 62%, and the clay content was about 7%. The LL value of stabilized subgrade sample was 27. The stabilized subgrade is a non-plastic soil. Figure 13 shows particle size distribution curves of base and stabilized subgrade.

Table 4. Summary of material properties SH 121 Fort Worth, Texas

Parameters	**Materials**	
Material Description	Base	Stabilized Subgrade
Depth mm (in.)	0-200 (0-8)	0-100 (0-4)
Gravel Content (%) (> 4.75mm)	46.3	10.2
Sand Content (%) (4.75mm – 75μm)	37.2	62.4
Silt Content (%) (75μm – 2μm)	12.9	20.7
Clay Content (%) (< 2μm)	3.6	6.7
Coefficient of Uniformity (C_u)	501.8	40.6
Coefficient of Curvature (C_c)	6.3	5.8
Liquid Limit, LL (%)	21.0	26.5
Plasticity Index, PI	9.0	N.P.
AASHTO	A-1-b	A-2-4
USCS	GM	SM
Water Content (%)	3.9	15.4

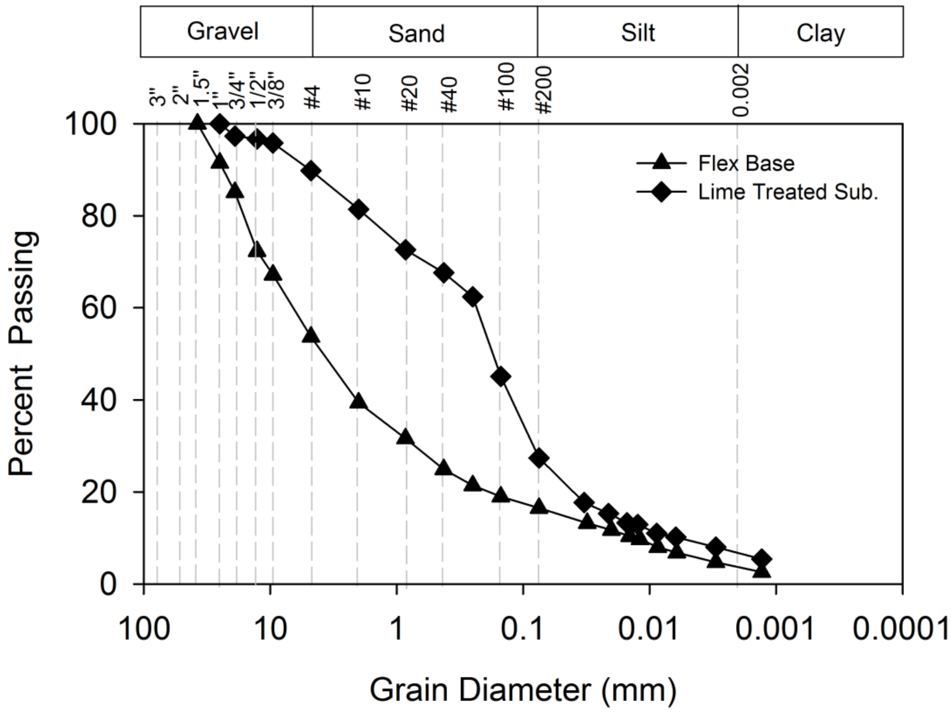


Figure 13. Particle size distribution curves for subgrade materials SH 121 Fort Worth, Texas

4.1.2.2 pH of Stabilized and Natural Subgrade

The pH value of stabilized subgrade was 9.2.

4.1.2.3 SEM Analysis

The energy dispersive spectrometry (EDS) map of stabilized subgrade is shown in Figure 14. The majority elements were silica (Si), alumina (Al), and oxygen (O). Calcium (Ca) is distributed in a few concentrated pockets. Additional elements identified include iron (Fe) and magnesium (Mg).

Figure 15 and Figure 16 compare element concentrations with different magnifications including Al, Si, O, S, Mg, Ca, K, and C for the stabilized subgrade. The sample at 30 × magnifications shows higher concentrations of Ca than that the sample at 150 × magnification. The sample at 500 × magnification shows higher concentrations of Al, O, and Si than the sample at 150× magnification. All SEM images are presented in Figure 17 and Appendix A.

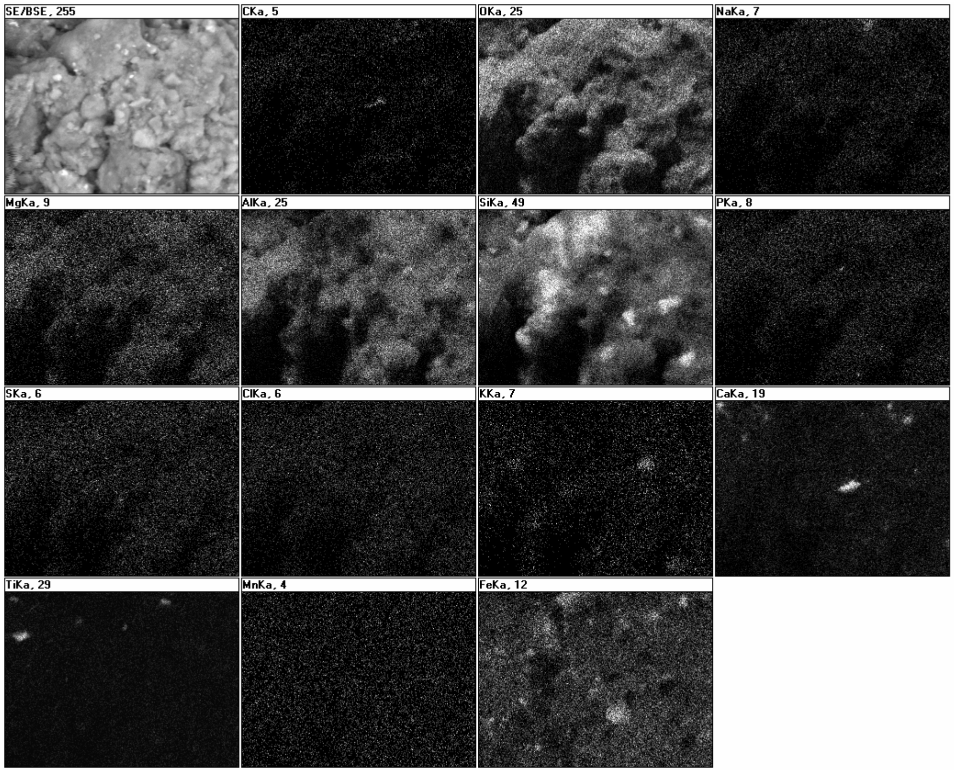


Figure 14. EDS map of stabilized subgrade sample (1500×) SH 121 Fort Worth, Texas

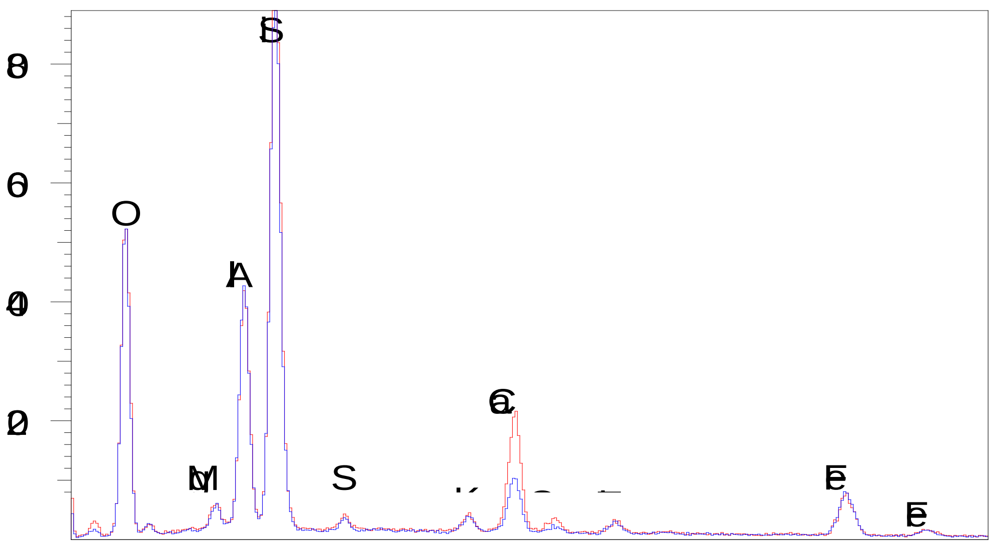


Figure 15. EDS intensity counts for stabilized subgrade sample (red line: 30×; blue line: 150×) SH 121, Fort Worth, Texas

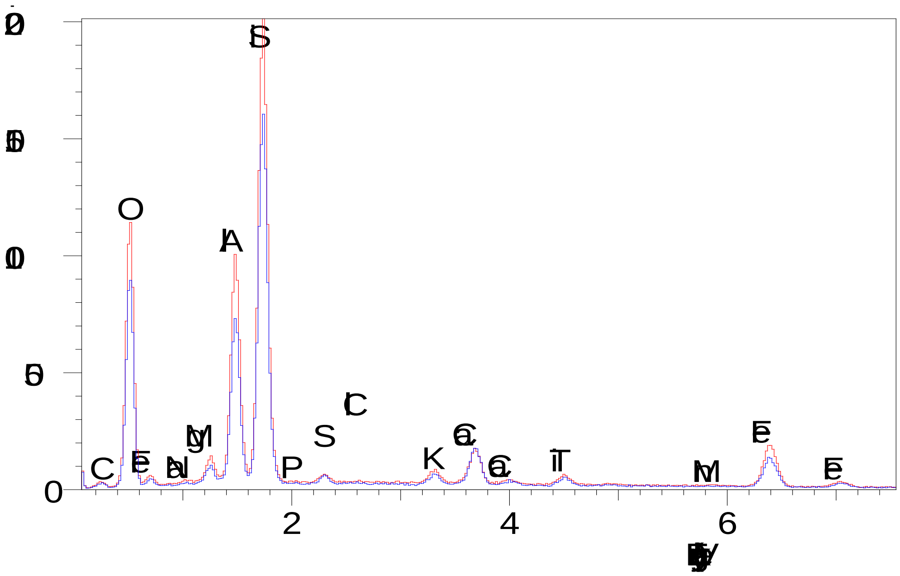


Figure 16. EDS intensity counts for stabilized subgrade sample (red line: 500× magnification, blue line: 150× magnification) SH 121, Fort Worth, Texas

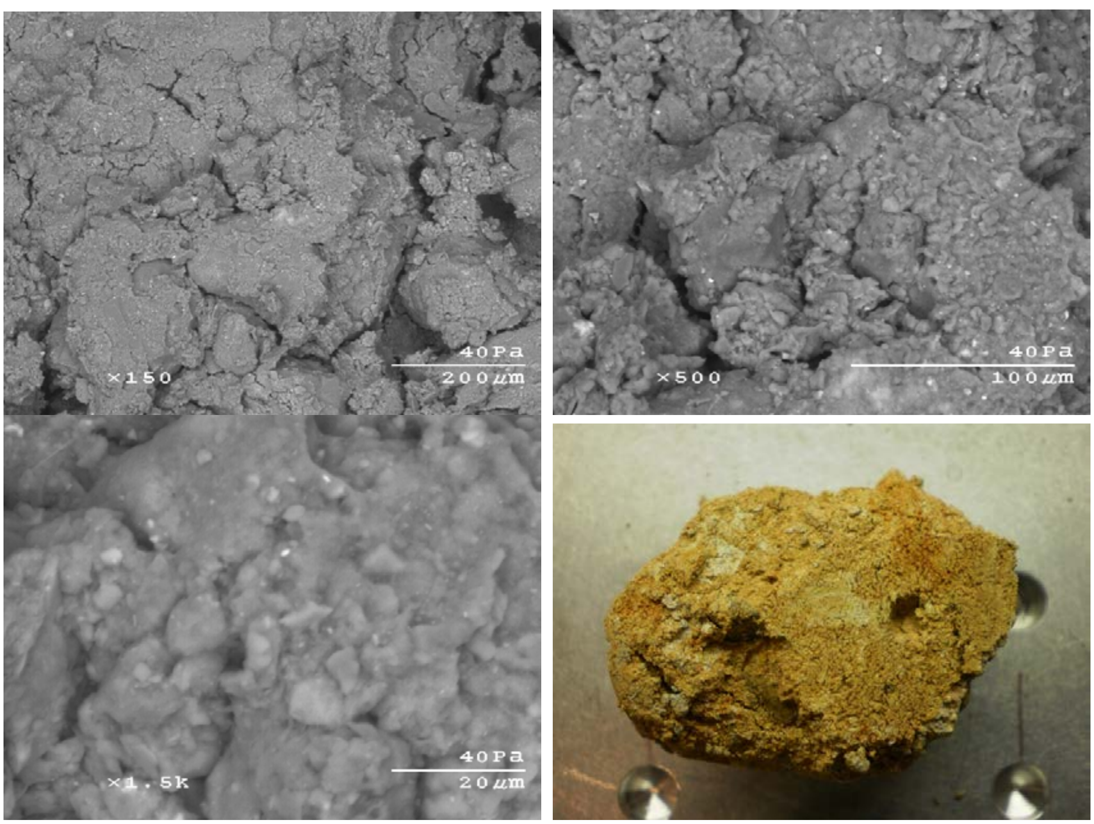


Figure 17. SEM images of stabilized subgrade SH 121, Fort Worth, Texas

4.1.2.4 Stiffness and Strength

CBR values of stabilized and natural subgrade were converted from DPI using Equation (1). The DCP-CBR profile is shown in Figure 18. The major observations are: (1) the average CBR of the stabilized subgrade was about 95 (2) the average CBR increases as the depth increases, and (3) the top 50 mm (2 in.) stabilized subgrade has a lower CBR ranging from 8 to 20.

Back calculated subgrade elastic moduli (E_{FWD}) and deflections (D_0) are presented in Figure 19. In the back calculation, the applied load was 57.7 KN (12965 lb). Poison's ratios were assumed to be 0.35, 0.35, 0.40, and 0.40 for ACC surface layer, flex base, stabilized subgrade, and natural subgrade layer, respectively. Detailed assumptions of seed values and layer thickness are summarized in Appendix B. The key findings are: (1) the average D_0 was about 0.32 mm under the applied average load. As D_0 decreases, back calculated E_{FWD} of both stabilized and natural subgrade increase; (2) the average E_{FWD} was 262 MPa for natural subgrade and increased to 1129 MPa for stabilized subgrade; (3) the average E_{FWD} of stabilized subgrade was about 4.3 times that of the natural subgrade; and (4) the values of E_{FWD} of stabilized and natural subgrade varied significantly indicating non-uniform subgrade properties.

Figure 20 presents the stress-strain relationships of tests at test points 4 and 7. The values of E_{V1} and E_{V2} were calculated in the first circle and after reloading. The uncorrected modulus of soil reaction k′u was calculated using deflection under a load of 69.0 kPa as shown in Figure 21 and Figure 22. The correction of k′u was made using the curve in Figure 21. The average LWD elastic modulus (E_{LWD}) was presented in Table 5. The average E_{LWD} of stabilized subgrade was equal to 0.4 E_{V1} and 0.2 E_{V2}. The elastic modulus ratio between stabilized and natural subgrade is provided in Table 6. The mean value, standard deviation, and coefficient of variation of in situ test results were listed in
Table 7. All in situ test results are presented in Appendix C.

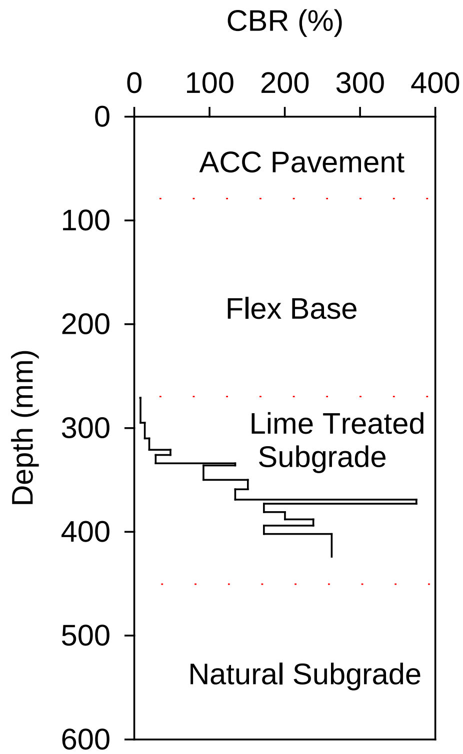


Figure 18. CBR – DCP profile at test point 4 SH 121 Fort Worth, Texas

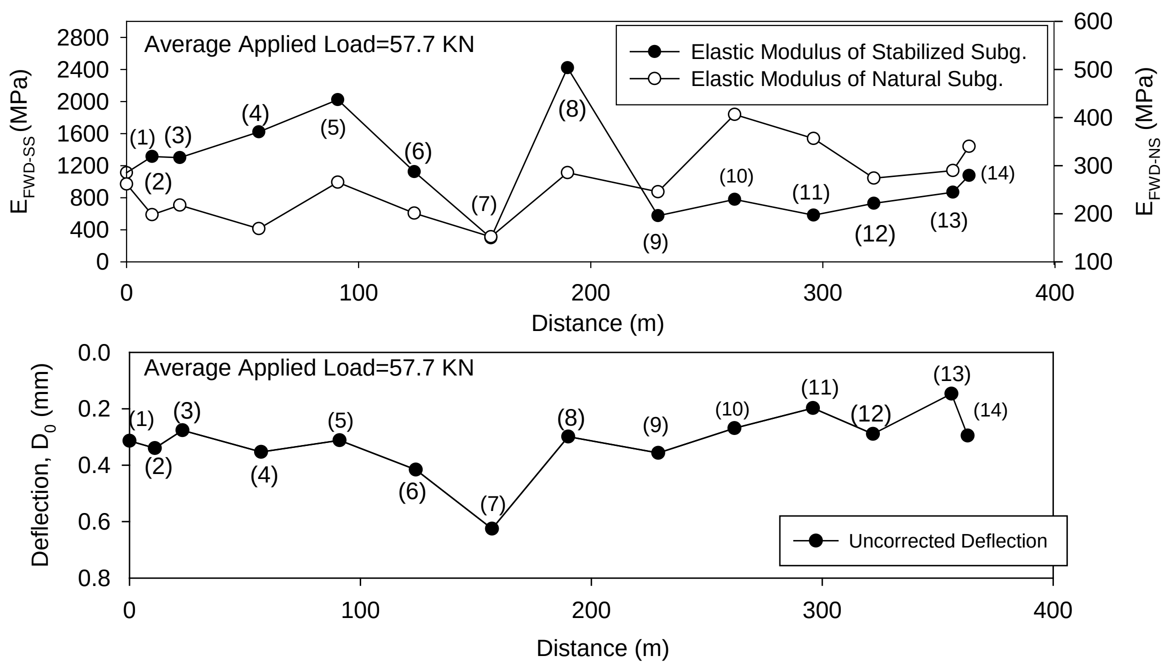


Figure 19. Back calculated FWD elastic modulus of stabilized and natural subgrade, and deflections under the loading plate SH 121 Fort Worth, Texas

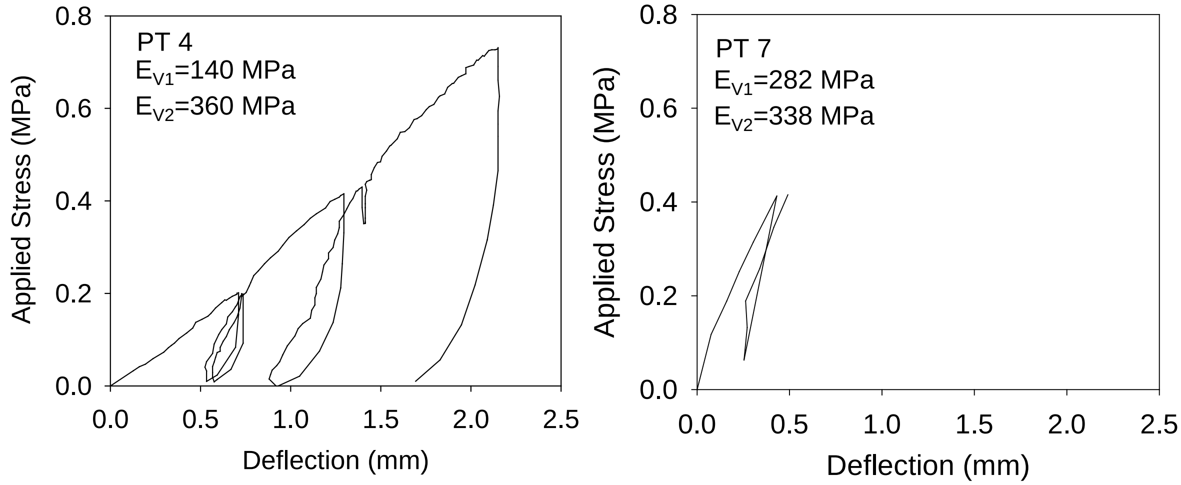


Figure 20. Stress – deflection curves from plate load tests at points 4 and 7 SH 121 Fort Worth, Texas

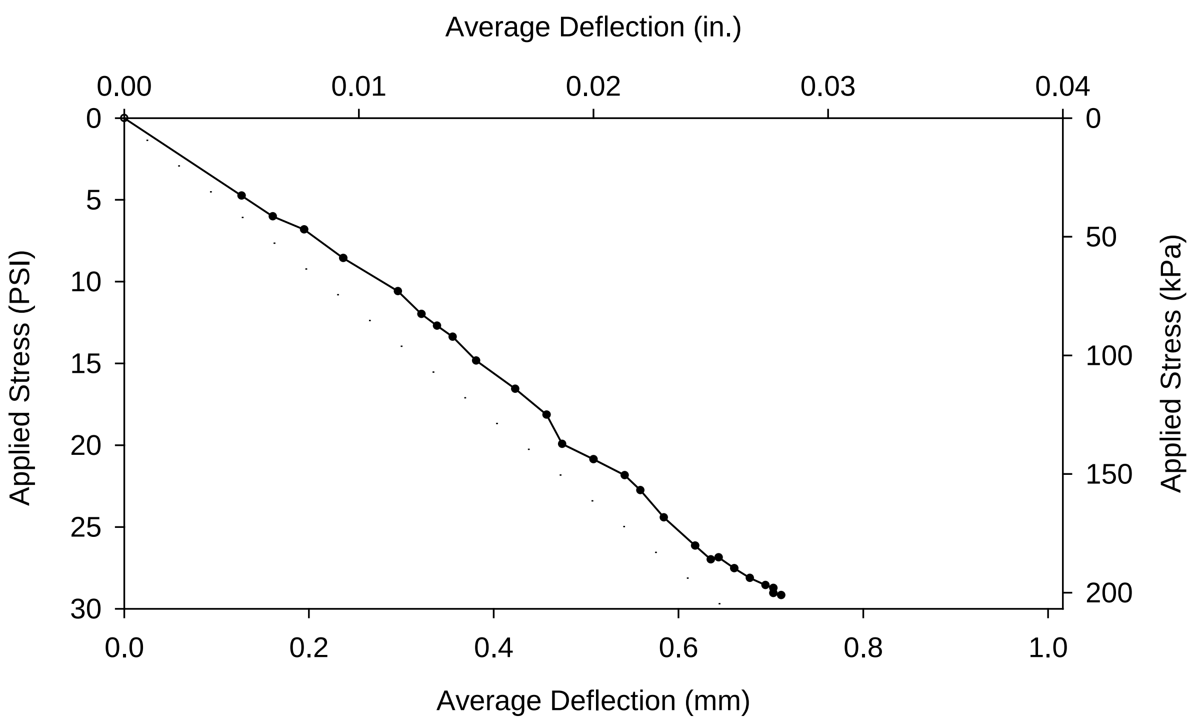


Figure 21. Stress – strain curves for obtaining K_U at point 4 SH 121 Fort Worth, Texas

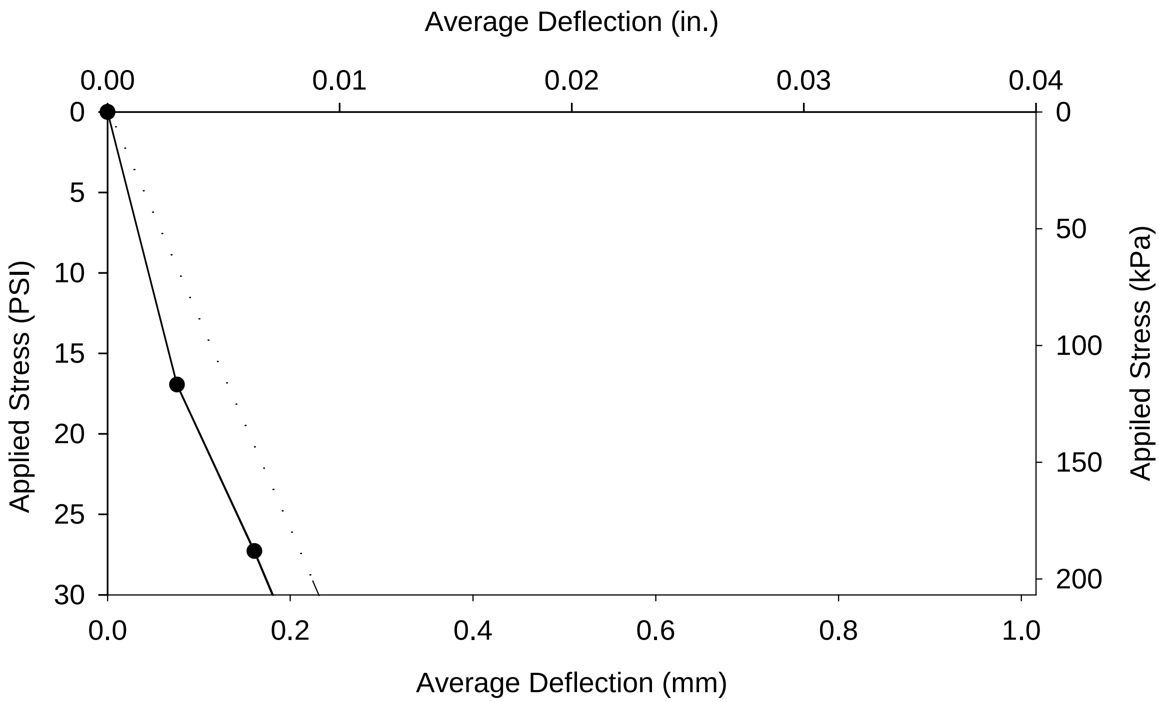


Figure 22. Stress – deflection curves for obtaining Ku at point 8 SH 121 Fort Worth, Texas

Table 5. Summary of LWD test results SH 121 Fort Worth, Texas

Test Point	**Material Type**	**Depth of Measurement**	**E_{LWD}**	**Ave. E_{LWD}**
			MPa	MPa
PT 4	Base	Top of base	93	108
PT 11	Base	25 mm from top of base	125	
PT 4	Base	75 mm from top of base	73	
PT 7	Base	100 mm from top of base	140	
PT 4	Stab. subgrade	Top of stab. subgrade	51	69
PT7	Stab. subgrade	Top of stab. subgrade	87	
PT 11	Stab. subgrade	Top of stab. subgrade	70	

Table 6. Summary of elastic modulus ratio between stabilized and natural subgrade SH 121 Fort Worth, Texas

Ratio of Stab. Subg./Nat. Subg.	
E_{FWD}	4.3

Table 7. Summary statistics of test results from in situ testing SH 121 Fort Worth, Texas

Statistic	Flex Base	Stabilized Subgrade						Nat. Subg.	FWD Def.
Measurement	E_{LWD}	CBR	E_{FWD}	E_{LWD}	E_{V1}	E_{V2}	k_U	E_{FWD}	$D_{0\text{-Cor.}}$
	MPa	%	MPa	MPa	MPa	MPa	kPa/mm	MPa	mm
Number of Measurement (n)	4	1	14	3	2	2	2	14	14
Mean Value (μ)	108	119	1129	69	211	349	182	262	0.32
Standard Deviation (σ)	30	—	583	18	100	16	—	72	0.11
Coefficient of Variation COV(%)	28	—	52	26	48	4	—	28	33

4.2 FM 1709, TX

4.2.1 Site Description

This project was located on the west bound of FM 1709 in Fort Worth, Tarrant County, Texas. The general location of this site is shown in Figure 23. This road is a six-lane roadway. The old pavement was constructed in 1987, and originally consisted of a 100 mm (4 in.) thick asphalt concrete (AC), a 150 mm (6 in.) flex base, and 150 mm (6 in.) lime stabilized subgrade. A 50 mm (2 in.) HMA overlay was placed in 2007. The pavement currently consists of a 150 mm (6 in.) thick asphalt concrete (AC), 200 mm (8 in.) flex base, and 150 mm (6 in.) lime stabilized subgrade. The length of this test section is approximately 300 m (984 ft). The ISU research team conducted in situ testing on August 4, 2010 with assistance and traffic control provided by Texas DOT.

The plan view of in situ test locations is shown in Figure 24. The research team preformed FWD tests on the surface of ACC pavement at intervals of about 40 m from test points 1 to 7. DCP tests were conducted at test point 1. After coring, LWD and PLT tests were performed on the top of stabilized subgrade at test point 1. Bag samples of base and stabilized subgrade were collected at test point 1.

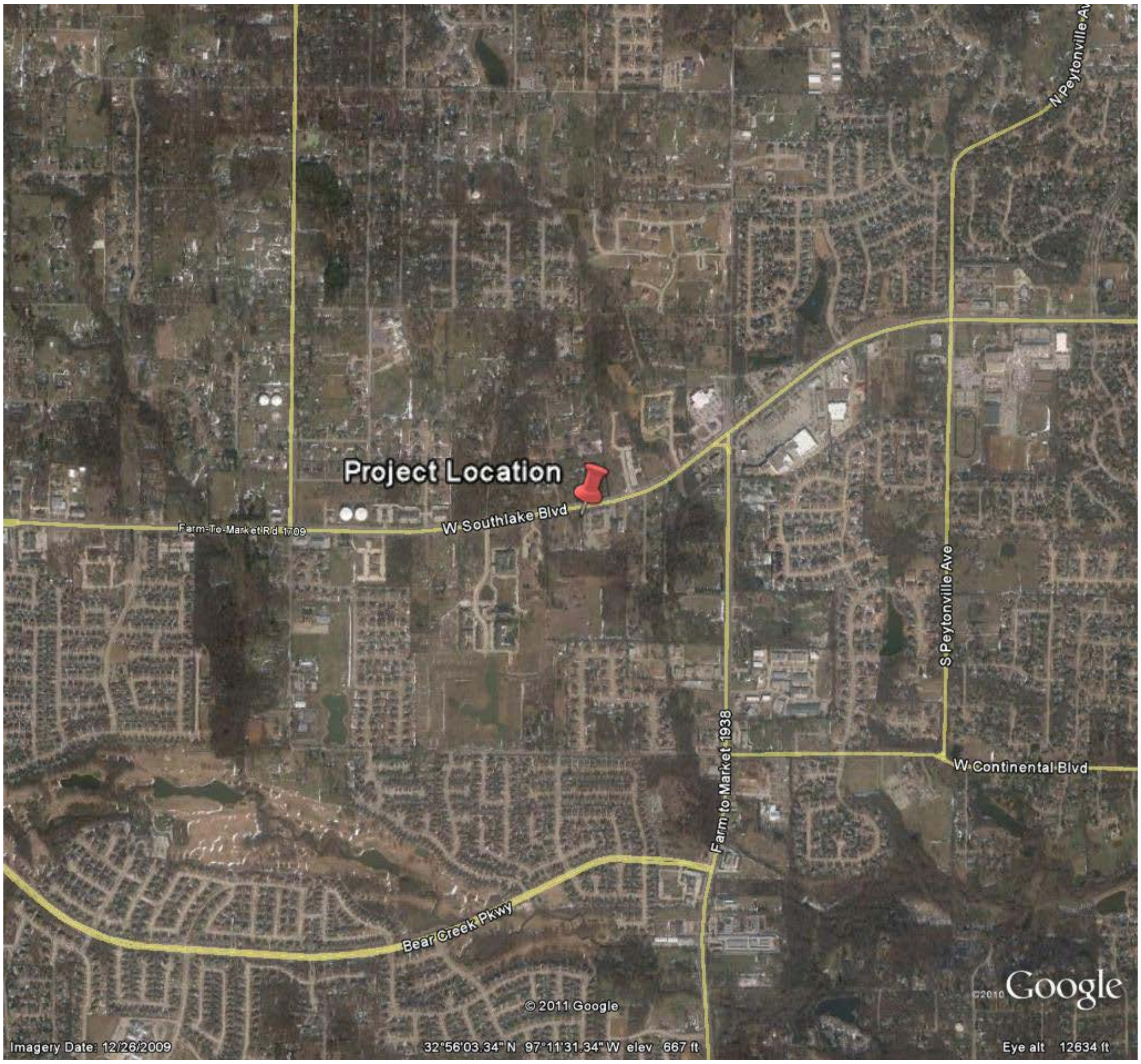


Figure 23. Project location of FM 1709 Fort Worth, Texas

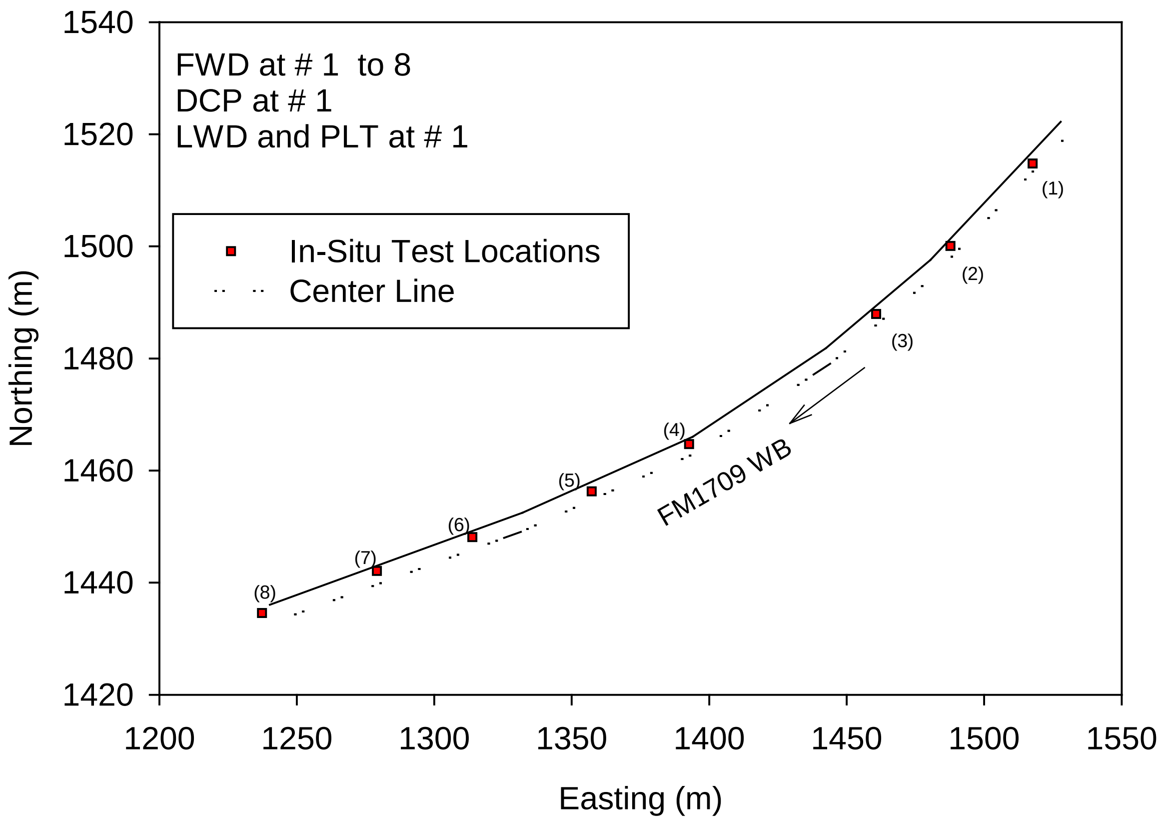


Figure 24. Test section plan layout based on RTK GPS FM 1709 Fort Worth, Texas

Figure 25. Site overview FM 1709 Fort Worth, Texas

4.2.2 Test Results and Analysis

4.2.2.1 Material Properties of Base and Subgrade

According to USCS and AASHTO, the flexible base was classified as GM and A-1-b, and the stabilized subgrade was classified as SM and A-4. Table 8 provides material properties of the subgrade. The stabilized subgrade is a non-plastic soil. Figure 26 shows particle size distribution curves of the base and subgrade.

Table 8. Summary of material properties FM 1709 Fort Worth, Texas

Parameter	**Materials**	
Material Description	Base	Stabilized Subgrade
Depth mm (in.)	0-200 (0-8)	0-75 (0-3)
Gravel Content (%) (> 4.75mm)	42.8	4.2
Sand Content (%) (4.75mm – 75µm)	37.1	55.2
Silt Content (%) (75µm – 2µm)	15.4	36.9
Clay Content (%) (< 2µm)	4.7	3.7
Coefficient of Uniformity (C_u)	856.4	14.1
Coefficient of Curvature (C_c)	10.0	2.2
Liquid Limit, LL (%)	21.2	—
Plasticity Index, PI	7.5	N.P.
AASHTO	A-1-b	A-4
USCS	GM	SM
Water Content (%)	7.0	17.3

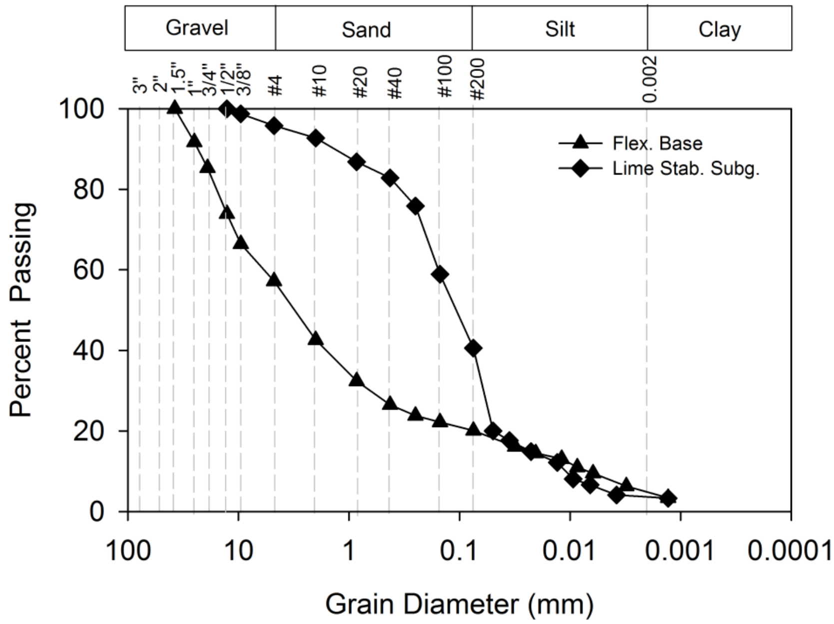


Figure 26. Particle size distribution curves for subgrade materials FM 1709 Fort Worth, Texas

4.2.2.2 pH of Stabilized and Natural Subgrade

The pH value of stabilized sample was 9.6.

4.2.2.3 SEM Analysis

The energy dispersive spectrometry (EDS) map of stabilized subgrade is shown in Figure 27 and Figure 28. The majority elements were calcium (Ca), silica (Si), alumina (Al), and oxygen (O). These elements commonly exist in lime stabilized subgrade. Additional elements were iron (Fe), potassium (K), and Sodium (Na).

Figure 29 shows element concentration in Al, Si, O, S, Mg, Ca, K, and C for stabilized subgrade. The stabilized subgrade sample has higher concentration of Si, Al, O, and Ca, and less concentration of C, Fe, and Mg. All SEM images are presented in Figure 30 and Appendix A.

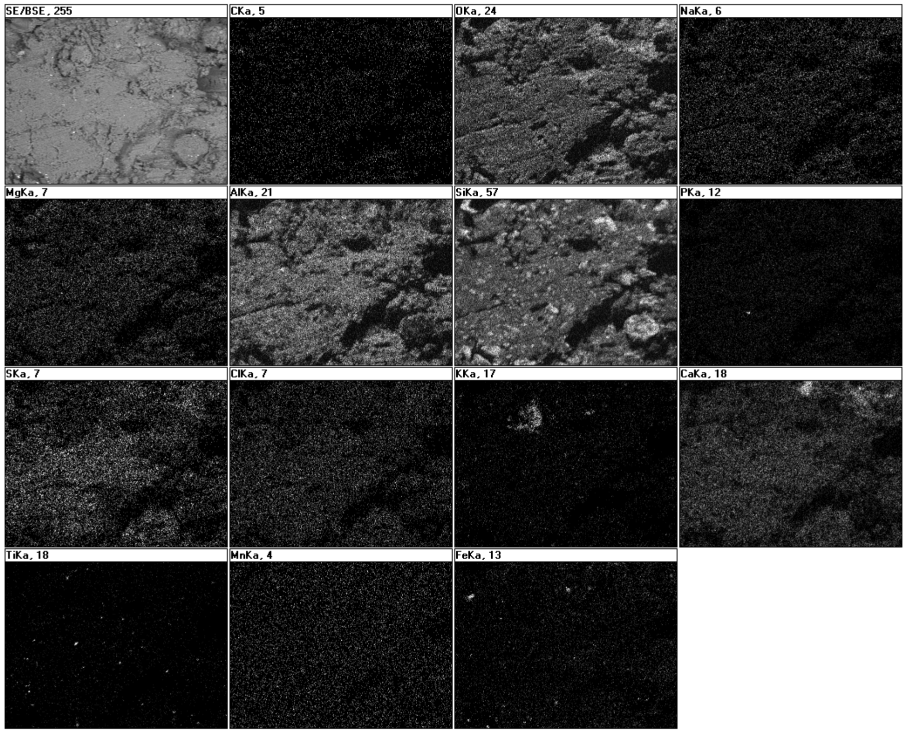


Figure 27. EDS map of stabilized subgrade sample (150 ×) FM 1709 Fort Worth, Texas

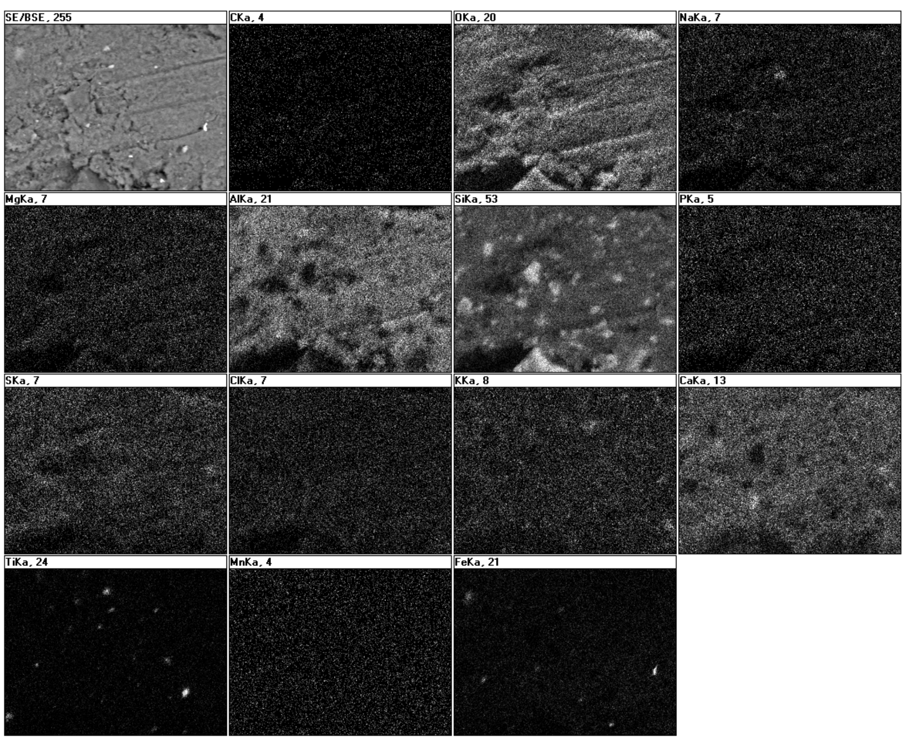


Figure 28. EDS map of stabilized subgrade sample (800 ×) FM 1709 Fort Worth, Texas

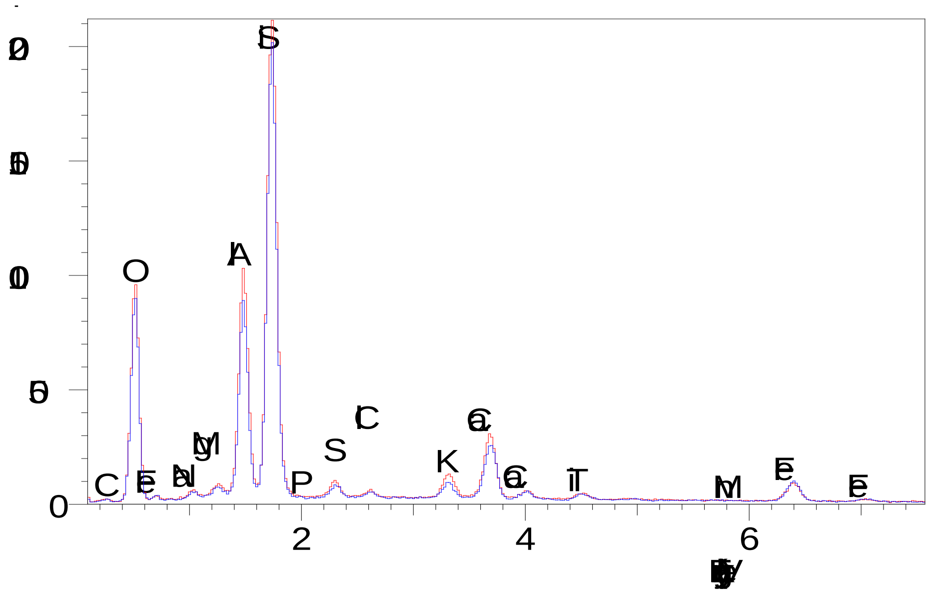


Figure 29. EDS intensity counts for stabilized subgrade sample (red line: 500×; blue line: 150×) FM 1709 Fort Worth, Texas

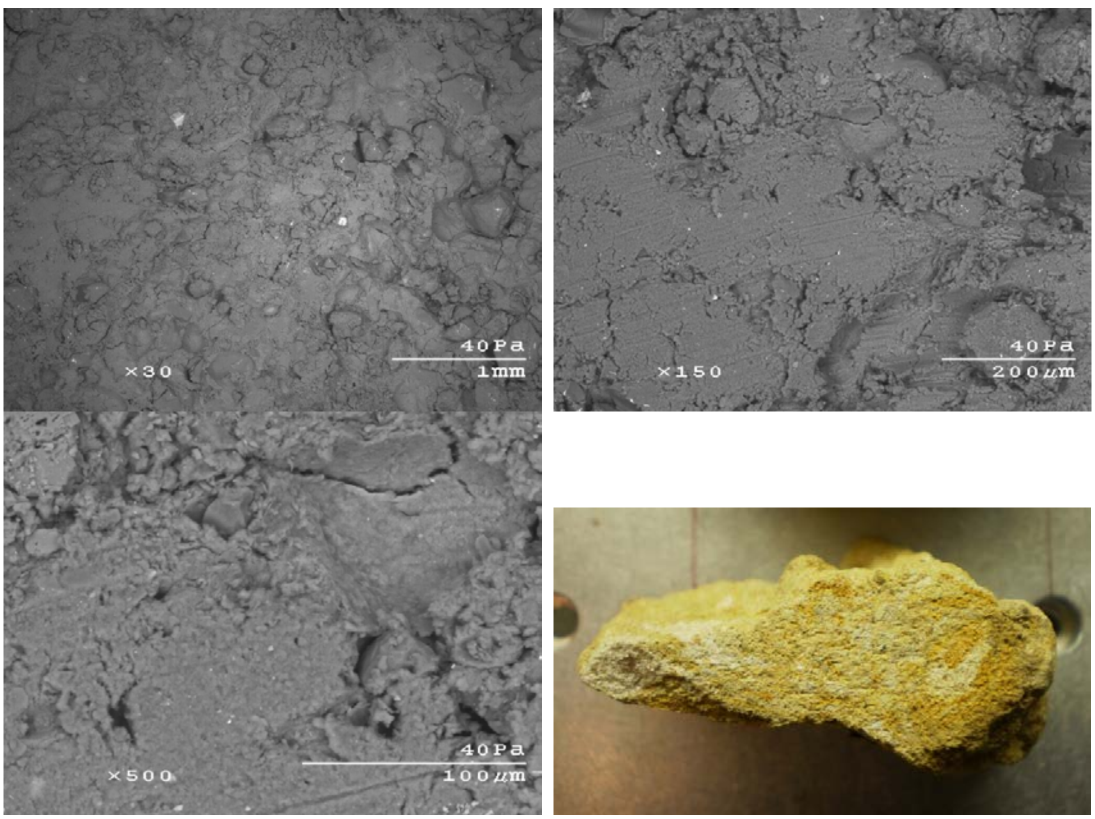


Figure 30. SEM images of stabilized subgrade FM 1709 Fort Worth, Texas

4.2.2.4 Stiffness and Strength

CBR values of stabilized and natural subgrade were converted from DPI using Equation (1). The DCP-CBR profile and cumulative drops versus CBR are shown in Figure 31. The following observations were: (1) the average CBR of the stabilized subgrade was 53, (2) the average CBR of the natural subgrade was 24, (3) the CBR of the stabilized subgrade was 2.2 times higher than the natural subgrade, and (4) the top 50 mm (2 in.) layer of stabilized subgrade has a lower CBR ranging from 10 to 30.

Back calculated subgrade elastic moduli (E_{FWD}) and deflections (D_0) are presented in Figure 32. In the back calculation, the applied test load was 56.0 KN (12573 lb). Poison's ratios were assumed to be 0.35, 0.35, 0.40, and 0.40 for ACC surface layer, flex base, stabilized subgrade, and natural subgrade layer respectively. Stabilized subgrade moduli were calculated based on designed or effective stabilized subgrade thickness obtained from DCP profiles. Detailed assumptions of seed values and layer thickness are summarized in Appendix B. The key findings are: (1) the average D_0 was about 0.45 mm under the applied average load. As D_0 decreases, back-calculated E_{FWD} of stabilized and natural subgrade increase; (2) the average E_{FWD} was 127 MPa for natural subgrade and increased to 396 MPa for stabilized subgrade; (3) the average E_{FWD} of stabilized subgrade was about 3.1 times higher than the natural subgrade; (4) for those test points, the values of E_{FWD} of stabilized and natural subgrade varied significantly indicating non-uniform subgrade soil properties.

Figure 33 presents the stress-deflection relationship of the plate load test at test point 1. The values of E_{V1} and E_{V2} were calculated in the first cycle and after reloading. The uncorrected modulus of soil reaction k′u was calculated using deflection under a load of 69.0 kPa as shown in Figure 34. The correction of k′u was made using the curve in Figure 34. The average LWD elastic modulus (E_{LWD}) of stabilized subgrade was equal to 1.4 E_{V1} and 1.0 E_{V2}.

Table 9 provides the elastic modulus ratio between stabilized and natural subgrade. The mean value, standard deviation, and coefficient of variation of in situ test results were listed in Table 9. All in situ test results are presented in Appendix C.

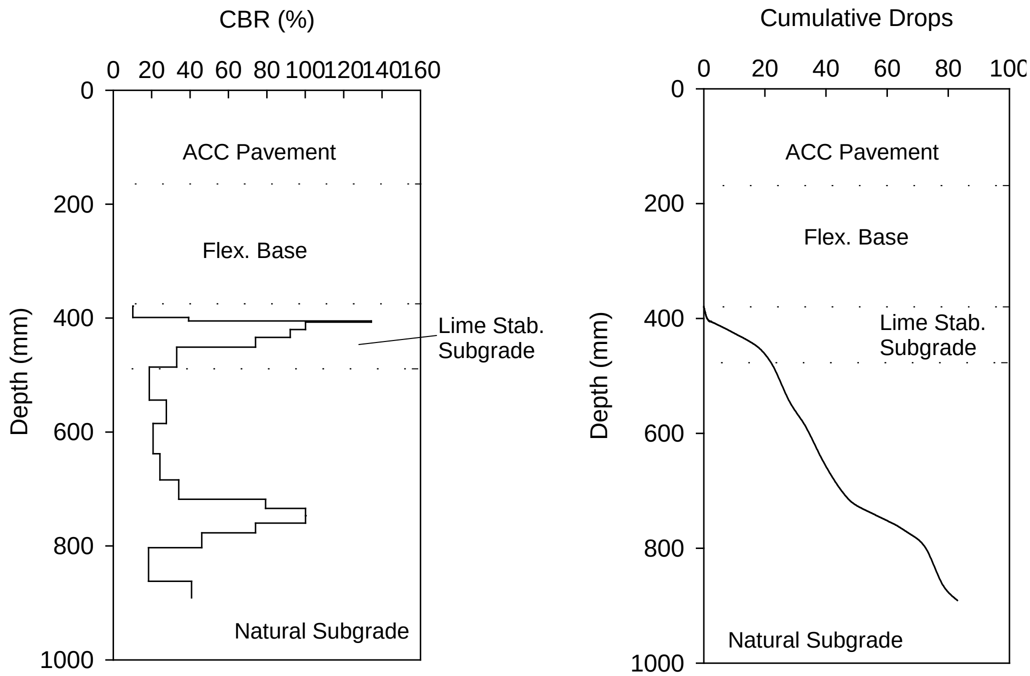


Figure 31. CBR – DCP profile and cumulative drops versus CBR at test point 1 FM 1709 Fort Worth, Texas

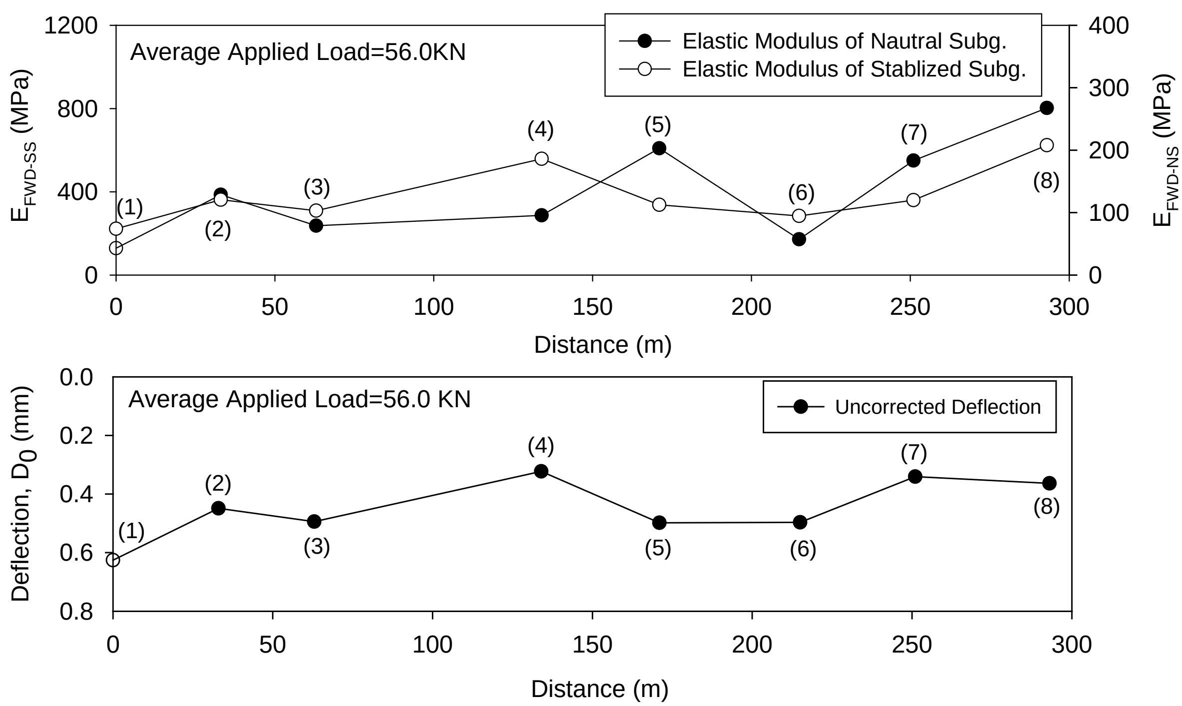


Figure 32. Back-calculated FWD elastic modulus of stabilized and natural subgrade, and deflections under the loading plate FM 1709 Fort Worth, Texas

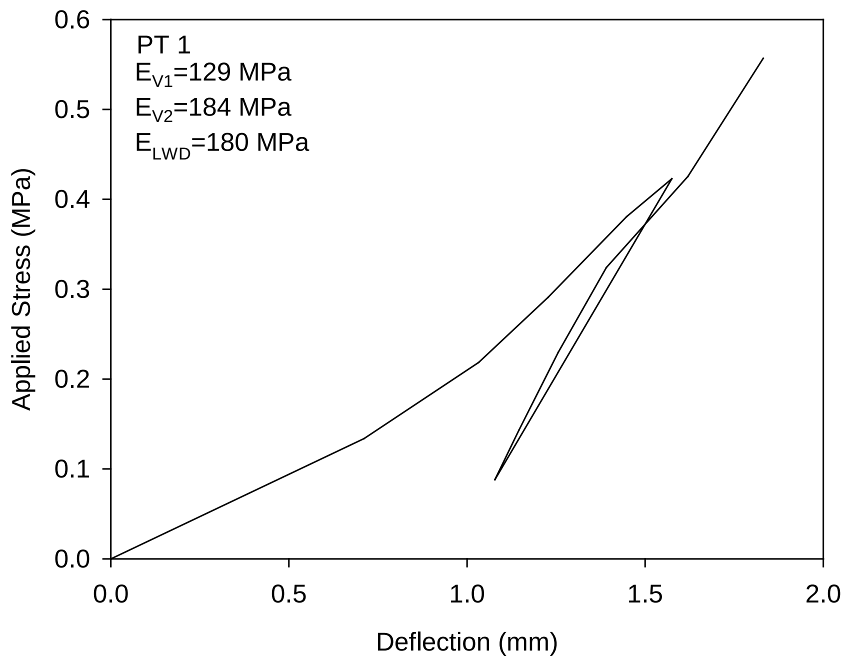


Figure 33. Corrected stress – strain curve from plate load test at point 1 FM 1709 Fort Worth, Texas

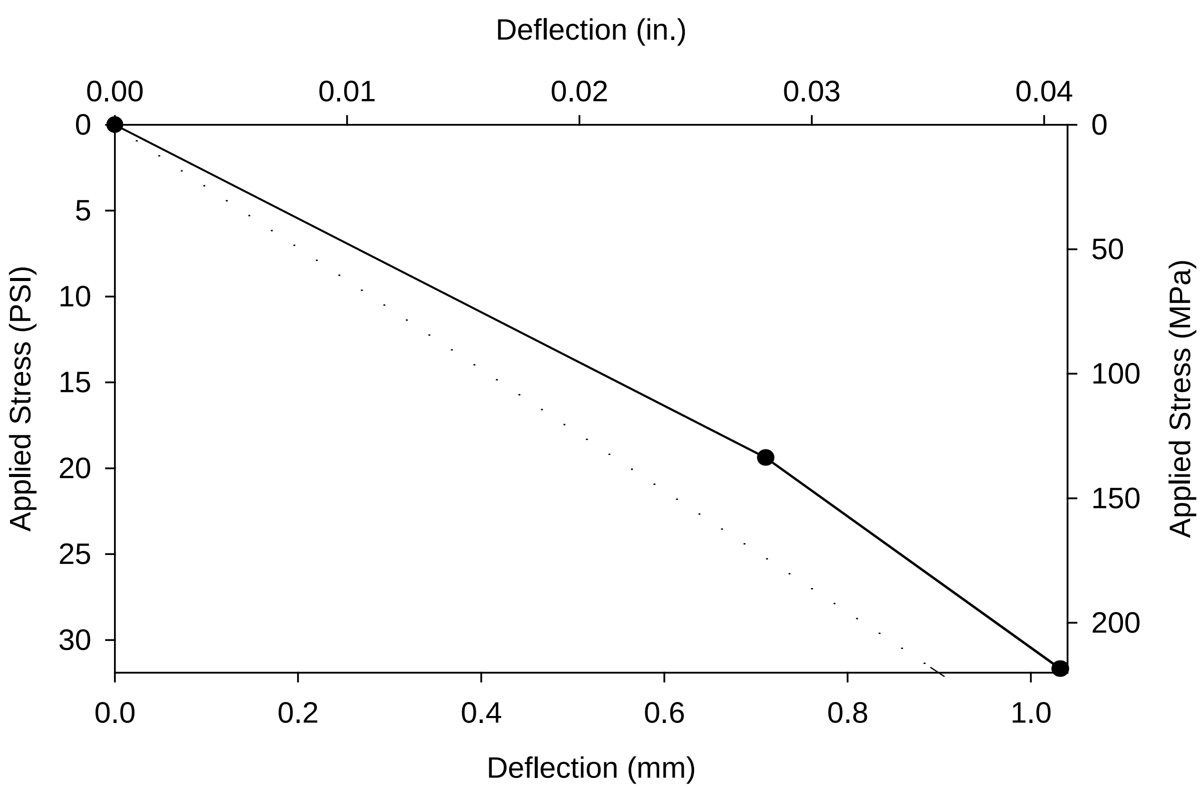


Figure 34. Stress – strain curves for obtaining K_U at point 1 FM 1709 Fort Worth, Texas

Table 9. Summary of elastic modulus ratio between stabilized and natural subgrade

Stab. Subg./Nat. Subg. Ratio	
CBR	E_{FWD}
2.2	3.1

Table 10. Summary statistics of test results from in situ testing FM 1709 Fort Worth, Texas

Statistic	**Stabilized Subgrade**							**Natural Subgrade**		**FWD Def.**
Measurement	CBR	E_{LWD}	E_{V1}	E_{V2}	E_{FWD}	k_U	Thi.	CBR	E_{FWD}	D_0
	%	MPa	MPa	MPa	MPa	kPa/mm	mm	%	MPa	mm
Number of Measurement (n)	1	1	1	1	8	1	1	1	8	8
Mean Value (μ)	53	180	129	184	396	99	100	24	127	0.45
Standard Deviation (σ)	—	—	—	—	237	—	—	—	46	0.10
Coefficient of Variation COV (%)	—	—	—	—	60	—	—	—	36	23

4.3 US 287, TX

4.3.1 Site Description

This project was located on the south bound lane of US 287 in Mansfield, Tarrant County, Texas. The general location of this site is shown in Figure 35. The road is a four-lane U.S. Highway. The old pavement was constructed in 1982, and originally consisted of a 38 mm (1.5 in.) thick asphalt concrete (AC), 280 mm (11 in.) flex base, and 356 mm (14 in.) lime stabilized subgrade. An HMA overlay with a thickness of 50 mm (2 in.) was placed in 2008. The pavement currently consists of an 89 mm (3.5 in.) thick asphalt concrete (AC), 280 mm (11 in.) flex base, and 356 mm (14 in.) lime stabilized subgrade. The length of this test section is approximately 600 m (1969 ft). The ISU research team conducted in situ testing on August 5, 2010 with assistance and traffic control provided by Texas DOT.

The plan view of in situ test locations is shown in Figure 36. The research team preformed FWD tests on the surface of the ACC pavement at intervals of about 20 to 30 m from test points 1 to 19. DCP were conducted at test points 12, 15, and 16. After coring, LWD and a PLT test was performed on the top of stabilized subgrade at test point 12. Bag samples of base and stabilized subgrade were collected at test point 12.

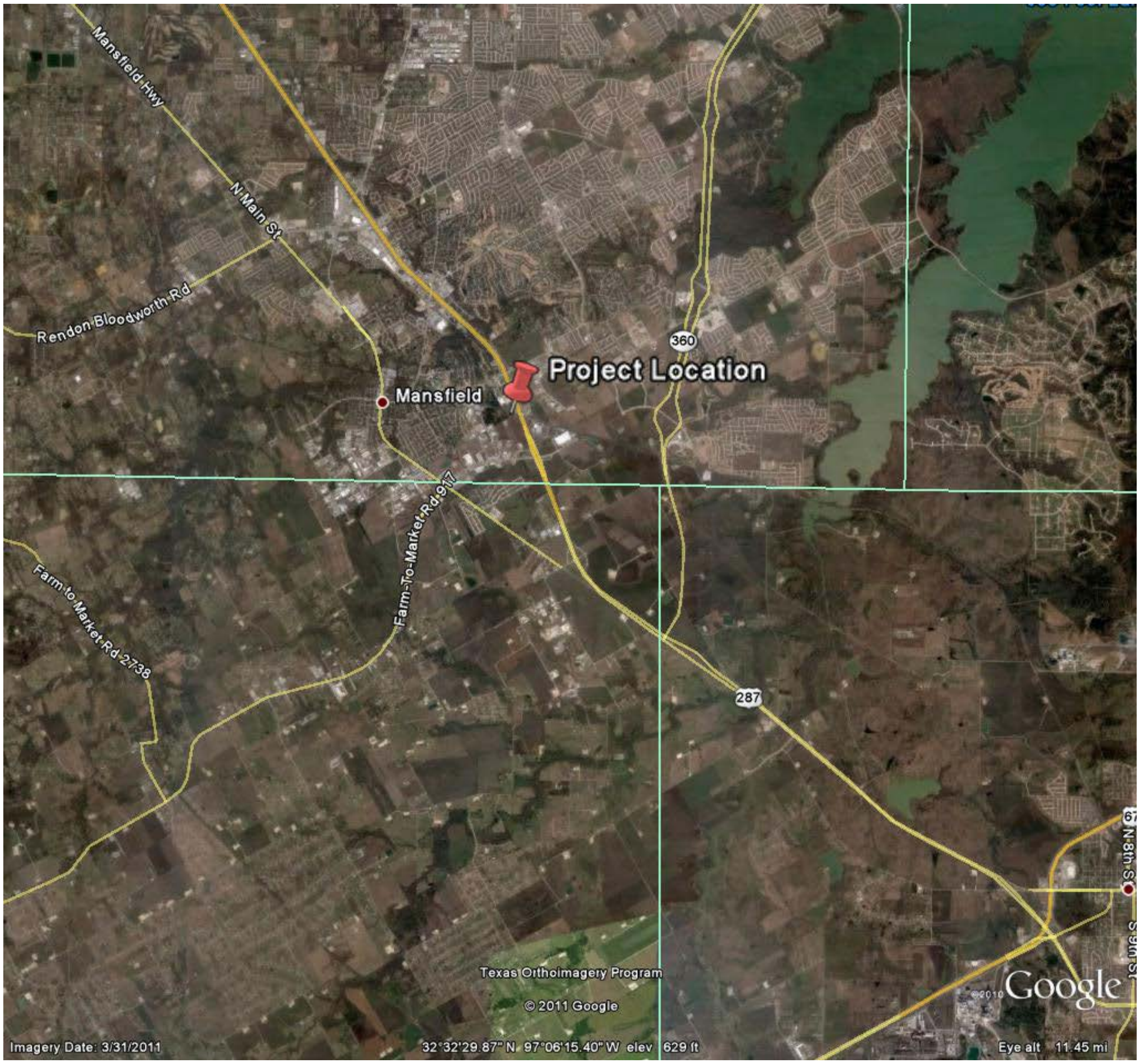


Figure 35. Project location of US 287 Mansfield, Texas

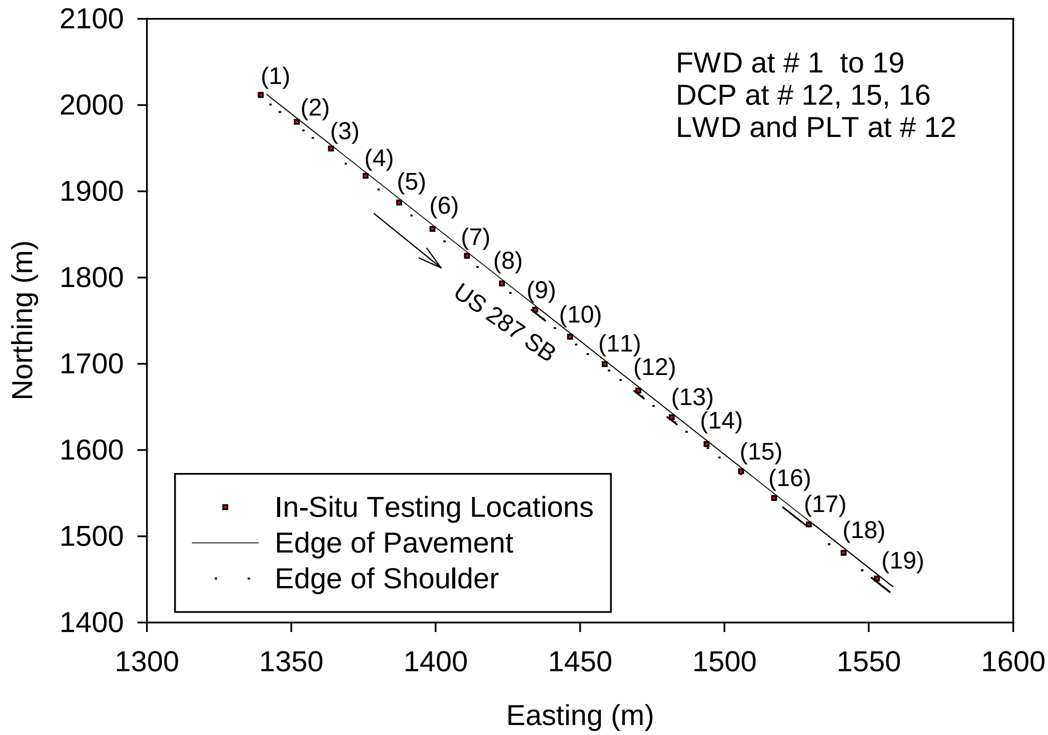


Figure 36. Test section plan layout US 287 Mansfield, Texas

Figure 37. Site overview US 287 Mansfield, Texas

4.3.2 Test Results and Analysis

4.3.2.1 Material Properties of Base and Subgrade

The base and stabilized subgrade samples were taken from different depths at test point 12 from the top to a depth of 200 mm (8 in.). USCS and AASHTO classifications of the top 50 mm (2 in.) stabilized subgrade is ML and A-4, and the stabilized subgrade from a depth of 50-200 mm (2-8 in.) is SM and A-4. It was observed that the top 50 mm (2 in.) stabilized soil was different than the stabilized subgrade from a depth of 50-200 mm (2-8 in.). Table 11 provides material properties of base and stabilized subgrade. The average PI value of the top 50 mm (2 in.) stabilized subgrade samples is higher than the stabilized subgrade from a depth of 50-200 mm (2-8 in.). Figure 38 shows particle size distribution curves of base and subgrade materials at varied depths. Test results show the soil type of subgrade has been modified after treatment.

Table 11. Summary of material properties US 287 Mansfield, Texas

Parameter	Materials			
Material Description	Base	Stab. Subgrade	Stab. Subgrade	Stab. Subgrade
Depth mm (in.)	0-280 (0-11)	0-50 (0-2)	50-150 (2-6)	150-200 (6-8)
Gravel Content (%) (> 4.75mm)	51.1	6.4	2.6	17.2
Sand Content (%) (4.75mm – 75µm)	32.6	36.7	56.8	47.1
Silt Content (%) (75µm – 2µm)	11.8	31.6	26.9	29
Clay Content (%) (< 2µm)	4.5	25.3	13.7	6.7
Coefficient of Uniformity (C_u)	692.7	286	346.8	262.1
Coefficient of Curvature (C_c)	17.0	0.21	0.22	1.24
Liquid Limit, LL (%)	17.0	54.8	54.4	54.6
Plasticity Index, PI	6.6	20.0	12.9	13.4
AASHTO	A-1-b	A-4	A-4	A-4
USCS	GM	ML	SM	SM
Water Content (%)	6.5	33.3	35.4	36.7

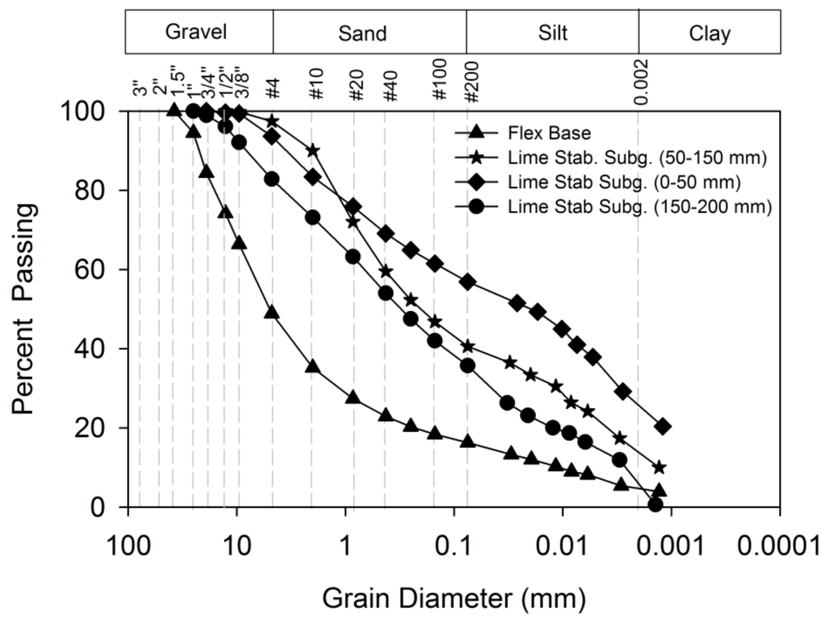


Figure 38. Particle size distribution curves for subgrade materials US 287 Mansfield, Texas

4.3.2.2 pH of Stabilized and Natural Subgrade

Table 12 shows pH values of stabilized subgrade from a depth of 0-200 mm (0-8 in.). It decreases gradually from the top to bottom of stabilized subgrade.

Table 12. Summary of pH value of subgrade US 287 Mansfield, Texas

Depth mm (in.)	pH
0-50 (0-2)	8.2
50-150 (2-6)	8.7
150-200 (6-8)	9.2

4.3.2.3 SEM Analysis

The energy dispersive spectrometry (EDS) map of stabilized subgrade is shown in Figure 39. The majority elements were calcium (Ca), silica (Si), alumina (Al), and oxygen (O). These elements commonly exist in lime stabilized subgrade. Additional elements were iron (Fe), potassium (K), and Sodium (Na).

Figure 40 and Figure 41 compares element concentration in Al, Si, O, S, Mg, Ca, K, and C for stabilized subgrade. The sample shows higher concentration of Ca, Si, Al, and O, and less concentration of Fe, S, and Mg. All SEM images are presented in Figure 42 to Figure 48 and Appendix A.

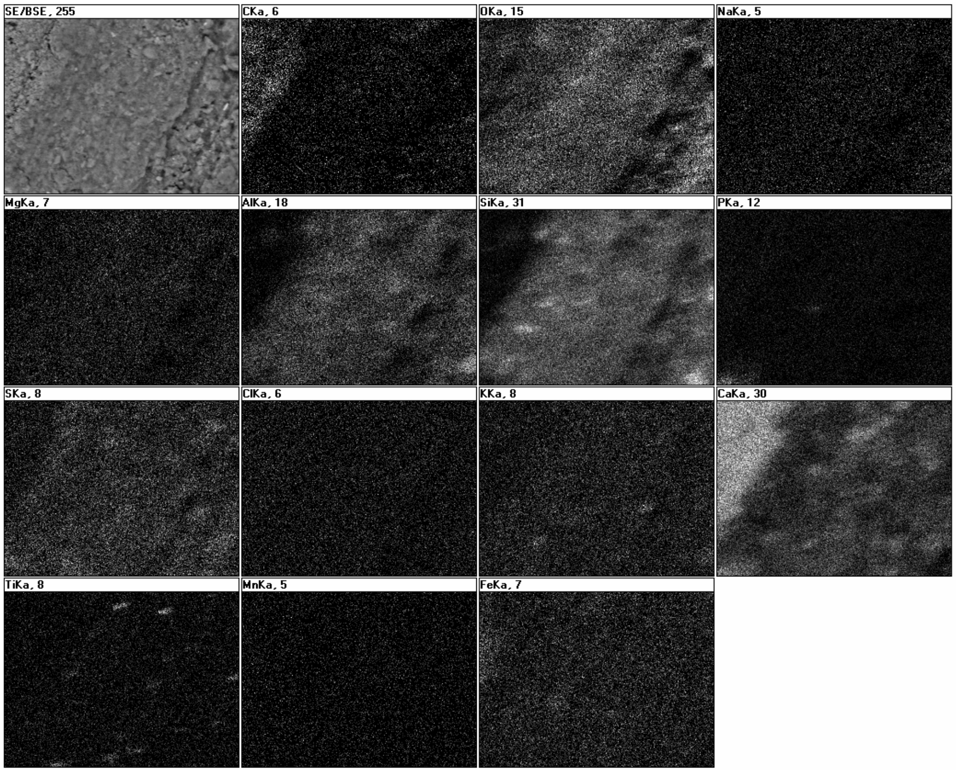


Figure 39. EDS map of stabilized subgrade sample (1000 × magnification) US 287 Mansfield, Texas

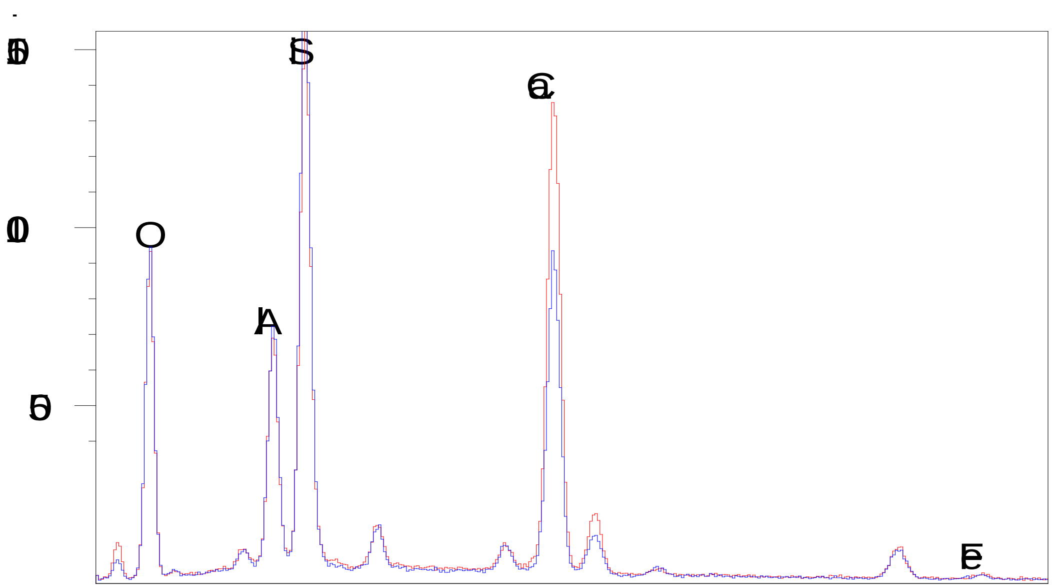


Figure 40. EDS intensity counts for stabilized subgrade sample (red line: 500×; blue line: 150×) US 287 Mansfield, Texas

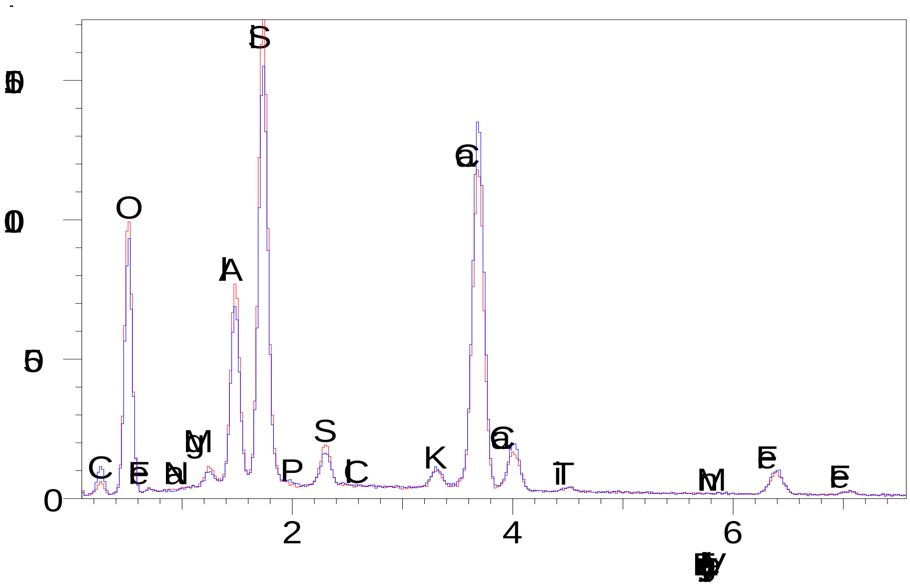


Figure 41. EDS intensity counts for stabilized subgrade sample (red line: 1000×; blue line: 500×) US 287 Mansfield, Texas

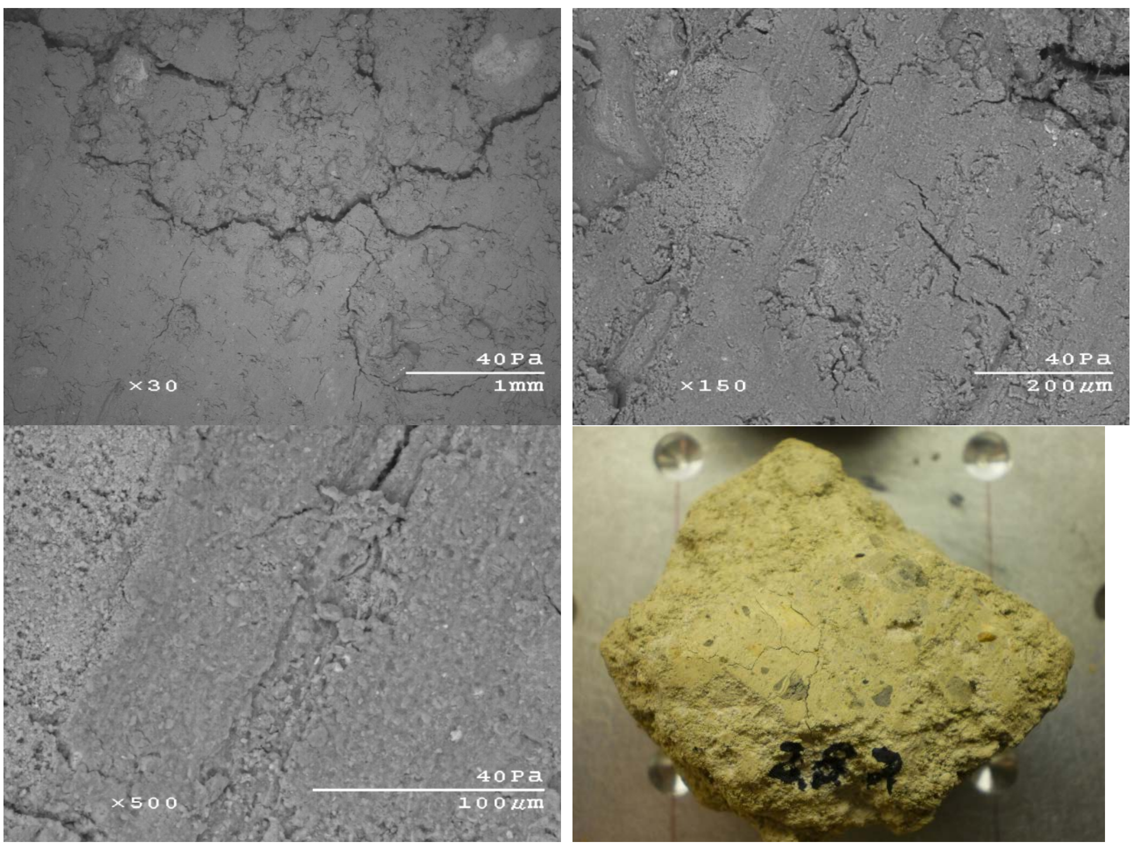


Figure 42. SEM images of stabilized subgrade US 287 Mansfield, Texas

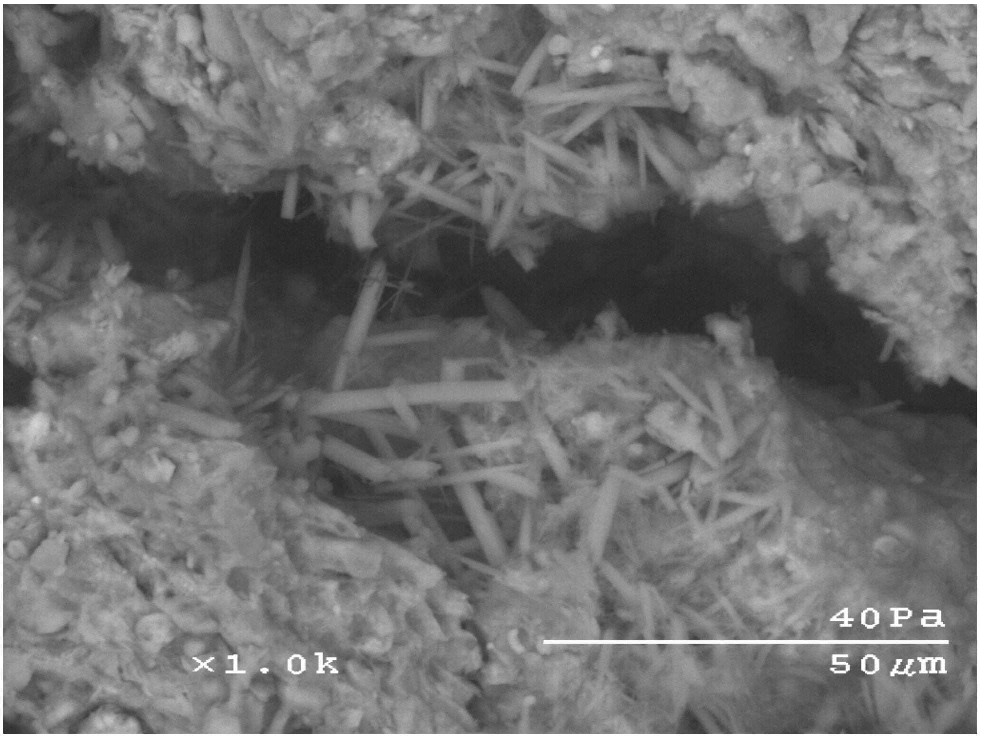


Figure 43. SEM image of stabilized subgrade in area b (1000 ×) US 287 Mansfield, Texas

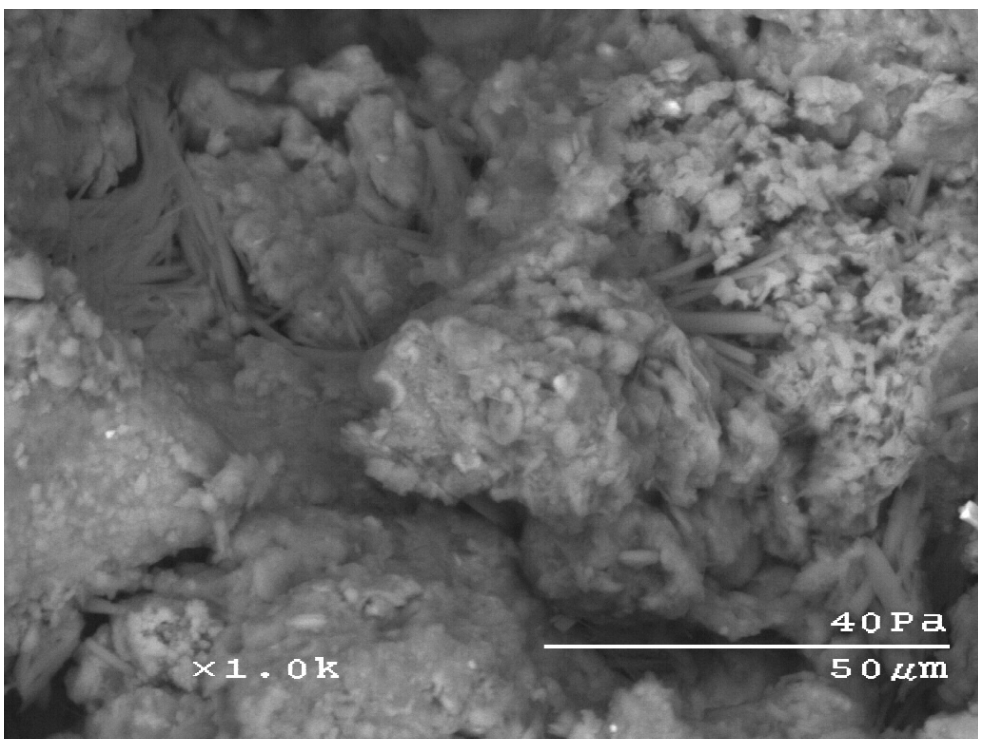


Figure 44. SEM image of stabilized subgrade (1000 ×) US 287 Mansfield, Texas

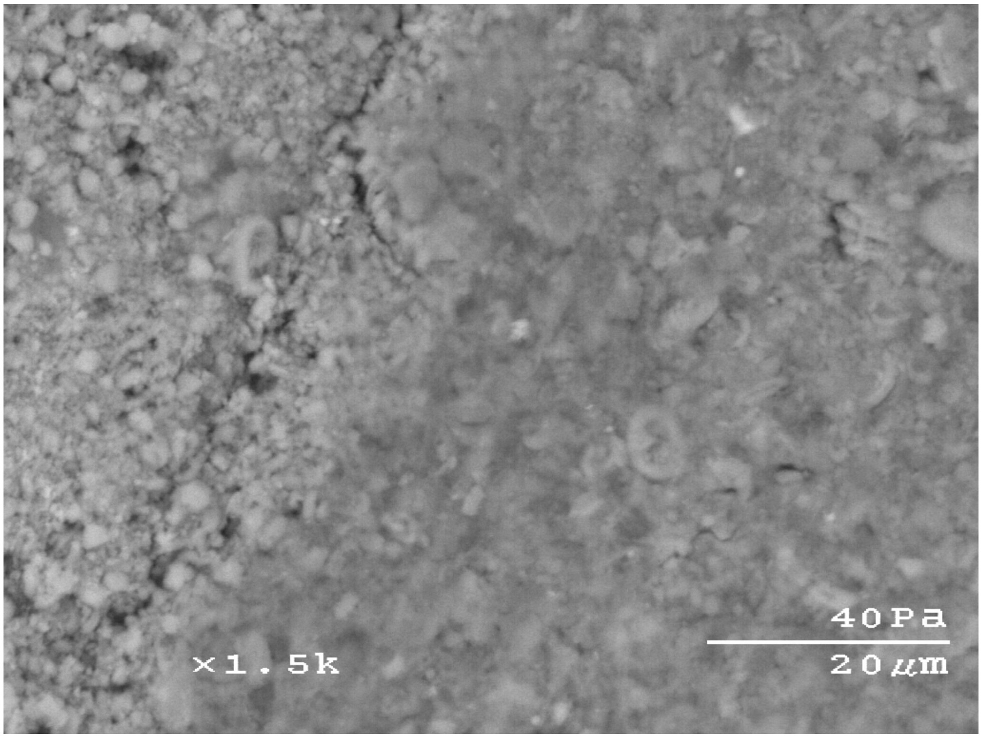


Figure 45. SEM image of stabilized subgrade (1500 ×) US 287 Mansfield, Texas

4.3.2.4 Stiffness and Strength

CBR values of stabilized and natural subgrade are converted from DPI using Equation (1). DCP-CBR profile and cumulative drops versus CBR are shown in Figure 46. The major observations are: (1) the average CBR of the stabilized subgrade was 163%, (2) the average CBR of the natural subgrade was 22%, (3) the CBR of the stabilized subgrade was 740% of the natural subgrade, (4) the top and bottom layer of stabilized subgrade has a lower CBR than the middle layer, and (5) from DCP profiles, the actual treatment thickness was thicker than the design value.

Back calculated subgrade elastic moduli (E_{FWD}) and deflections (D_0) were presented in Figure 47. In the back calculation, the applied test load was 57.0 KN (12785 lb). The assumptions of poison's ratio were 0.35, 0.35, 0.40, and 0.40 for ACC surface layer, flex base, stabilized subgrade, and natural subgrade layer respectively. Stabilized subgrade moduli were calculated based on designed or effective stabilized subgrade thickness obtained from DCP profiles. Detailed assumptions of seed values and layer thickness are summarized in Appendix B. The key findings are: (1) the average D_0 was about 0.34 mm under the applied average load. As D_0 decreases, back calculated E_{FWD} of both stabilized and natural subgrade increase; (2) the average E_{FWD} was 111 MPa for natural subgrade and increased to 926 MPa for stabilized subgrade; (3) the average E_{FWD} of stabilized subgrade was 8.3 times higher than the natural subgrade; (4) the values of E_{FWD} of natural and stabilized subgrade varied significantly indicating non-uniform subgrade soil properties.

Figure 48 presents the stress-strain relationship of test at test point 12. The values of E_{V1} and E_{V2} were calculated in the first cycle and after reloading. The uncorrected modulus of soil reaction k′u was calculated using deflection under a load of 69.0 kPa as shown in Figure 49. The average LWD elastic modulus (E_{LWD}) of stabilized subgrade was presented in Table 13, which is equal to 0.4 E_{V1} and 0.3 E_{V2}. Table 14 provides the elastic modulus ratio between stabilized and natural subgrade. The mean value, standard deviation, and coefficient of variation of in situ test results were listed in Table 14. All in situ test results are presented in Appendix C.

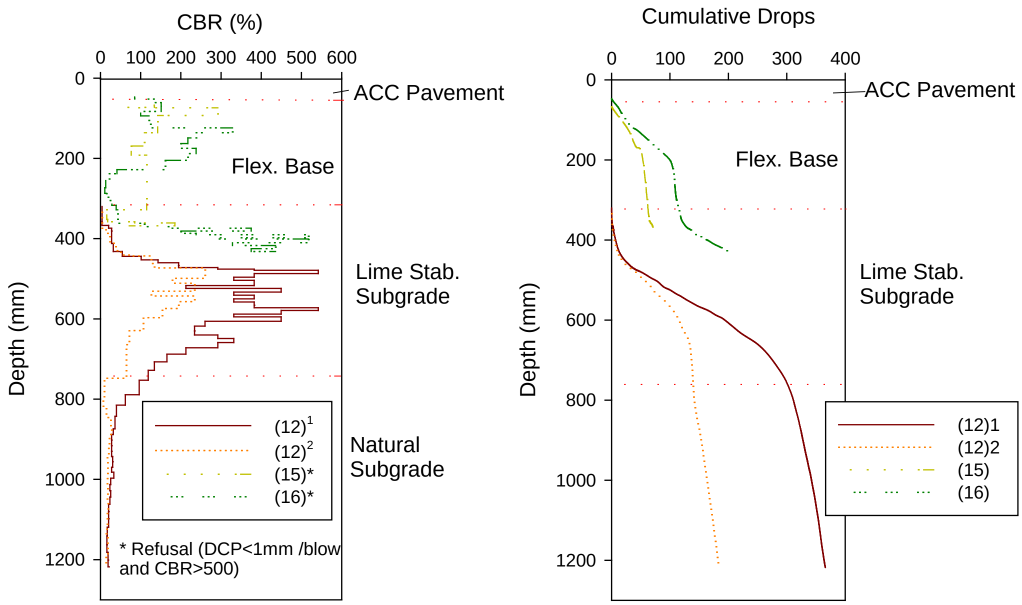


Figure 46. CBR – DCP profile and cumulative drops versus CBR of test points US 287 Mansfield, Texas

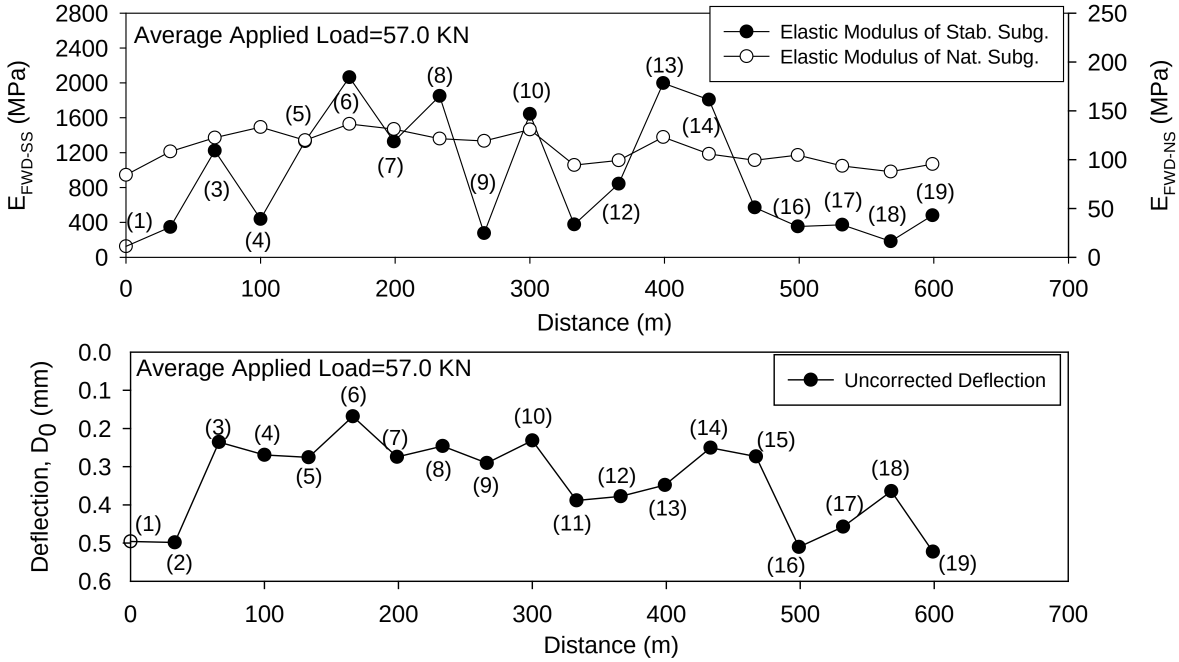


Figure 47. Back calculated FWD elastic modulus of stabilized and natural subgrade, and deflections under the loading plate US 287 Mansfield, Texas

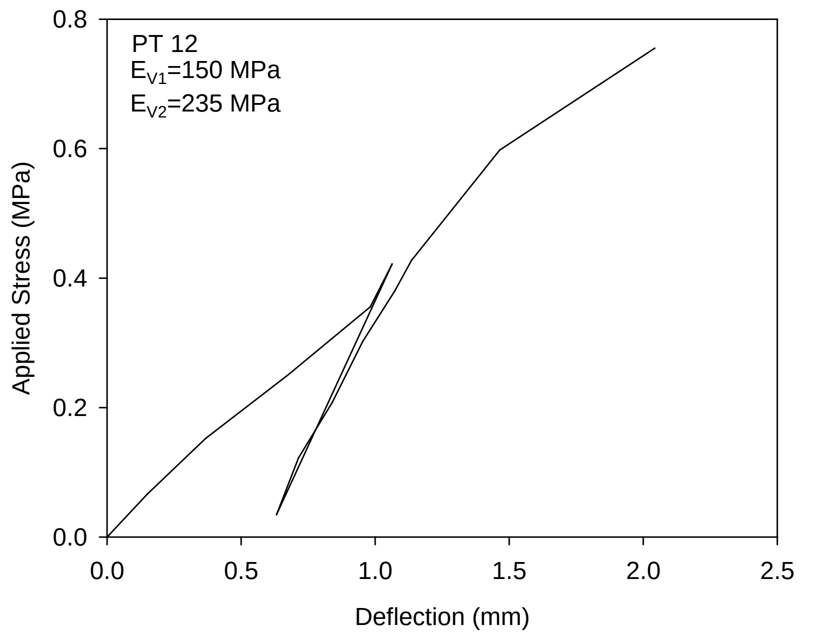


Figure 48. Corrected stress – strain curve from plate load test at point 12 US 287 Mansfield, Texas

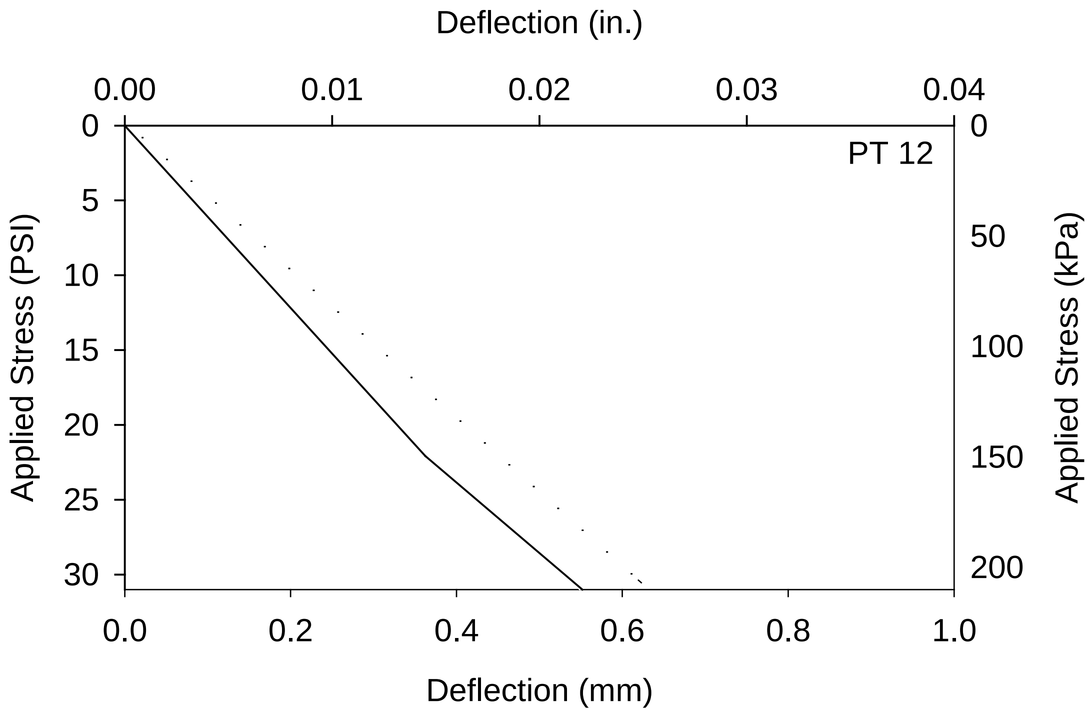


Figure 49. Stress – strain curves for obtaining K_U at point 12 US 287 Mansfield, Texas

Table 13. Summary of LWD test results US 287 Mansfield, Texas

Test Point	**Material Type**	**Depth of Measurement**	**E_{LWD}**	**Average E_{LWD}**
			MPa	MPa
PT 12	Base	Top of base	102	107
PT 12	Base	60 mm from top of base	112	
PT 12	Base	95 mm from top of base	102	
PT 12	Stab. Subgrade	Top of stabilized subgrade	65	65

Table 14. Summary of elastic modulus ratio between stabilized and natural subgrade

Stab. Subg./Nat. Subg. Ratio	
CBR	E_{FWD}
7.4	8.3

Table 15. Summary statistics of test results from in situ testing US 287 Mansfield, Texas

Statistic	Base		Stabilized Subgrade							Natural Subgrade	
	CBR	E_{LWD}	CBR	E_{FWD}	k_U	E_{LWD}	E_{V1}	E_{V2}	Thi.	E_{FWD}	CBR
	%	MPa	%	MPa	kPa/mm	MPa	MPa	MPa	mm	MPa	%
Number of Measurement (n)	2	1	2	19	1	1	1	1	1	19	1
Mean Value (μ)	97	107	163	926	126	65	150	235	400	111	22
Standard Deviation (σ)	52	—	18	685	—	—	—	—	—	17	—
Coefficient of Variation COV (%)	53	—	11	74	—	—	—	—	—	15	—

4.4 US 183, OK

4.4.1 Site Description

This project was located on US 183 south of Clinton in Washita County, Oklahoma. The general location of this site is shown in Figure 50. This road is a four-lane U.S. Highway. The design life of pavement is 20 years based on an equivalent single axle loads (ESALS) of 10.6 million. The annual average daily traffic was 4400 in 1998 and is estimated to be 6600 by 2018. The road was constructed in 1999 and consisted of a nominal 254 mm (10 in.) thick asphalt concrete (AC) surface overlying a nominal 203 mm (8 in.) lime stabilized subgrade. In 2009, an HMA overlay was placed with nominal thickness of 50 mm (2 in.). The pavement currently consists of a 300 mm (12 in.) thick asphalt concrete (AC), and 203 mm (8 in.) lime stabilized subgrade (Figure 51). No base layer was presented between subgrade and ACC pavement. The length of this test section is approximately 300 m (984 ft) and was in the southbound lane. Construction records indicate that the subgrade was stabilized with 5% lime from station 385+00 to 641+00. The ISU research team conducted in situ testing between station 407+00 to 414+00 on September 28, 2010 with assistance and traffic control provided by Oklahoma DOT.

The plan view of in situ test locations based on RTK PGS is shown in Figure 52. The research team preformed FWD tests on the surface of the ACC pavement at intervals of about 10 m from test points 1 to 25. DCP tests were conducted at test points 1, 3, 8, 9, 12, 15, 18, and 25. After coring, LWD and PLT tests were performed on the top of the stabilized subgrade at test point 8. Bag samples were collected at test point 8 from the top of the subgrade to a depth of 300 mm (12 in.) at intervals of 50 to 75 mm (2 to 3 in.). Natural subgrade samples were also collected at test points 26, 27, and 28.

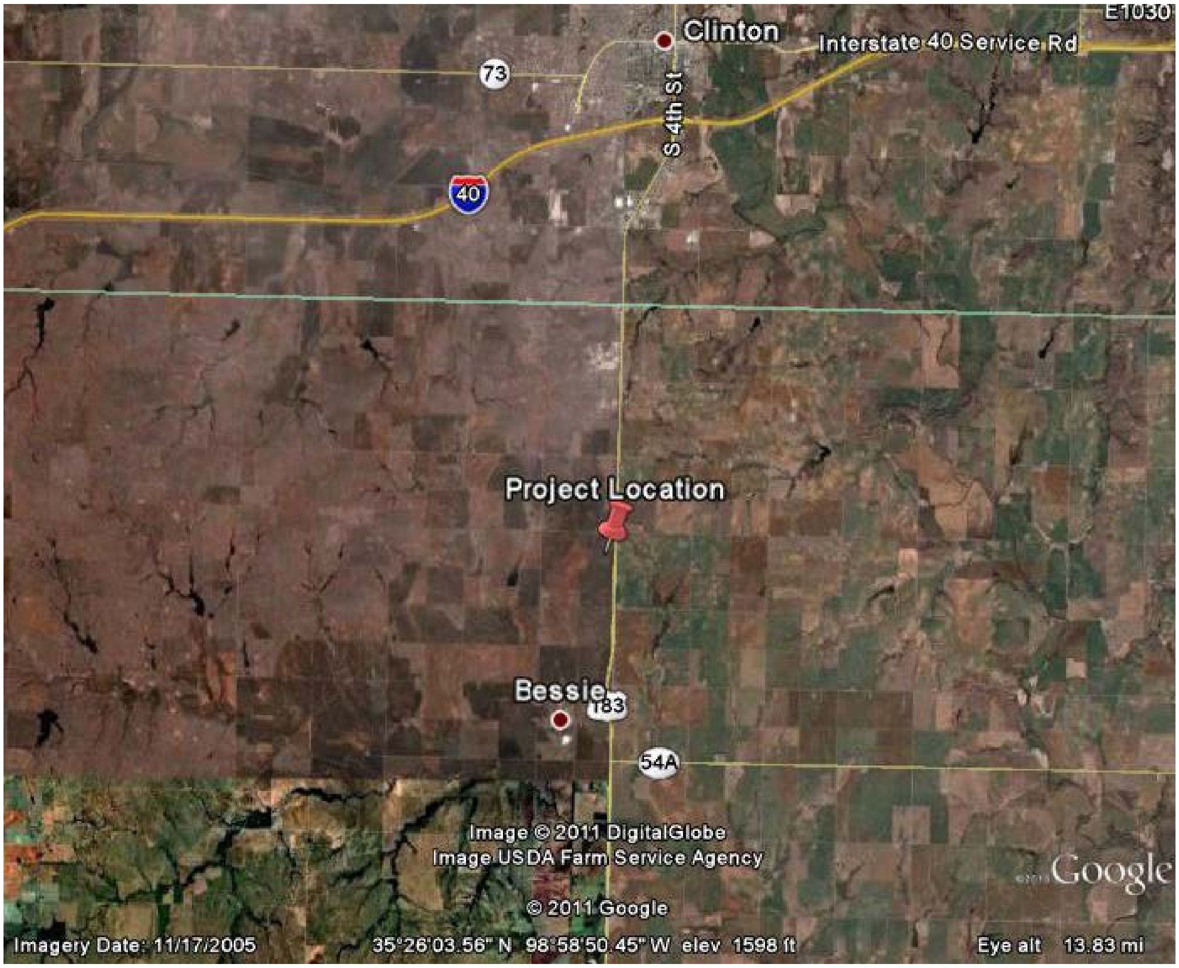


Figure 50. Project location of US 183 Clinton, Oklahoma

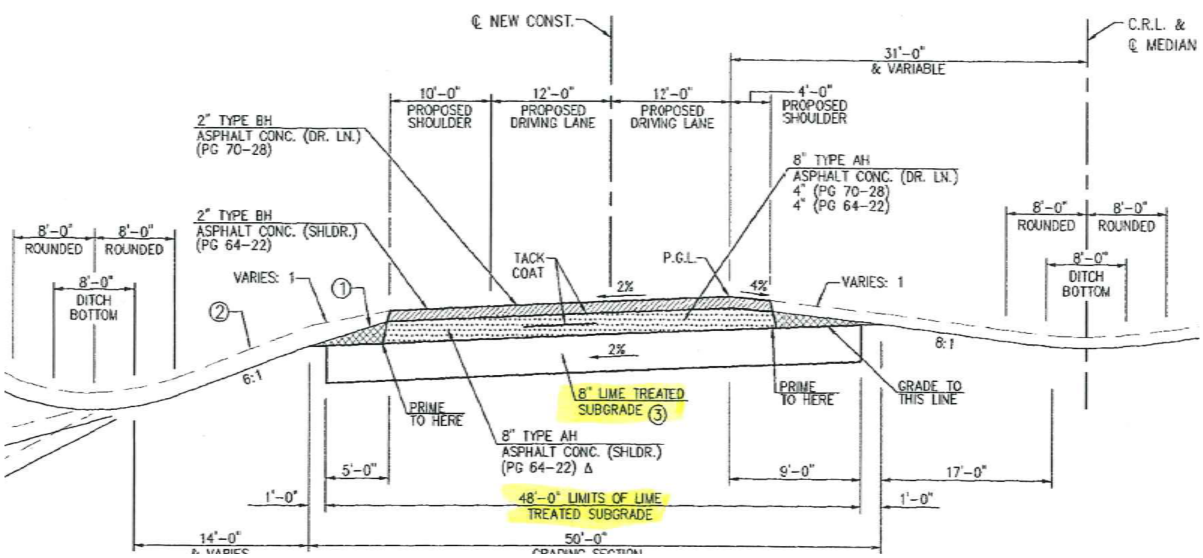


Figure 51. Original design cross section US 183 Clinton, Oklahoma

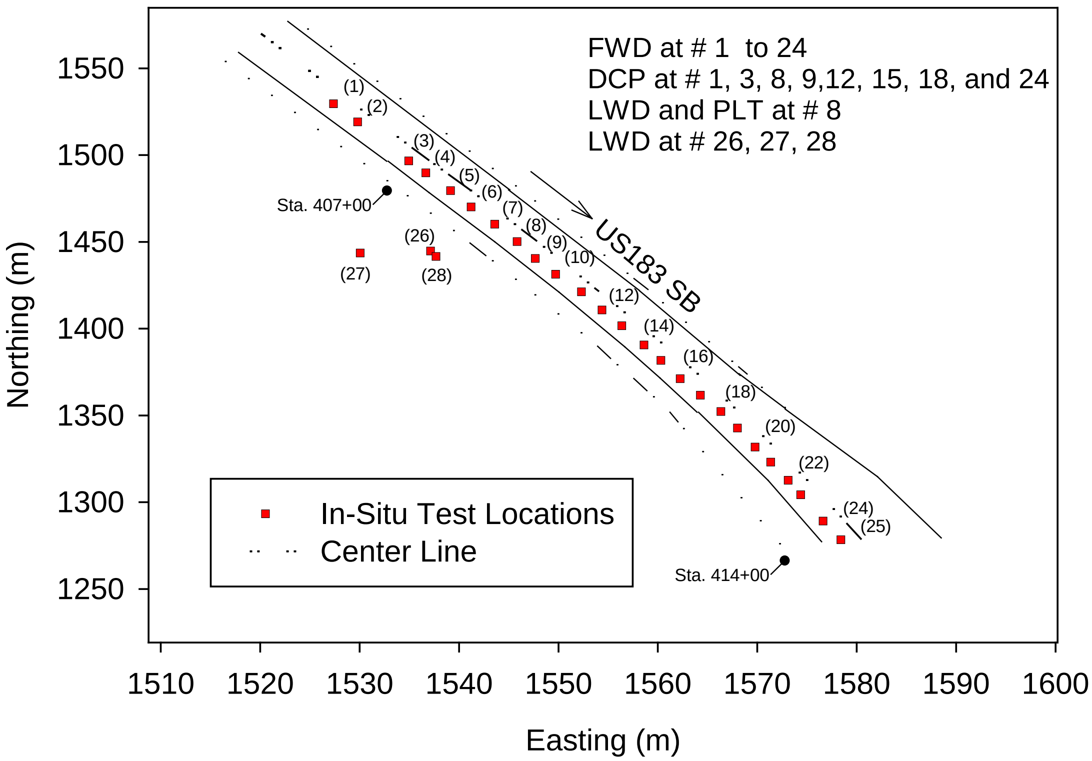


Figure 52. Test section plan layout US 183 Clinton, Oklahoma

Figure 53. Site overview US 183 Clinton, Oklahoma

4.4.2 Test Results and Analysis

4.4.2.1 Material Properties of Subgrade

Stabilized subgrade samples were taken from different depths at test point 8. The natural subgrade sample was taken at test point 26. In accordance with USCS and AASHTO classification, the natural subgrade was classified as ML and A-4 and the stabilized subgrade was classified as SM and A-4 or A-2-4. A summary of material properties of the subgrade is provided in Table 16. Comparing the natural subgrade to the stabilized subgrade, the sand content increased from about 14% to 40%. Further, the clay content decreased from about 16% to 5% and the silt content decreased from about 68% to about 30%. LL values of stabilized and natural subgrade samples were approximately equal. PI values of the stabilized subgrade samples were reduced about 3 to 4 points. The moisture content was around 20% for the stabilized subgrade and 10% for the natural subgrade. Figure 54 shows particle size distribution curves of different subgrade layers. Construction records show that the in situ moisture content and density results produced about 102 percent relative compaction at about 3 percent below optimum moisture content (results provided in Appendix G).

Table 16. Summary of material properties US 183 Clinton, Oklahoma

Parameter	Materials					
Material Description	Natural Sub.	Stab. Sub.	Stab. Sub.	Stab. Sub.	Stab. Sub.	Sub.
Depth mm (in.)	—	0-90 (0-3)	90-140 (3-5)	140-191 (5-7)	191-254 (7-9.5)	254-305 (9.5-11.5)
Gravel (%) (> 4.75mm)	1.5	25.1	28.8	25.9	19.4	13.9
Sand (%) (4.75mm-75µm)	14.2	39.4	42.7	39.5	39.4	31.5
Silt (%) (75µm–2µm)	68.4	30.6	24.7	30	35.2	46.2
Clay (%) (< 2µm)	15.9	4.9	3.8	4.6	6	8.4
C_u	—	286	407	321	184.5	57.9
C_c	—	0.2	0.5	0.2	0.3	0.7
Liquid Limit, LL (%)	33.9	34.7	37	34.5	35.9	30.5
Plasticity Index, PI	10.2	6.5	8.8	5.4	4.5	6.7
AASHTO	A-4	A-4	A-2-4	A-2-4	A-4	A-4
USCS	ML	SM	SM	SM	SM	ML
Water Content (%)	9.9	22.2	22.3	21.0	21.0	18.0

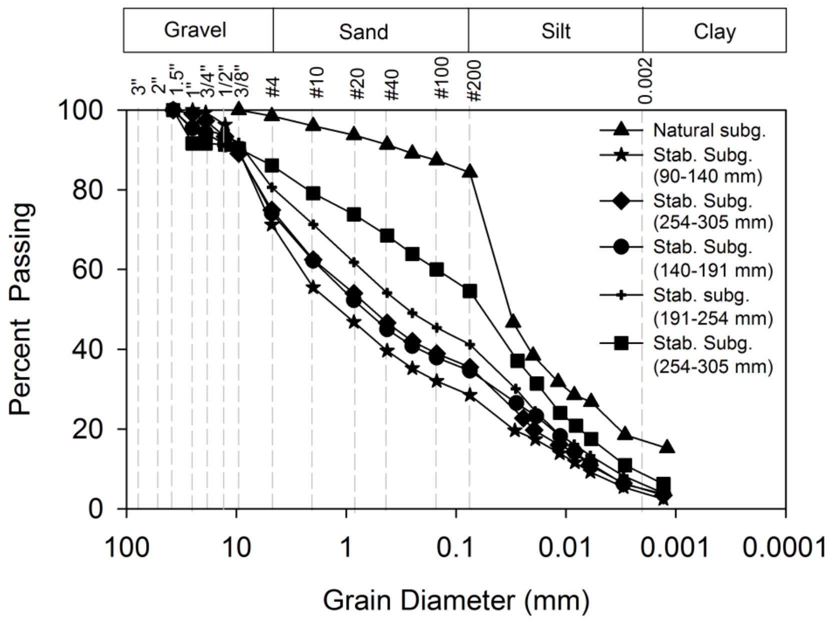


Figure 54. Particle size distribution curves for subgrade materials US 183 Clinton, Oklahoma

4.4.2.2 pH of Stabilized and Natural Subgrade

Figure 55 shows the pH profile of the subgrade layers at test point 8. The pH values of stabilized subgrade varied from 8.1 to 8.9 and for the natural subgrade from 7.9 to 8.3.

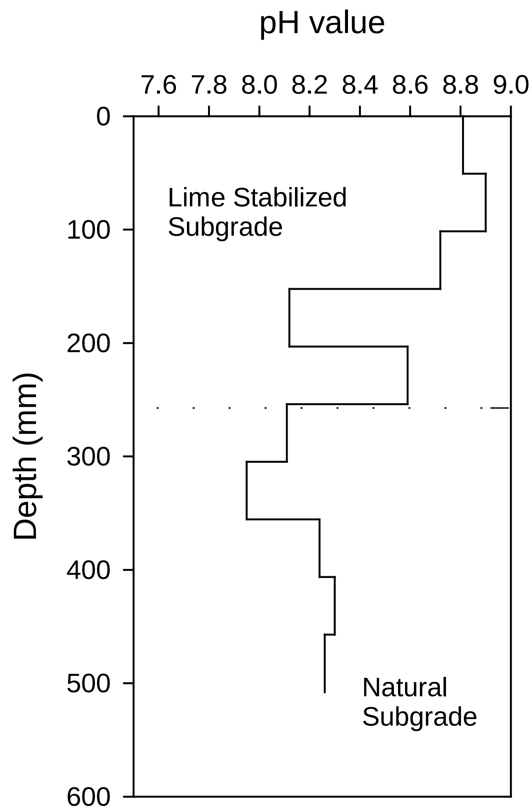


Figure 55. pH profile of subgrade US 183 Clinton, Oklahoma

4.4.2.3 SEM Analysis

The energy dispersive spectrometry (EDS) map of stabilized subgrade is shown in Figure 56. The major elements identified include calcium (Ca), silica (Si), alumina (Al), and oxygen (O). These elements commonly exist in lime stabilized subgrade. Additional elements present were iron (Fe), potassium (K), and sodium (Na).

Figure 57 compares elemental concentration in Al, Si, O, S, Mg, Ca, K, and C for the stabilized and natural subgrade. The Natural subgrade sample shows lower concentrations of Ca and higher concentrations of Si, Al, O, and Mg.

SEM images of the natural and stabilized subgrade samples at 5000 to 15000 × magnification are shown in Figure 58 and Figure 60. SEM images of natural and stabilized subgrade samples at 15000×maginification are shown in Figure 59 and Figure 61. The natural subgrade sample shows thin clay particles and some pore space. The stabilized subgrade sample shows particles what appear to be thin needle-like reaction products. Additional SEM images are presented in Appendix A.

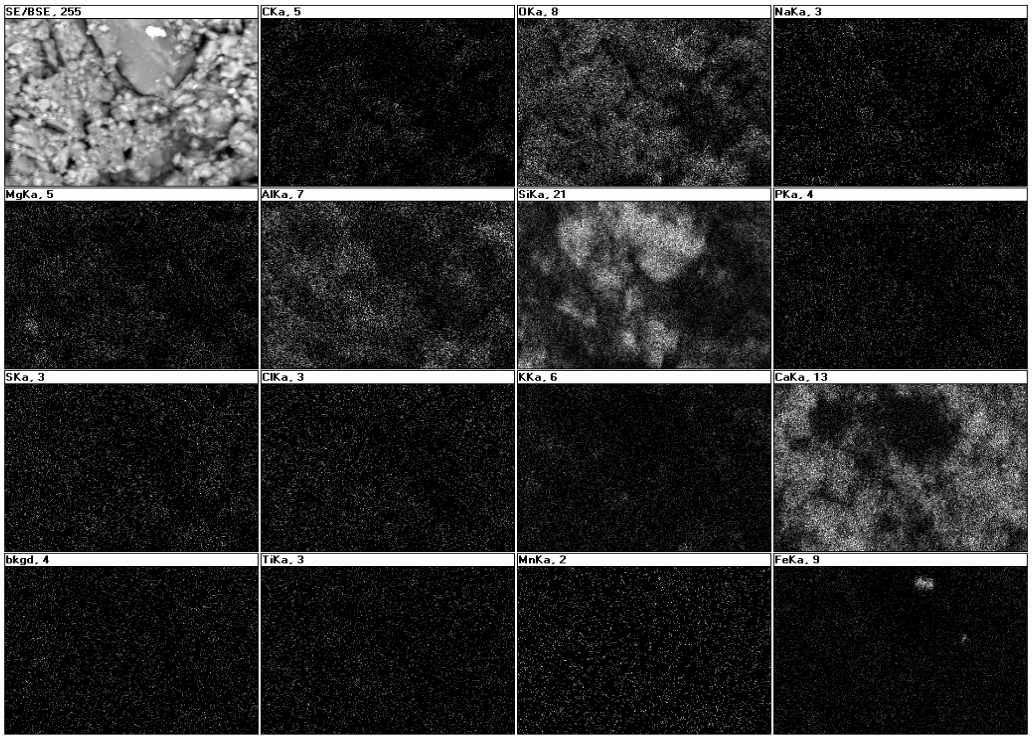


Figure 56. EDS map of stabilized subgrade sample (1500 ×) US 183 Clinton, Oklahoma

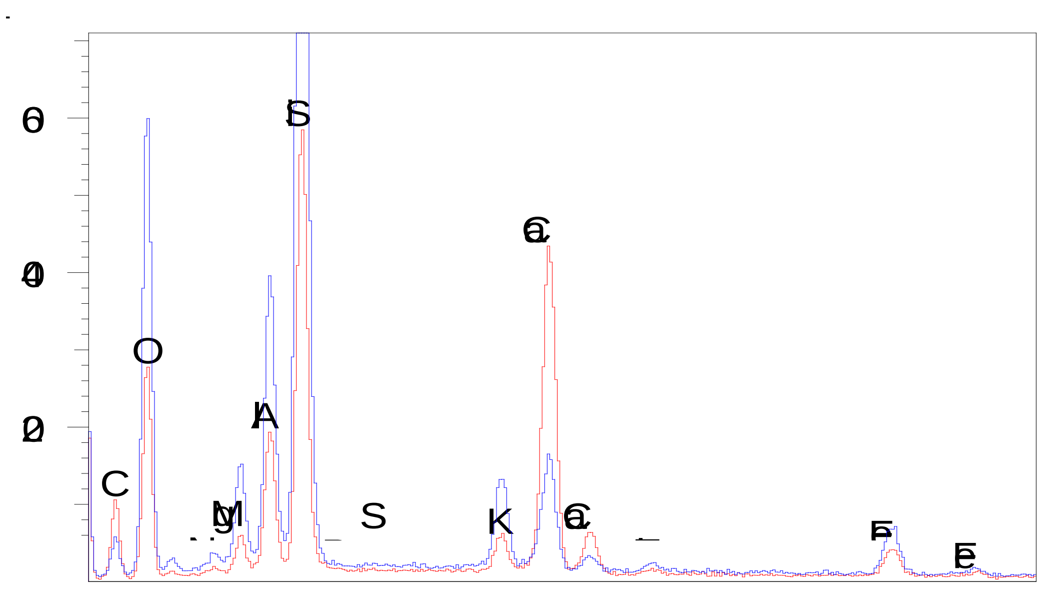


Figure 57. EDS intensity counts for stabilized subgrade sample (red line: 30×) and natural subgrade sample (blue line: 30×) US 183 Clinton, Oklahoma

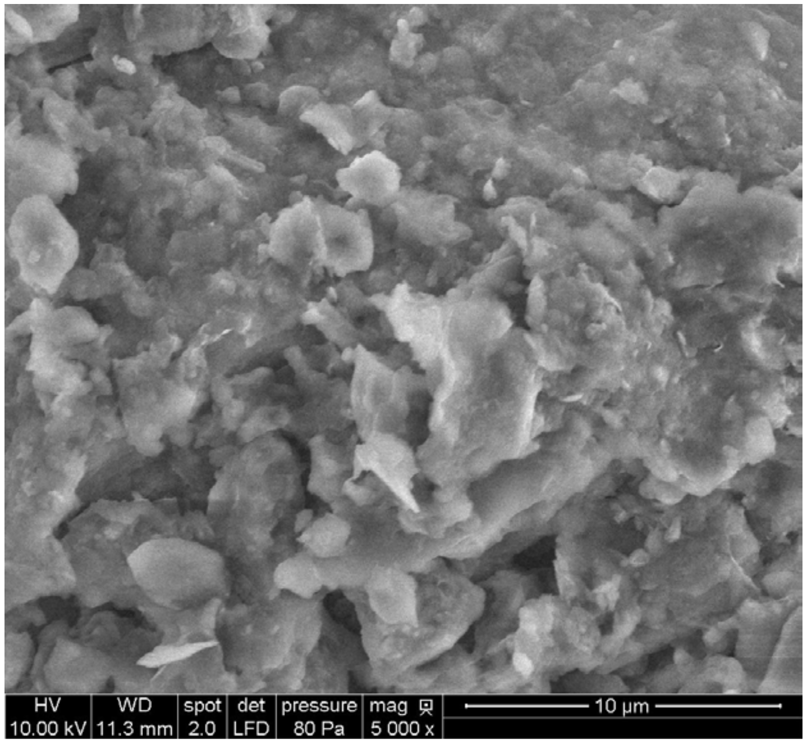


Figure 58. SEM image of natural subgrade sample (5000×) US 183 Clinton, Oklahoma

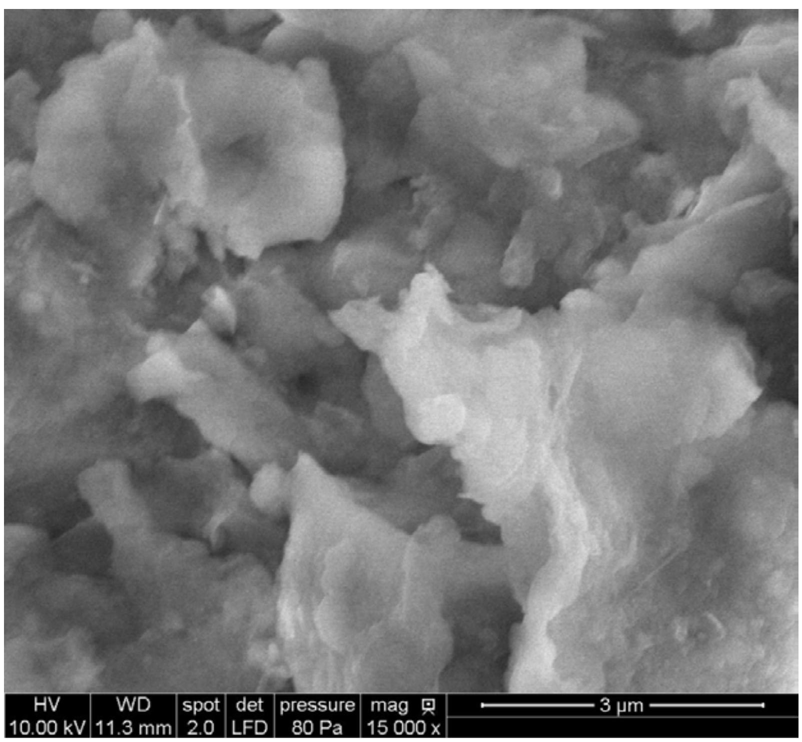


Figure 59. SEM image of natural subgrade (15000×) US 183 Clinton, Oklahoma

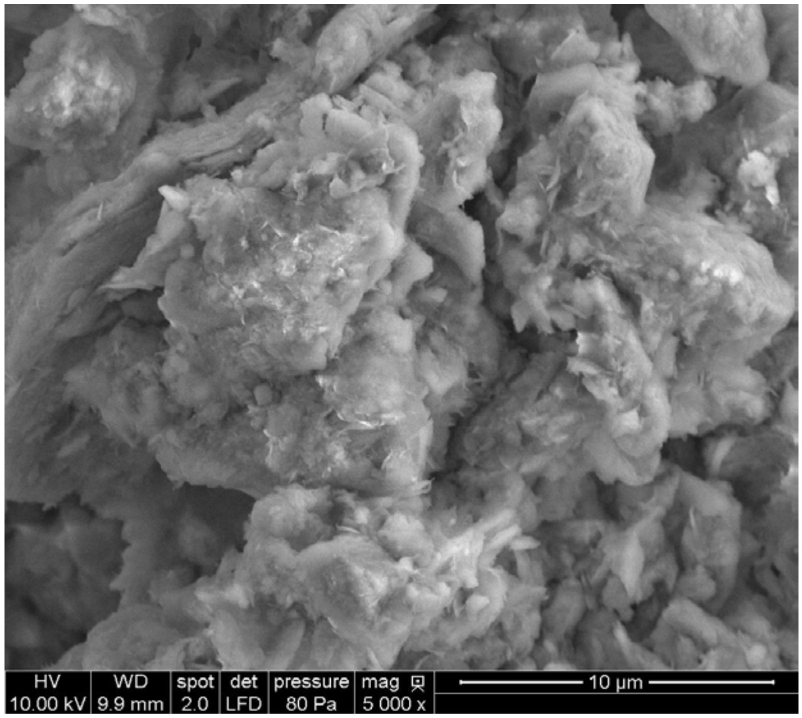


Figure 60. SEM image of stabilized subgrade sample (5000×) US 183 Clinton, Oklahoma

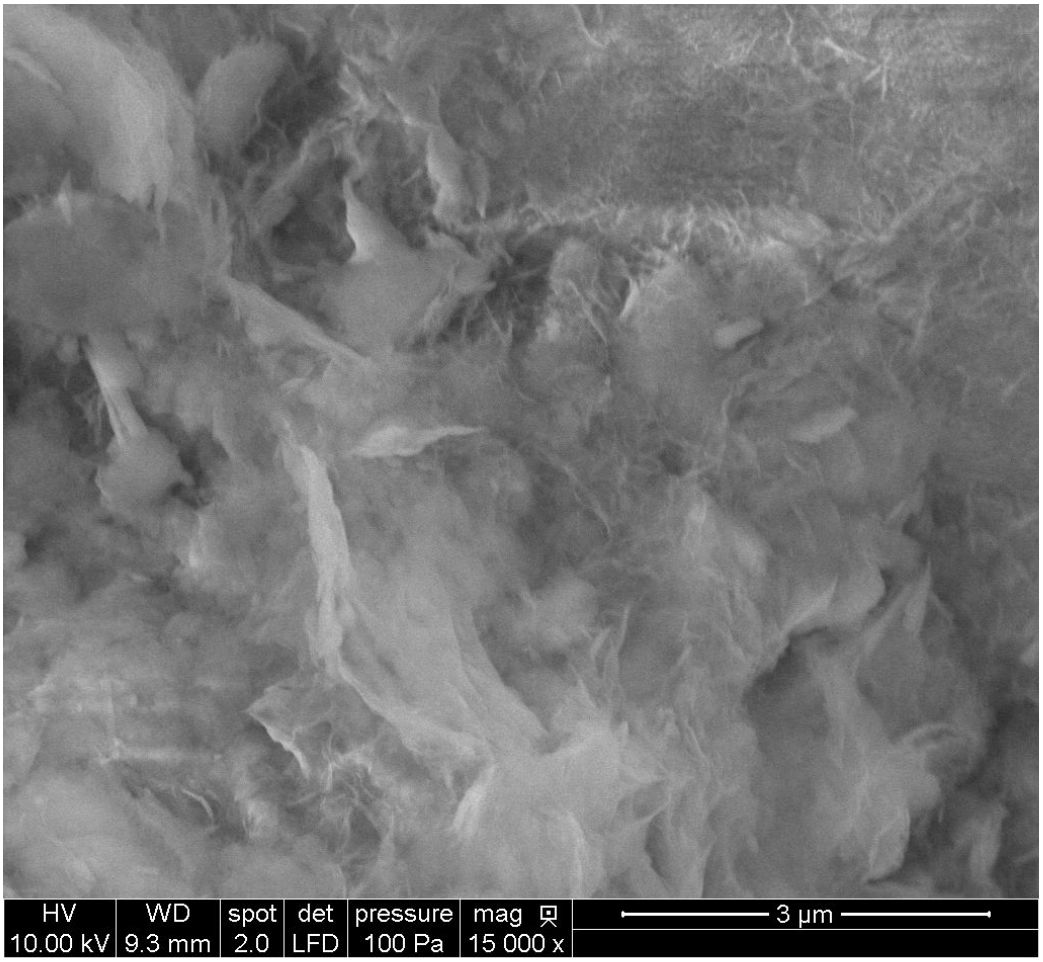


Figure 61.SEM image of stabilized sample (15000×) US 183 Clinton, Oklahoma

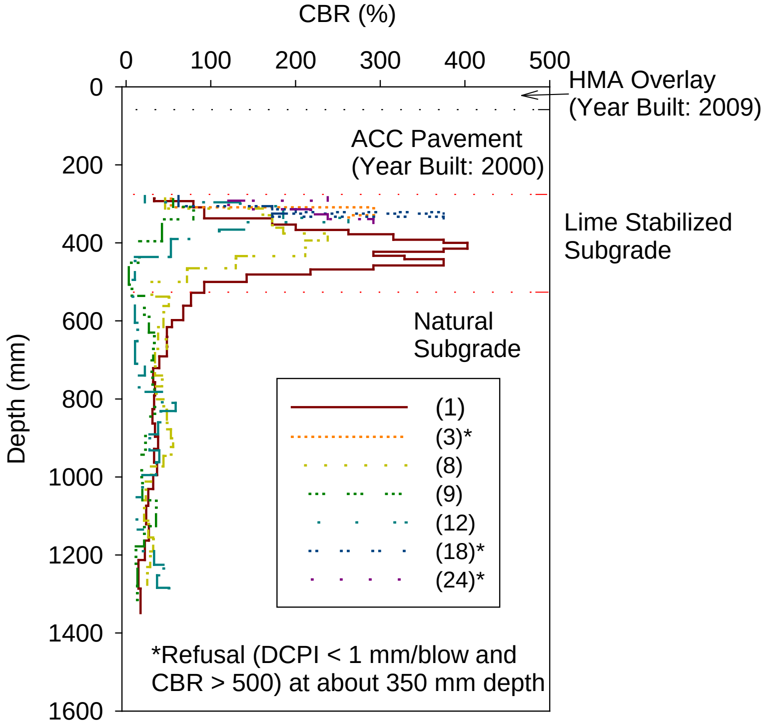
4.4.2.4 Stiffness and Strength

CBR-DCP profile and cumulative drops versus CBR are shown Figure 62. Average CBR values for both the natural and stabilized subgrades and the effective stabilized subgrade thickness are shown in Figure 63. The major observations are: (1) based on the effective treatment thickness, the average CBR of the stabilized subgrade was 133, (2) the average CBR of the natural subgrade ranged from 21 to 34, (3) the CBR of the stabilized subgrade was 2.70-6.3 time greater than the natural subgrade, (4) the top and bottom of the stabilized subgrade layer have lower CBR values than the middle of the layer, and (5) the actual treatment thickness was thicker than the design thickness at the test locations.

Back-calculated subgrade elastic moduli (E_{FWD}) value and deflections are presented in Figure 64. In the back-calculation, the applied test load was 57 KN (12800 lb). The assumptions of poison's ratio were 0.35, 0.40, and 0.40 for ACC surface layer, stabilized subgrade, and natural subgrade layer, respectively. Stabilized subgrade moduli were calculated based the effective stabilized subgrade thickness obtained from DCP profiles. Detailed assumptions of seed values and layer thickness value are summarized in Appendix B. The temperature of the middle depth of the ACC pavement layer is 24°C (75°F). Deflections under the loading plate (D_0) were adjusted to a standard temperature of 20°C (68°F) using Equation (2). The key findings from FWD testing are as follows: (1) the average D_0 was about 0.15 mm for an average applied load of 57 KN (12814 lb); (2) the average E_{FWD} value was 144 MPa for the natural subgrade and 1794 MPa for the stabilized subgrade layer; (3) the average E_{FWD} of stabilized subgrade was about 12 times higher than the natural subgrade; and (4) the values of E_{FWD} of natural and stabilized subgrade varied significantly indicating non-uniform subgrade soil properties.

Figure 65 presents the stress-deflection response from the PLT test at test point 8. E_{V1} and E_{V2} were calculated from the first load cycle and from the reload cycle. The uncorrected modulus of soil reaction k′u was calculated using the deflection under a plate stress of 69.0 kPa as shown in Figure 66.

Table 17 provides all LWD elastic modulus (E_{LWD}) at four test points. The average E_{LWD} was increased 8.6 times from 19 MPa for natural subgrade to164 MPa for stabilized subgrade. E_{LWD} of stabilized subgrade was equal to 0.5 E_{V1} and 0.3 E_{V2}. Table 18 provides the elastic modulus ratio between stabilized and natural subgrade. The mean value, standard deviation, and coefficient of variation of in situ test results were listed in Table 19. All in situ test results are presented in Appendix C.



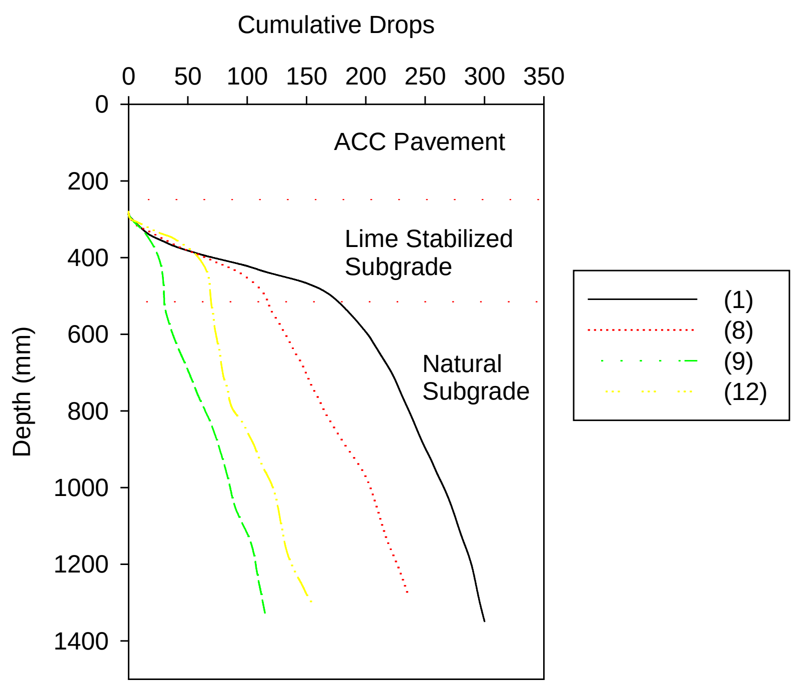


Figure 62. CBR – DCP profile and cumulative drops versus CBR of test points US 183 Clinton, Oklahoma

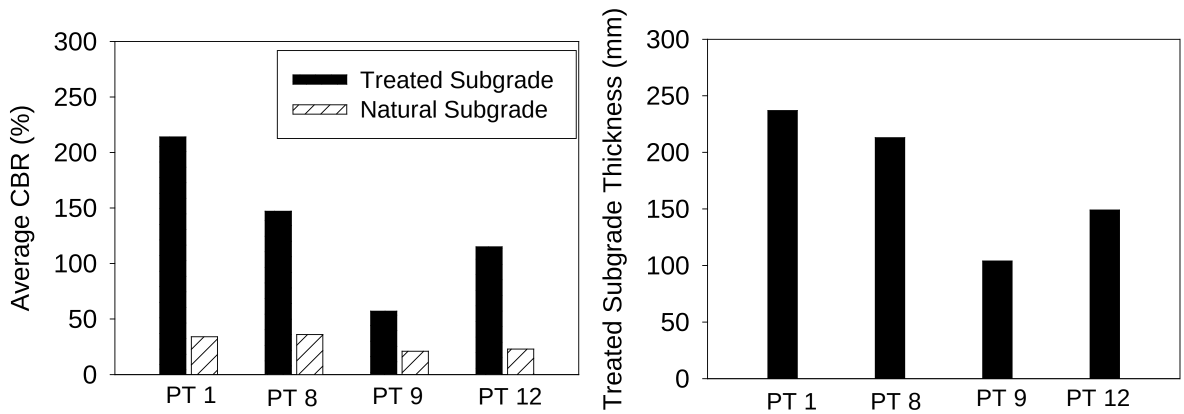


Figure 63. CBR and stabilized subgrade thickness from DCP profile US 183 Clinton, Oklahoma

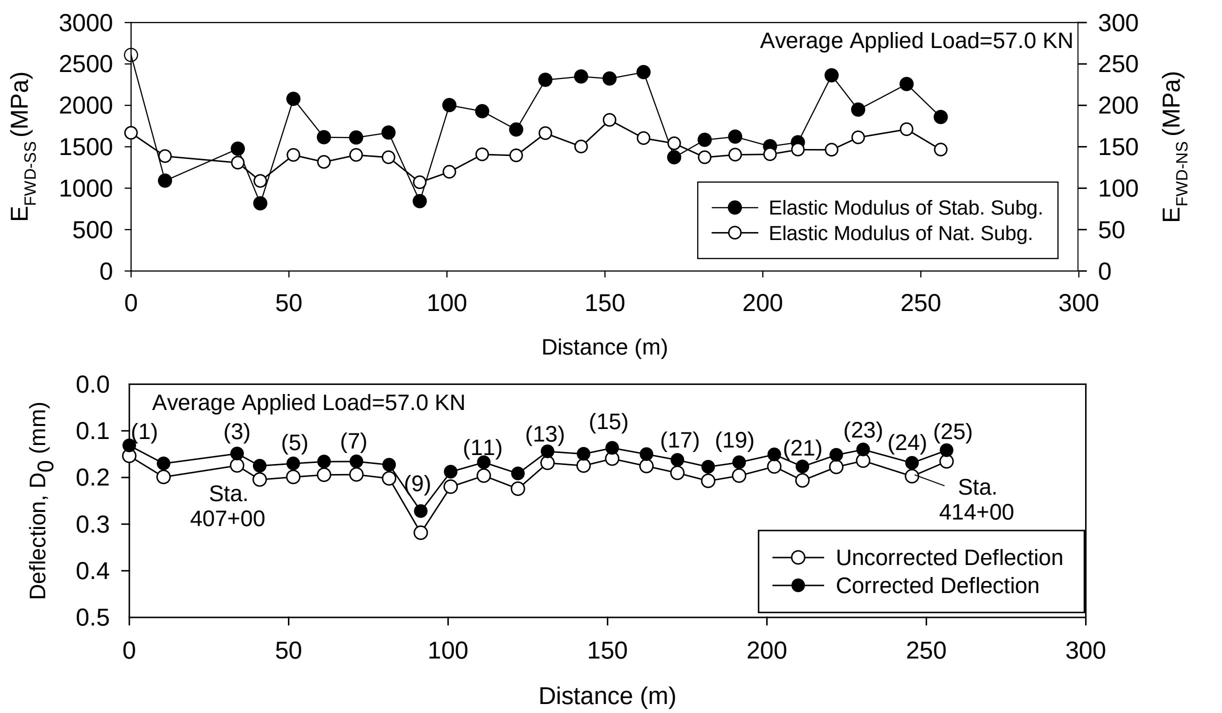


Figure 64. Back-calculated FWD elastic modulus of stabilized and natural subgrade, and deflections under the loading plate US 183 Clinton, Oklahoma

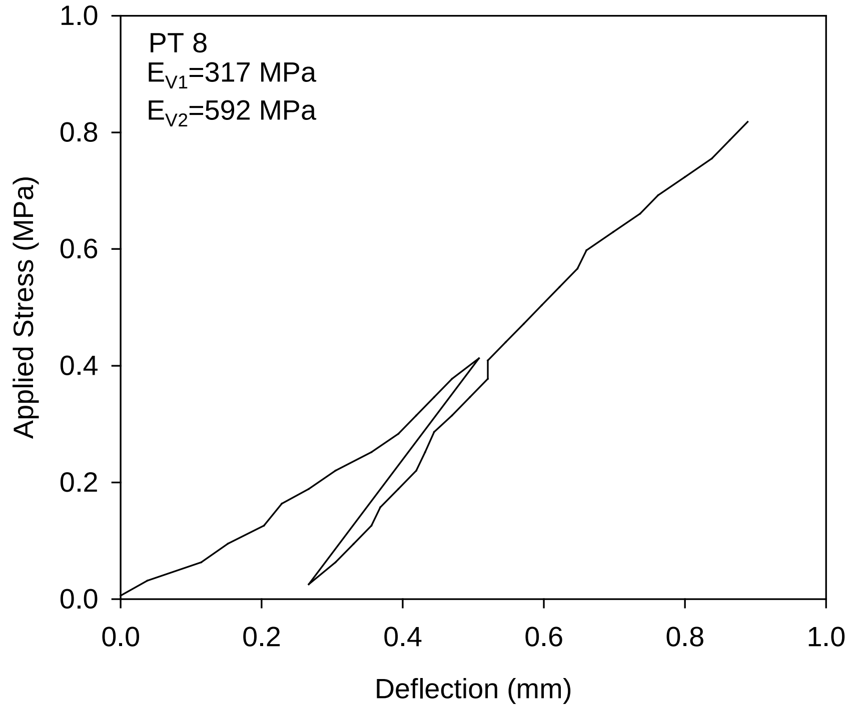


Figure 65. Stress – strain curves from plate load test at point 8 US 183 Clinton, Oklahoma

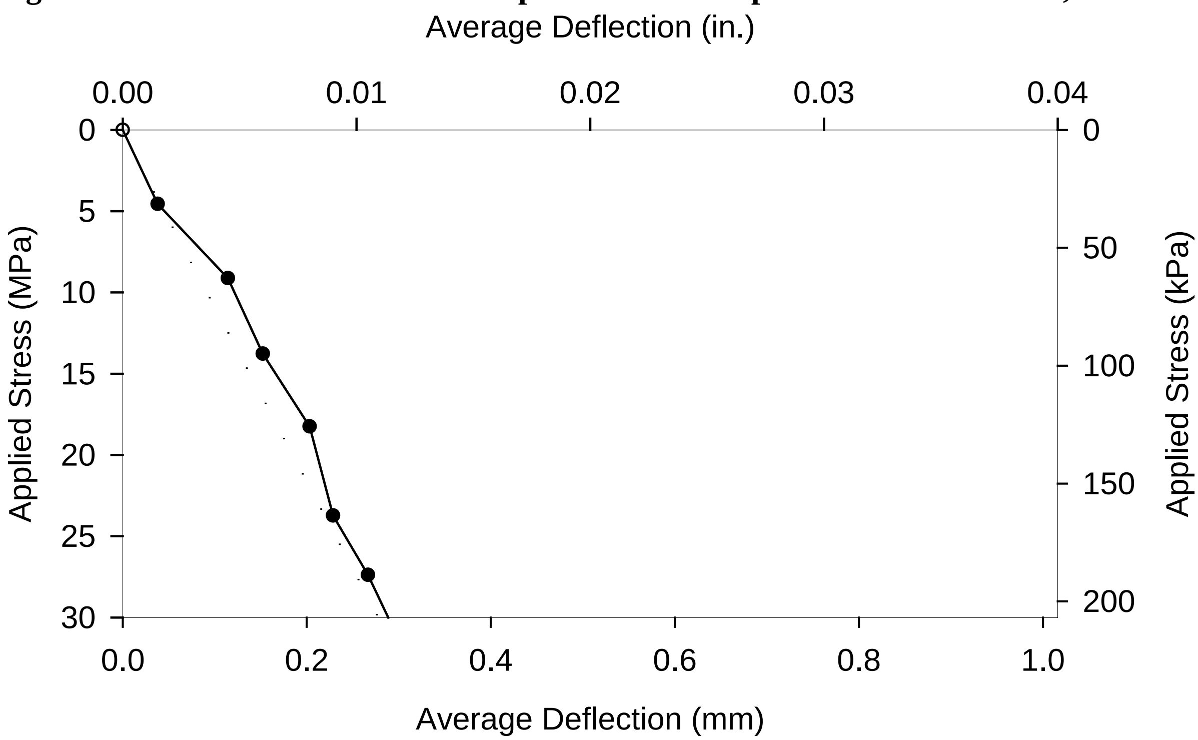


Figure 66. Stress – deflection curves for obtaining K_U at point 8 US 183 Clinton, Oklahoma

Table 17. Summary of LWD test results US 183 Clinton, Oklahoma

Test Point	Material Type	Depth of Measurement	E_{LWD}	Average E_{LWD}
			MPa	MPa
8	Stabilized Subgrade	Top of stabilized subgrade	164	164
26	Natural Subgrade	Top of natural subgrade	20	15
27	Natural Subgrade	Top of natural subgrade	13	
28	Natural Subgrade	Top of natural subgrade	13	

Table 18. Summary of elastic modulus ratio between stabilized and natural subgrade US 183 Clinton, Oklahoma

Stab. Subg./Nat. Subg. Ratio		
CBR	E_{FWD}	E_{LWD}
4.5	12.3	8.5

Table 19. Summary statistics of test results from in situ testing US 183 Clinton, Oklahoma

Statistic	Stabilized Subgrade							Natural Subgrade			FWD Def
Measurement	CBR	E_{FWD}	E_{LWD}	E_{V1}	E_{V2}	k_U	Thi.	CBR	E_{FWD}	E_{LWD}	$D_{0\text{-Cor.}}$
	%	MPa	MPa	MPa	MPa	kPa/mm	mm	%	MPa	MPa	mm
Number of Measurement (n)	4	25	1	1	1	1	4	4	25	3	25
Mean Value (μ)	133	1794	164	317	592	202	176	29	144	19	0.17
Standard Deviation (σ)	65	480	—	—	—	—	61	8	18	5	0.03
Coefficient of Variation COV (%)	49	27	—	—	—	—	34	27	12	25	17

4.5 SH 99, OK

4.5.1 Site Description

This project was located on SH 99 north of Seminole in Seminole County, Oklahoma. The general location of this site is shown in Figure 67. This road is a four-lane State Highway. The design life of pavement is 20 years, and annual average daily traffic was 6800 in 1991 and estimated to be 12000 in 2011. The road was constructed in 1999. The length of this test section is approximately 500 m (1640 ft). The pavement consisted of a nominal 254 mm (10 in.) thick asphalt concrete (AC), 152 mm (6 in.) aggregate base, and 203 mm (8 in.) subgrade stabilized with fly ash (Figure 68). The ISU research team conducted in situ testing between station 5110+00 to 5126+00 on September 29, 2010 with assistance and traffic control provided by Oklahoma DOT.

The plan view of in situ test locations is shown in Figure 69. The research team preformed FWD tests on the surface of ACC pavement at intervals of about 11 m from test points 1 to 45. DCP tests were conducted at test points 1, 43, 44, and 45. After coring, LWD and PLT tests were performed on the top of the stabilized subgrade at test point 45. LWD and DCP tests were also performed at control test point 46. Bag samples of base and stabilized subgrade were collected at test point 45 to a depth of 75 mm (3 in.) into the stabilized subgrade while the natural subgrade was collected from control test point 46.

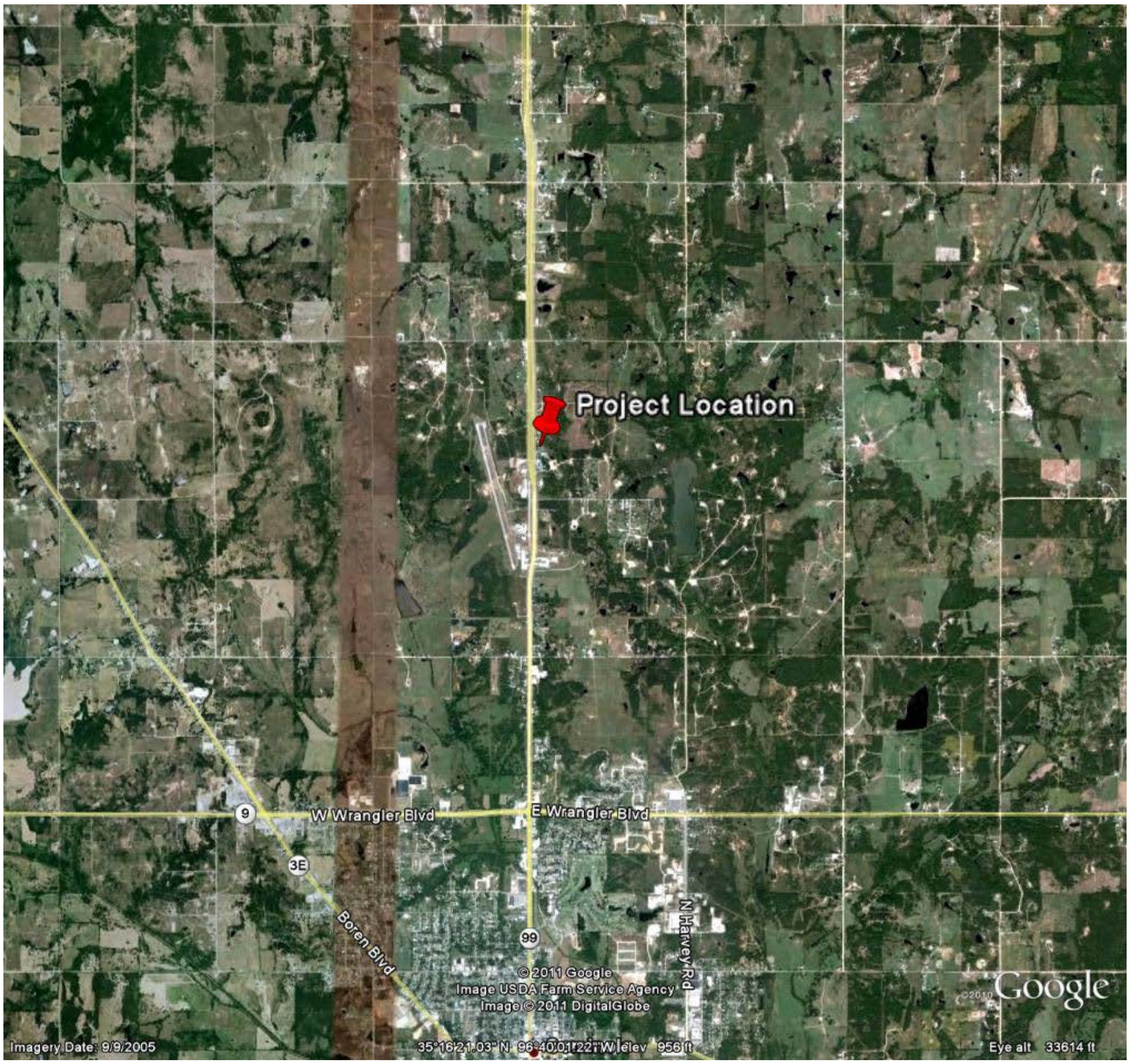


Figure 67. Project location of SH 99 Seminole County, Oklahoma

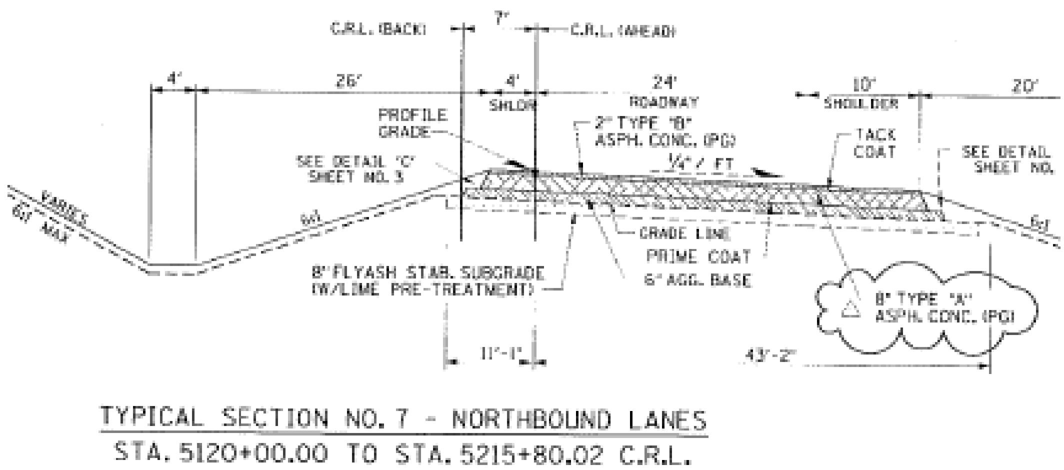


Figure 68. Typical cross section SH 99 Seminole County, Oklahoma

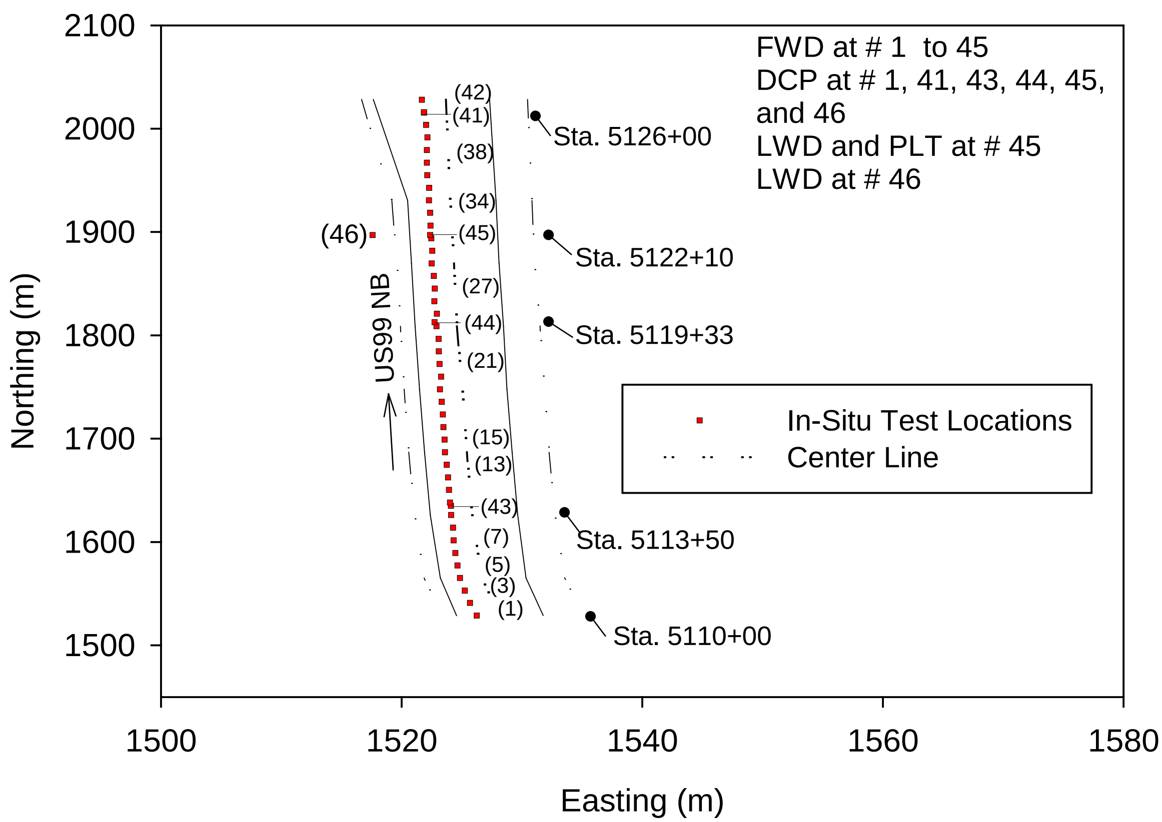


Figure 69. Test section plan layout with RTK GPS test points SH 99 Seminole County, Oklahoma

Figure 70. Site overview north bound lane SH 99 Seminole County, Oklahoma

4.5.2 Test Results and Analysis

4.5.2.1 Material Properties of Base and Subgrade

In accordance with USCS and AASHTO classification systems, the natural subgrade classify as ML and A-4-0, and the stabilized subgrade as SM and A-4-0. Table 19 provides a summary of the material index values. Compared to the natural subgrade, the sand content of the stabilized subgrade increased from about 49% to 58%. The stabilized subgrade was a non-plastic. The moisture content for the stabilized subgrade was about 21% and for the natural subgrade about 12%. Figure 71 shows particle size distribution curves of different subgrade layers. Original construction records showing in situ density and moisture content of some test points were recorded during construction shown in Appendix G.

Table 20. Summary of material properties SH 99 Seminole County, Oklahoma

Parameter	Materials		
Material Description	Base	Stabilized Subgrade	Natural Subgrade
Depth mm (in.)	0-150 (0-6)	0-200 (0-8)	—
Gravel Content (%) (> 4.75mm)	64.7	6.7	3.9
Sand Content (%) (4.75mm – 75µm)	29.0	48.6	58.4
Silt Content (%) (75µm – 2µm)	5.1	35.4	27.2
Clay Content (%) (< 2µm)	1.2	9.3	10.5
Coefficient of Uniformity (Cu)	37.9	68.8	84.7
Coefficient of Curvature (Cc)	2.6	2.0	2.0
Liquid Limit, LL (%)	16.1	—	22.3
Plasticity Index, PI	4.5	N.P.	4.9
AASHTO	A-1-a	A-4-0	A-4-0
USCS	GW-GM	SM	SM
Water Content (%)	3.4	20.6	11.7

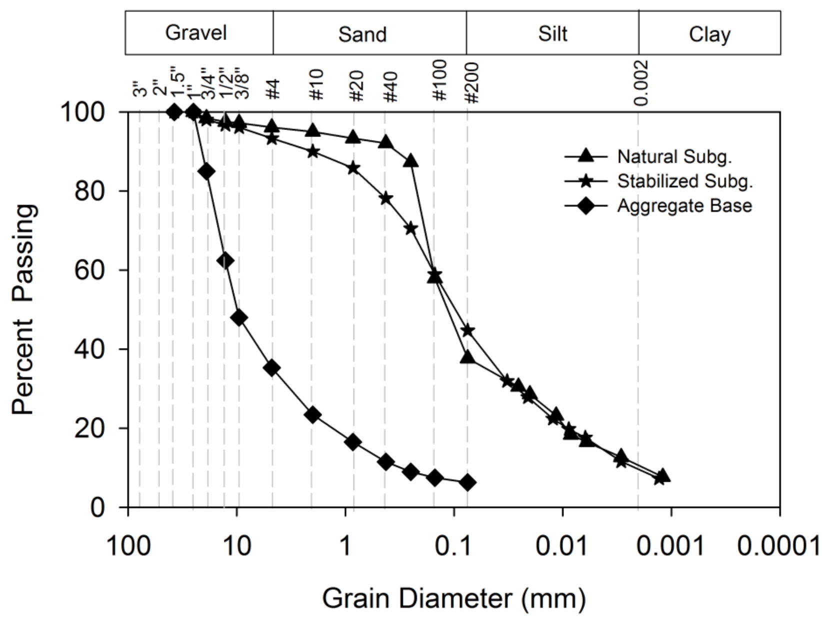


Figure 71. Particle size distribution curves for subgrade materials SH 99 Seminole County, Oklahoma

4.5.2.2 pH of Stabilized and Natural Subgrade

Table 21 shows pH values of natural and stabilized subgrade. Similar to the other test sites, the pH is higher in the stabilized layer.

Table 21. Summary of pH value of subgrade SH 99 Seminole County, Oklahoma

Depth	pH value
Natural subgrade	8.2
Stabilized subgrade	9.2

4.5.2.3 SEM Analysis

An energy dispersive spectrometry (EDS) map of stabilized subgrade is shown in Figure 72. The majority elements were calcium (Ca), silica (Si), alumina (Al), and oxygen (O). Additional present elements were iron (Fe), potassium (K) and Sodium (Na). Figure 73 shows elemental concentration for Al, Si, O, S, Mg, Ca, K, and C for the natural subgrade. Figure 74 shows elemental concentrations for the stabilized subgrade. The stabilized subgrade sample shows higher concentrations of O, Ca, and Al than the natural subgrade sample. All SEM images are presented in Figure 75, Figure 76, and Appendix D. Figure 76 shows evidence of fly ash particles with the soil matrix.

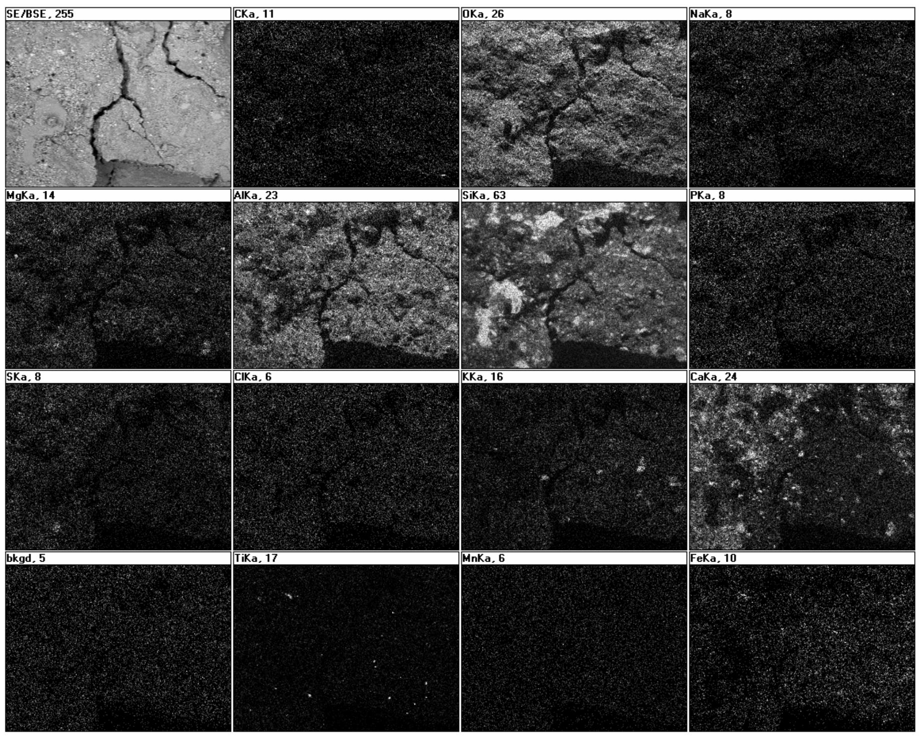


Figure 72. EDS map of stabilized subgrade sample (150 ×) SH 99 Seminole County, Oklahoma

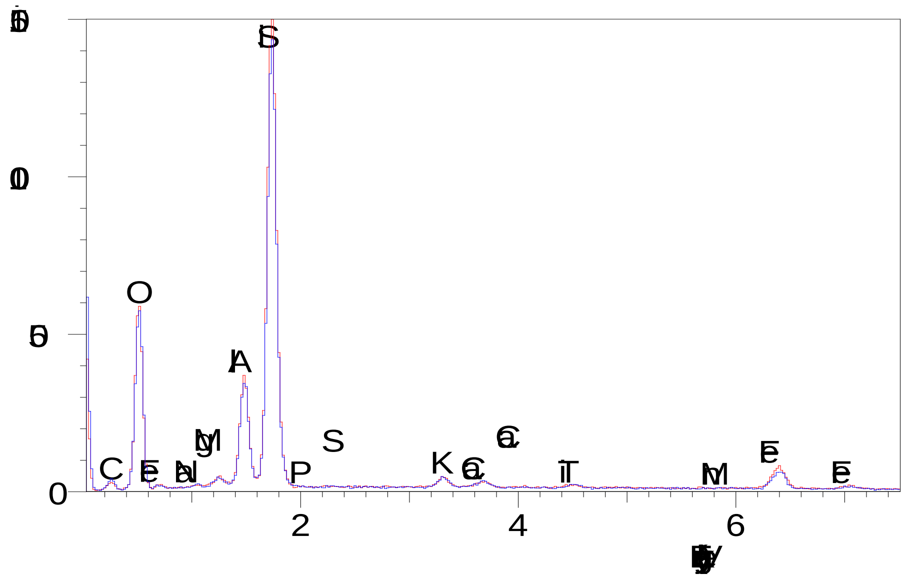


Figure 73. EDS intensity counts for natural subgrade sample (red line: 500×; blue line: 30×) SH 99 Seminole County, Oklahoma

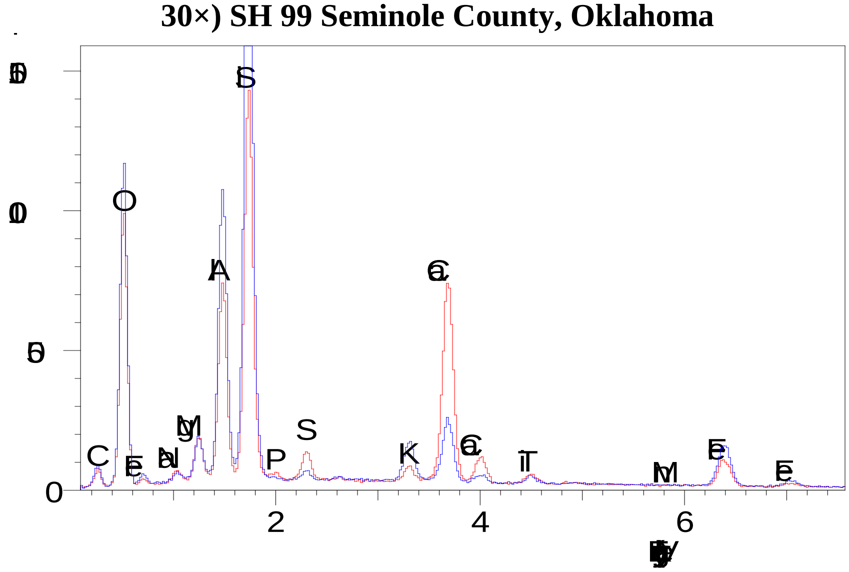


Figure 74. EDS intensity counts for stabilized subgrade sample in area a (blue line: 500×) and stabilized subgrade sample in area b (red line: 500×) SH 99 Seminole County, Oklahoma

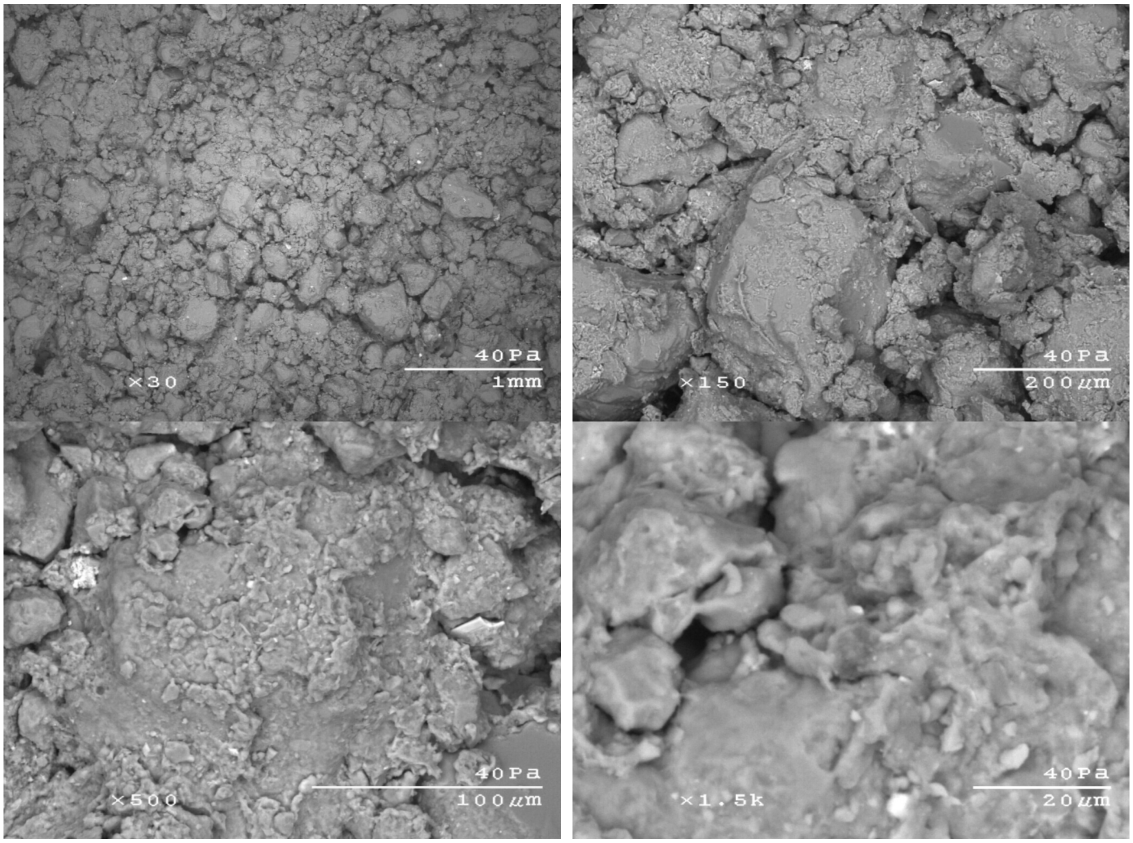


Figure 75. SEM images of natural subgrade SH 99 Seminole County, Oklahoma

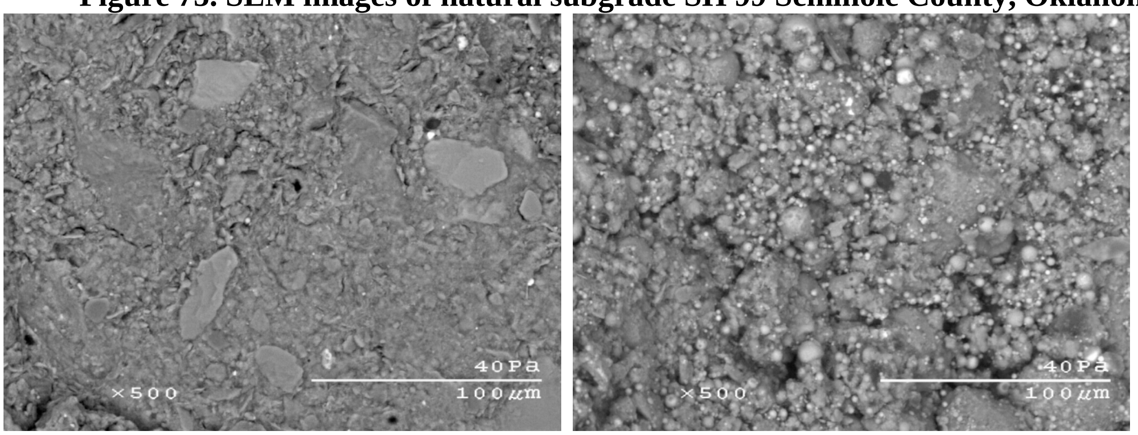


Figure 76. SEM images of stabilized subgrade in area a and b SH 99 Seminole County, Oklahoma

4.5.2.4 Stiffness and Strength

CBR-DCP profile and cumulative drops versus CBR are shown in Figure 77. The average CBR of both the natural and stabilized subgrade, and effective stabilized subgrade thickness are shown in Figure 78. Key observations from this analysis are as follows: (1) based on the effective treatment thickness, the average CBR of the stabilized subgrade was 103, (2) the average CBR of the natural subgrade was 27, (3) The average CBR of the stabilized subgrade was 3.8 times greater than the natural subgrade, (4) the top and bottom of the stabilized subgrade layer have comparatively lower CBR than the middle of the layer, and (5) the actual average treatment thickness was about 220 mm (8.8 in.), which was thicker the design value.

Back-calculated subgrade elastic moduli (E_{FWD}), uncorrected deflections, and corrected deflections are presented in Figure 79. An average applied test load was 57 KN (12876 lb) was used in the back-calculation. Poison's ratio was assumed to be 0.35, 0.35, 0.40, and 0.40 for ACC surface layer, aggregate stabilized subgrade, and natural subgrade layer respectively. Stabilized subgrade moduli were calculated based on designed or effective stabilized subgrade thicknesses obtained from the DCP profiles. Detailed assumptions of seed values and layer thicknesses are summarized in Appendix B. At the time of testing, the temperature at the middle depth of ACC pavement was 11 ^{0}C (52 ^{0}F). Deflections under the loading plate (D_0) were adjusted to a standard temperature of 20 ^{0}C (68 ^{0}F) using Equation (2). Key findings from the FWD testing are as follows: (1) the average corrected D_0 was about 0.21 mm under average applied load; (2) the average E_{FWD} was 238 MPa for natural subgrade and increased to 369 MPa for stabilized subgrade; (3) the average E_{FWD} of the stabilized subgrade was about 1.6 times higher compared to the natural subgrade; and (4) the values of E_{FWD} of natural and stabilized subgrade varied significantly indicating non-uniform subgrade soil properties.

Figure 80 presents the stress-strain relationship at point 45. The values of E_{V1} and E_{V2} were calculated in the first circle and after reloading. The uncorrected modulus of soil reaction k′u was calculated using deflection under a load of 69.0 kPa as shown in Figure 81. The average LWD elastic modulus (E_{LWD}) was 410% greater than natural subgrade. The E_{LWD} of stabilized subgrade was equal to 1.7 E_{V1} and 0.7 E_{V2}. The E_{LWD} of stabilized subgrade was 0.3 $E_{FWD.}$

Table 22 lists all LWD test results at points 45 and 46. Table 23 provides the elastic modulus ratio between stabilized and natural subgrade. The mean value, standard deviation, and coefficient of variation of in situ test results were listed in Table 24. All in situ test results are presented in Appendix C.

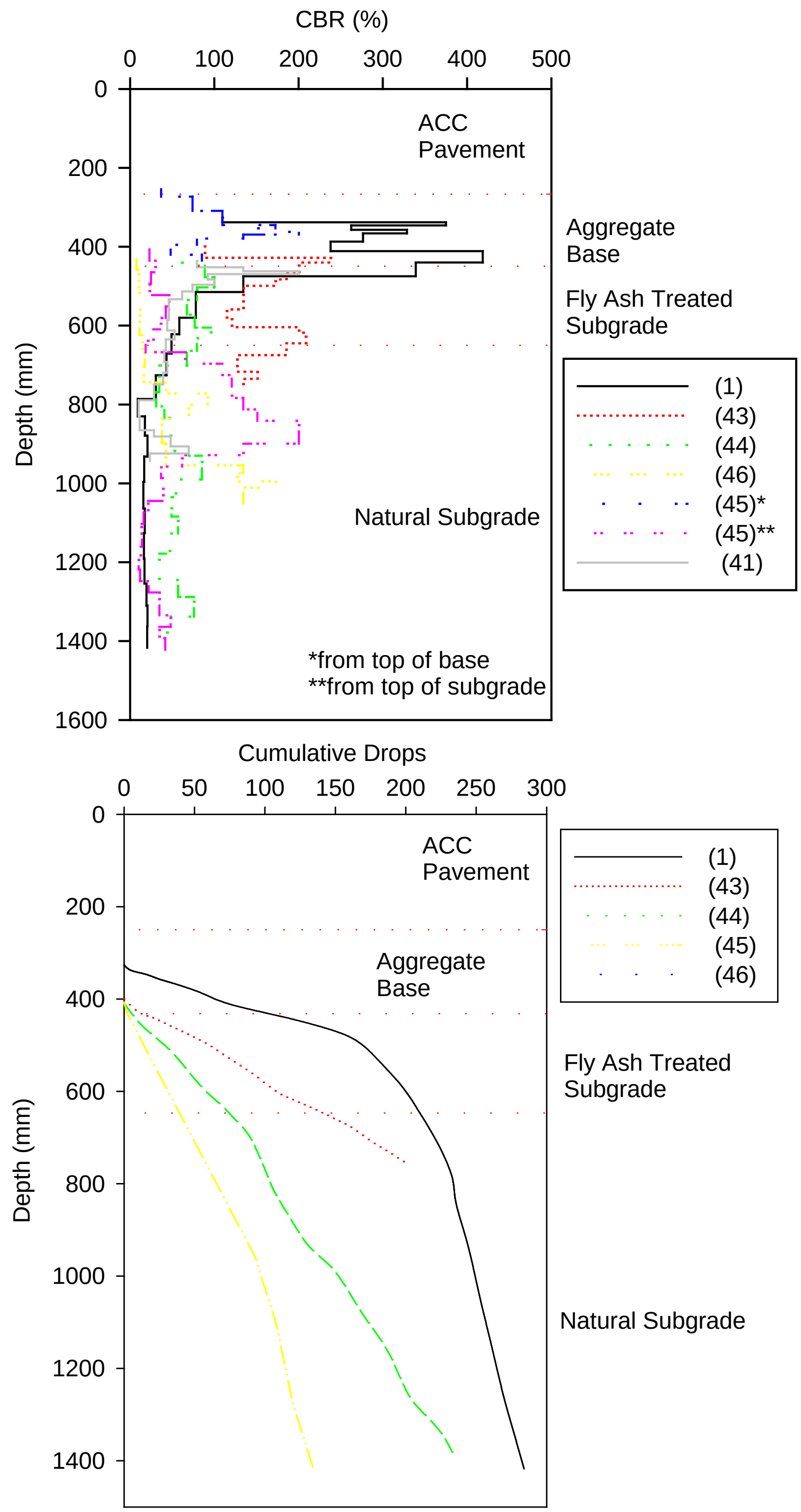


Figure 77. CBR – DCP profile of test points SH 99 Seminole County, Oklahoma

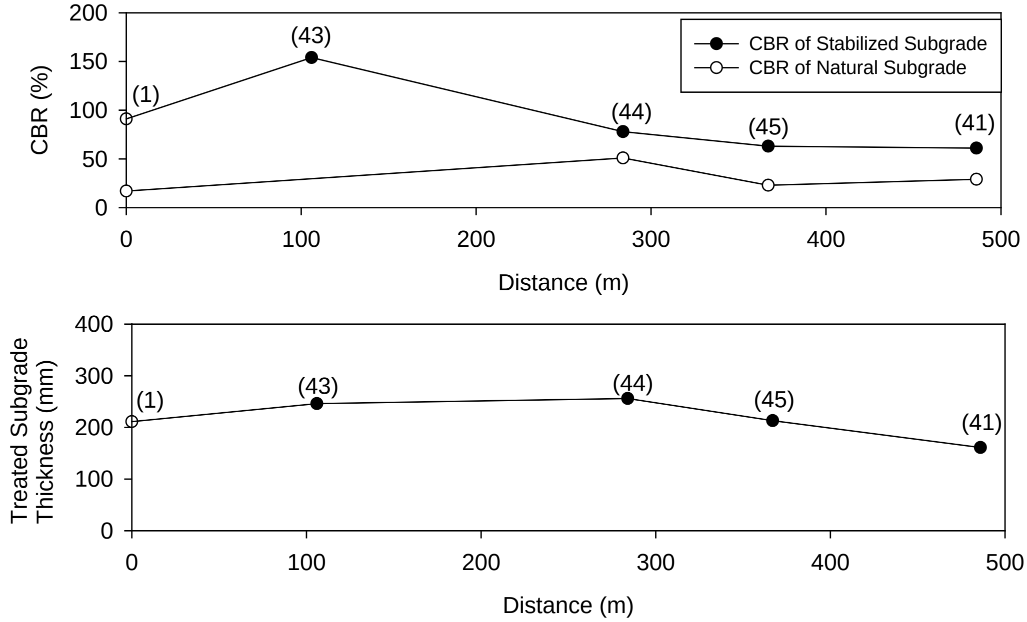


Figure 78. CBR and stabilized subgrade thickness from DCP profile SH 99 Seminole County, Oklahoma

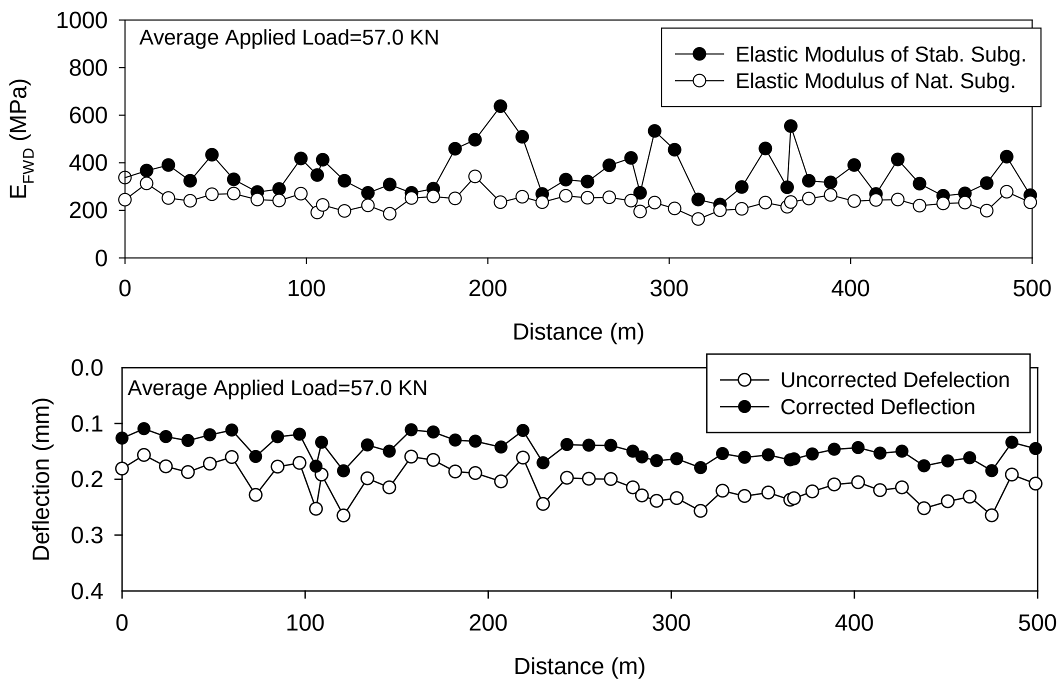


Figure 79. Back-calculated FWD elastic modulus of stabilized and natural subgrade, and deflections under the loading plate SH 99 Seminole County, Oklahoma

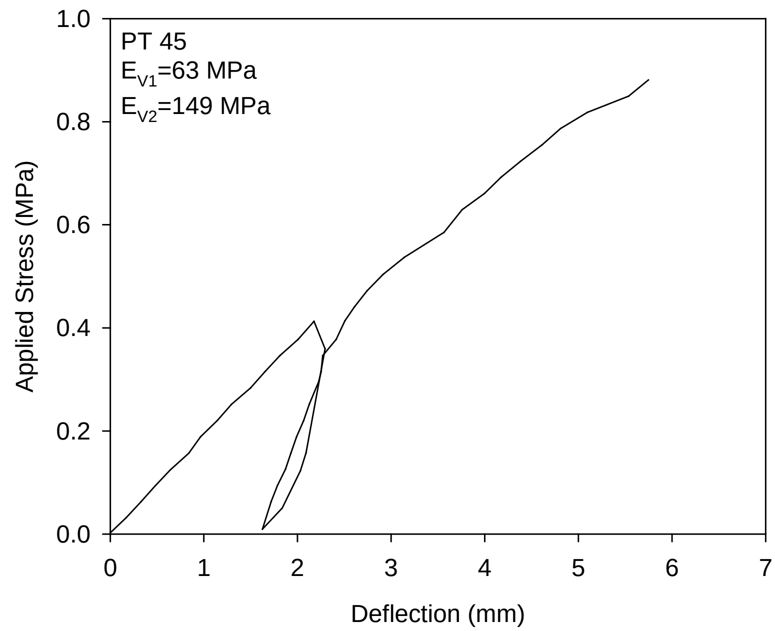


Figure 80. Stress – strain curves from plate load test at point 45 SH 99 Seminole County, Oklahoma

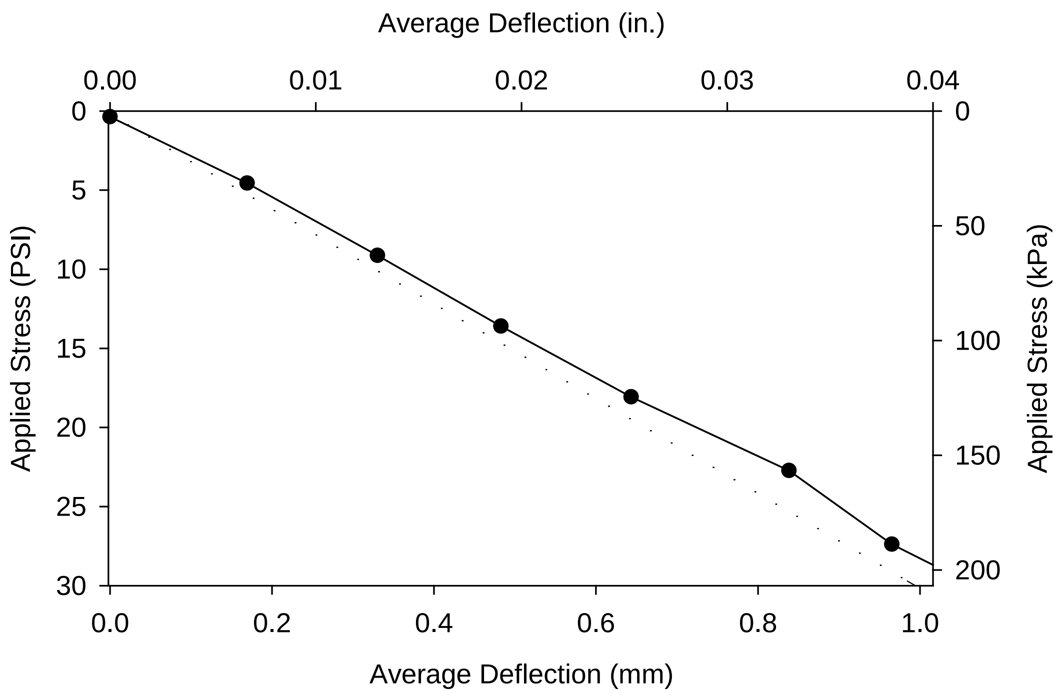


Figure 81. Stress – strain curves for obtaining K_U at point 45

Table 22. Summary of LWD test results SH 99 Seminole County, Oklahoma

Test Point	Material Type	Depth of Measurement	E_{LWD}	Average E_{LWD}
			MPa	MPa
45	Stabilized subgrade	Top of Stabilized Subgrade	80	65
45	Stabilized subgrade	63 mm from top of stabilized subgrade	50	
46	Natural Subgrade	Top of natural subgrade	16	16

Table 23. Summary of elastic modulus ratio between stabilized and natural subgrade SH 99 Seminole County, Oklahoma

Stab. Subg./Nat. Subg. Ratio		
CBR	E_{FWD}	E_{LWD}
3.8	1.6	4.1

Table 24. Summary statistics of test results from in situ testing SH 99 Seminole County, Oklahoma

Statistic	Stabilized Subgrade							Natural Subgrade		
Measurement	CBR	E_{FWD}	E_{LWD}	E_{V1}	E_{V2}	k_U	Thi.	CBR	E_{FWD}	E_{LWD}
	%	MPa	MPa	MPa	MPa	kPa/mm	mm	%	MPa	MPa
Number of Measurement (n)	5	45	2	1	1	1	5	5	45	1
Mean Value (μ)	103	369	65	63	149	78	220	27	238	16
Standard Deviation (σ)	60	132	21	—	—	—	37	17	32	—
Coefficient of Variation COV (%)	58	36	32	—	—	—	17	63	14	—

4.6 US 59, OK

4.6.1 Site Description

This project was located on US 59 north of Panama, in Le Flore County, Oklahoma. The general location of this site is shown in Figure 82. This road is a four-lane U.S. Highway. The design life of pavement is 20 years, equivalent single axle loads (ESALS) was 12.26 million and annual average daily traffic was 7500 in 1996 and estimated to be 13250 in 2016. The road was constructed in 2000. The length of this test section is approximately 500 m (1640 ft) from station 588+40 to 601+50. The pavement consisted of a nominal 254 mm (10 in.) thick asphalt concrete (AC), and 254 mm (10 in.) aggregate base, and 203 mm (8 in.) subgrade stabilized with fly ash (Figure 83). The ISU research team conducted in situ testing on September 30, 2010 with assistance and traffic control provided by Oklahoma DOT.

The plan view of the in situ test locations is shown in Figure 84. The research team preformed FWD tests on the surface of ACC pavement at intervals of about 15 m from test points 1 to 31. Five DCP tests were conducted at test points 4 (Sta.600+00), 12 (Sta. 596+00), 16 (Sta. 594+00), 20 (Sta. 592+00), 24 (Sta. 590+00), and 28 (Sta. 588+40). The control points 32, 33, and 34 were selected adjacent to test point 24. After coring, LWD and PLT test were performed on the top of stabilized subgrade at test point 24. LWD and DCP tests were also performed at the control points. Natural subgrade bag samples were collected at test point 24, 31, 32, and 33.

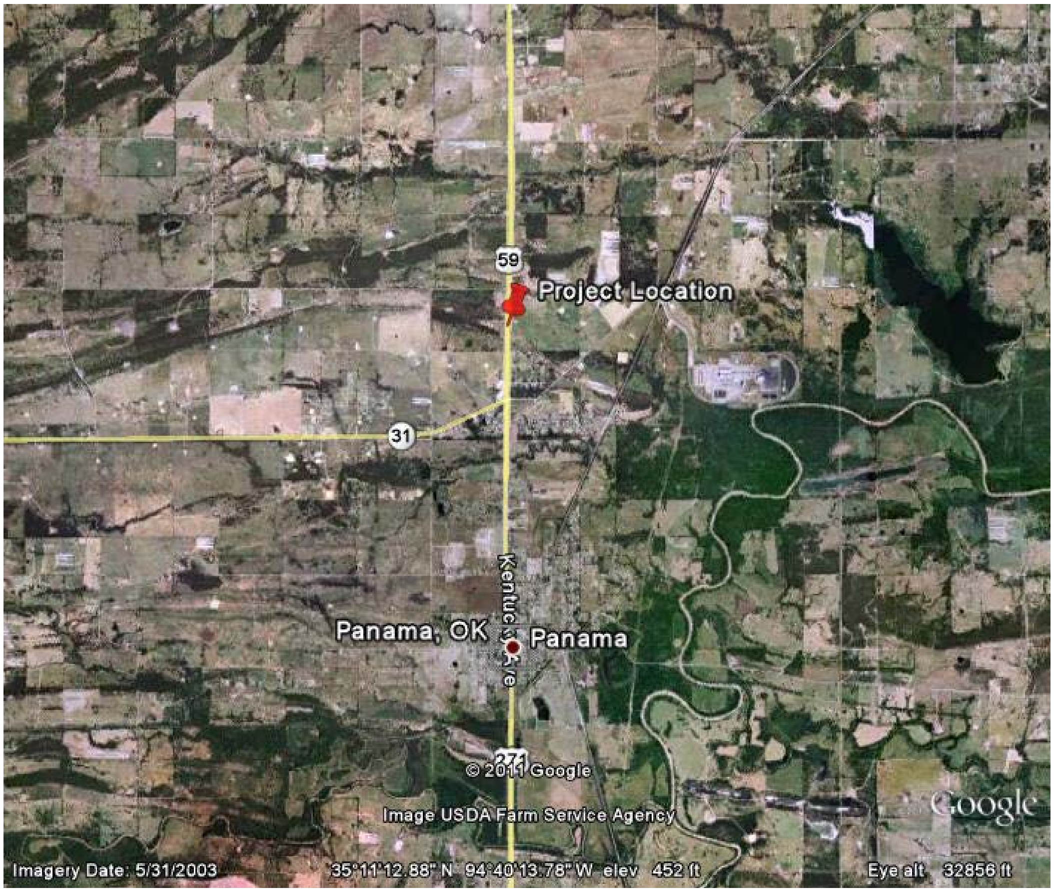


Figure 82. Project location of US 59 Le Flore County, Oklahoma

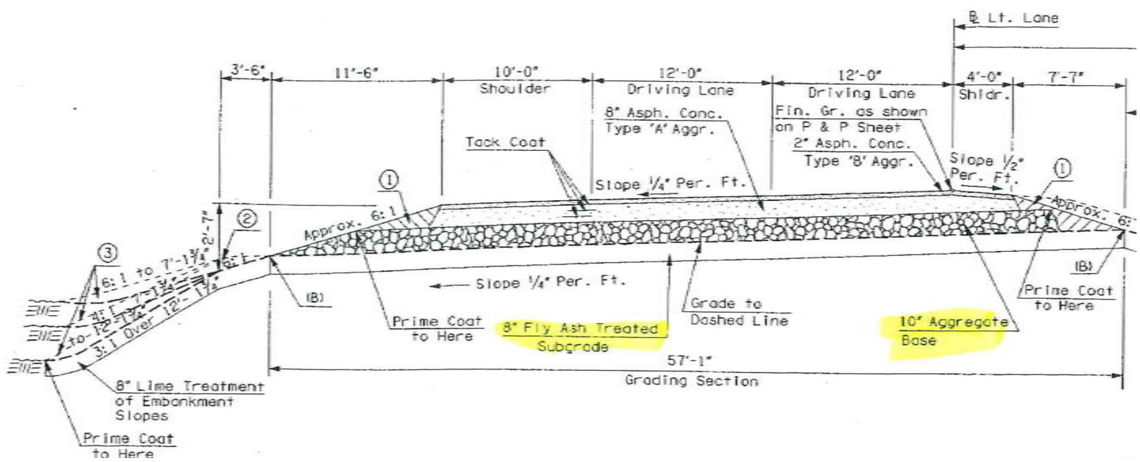


Figure 83. Typical cross section US 59 Le Flore County, Oklahoma

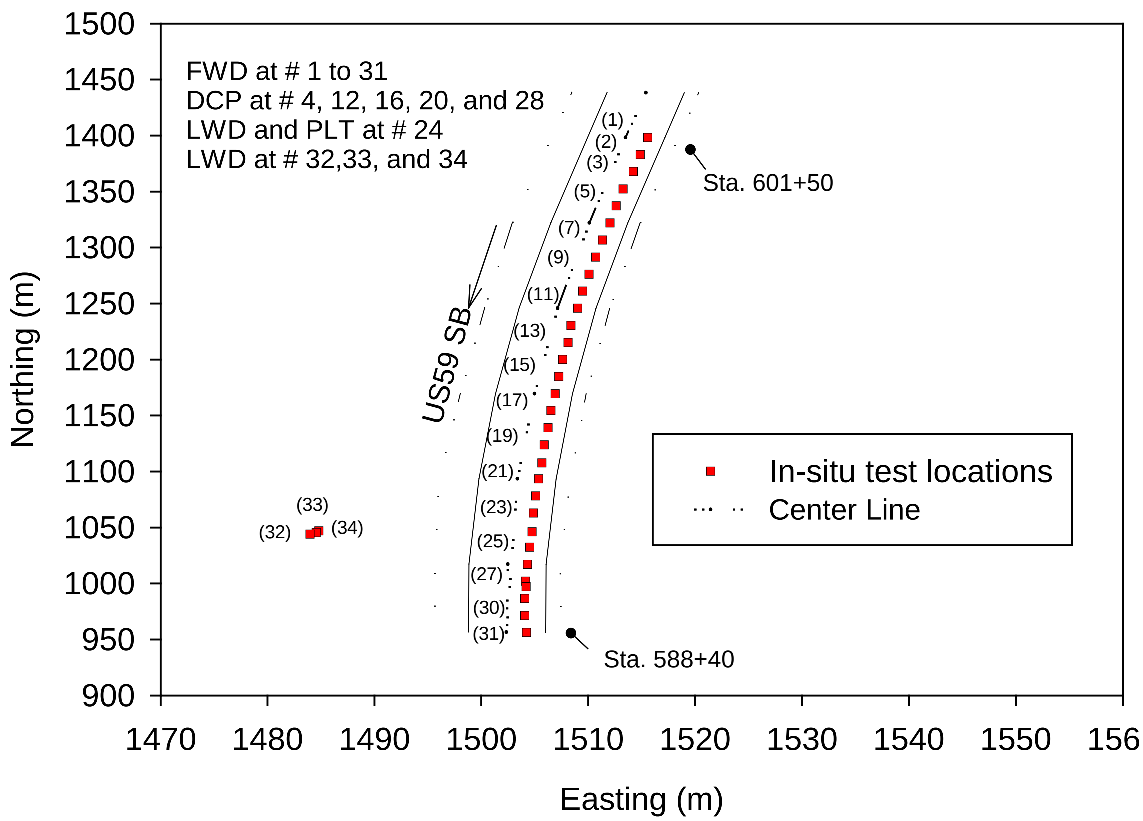


Figure 84. Test section plan layout with RTK GPS test points US 59 Le Flore County, Oklahoma

Figure 85. Site overview US 59 Le Flore County, Oklahoma

4.6.2 Test Results and Analysis

4.6.2.1 Material Properties of Base and Subgrade

Table 24 provides material index values for the aggregate base and subgrade layer. Figure 86 shows the grain-size distribution curves. The natural subgrade was classified as ML and A-4 (0), and the top 100 mm (4 in.) stabilized subgrade was classified as SM and A-4 (0). Compared to the natural subgrade, the sand content of the stabilized subgrade increased from about 31% to 48%. The clay content decreased from about 28 % to 4%, and the silt content decreased from about 38% to about 32%. LL of stabilized subgrade samples was reduced from about 46 to 33. PI value was 6 for stabilized subgrade and 25 for natural subgrade in situ density and moisture content of some test points were recorded during construction shown in Appendix G.

Table 25. Summary of material properties US 59 Le Flore County, Oklahoma

Parameters	Materials		
Material Description	Base	Stabilized Subgrade	Natural Subgrade
Depth mm (in.)	0-254 (0-10)	0-200 (0-8)	—
Gravel Content (%) (> 4.75mm)	49.7	16.1	3.6
Sand Content (%) (4.75mm – 75μm)	31.1	48.2	30.5
Silt Content (%) (75μm – 2μm)	15.2	31.5	37.7
Clay Content (%) (< 2μm)	4.0	4.2	28.2
Coefficient of Uniformity (C_u)	446.7	110.3	—
Coefficient of Curvature (C_c)	5.2	0.4	—
Liquid Limit, LL (%)	24.7	32.7	45.9
Plasticity Index, PI	9.7	5.6	24.7
AASHTO	A-1-b	A-4	A-4
USCS	GM	SM	ML
Water Content (%)	5.0	17.7	13.2

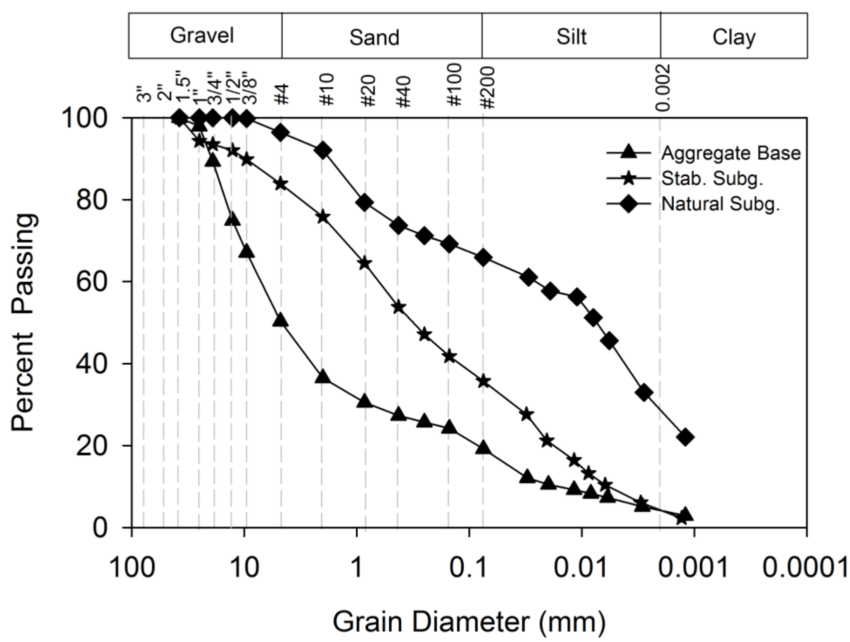


Figure 86. Particle size distribution curves for subgrade materials US 59 Le Flore County, Oklahoma

4.6.2.2 pH of Stabilized and Natural Subgrade

Table 26 provides pH values of natural and stabilized subgrade.

Table 26. Summary of pH value of subgrade US 59 Le Flore County, Oklahoma

Depth	pH value
Natural subgrade	8.4
Fly ash stabilized subgrade	8.9

4.6.2.3 SEM Analysis

The energy dispersive spectrometry (EDS) map of stabilized subgrade is shown in Figure 87. The majority elements were calcium (Ca), silica (Si), alumina (Al), and oxygen (O). Additional elements present include iron (Fe), potassium (K), and Sodium (Na). Figure 88 compares elemental concentration in Al, Si, O, S, Mg, Ca, K, and C for the stabilized subgrade sample in area a and b. The sample shows high concentration of Si, Al, and O in both areas, and a lower concentration of Ca in area a. All SEM images are presented in Figure 89 and Appendix D. Figure 89 shows evidence of an unreacted fly ash particle in the soil matrix.

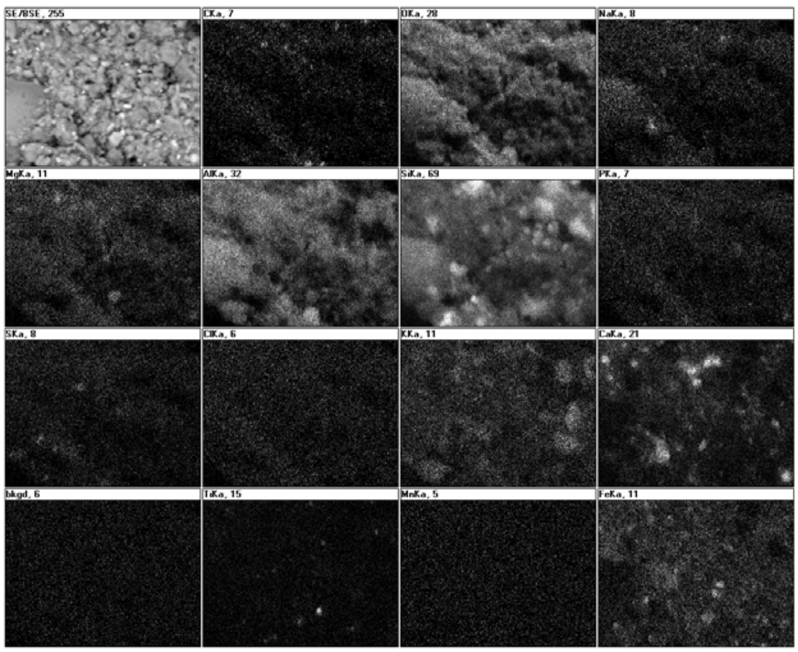


Figure 87. EDS map of stabilized subgrade sample (1500 ×) US 59 Le Flore County, Oklahoma

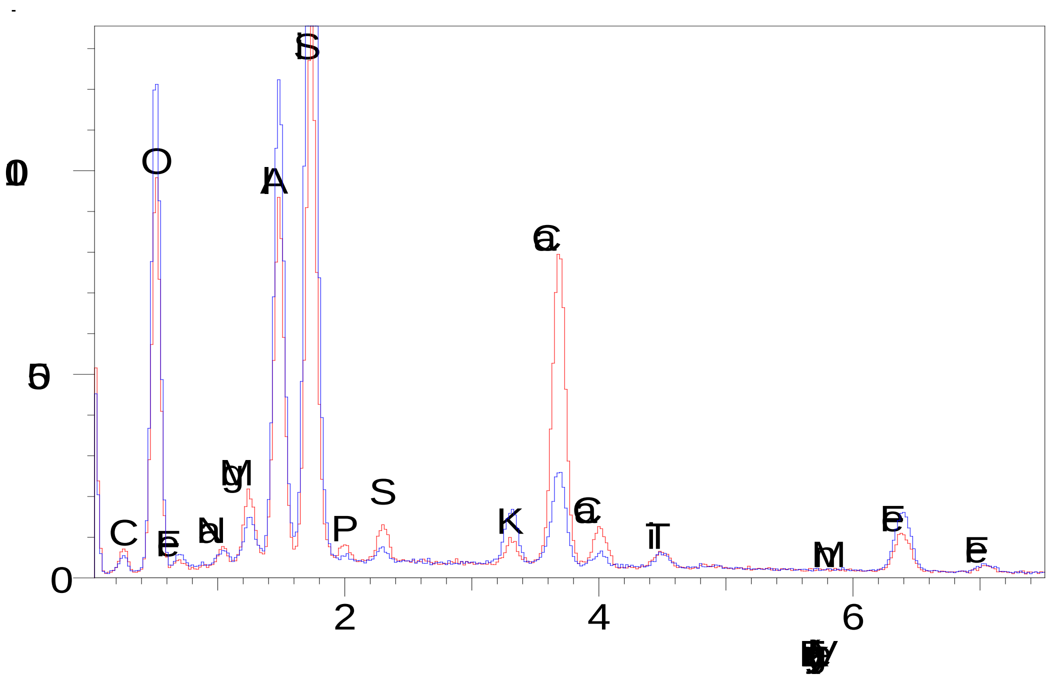


Figure 88. EDS intensity counts for stabilized subgrade sample in area a (blue line: 500×) and stabilized subgrade sample in area b (red line: 500×) US 59 Le Flore County, Oklahoma

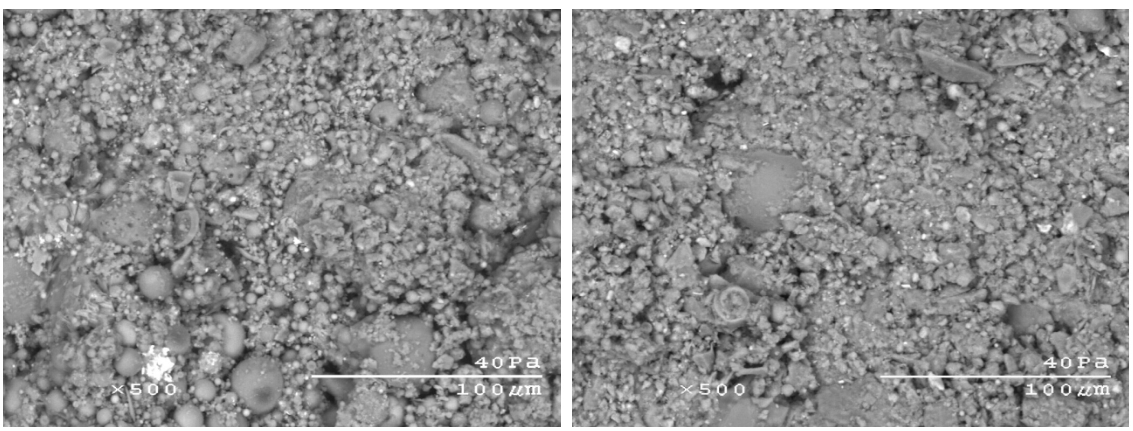


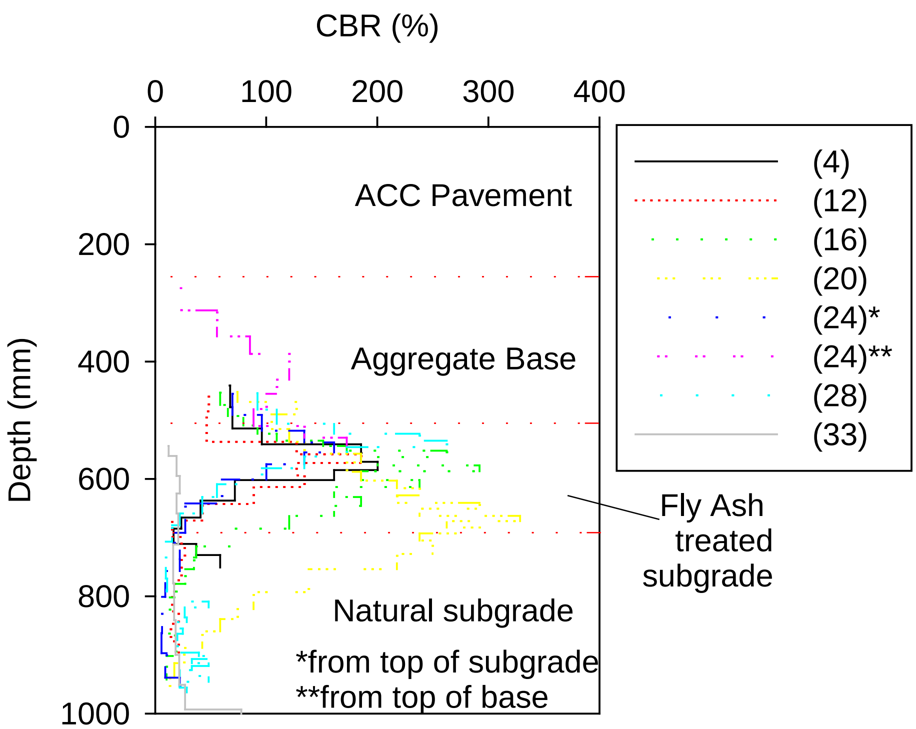
Figure 89. SEM of stabilized subgrade US 59 Le Flore County, Oklahoma

4.6.2.4 Stiffness and Strength

DCP profiles and cumulative drops versus CBR are shown in Figure 90. Average CBR of both natural and stabilized subgrade, and effective stabilized subgrade thickness are shown in Figure 91. Major observations from this testing include: (1) based on the effective treatment thickness, the average CBR of the stabilized subgrade was 139, (2) the average CBR of the natural subgrade was 23, (3) the CBR of the stabilized subgrade was 6.4 times higher than the natural subgrade, (4) the top and bottom portions of the stabilized subgrade layer yield lower CBR values than the middle, and (5) from DCP profiles, the actual average treatment thickness at the test locations was about 150 mm (6 in.), which was thinner than the design value.

Back-calculated subgrade elastic moduli (E_{FWD}) and deflections (D_0) are presented in Figure 92. An average applied test load was 57 KN (12906 lb) was used as part of the back-calculation. Poison's ratio values were assumed to be 0.35, 0.35, 0.40, and 0.40 for ACC surface layer, aggregate base, stabilized subgrade and natural subgrade layer, respectively. Stabilized subgrade moduli were calculated based on designed or effective stabilized subgrade thicknesses obtained from DCP profiles. Detailed assumptions of seed values and layer thicknesses are summarized in Appendix B. At the middle depth of the ACC pavement layer, the measured temperature at the time of testing was as 18°C (65°F). Deflections under the loading plate (D_0) were adjusted to a standard temperature of 20°C (68°F) using equation (2). Key findings from the FWD testing include: (1) the average corrected D_0 was about 0.20 mm under average applied load of 57 KN (12906 lb). (2) the average E_{FWD} was 383 MPa for natural subgrade and increased to 819 MPa for stabilized subgrade; (3) the average E_{FWD} of stabilized subgrade was about 230% of the natural subgrade; and (4) the values of E_{FWD} of stabilized and natural varied significantly indicating non-uniform subgrade soil properties.

Figure 93 presents the stress-deflection relationship at point 24. The values of E_{V1} and E_{V2} were calculated in the first load cycle and after reloading. The uncorrected modulus of soil reaction k′u was calculated using the average deflection under a plate contact stress of 69.0 kPa as shown in Figure 94. The LWD elastic modulus (E_{LWD}) for the stabilized subgrade was equal to 0.6 E_{V1} and 0.4 E_{V2}. The E_{LWD} of stabilized subgrade was $0.1E_{FWD.}$ Table 28 provides the elastic modulus ratio between stabilized and natural subgrade. The mean value, standard deviation, and coefficient of variation of in situ test results were listed in Table 29. All in situ test results are presented in Appendix C.



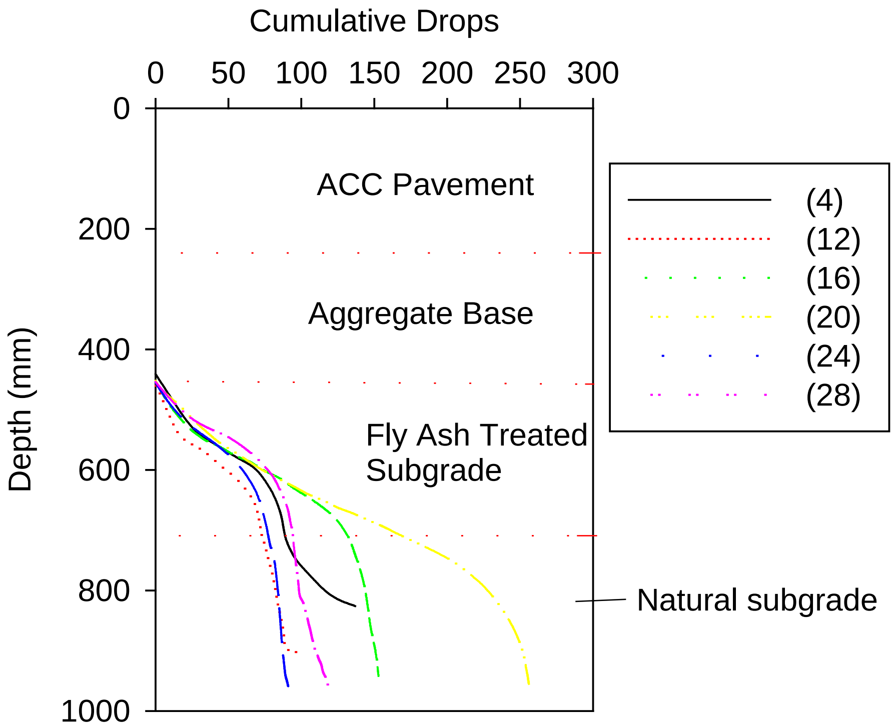


Figure 90. CBR – DCP profile and cumulative drops versus CBR of test points US 59 Le Flore County, Oklahoma

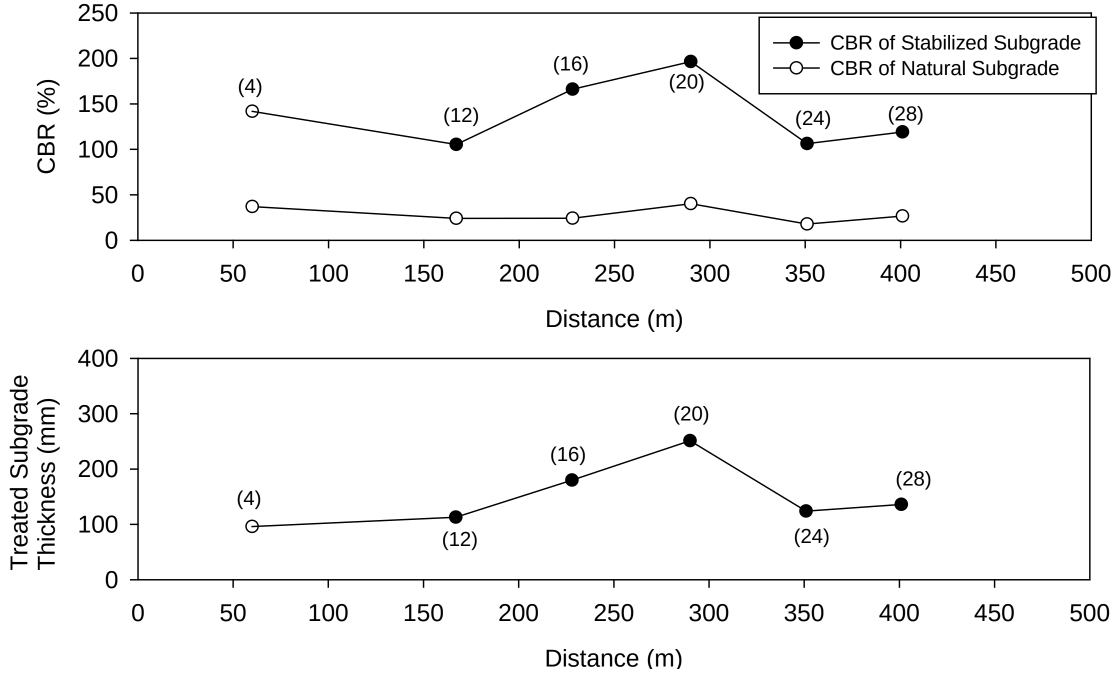


Figure 91. CBR and stabilized subgrade thickness from CBR-DCP profile

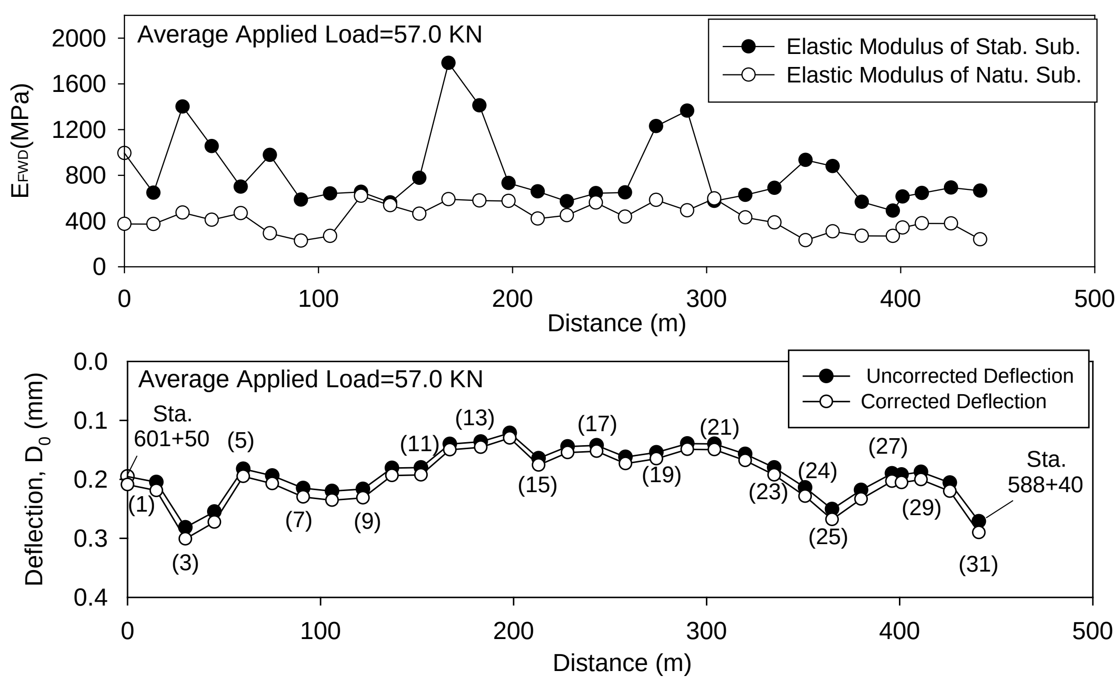


Figure 92. Back-calculated FWD elastic modulus of stabilized and natural subgrade, and deflections under the loading plate US 59 Le Flore County, Oklahoma

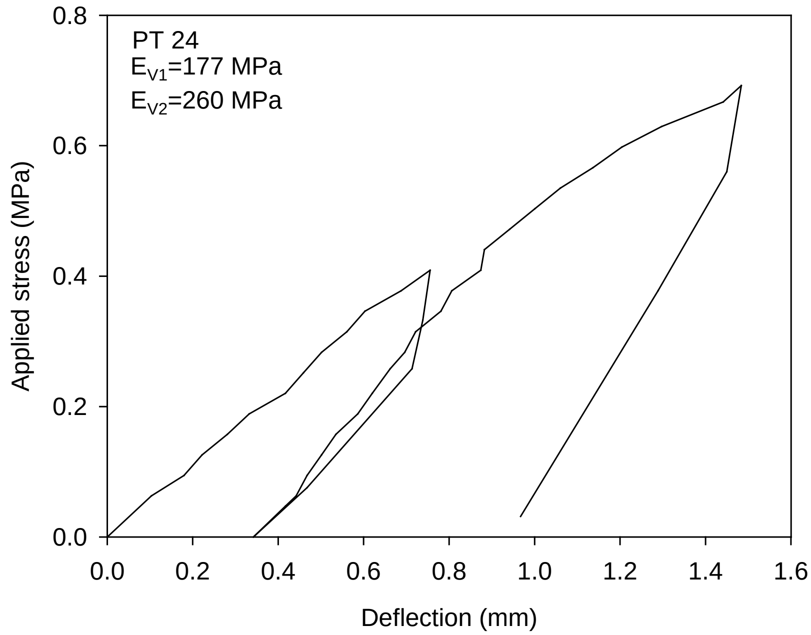


Figure 93. Stress – strain curves from plate load test at point 24 US 59 Le Flore County, Oklahoma

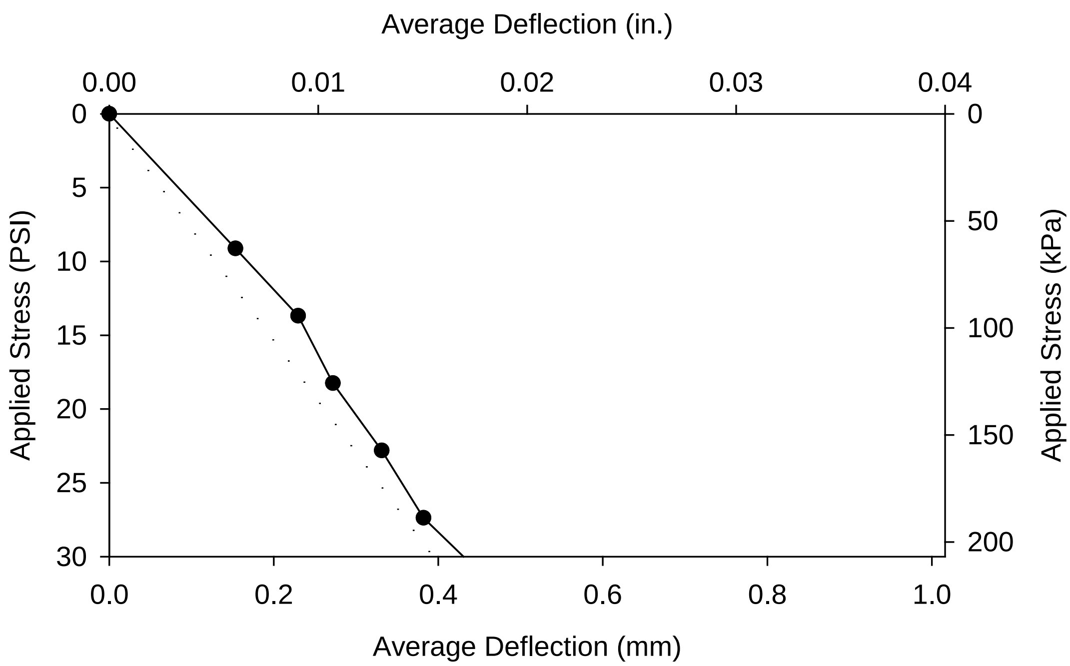


Figure 94. Stress – strain curves for obtaining K_U at point 24 US 59 Le Flore County, Oklahoma

Table 27. Summary of LWD test results US 59 Le Flore County, Oklahoma

Test Point	Material Type	Depth of Measurement	E_{LWD}	Average E_{LWD}
			MPa	MPa
24	Base	Top of base	126	126
24	Stabilized Subgrade	Top of stabilized subgrade	105	105
32	Natural Subgrade	Top of natural subgrade	26	20
33	Natural Subgrade	Top of natural subgrade	13	
34	Natural Subgrade	Top of natural subgrade	20	

Table 28. Summary of elastic modulus ratio between stabilized and natural subgrade US 59 Le Flore County, Oklahoma

Stab. Subg./Nat. Subg. Ratio		
CBR	E_{FWD}	E_{LWD}
6.4	2.3	5.3

Table 29. Summary statistics of test results from in situ testing US 59 Le Flore County, Oklahoma

Statistic	Base	Stabilized Subgrade							Natural Subgrade		
Meas.	E_{LWD}	CBR	E_{FWD}	E_{LWD}	E_{V1}	E_{V2}	k_U	Thi.	CBR	E_{FWD}	E_{LWD}
	MPa	%	MPa	MPa	MPa	MPa	kPa/mm	mm	%	MPa	MPa
Number of Meas. (n)	1	6	31	1	1	1	1	6	6	31	3
Mean Value (μ)	126	139	819	105	177	261	164	150	23	383	20
Standard Deviation (σ)	—	36	316	—	—	—	—	57	—	110	8
Coefficient of Variation COV (%)	—	26	39	—	—	—	—	38	—	29	33

4.7 US 75 SB, KS

4.7.1 Site Description

This project was located on US 75 south of Lyndon in Osage County, Kansas. The general location of this site is shown in Figure 95. This road is a two-lane U.S. Highway, and was constructed in 1995. The length of this test section is approximately 700 m (2297 ft). The designed pavement consisted of a nominal 330 mm (13 in.) thick asphalt concrete (AC), 50 mm (2 in.) thick base, and 100 mm (4 in.) lime stabilized subgrade. The subgrade was stabilized with 5% lime according to the design records. The ISU research team conducted in situ testing near the milepost 123 on November 2, 2010 with assistance and traffic control provided by Kansas DOT.

The plan view of in situ test locations is shown in Figure 96. The research team preformed FWD tests on the surface of ACC pavement at intervals of about 10 m from points 1 to 30 and 20 m from points 31 to 50. DCP tests were conducted at test points 4, 11, 20, 28, 34, and 45. After coring, LWD tests were performed at different depths within the stabilized subgrade, and a PLT test was performed on the top of the stabilized subgrade at test point 18. Bag samples of materials were collected at test point 18 from the top of the stabilized subgrade to a depth of 250 mm (10 in.) at intervals about 50 mm (2 in.). Undisturbed Shelby tube samples were collected at test point 18 from the top of the stabilized subgrade to a depth of 990 mm (39 in.). Bag and tube samples were carefully sealed and transported to ISU laboratory for analysis.

Figure 95. Project location of US 75 Osage County, Kansas

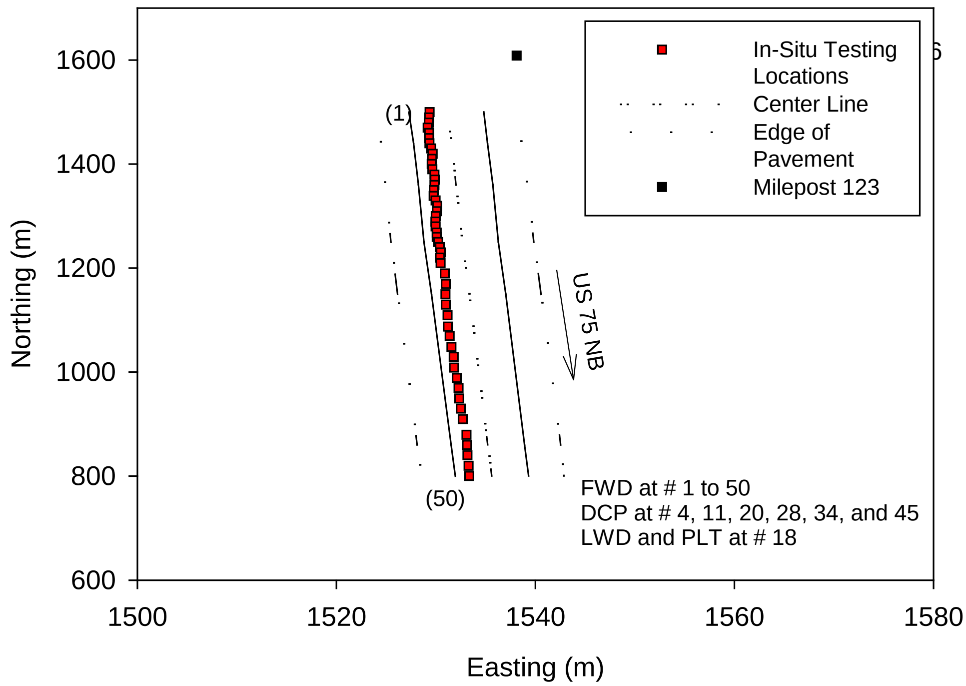


Figure 96. Test section plan layout with RTK PGS locations US 75 Osage County, Kansas

Figure 97. Site overview south bound lane US 75 Osage County, Kansas

4.7.2 Test Results and Analysis

4.7.2.1 Material Properties of Base and Subgrade

The stabilized subgrade samples were taken at test point 18 from the top to a depth of 250 mm (10 in.) subgrade at intervals of about 50 mm (2 in.). The natural subgrade sample was collected from Shelby tube at test point 18. According to USCS and AASHTO, the natural subgrade was classified as ML and A-4, and the top 50 mm (2 in.) stabilized subgrade was classified as SM and A-2. The bottom 50-100 mm (2-4 in.) stabilized subgrade soil was classified as ML and A-4 as same as the soil type of natural subgrade. Table 30 provides material properties of subgrade, and it is shown that gravel, sand, silt, and clay content were largely different between natural and the top 50 mm (2 in.) stabilized subgrade. The average LL values of natural and stabilized subgrade samples were approximately equal. The average PI values of the top 50 mm (2 in.) stabilized subgrade samples were about 19 smaller than natural subgrade. PI values of the bottom 50 mm (2 in.) stabilized subgrade samples were about 5 smaller than natural subgrade. Figure 98 shows particle size distribution curves of different subgrade layers. Test results show the soil type of subgrade has been modified after treatment.

Table 30. Summary of material properties US 75 Osage County, Kansas

Parameter	Materials					
Material Description	Natural Sub.	Base	Stab. Sub.	Stab. Sub.	Sub.	Sub.
Depth mm (in.)	838-990 (33-39)	0-50 (0-2)	0-50 (0-2)	50-100 (2-4)	100-150 (4-6)	150-250 (6-10)
Gravel (%) (> 4.75mm)	0.4	48.3	22.5	11.4	1.0	0.4
Sand (%) (4.75mm – 75µm)	2.9	40.2	51.9	25.2	7.6	4.7
Silt (%) (75µm – 2µm)	30.3	8.9	19.9	36.7	51.1	55.6
Clay (%) (< 2µm)	66.4	2.6	5.7	26.7	40.3	39.3
C_u	—	149.3	481.8	—	—	—
C_c	—	15.0	6.6	—	—	—
Liquid Limit, LL (%)	56.1	—	54.0	55.6	57.5	56.1
Plasticity Index, PI	33.1	—	14.0	28.3	34.8	33.0
AASHTO	A-4	A-1-a	A-2	A-4	A-4	A-4
USCS	ML	GP-GM	SM	ML	ML	ML
Water Content (%)	23.8	32.4	29.9	25.1	25.2	25.5

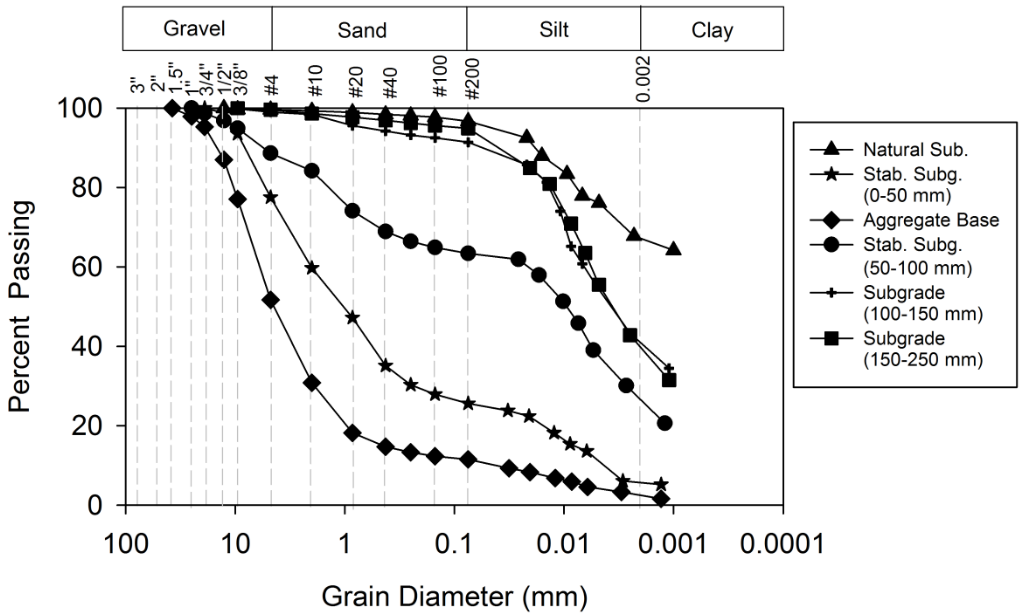


Figure 98. Particle size distribution curves for subgrade materials US 75 Osage County, Kansas

4.7.2.2 pH of Stabilized and Natural Subgrade

Figure 99 shows the pH profile of the subgrade layers at test point 8. The pH values of stabilized subgrade ranged from about 7.7 to 8.8. It gradually decreased from the top of stabilized subgrade to the bottom of stabilized subgrade. Below the stabilized subgrade, the pH values were relatively constantly to a depth of 400 mm. Then the pH value decrease from 7.5 to 6.5 to a depth of 1000 mm.

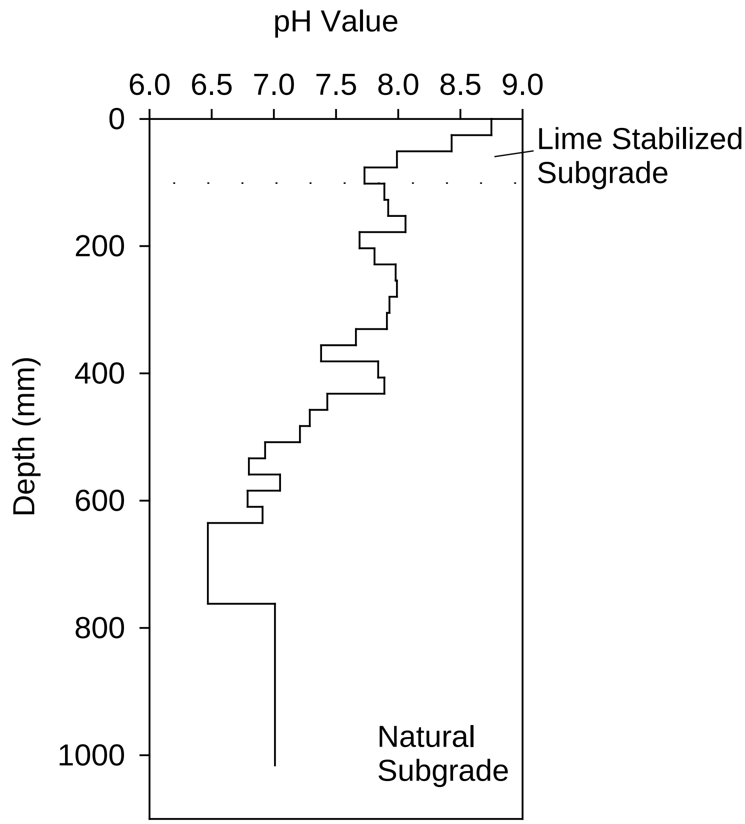


Figure 99. pH profile of subgrade US 75 Osage County, Kansas

4.7.2.3 SEM Analysis

Energy dispersive spectrometry (EDS) maps of the natural subgrade are shown in Figure 100 and Figure 101. The majority elements identified include silica (Si), alumina (Al), and oxygen (O). Additional present elements were iron (Fe), potassium (Mg), and Sodium (Na).
An EDS map of the stabilized subgrade is shown in Figure 101. The majority elements identified include calcium (Ca), Si, Al, phosphorus (P), and O. The mineral Ca was found in high concentration in only a small area of the map. Additional elements identified include Fe, potassium (K) and Na.

Figure 102, Figure 103, and Figure 104 compare elemental concentrations of Al, Si, O, S, Mg, Ca, K, P, and C for the natural and stabilized subgrade samples. The natural subgrade sample shows lower concentrations of Ca and P, and higher concentration of Si, Al, and O. All SEM images are presented in Figure 105, Figure 106, and Appendix D.

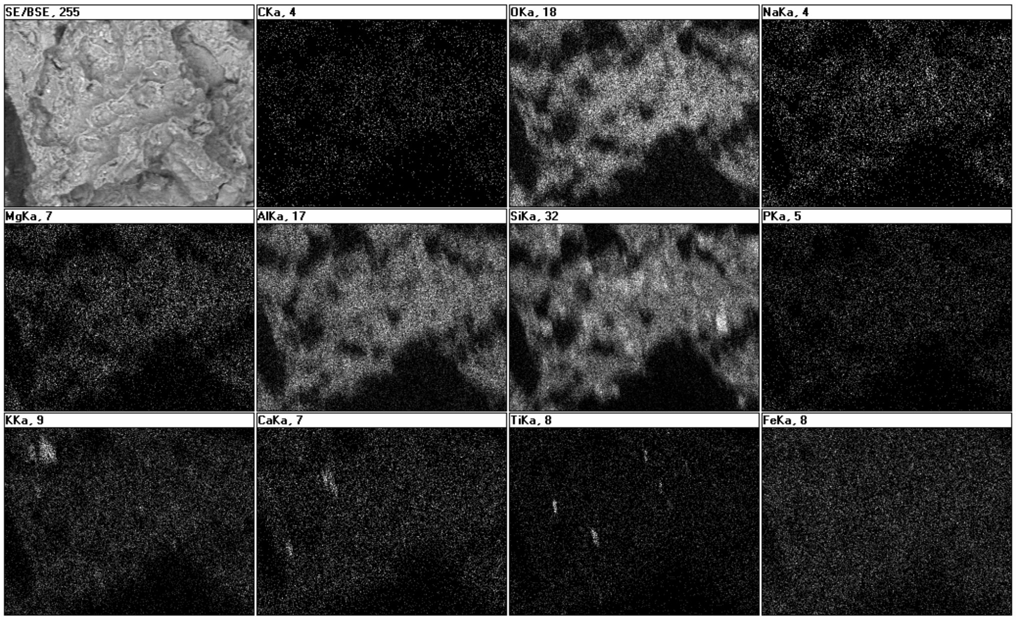


Figure 100. EDS map of natural subgrade sample (500 ×) US 75 Osage County, Kansas

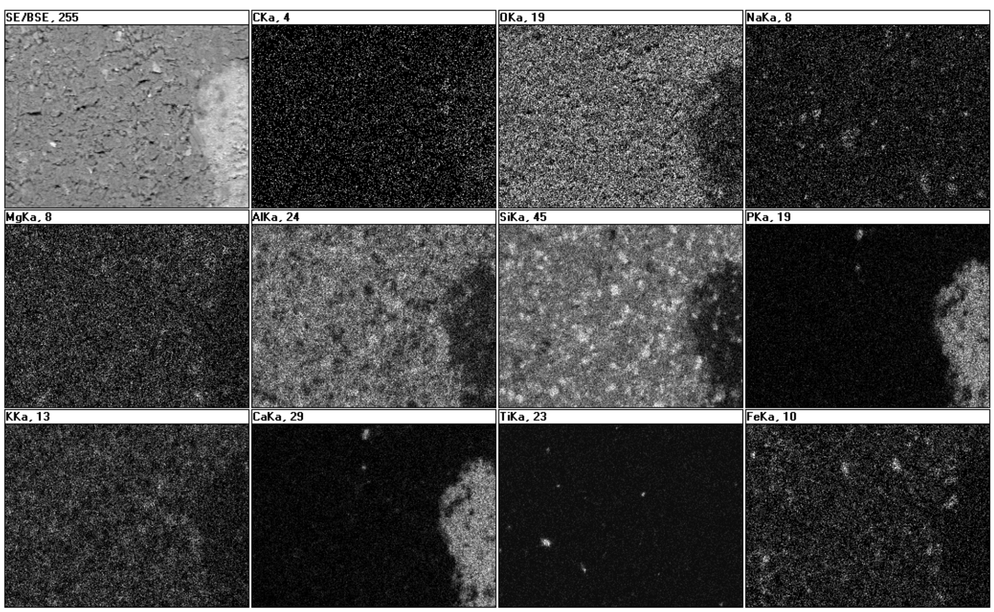


Figure 101. EDS map of stabilized subgrade sample (500 ×) US 75 Osage County, Kansas

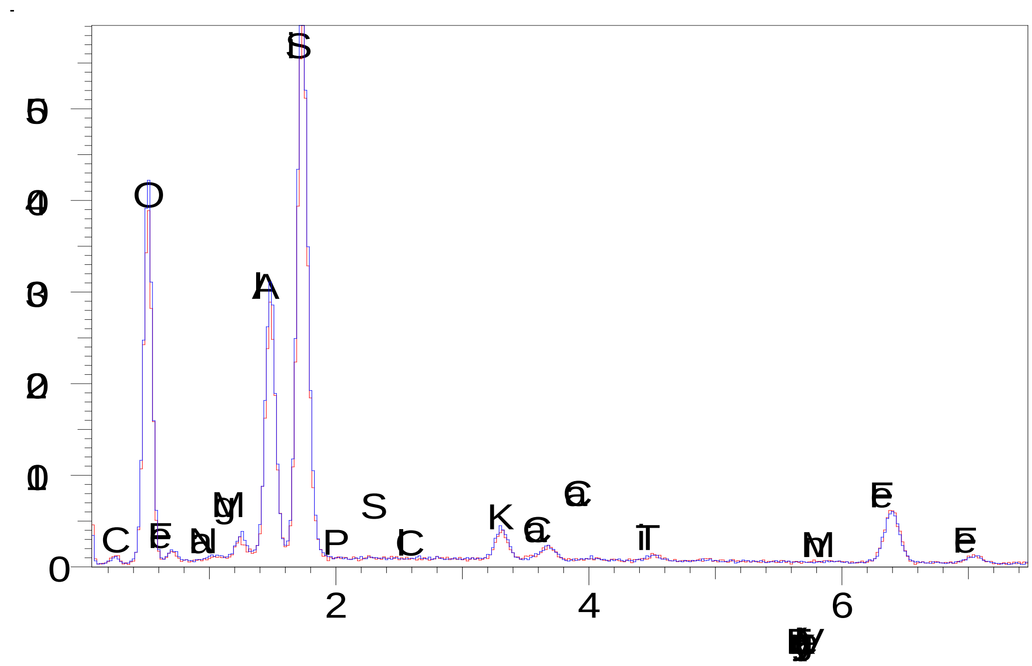


Figure 102. EDS intensity counts for natural subgrade sample (red line: 30×; blue line: 150×) US 75 Osage County, Kansas

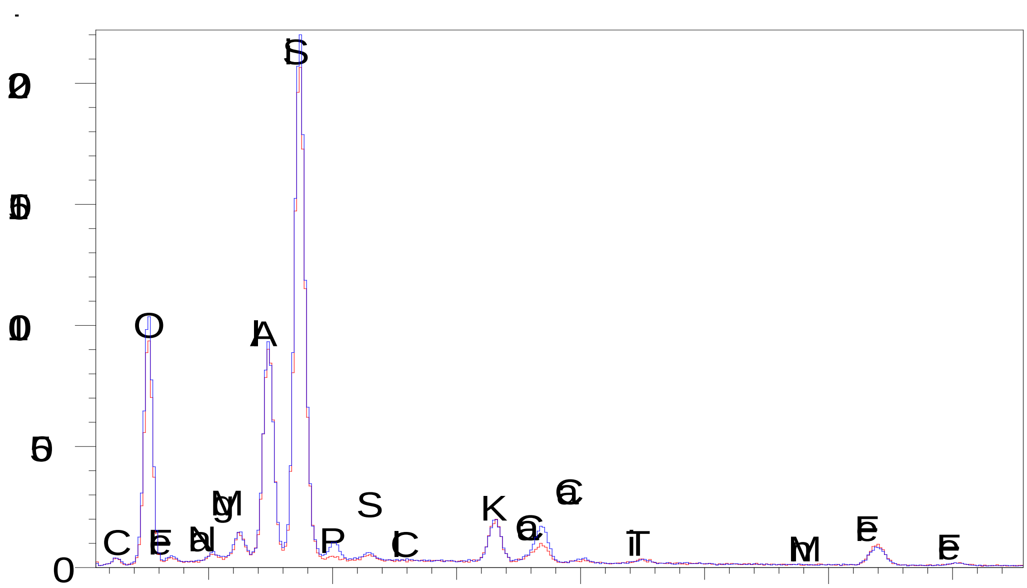


Figure 103. EDS intensity counts for stabilized subgrade sample (red line: 30×; blue line: 150×) US 75 Osage County, Kansas

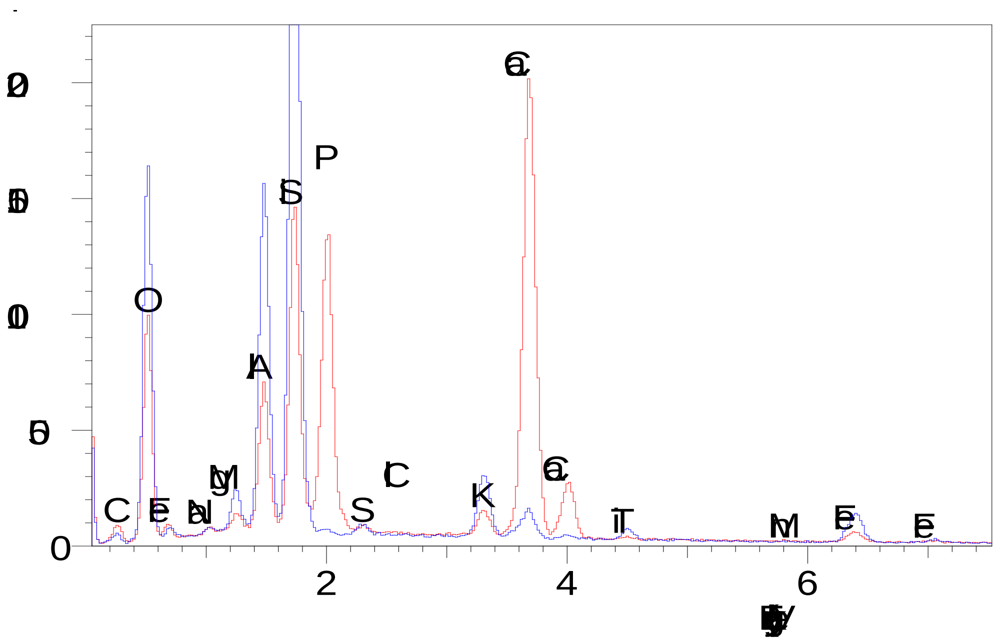


Figure 104. EDS intensity counts for stabilized subgrade sample in area a (red line: 1500×) and in area b (blue line: 1500×) US 75 Osage County, Kansas

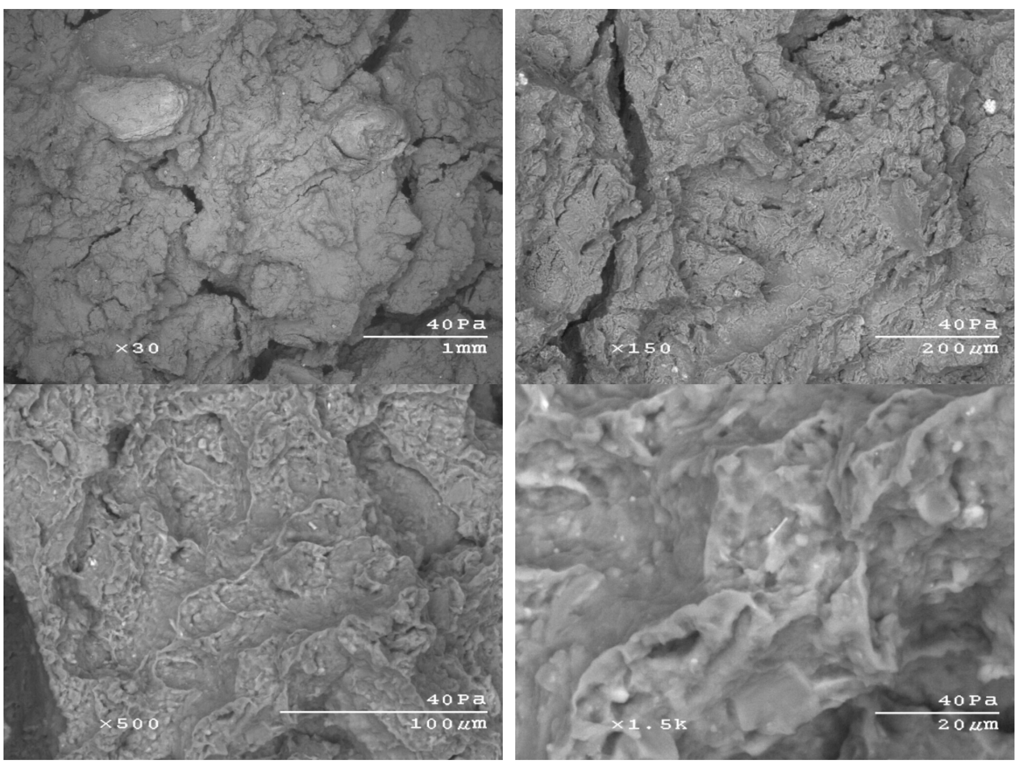


Figure 105. SEM images of natural subgrade US 75 Osage County, Kansas

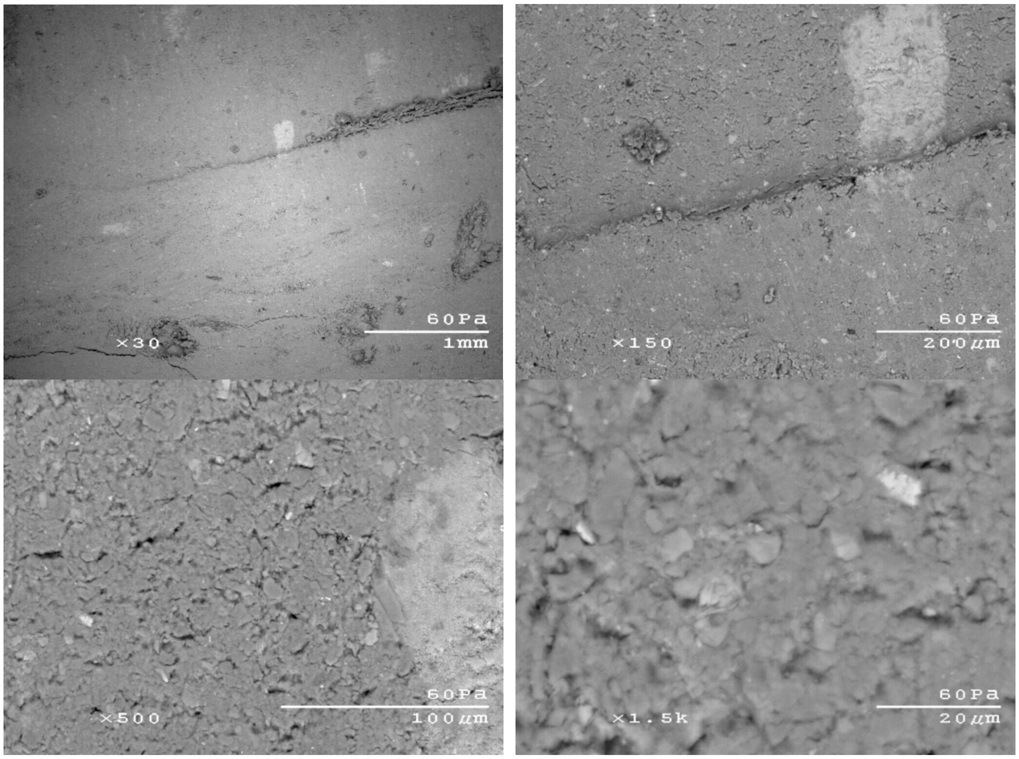


Figure 106. SEM images of stabilized subgrade US 75 Osage County, Kansas

4.7.2.4 Stiffness and Strength

DCP profiles and cumulative drops versus CBR are shown in Figure 107. The average CBR of both natural and stabilized subgrade, and effective stabilized subgrade thickness are shown in Figure 108. Major observations derived from the DCP testing include: (1) based on the effective treatment thickness, the average CBR of the stabilized subgrade was 30, (2) the average CBR of the natural subgrade was 11, (3) the average CBR of the stabilized subgrade was 2.7 times higher than the natural subgrade, (4) the subgrade did not shown significant strength improvement within the design thickness at test point 11, (5) the subgrade shows minimum strength improvement at test points 20, 28, and 45, and (6) the effective treatment thickness was thinner than the design value.

Back-calculated subgrade elastic moduli (E_{FWD}) and surface deflections were presented in Figure 109. An applied test load of 57.9 KN (13020 lb) was used in the back-calculation. Poison's ratio was assumed to be 0.35, 0.35, 0.40, and 0.40 for ACC surface layer, aggregate base, stabilized subgrade and natural subgrade layer respectively. Stabilized subgrade moduli were calculated based on designed or effective stabilized subgrade thickness obtained from DCP profiles. Detailed assumptions of seed values and layer thickness are summarized in Appendix B. Deflections under the loading plate were adjusted to a standard temperature of 20°C (68°F) using Equation (2). The temperature of middle depth of ACC pavement was measured as 9.8°C (49.7°F) prior to FWD testing. The key findings are: (1) the average D_0 and $D_{0\text{-cor}}$ were about 0.13 mm and 0.19 under average applied load; (2) the average E_{FWD} was 323 MPa for natural subgrade and 711 MPa for stabilized subgrade; (3) the average E_{FWD} of stabilized subgrade was about 2.2 times higher than the natural subgrade; and (4) the values of E_{FWD} of natural and stabilized subgrade varied significantly indicating non-uniform subgrade soil properties.

Figure 110 presents the stress-deflection relationship at point 18. The values of E_{V1} and E_{V2} were calculated from the first load cycle and after reloading. The uncorrected modulus of soil reaction k′u was calculated using deflection under a plate contact stress of 69.0 kPa as shown in Figure 111. The average E_{LWD} was 37 MPa for stabilized subgrade. The average E_{LWD} of stabilized subgrade was equal to 2.5 E_{V1} and 5.3 E_{V2}. The undrained shear strength (s_u) of the top subgrade (1-7 in.) did not showed strength improvement after treatment compared with underlying subgrade.

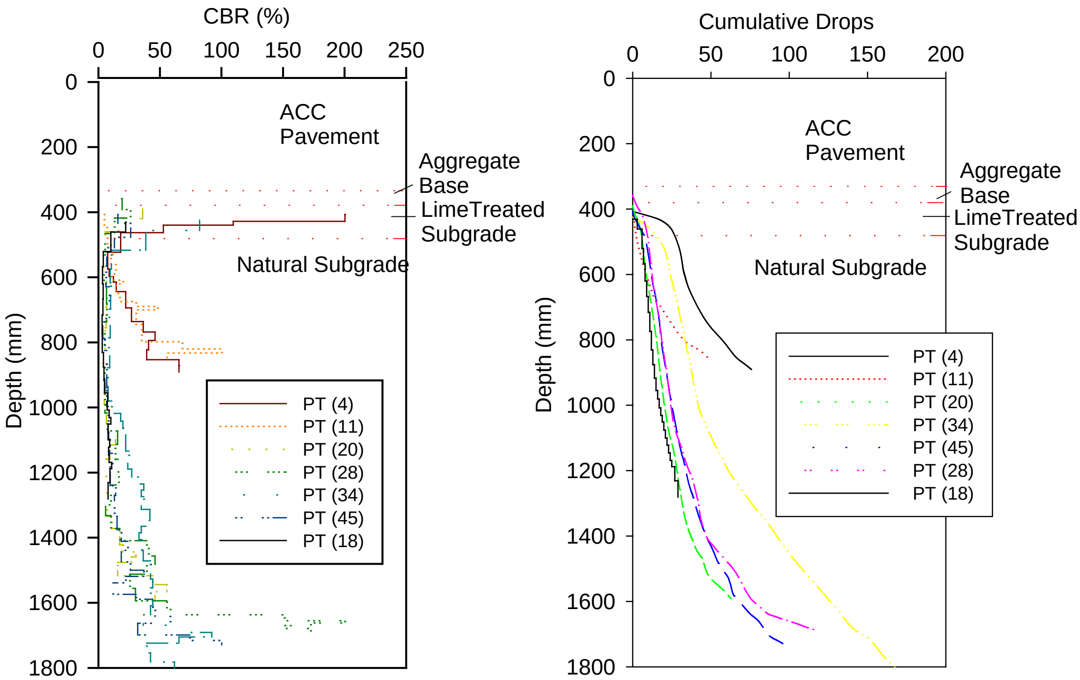


Figure 107. CBR – DCP profile of test points US 75 Osage County, Kansas

Table 31 lists all LWD test results. Table 32 provides the elastic modulus ratio between stabilized and natural subgrade. The mean value, standard deviation, and coefficient of variation of in situ test results were listed in Table 33. All in situ test results are presented in Appendix C.

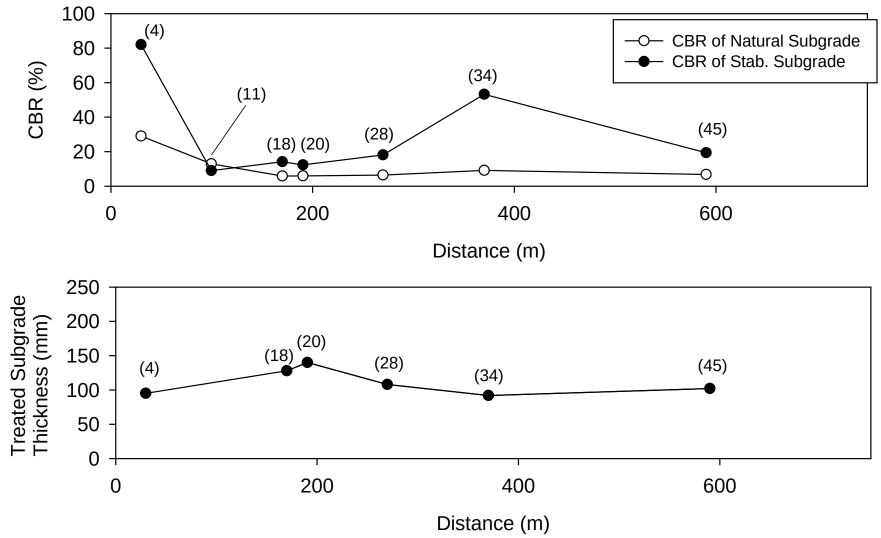


Figure 108. CBR of subgrade and stabilized subgrade thickness from DCP profile US 75 Osage County, Kansas

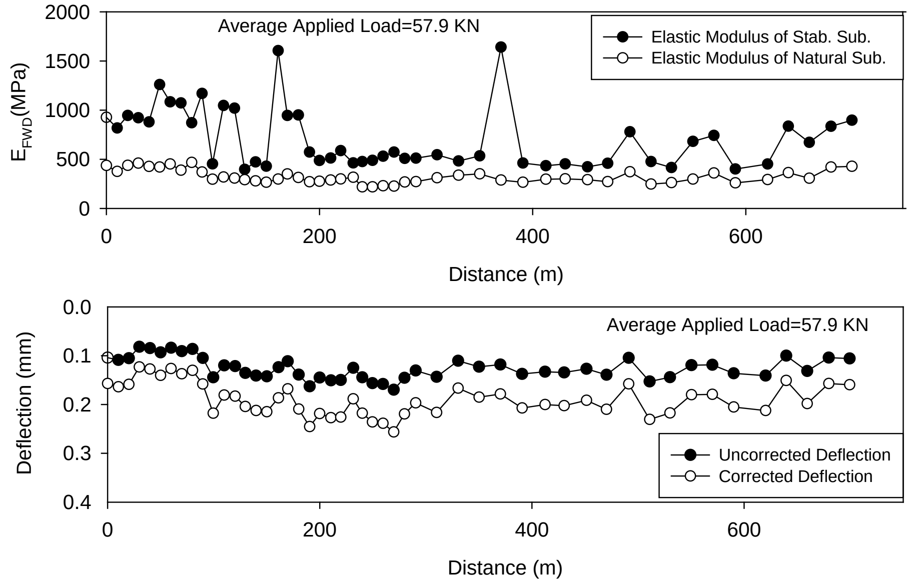


Figure 109. Back-calculated FWD elastic modulus of stabilized and natural subgrade, and deflections under the loading plate US 75 Osage County, Kansas

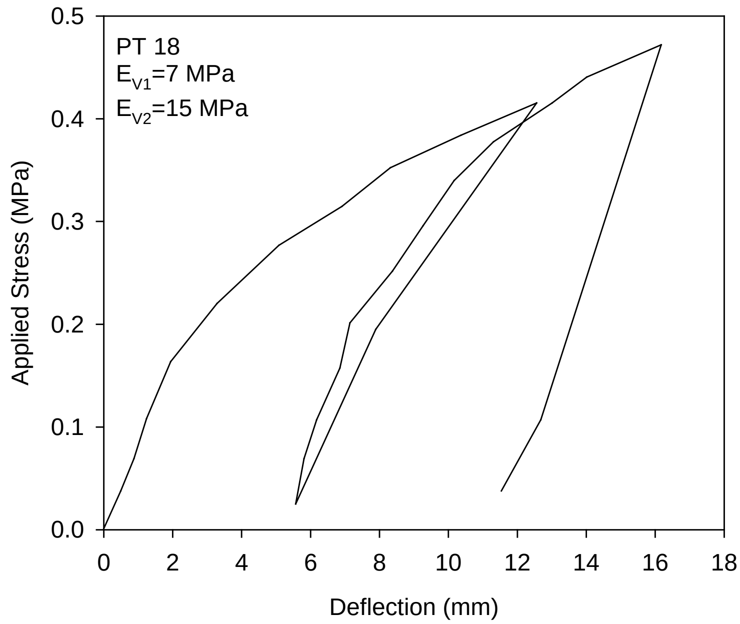


Figure 110. Corrected stress – deflection curves from plate load test at point 18 US 75 Osage County, Kansas

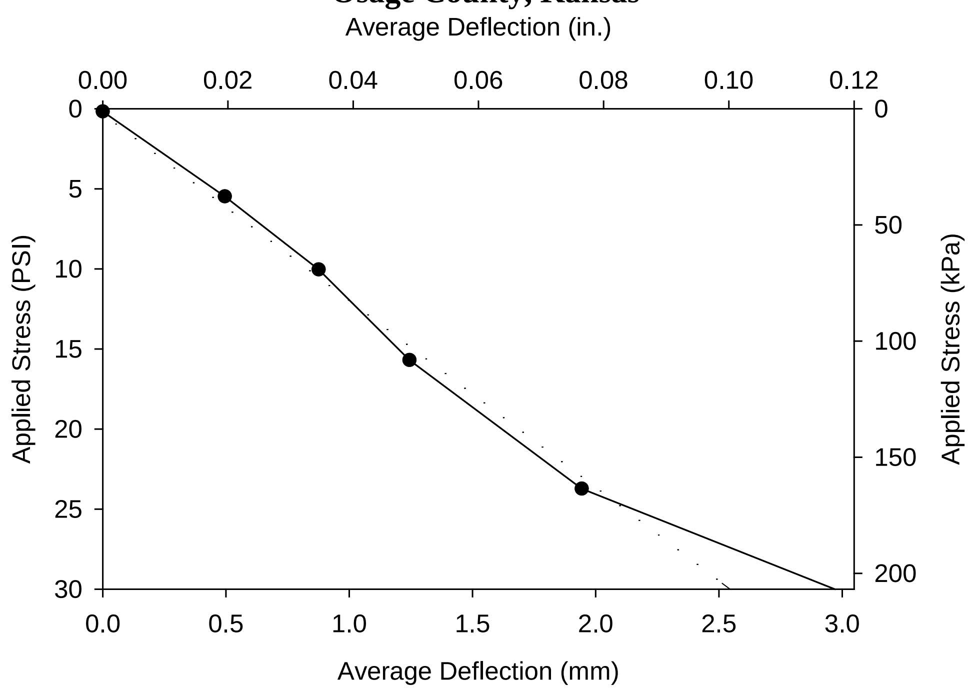


Figure 111. Stress – deflection curves for obtaining K_U at point 18 US 75 Osage County, Kansas

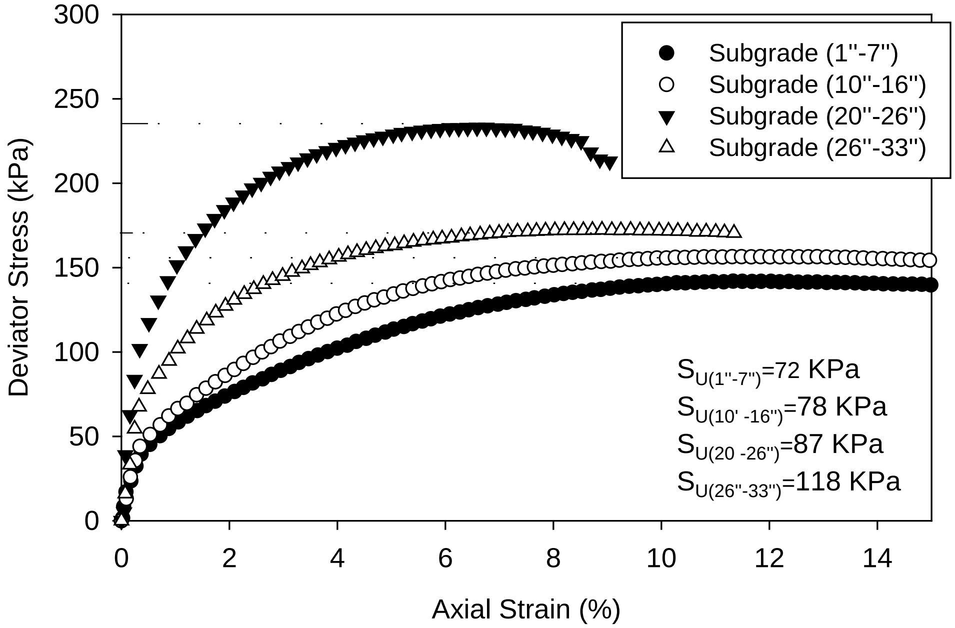


Figure 112. Unconsolidated – Undrained test of subgrade US 75 Osage County, Kansas

Table 31. Summary of LWD test results US 75 Osage County, Kansas

Test Point	**Material Type**	**Depth of Measurement**	**E_{LWD}**	**Average E_{LWD}**
			MPa	MPa
18	Stabilized subgrade	Top of stabilized subgrade	37	31
18	Stabilized subgrade	50 mm from top of stabilized subgrade	24	

Table 32. Summary of elastic modulus ratio between stabilized and natural subgrade US 75 Osage County, Kansas

Stab. Subg./Nat. Subg. Ratio	
CBR	E_{FWD}
2.7	2.2

Table 33. Summary statistics of test results from in situ testing US 75 Osage County, Kansas

Statistic	Stabilized Subgrade							Natural Subgrade		FWD Def
Measurement	CBR	E_{FWD}	E_{LWD}	E_{V1}	E_{V2}	k_U	Thi.	CBR	E_{FWD}	$D_{0\text{-Cor.}}$
	%	MPa	MPa	MPa	MPa	kPa/mm	mm	%	MPa	mm
Number of Measurement(n)	7	50	2	1	1	1	6	7	50	50
Mean Value (μ)	30	711	31	7	15	31	111	11	323	0.19
Standard Deviation (σ)	28	304	18	—	—	—	19	8	68	0.03
Coefficient of Variation COV (%)	93	43	48	—	—	—	17	73	21	16

4.8 US 75 NB, KS

4.8.1 Site Description

This project was located US 75 NB north of Hoyt, in Jackson County, Kansas. The general location of this site is shown in Figure 113. This road is a four-lane U.S. Highway. The road was constructed in 1995. The pavement consists of a nominal 229 mm (9 in.) thick Portland cement concrete (PCC), 102 mm (4 in.) cement stabilized aggregate base, and 152 mm (6 in.) lime stabilized subgrade. The length of this test section is approximately 220 m (721 ft). The ISU research team conducted in situ testing near milepost 176 on November 3, 2010 with assistance and traffic control provided by Kansas DOT.

The plan view of in situ test locations is shown in Figure 114. The research team preformed FWD tests on the surface of the PCC pavement at the center and joints of each slabs. DCP tests were conducted at test points 3, 11, 31, 43, 49, and 51. After coring, LWD and PLT test were performed at the top of stabilized subgrade at test point 25. Bag samples of subgrade were collected at test point 25. Natural subgrade samples were at test point 51. Undisturbed Shelby tube samples were collected at test point 25 from the top of the stabilized subgrade to a depth of about 330 mm (13 in.).

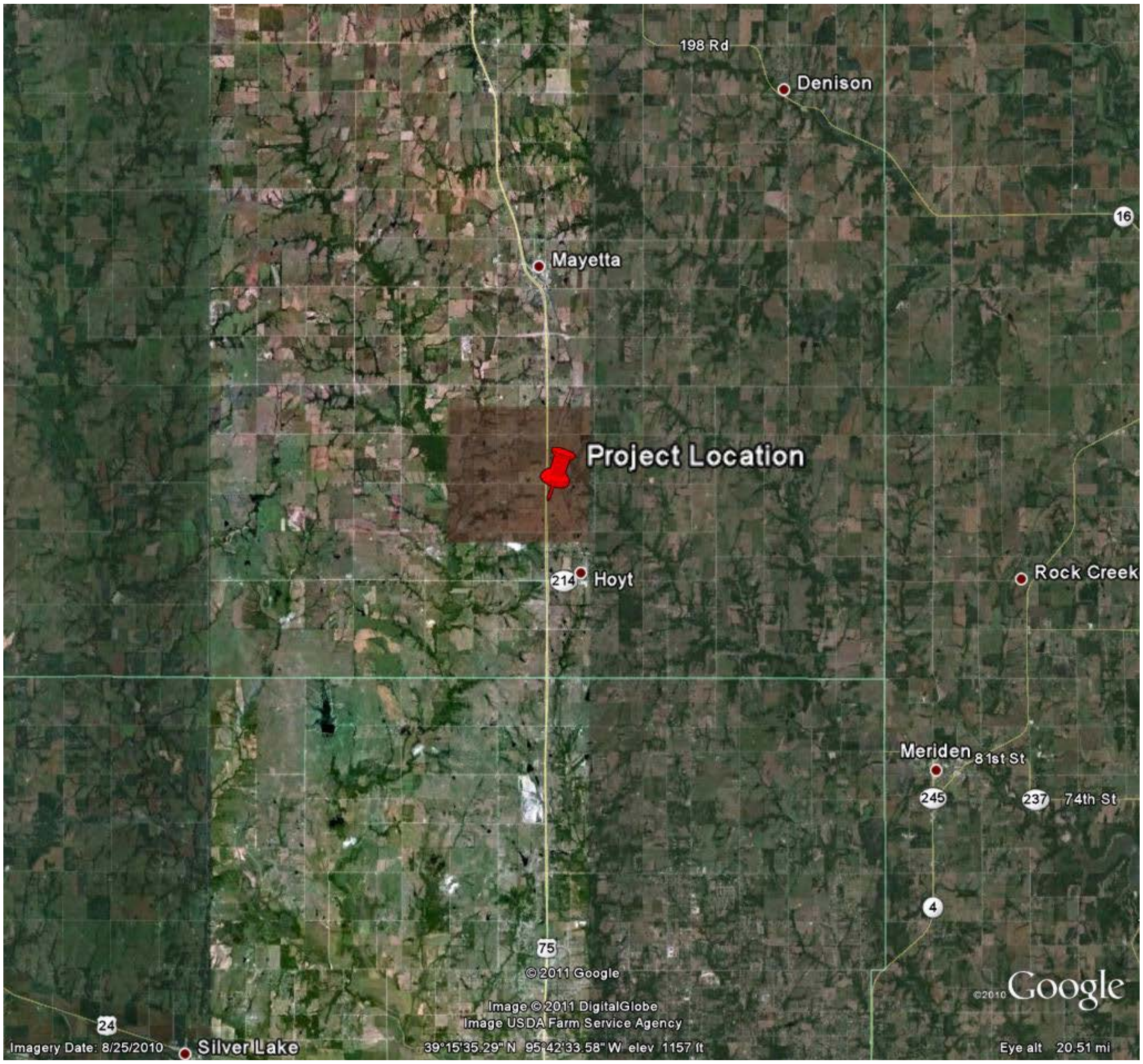


Figure 113. Project location of US 75 NB Jackson County, Kansas

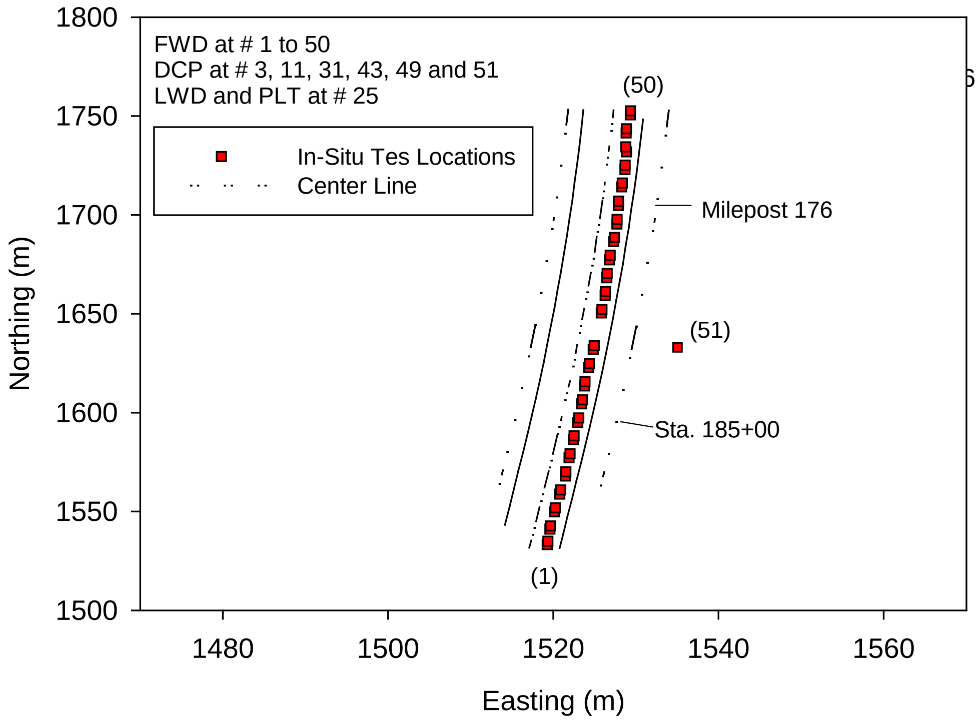


Figure 114. Test section plan layout with RTK GPS test locations US 75 NB Jackson County, Kansas

Figure 115. Site overview US 75 NB Jackson County, Kansas

4.8.2 Test Results and Analysis

4.8.2.1 Material Properties of Subgrade

Table 34 provides a summary of the material index values. The natural subgrade was classified as ML and A-4, and the stabilized subgrade as SM and A-2-4 for the top 50 mm (2 in.) and A-2 from 50 mm to 150 mm (2-6 in.). The average sand content of the stabilized subgrade increased from about 29% to 59% compared to the natural subgrade. The average clay content decreased from about 33% to 6%, while the average silt content decreased from about 36% to about 28%. The average LL was 45 for the stabilized subgrade and 52 for natural subgrade. The average PI was 8 for stabilized subgrade and 34 for natural subgrade. The average moisture content was about 28% for the stabilized subgrade and 19% for the natural subgrade. Figure 116 shows the particle size distribution curves of different subgrade layers.

Table 34. Summary of material properties US 75 NB Jackson County, Kansas

Parameter	Materials				
Material Description	Natural Sub.	Base	Stab. Sub.	Stab. Sub.	Stab. Sub.
Depth mm (in.)	—	0-100 (0-4)	0-50 (0-2)	50-100 (2-4)	100-150 (4-6)
Gravel (%) (> 4.75mm)	2.6	60.4	8.5	4.9	7.1
Sand (%) (4.75mm – 75µm)	28.5	33.3	64.3	56.5	54.9
Silt (%) (75µm – 2µm)	36.3	5.1	19.5	33.4	32.3
Clay (%) (< 2µm)	32.6	1.2	7.7	5.2	5.7
C_u	—	20.4	165.3	65.3	67.8
C_c	—	3.0	6.4	0.6	0.5
Liquid Limit, LL (%)	52.0	—	44.0	45.3	45.8
Plasticity Index, PI	34.3	—	5.9	8.9	8.6
AASHTO	A-4	A-1-a	A-2-4	A-4	A-4
USCS	ML	GW-GM	SM	SM	SM
Water Content (%)	18.7	10.4	27.0	29.0	28.3

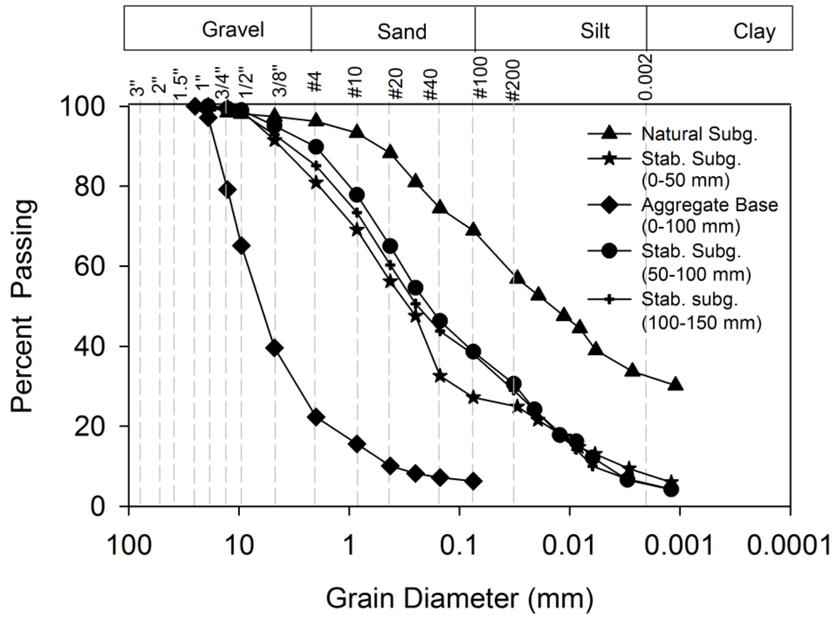


Figure 116. Particle size distribution curves for subgrade materials US 75 NB Jackson County, Kansas

4.8.2.2 pH of Stabilized and Natural Subgrade

Figure 117 shows the pH profile of subgrade materials at test point 25. The pH values of stabilized subgrade ranged from 8.7 to 9.4. pH gradually decreased from the top of the stabilized subgrade to the bottom of the stabilized subgrade. pH of natural subgrade ranged from 7.9 to 8.1.

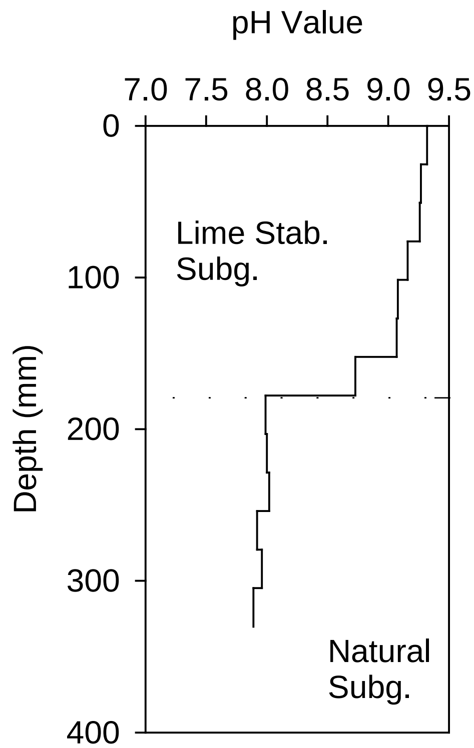


Figure 117. pH profile of subgrade US 75 NB Jackson County, Kansas

4.8.2.3 SEM Analysis

The energy dispersive spectrometry (EDS) map of natural subgrade is shown in Figure 118. The majority elements identified include silica (Si), alumina (Al), and oxygen (O). Additional elements present were potassium (K) and magnesium (Mg). EDS map of stabilized subgrade is shown in Figure 119. The majority elements identified includes Si, Al, K, and O. Calcium (Ca) was identified in a few regions of intense concentration in the sample. Iron (Fe), and Mg were also identified in the stabilized sample.

Figure 120 and Figure 121 compare elemental concentrations of Al, Si, O, S, Mg, Ca, K, and C for stabilized and natural subgrade. The stabilized subgrade sample shows higher concentrations of Ca and Fe than the natural subgrade. All SEM images are presented in Figure 122, Figure 123, and Appendix D.

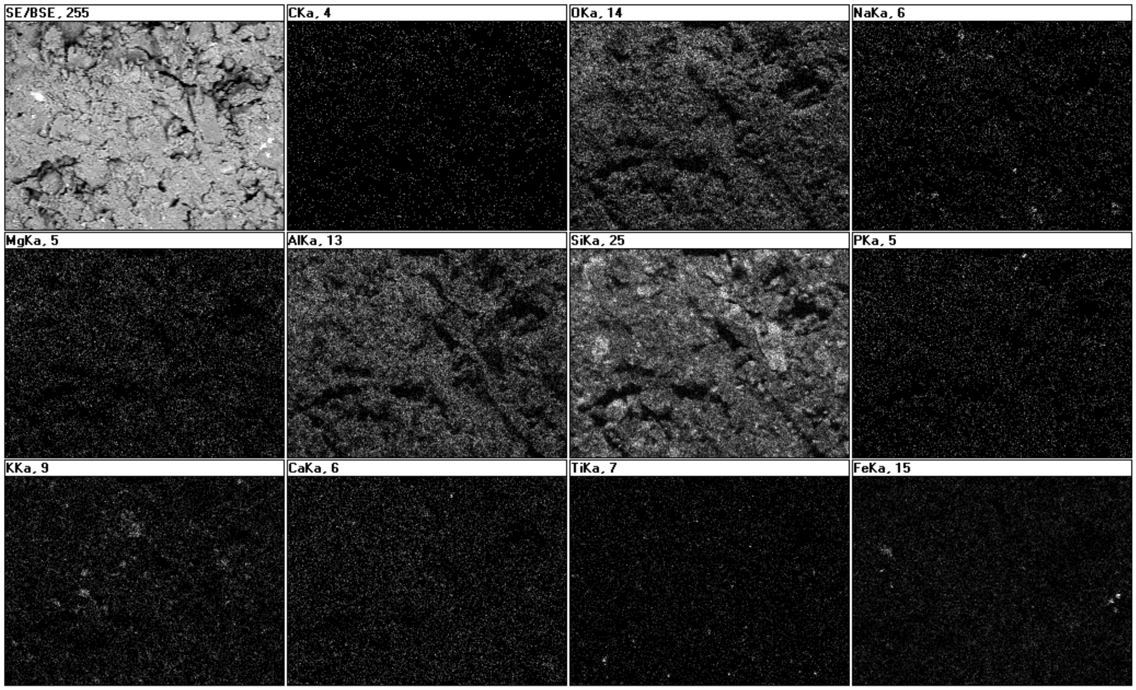


Figure 118. EDS map of natural subgrade sample (500 ×) US 75 NB Jackson County, Kansas

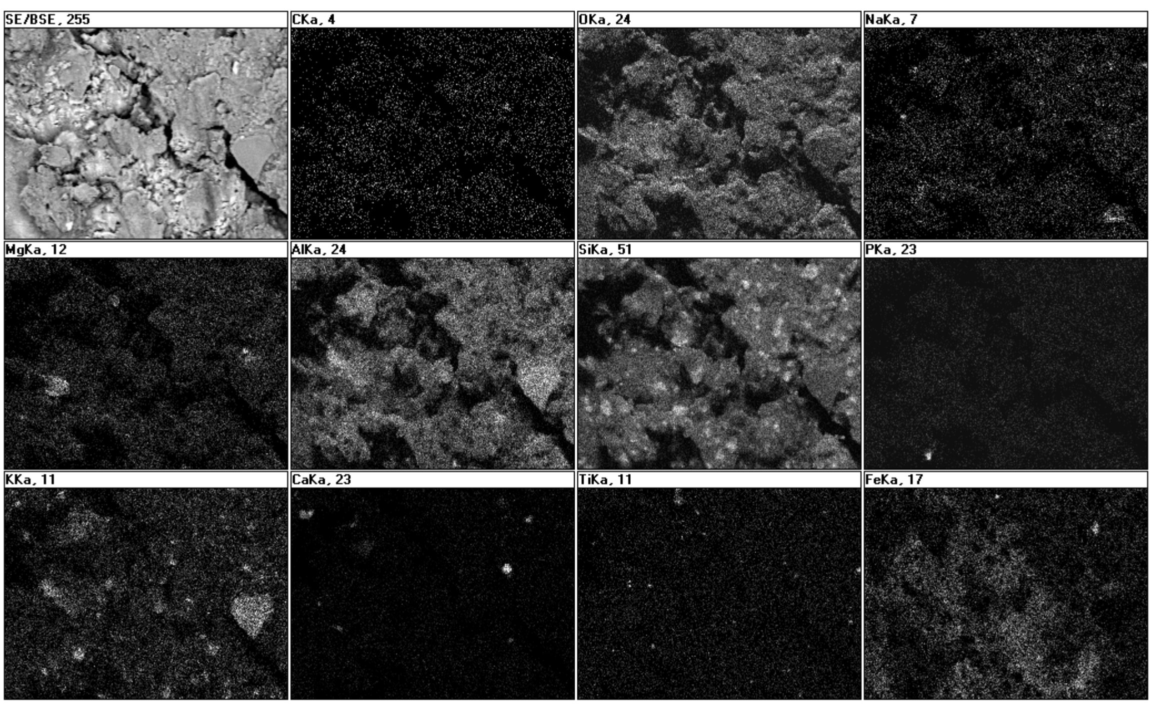


Figure 119. EDS map of stabilized subgrade sample (250 ×) US 75 NB Jackson County, Kansas

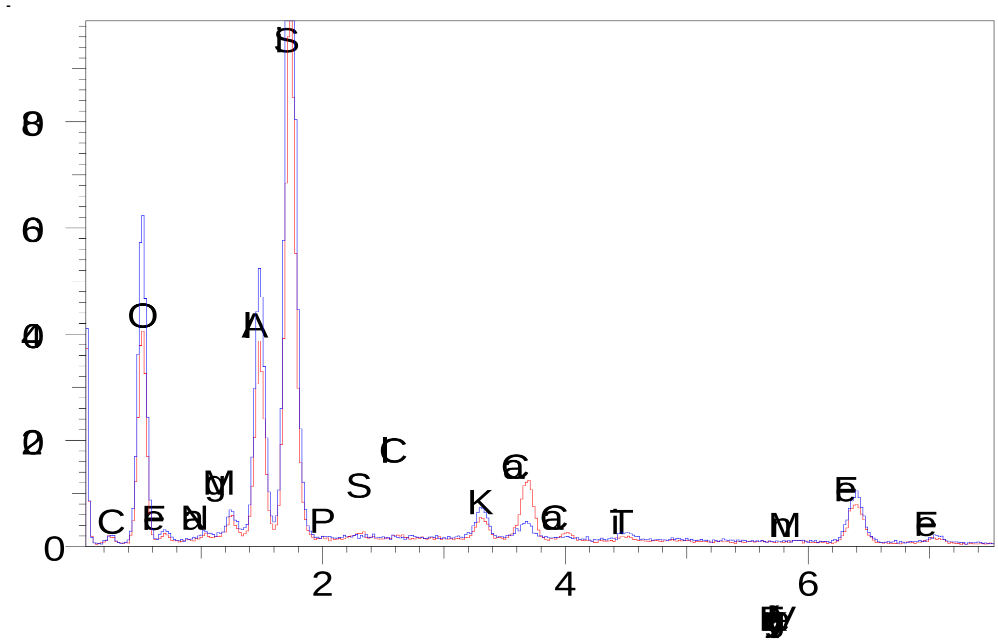


Figure 120. EDS intensity counts for stabilized subgrade sample (red line: 30×) and natural subgrade sample (blue line: 30×) US 75 NB Jackson County, Kansas

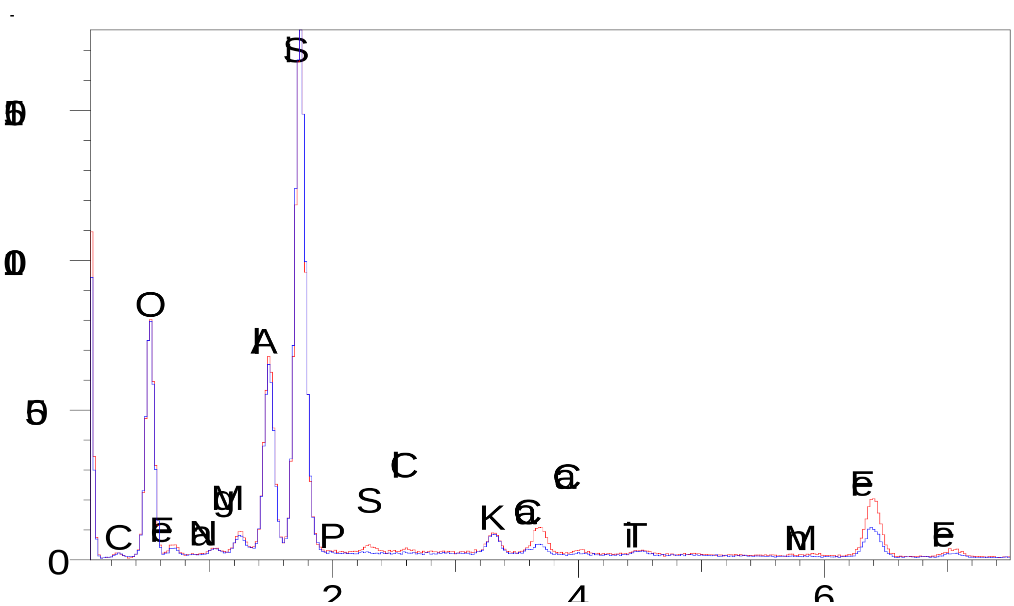


Figure 121. EDS intensity counts for stabilized subgrade sample (red line: 150×) and natural subgrade sample (blue line: 150×) US 75 NB Jackson County, Kansas

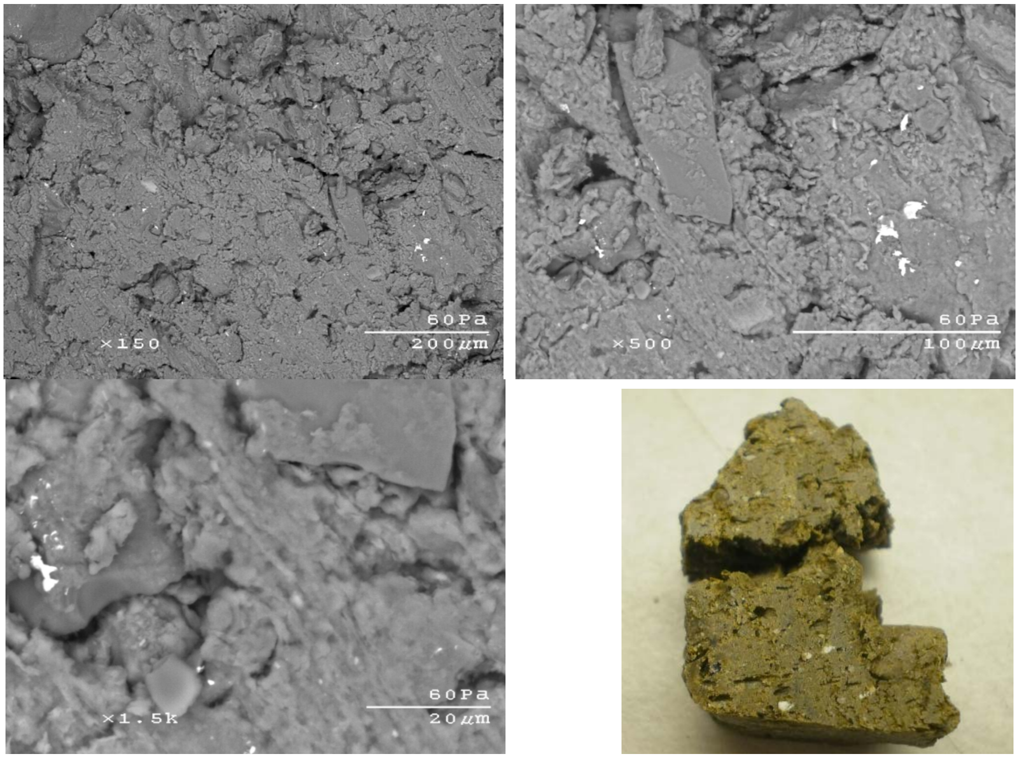


Figure 122. SEM images of natural subgrade US 75 NB Jackson County, Kansas

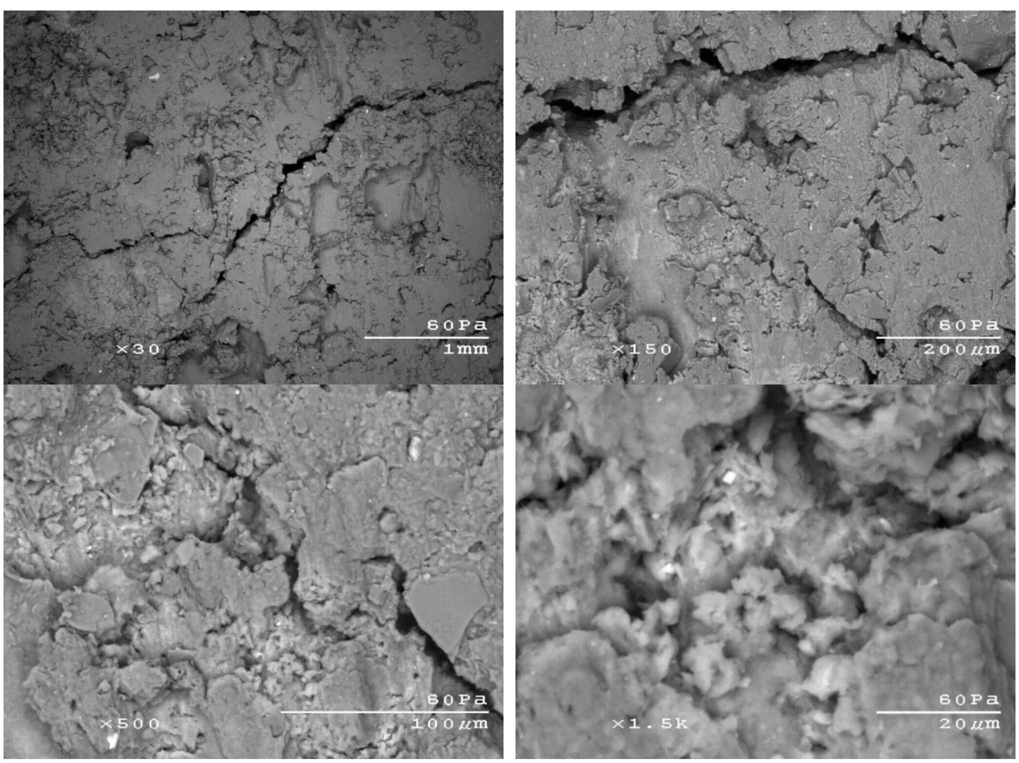


Figure 123. SEM images of stabilized subgrade US 75 NB Jackson County, Kansas

4.8.2.4 Stiffness and Strength

DCP profiles and cumulative drops versus CBR are shown in Figure 124. The average CBR of both natural and stabilized subgrade, and effective stabilized subgrade thickness are shown in Figure 125. The major observations from DCP testing include: (1) based on the effective treatment thickness, the average CBR of the stabilized subgrade was 20, (2) the average CBR of the natural subgrade was 7, (3) the average CBR of the stabilized subgrade was 2.9 times higher than the natural subgrade, and (4) the actual average treatment thickness was about 128 mm (5 in.).

ERIDA assumes a two layers system for PCC pavement to calculate composite subgrade moduli (E_{sg}) and PCC pavement (E_{PCC}). Figure 126 shows subgrade moduli (E_{sg}) and deflection.

Figure 127 presents the PLT stress-deflection relationship at point 25. The values of E_{V1} and E_{V2} were calculated in the first circle and after reloading. The uncorrected modulus of soil reaction k′u was calculated using deflection under a load of 69.0 kPa as shown in Figure 128. The correction of k′u was made using the curve in Figure 128. The average LWD elastic modulus (E_{LWD}) was 25 MPa for stabilized subgrade and 15 MPa for natural subgrade. The average E_{LWD} of stabilized subgrade was equal to 0.3 E_{V1} and 0.2 E_{V2}.

Table 35 lists all LWD test results. Table 36 provides the elastic modulus ratio between stabilized and natural subgrade. The mean value, standard deviation, and coefficient of variation of in situ test results were listed in Table 37. All in situ test results are presented in Appendix C.

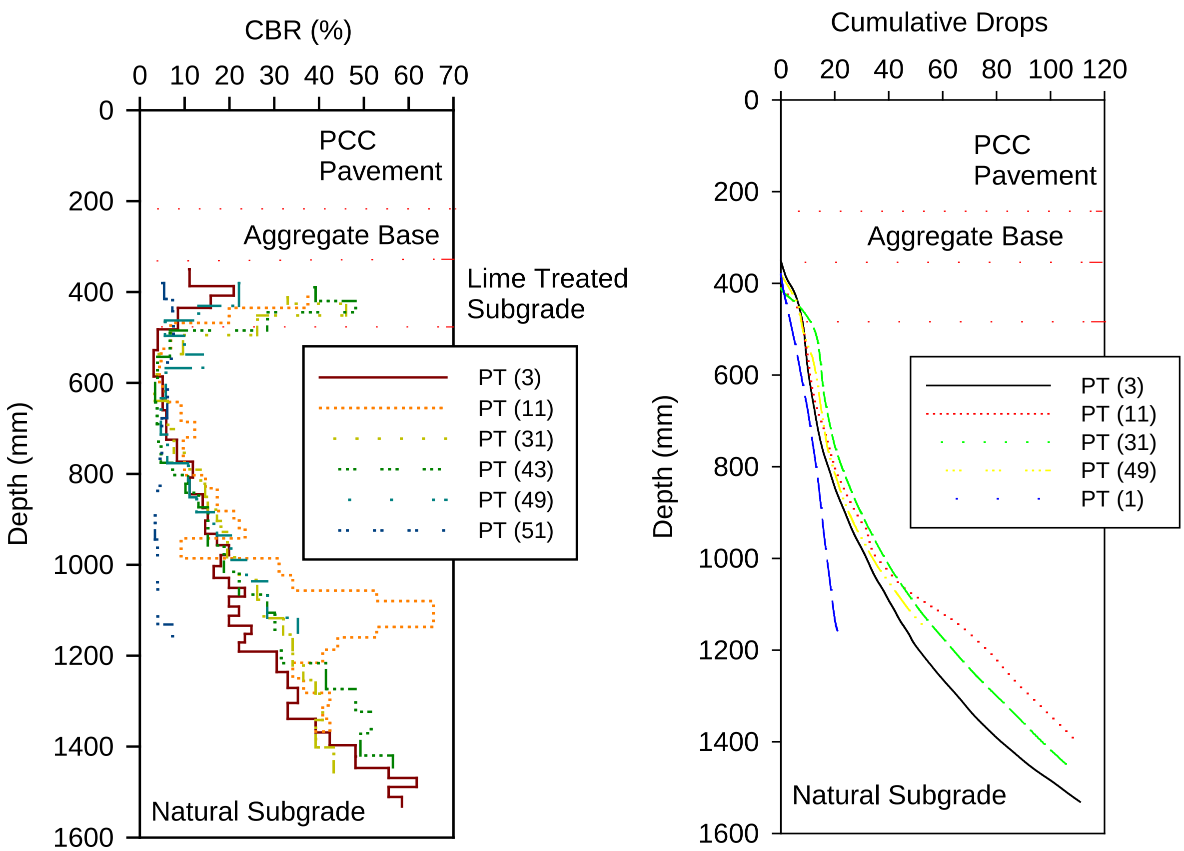


Figure 124. CBR – DCP profile and cumulative drops versus CBR of test points US 75 NB Jackson County, Kansas

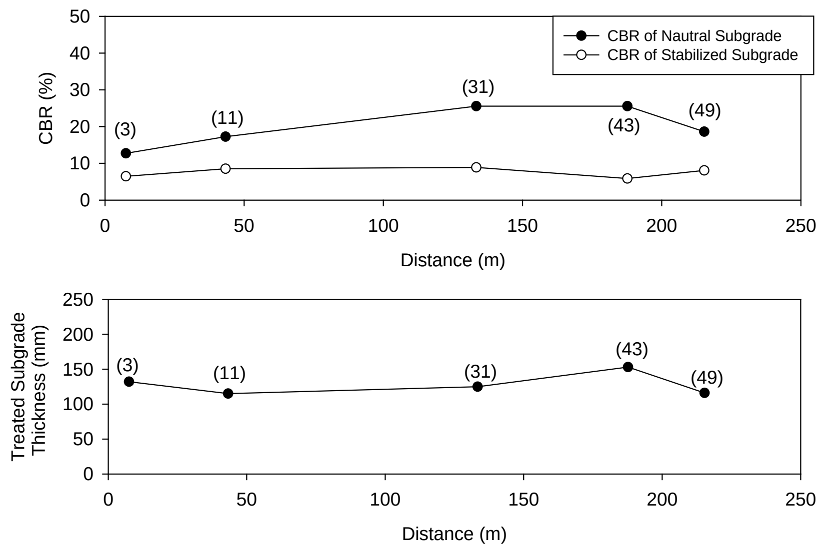


Figure 125. CBR and stabilized subgrade thickness from DCP profile

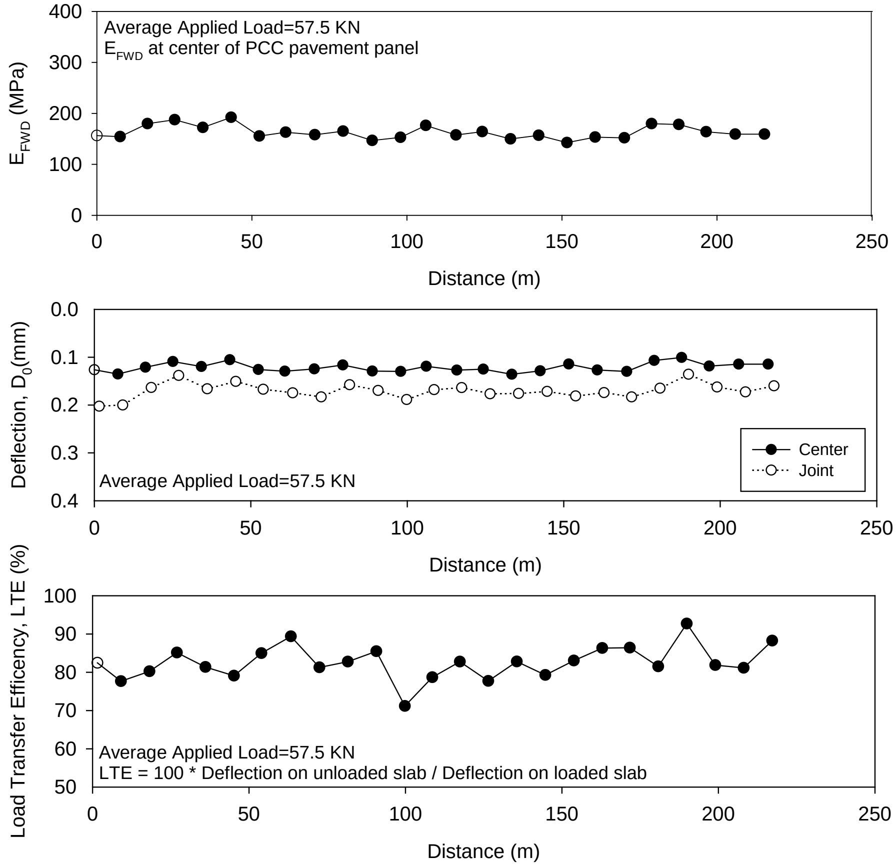


Figure 126. Back-calculated FWD elastic modulus of stabilized subgrade, deflections under the loading plate and load transfer efficiency at joints US 75 NB Jackson County, Kansas

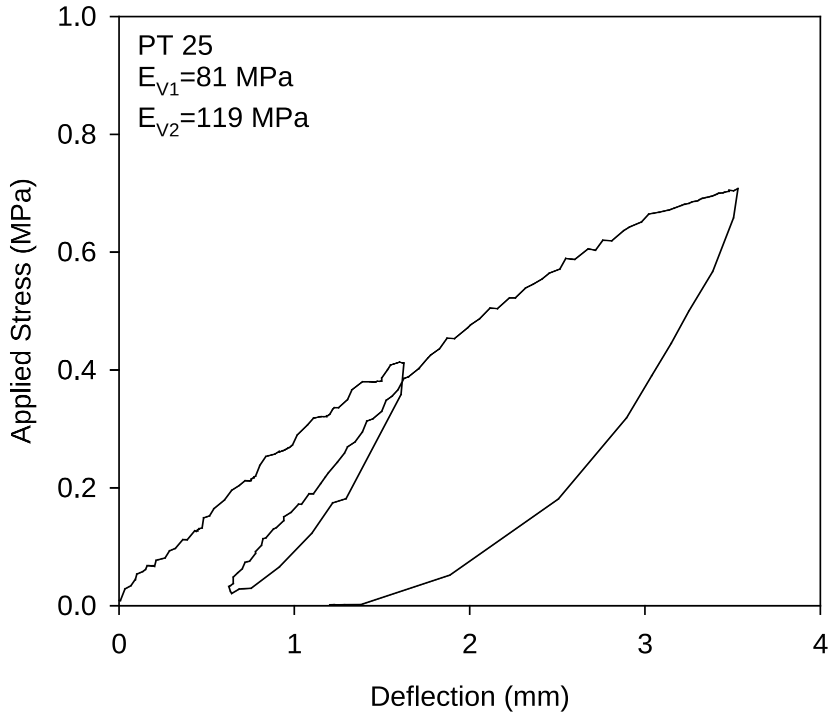


Figure 127. Stress – deflection curves from plate load test at point 25 US 75 NB Jackson County, Kansas

Table 35. Summary of LWD test results

Test Point	**Material Type**	**Depth of Measurement**	**E_{LWD}**	**Average E_{LWD}**
			MPa	MPa
25	Base	Top of base	81	81
25	Stabilized subgrade	Top of stabilized subgrade	91	91
51	Natural Subgrade	Top of natural subgrade	15	15

Table 36. Summary of elastic modulus ratio between stabilized and natural subgrade US 75 NB Jackson County, Kansas

Stab. Subg./Nat. Subg. Ratio	
CBR	E_{LWD}
2.9	6.1

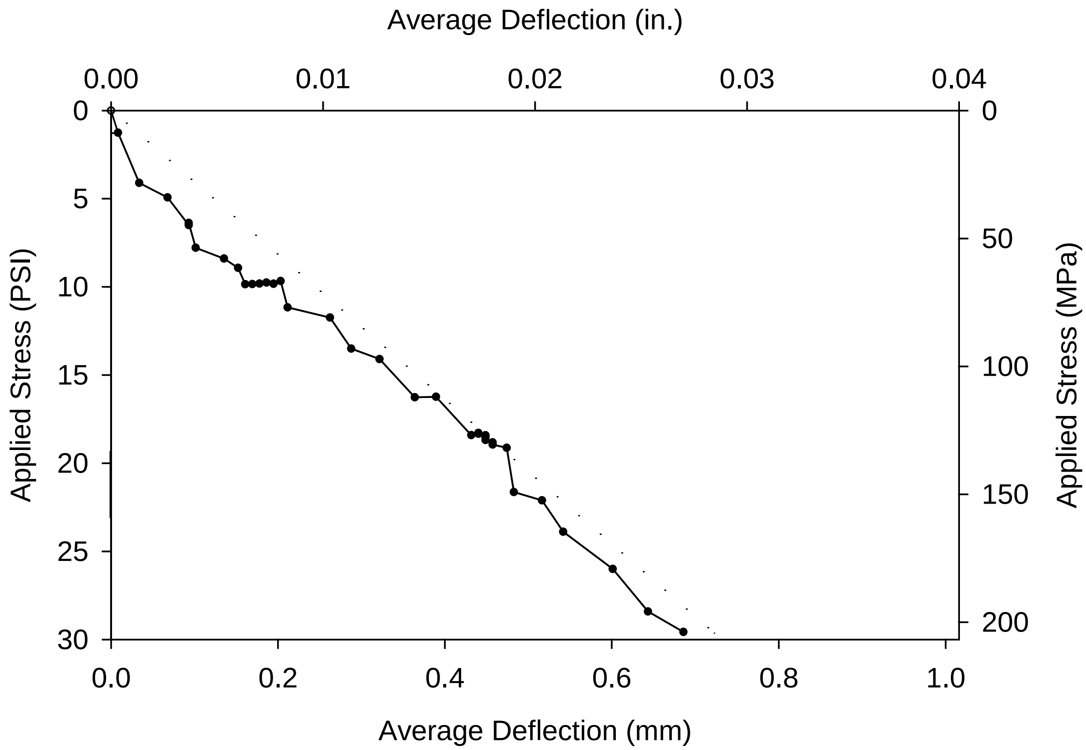


Figure 128. Stress – deflection curves for obtaining Ku at point 25 US 75 NB Jackson County, Kansas

Table 37. Summary statistics of test results from in situ testing

Statistic	**Base**	**Stabilized Subgrade**						**Natural Subgrade**		**FWD Def.**
Measurement	E_{LWD}	CBR	E_{LWD}	k_U	E_{V1}	E_{V2}	Thi.	CBR	E_{LWD}	D_0
	MPa	%	MPa	kPa/mm	MPa	MPa	mm	%	MPa	mm
Number of Measurement (n)	1	5	1	1	1	1	5	6	1	50
Mean Value (μ)	81	20	91	103	81	119	128	7	15	0.15
Standard Deviation (σ)	—	6	—	—	—	—	16	2	—	0.03
Coefficient of Variation COV (%)	—	30	—	—	—	—	13	29	—	20

4.9 K 7, KS

4.9.1 Site Description

This project was located on K 7 south of Doniphan, in Doniphan County, Kansas. The general location is shown in Figure 129. This road is a two-lane State Highway. The road was constructed in 2005. The length of this test section is approximately 515 m (1690 ft). The design pavement consists of a 229 mm (9 in.) thick asphalt concrete (AC) and 300 mm (12 in.) fly ash stabilized subgrade. No base layer was presented between the stabilized subgrade and ACC pavement. The subgrade was reportedly stabilized with 14 to18% fly ash. The ISU research team conducted in situ testing near milepost 223 on November 4, 2010 with assistance and traffic control provided by Kansas DOT.

The plan view of in situ test locations is shown in Figure 130. The research team preformed FWD tests on the surface of the ACC pavement at intervals of about 10 to 20 m from test points 1 to 31. DCP tests were conducted at test points 1, 3, 16, 29, 32, and 33. After coring, LWD and PLT tests were performed at the top of stabilized subgrade at test point 11. Bag samples were collected at test point 32 for natural soil and at test point 11 for stabilized subgrade.

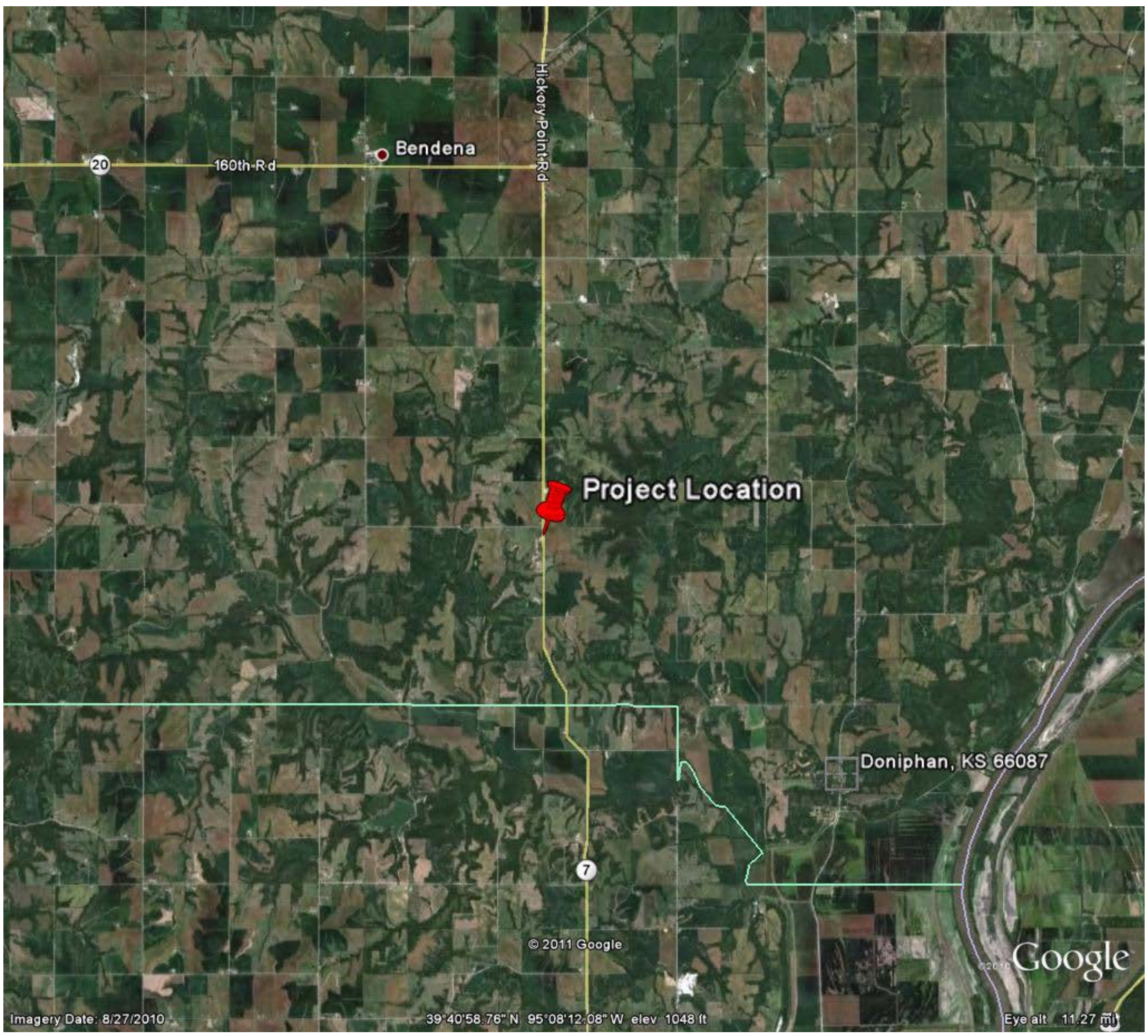


Figure 129. Project location of K 7 NB Doniphan County, Kansas

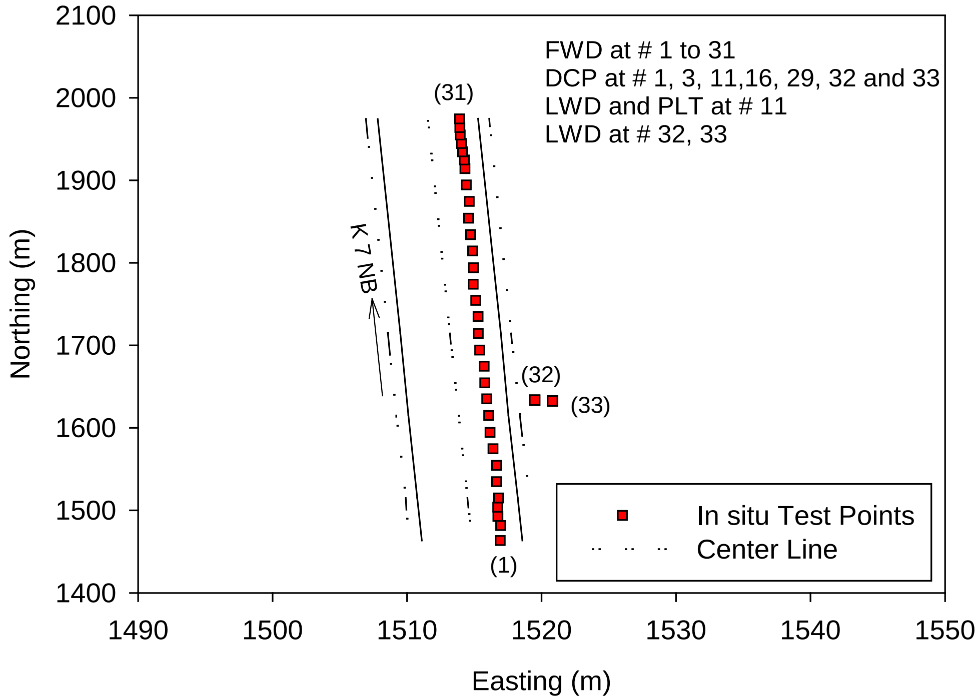


Figure 130. Test section plan layout with RTK GPS test locations K 7 NB Doniphan County, Kansas

Figure 131. Site overview K 7 NB Doniphan County, Kansas

4.9.2 Test Results and Analysis

4.9.2.1 Material Properties of Subgrade

Table 38 provides material index values for the subgrade materials. The stabilized subgrade samples were taken at test point 11 from the top of subgrade to a depth of 300 mm (12 in.) at intervals of 50 mm (2 in.). The natural subgrade was collected at test point 32. The natural subgrade was classified as ML and A-4, and the stabilized subgrade was classified as SM and A-2-4, except A-1-b for the stabilized subgrade from a depth of 51 mm to 102 mm. The average sand content of the stabilized subgrade increased from about 5 to 42% compared to the natural subgrade. Further, the average clay content decreased from about 20% for natural subgrade to 2% for stabilized subgrade, while the silt content decreased from about 74% to about 23%. The average LL value decreased from 38 for natural subgrade to 23 for stabilized subgrade, while the average PI decreased from 18 to 5. Figure 132 shows the particle size distribution curves. Test results show the soil type of subgrade has been modified after treatment.

Table 38. Summary of material properties K 7 NB Doniphan County, Kansas

Parameter	Materials					
Material Description	Natural Sub.	Stab. Sub.	Stab. Sub.	Stab. Sub.	Stab. Sub.	Stab. Sub.
Depth mm (in.)	—	0-51 (0-2)	51-102 (2-4)	101-151 (4-6)	151-203 (6-8)	203-254 (8-10)
Gravel (%) (> 4.75mm)	1.1	26.2	37.4	34.0	23.1	25.8
Sand (%) (4.75mm–75µm)	4.6	46.6	39.8	38.8	49.6	46.0
Silt (%) (75µm–2µm)	74.1	25.2	19.9	24.8	24.9	26.0
Clay (%) (< 2µm)	20.2	2.0	2.9	2.4	2.4	2.2
C_u	—	421.1	574.7	541.6	361.0	425.5
C_c	—	1.3	3.6	1.0	1.3	2.0
Liquid Limit, LL (%)	38.4	23.0	23.2	22.1	22.3	22.1
Plasticity Index, PI	18.3	5.2	5.6	4.4	5.6	4.5
AASHTO	A-4	A-2-4	A-1-b	A-2-4	A-2-4	A-2-4
USCS	ML	SM	SM	SM	SM	SM
Water Content (%)	17.2	9.7	6.7	8.6	8.8	7.9

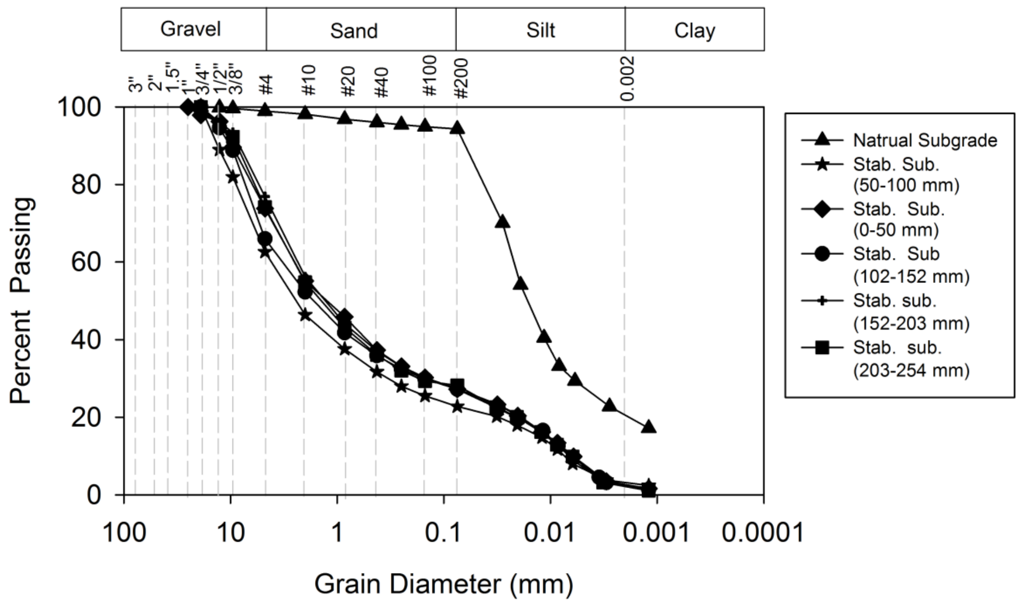


Figure 132. Particle size distribution curves for subgrade K 7 NB Doniphan County, Kansas

4.9.2.2 pH of Stabilized and Natural Subgrade

Figure 133 shows the pH profile of subgrade at test point 11. The pH profile of the stabilized subgrade increased gradually from the top to a depth of 300 mm subgrade. The pH of stabilized subgrade ranged from 7.8 to 8.3. pH of the natural subgrade was 7.4.

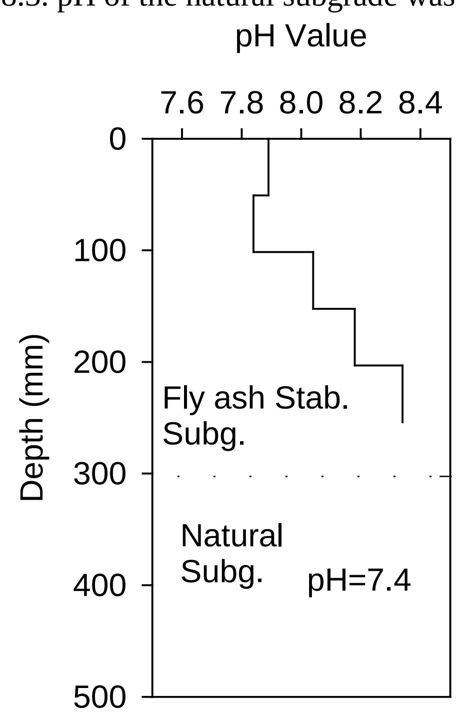


Figure 133. pH profile of subgrade K 7 NB Doniphan County, Kansas

4.9.2.3 SEM Analysis

An energy dispersive spectrometry (EDS) map of natural subgrade is shown in Figure 134. The majority elements identified include silica (Si), alumina (Al), and oxygen (O). Additional elements present were potassium (K), iron (Fe), and calcium (Ca).

The EDS map of stabilized subgrade is shown in Figure 135. The majority elements identified include Ca, Si, Al, and O. Ca was identified in this sample as having a high concentration compared to Al, O, and Si. Additional elements present were Fe, K, and magnesium (Mg).

Figure 136 compares elemental concentrations of Al, Si, O, S, Mg, Ca, K, and C for the stabilized and natural subgrade. The stabilized subgrade sample shows higher concentrations of Ca and C, lower concentrations of O, Al, and Si compared to the natural subgrade sample. All SEM images are presented in Figure 137, Figure 138, and Appendix D.

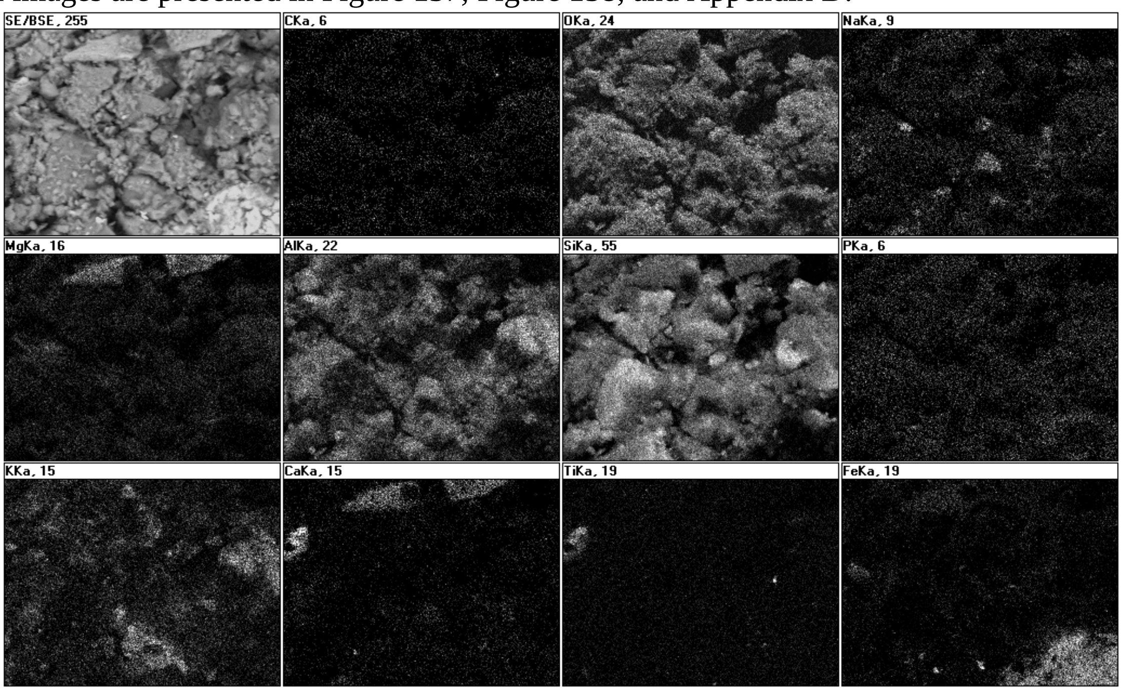


Figure 134. EDS map of natural subgrade sample (1000 ×) K 7 NB Doniphan County, Kansas

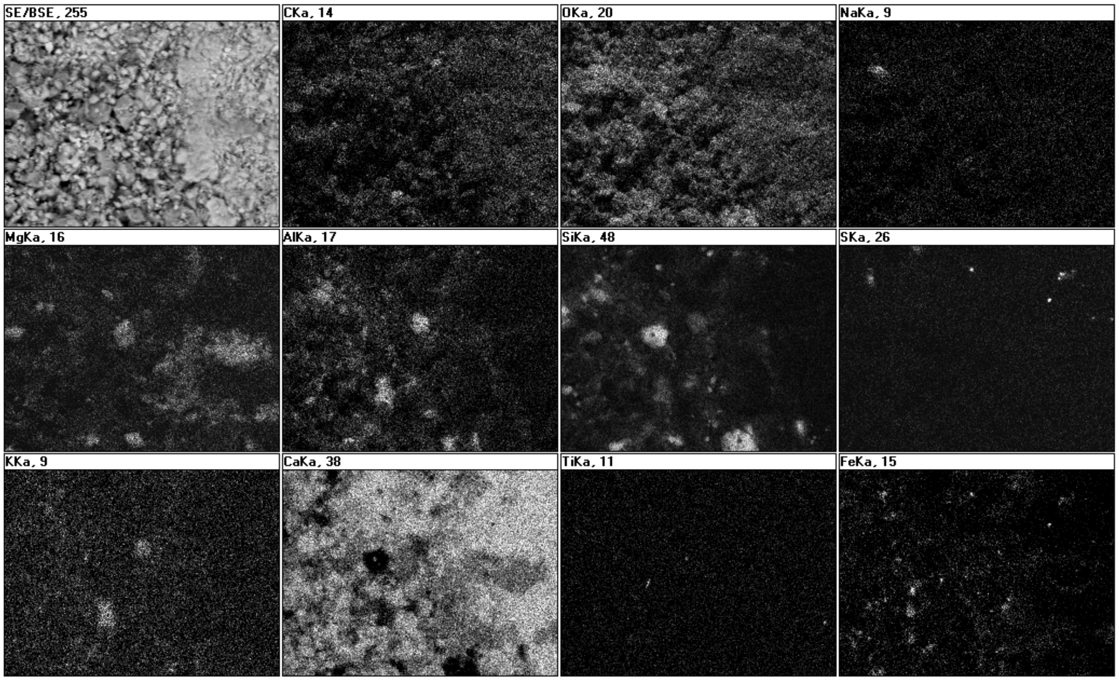


Figure 135. EDS map of stabilized subgrade sample (1000 ×) K 7 NB Doniphan County, Kansas

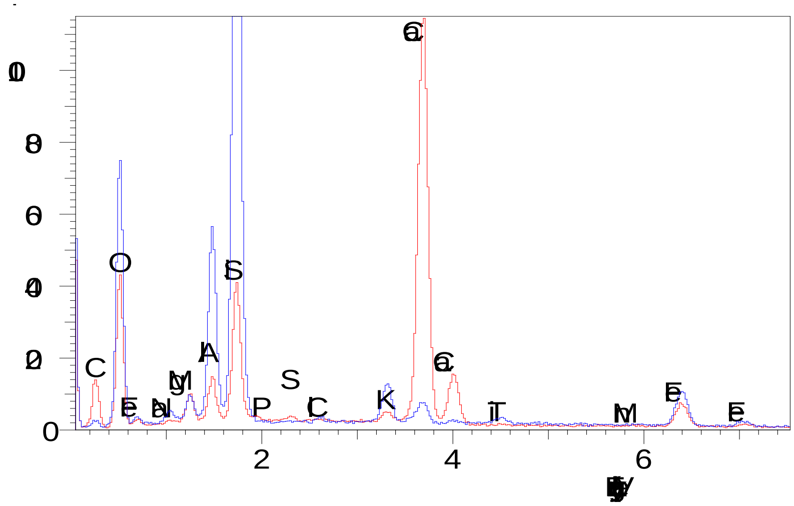


Figure 136. EDS intensity counts for stabilized subgrade sample (red line: 30×) and natural subgrade sample (blue line: 30×) K 7 NB Doniphan County, Kansas

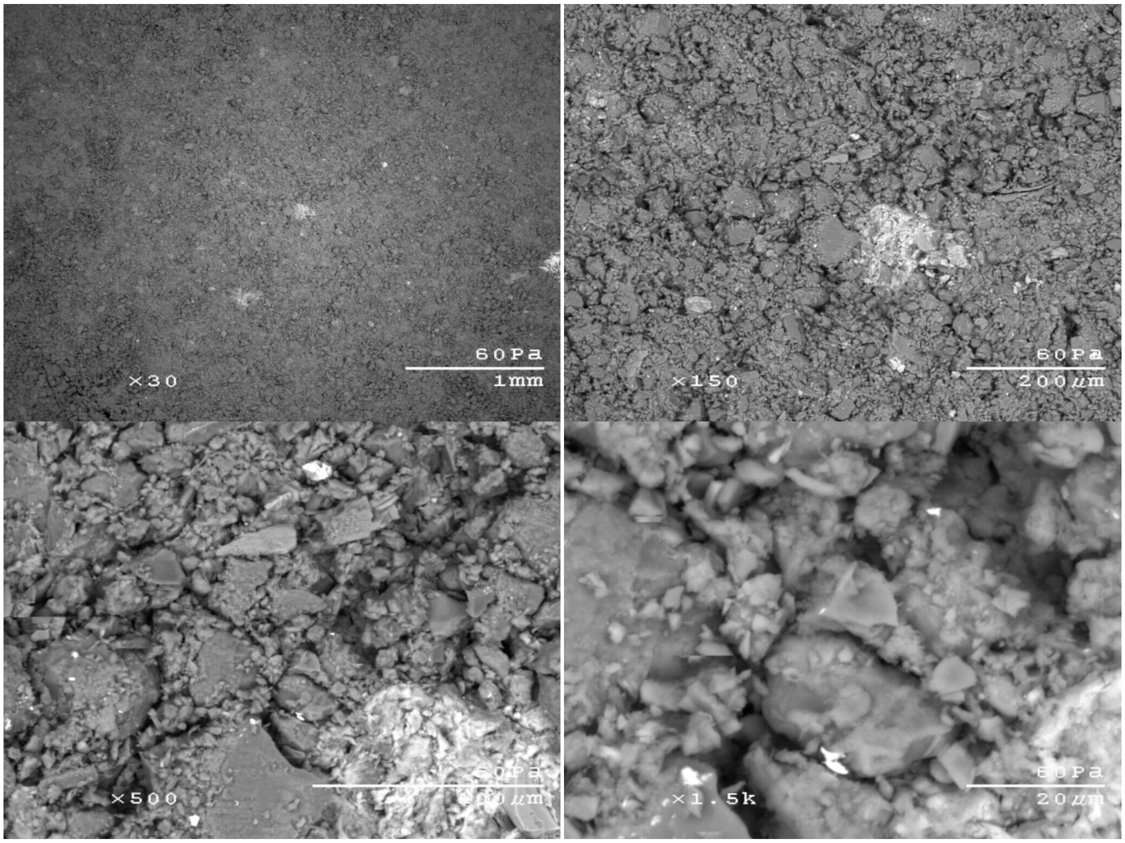


Figure 137. SEM images of natural subgrade K 7 NB Doniphan County, Kansas

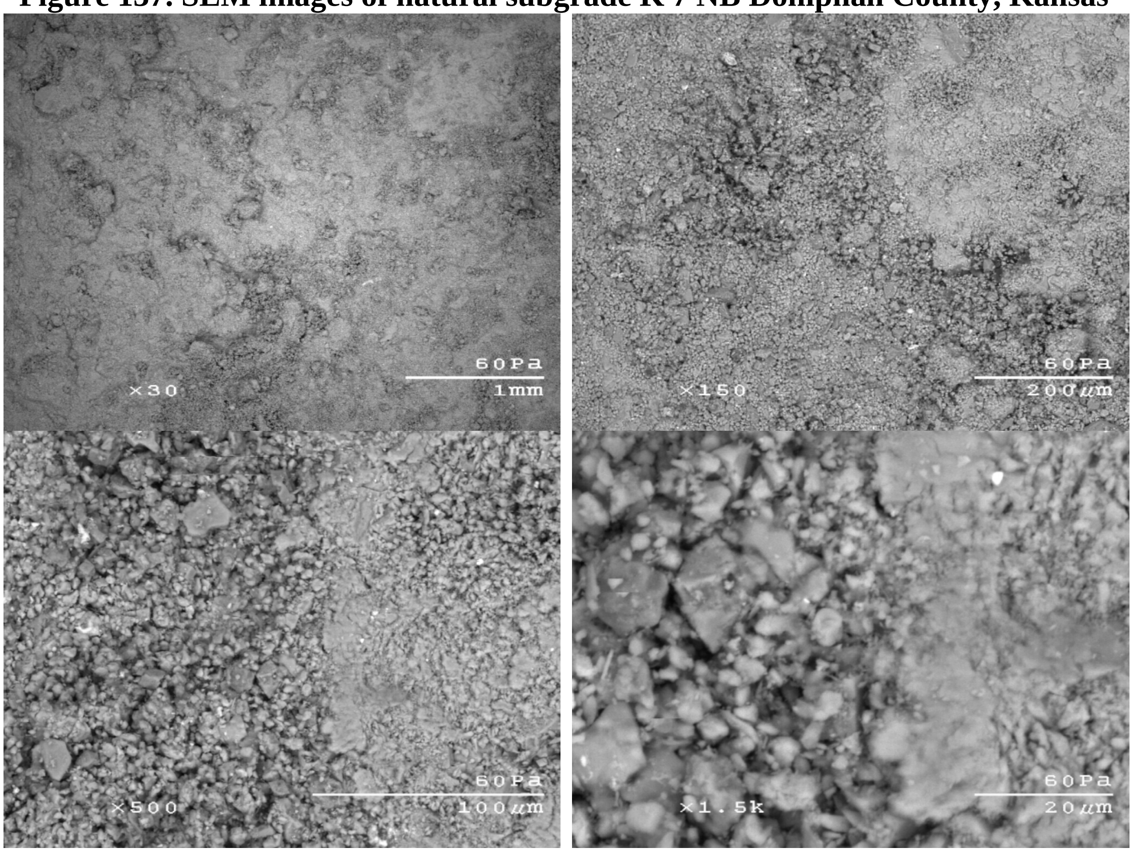


Figure 138. SEM images of stabilized subgrade K 7 NB Doniphan County, Kansas

4.9.2.4 Stiffness and Strength

DCP profiles and cumulative drops versus CBR are shown in Figure 139. The average CBR of both natural and stabilized subgrade, and effective stabilized subgrade thickness are shown in Figure 140. The major observations are: (1) based on the effective treatment thickness, the average CBR of the stabilized subgrade was 72, (2) the average CBR of the natural subgrade was 16, (3) the average CBR of the stabilized subgrade was 4.5 times higher than the natural subgrade, and (4) the top and bottom portions of the stabilized subgrade layer have lower CBR values than the middle of the layer.

Back-calculated subgrade elastic moduli and deflections are presented in Figure 141. An applied test load of 57.5 KN (12928 lb) was used in the FWD back-calculation. Poison's ratio was assumed to be 0.35, 0.40, and 0.40 for ACC surface layer, stabilized subgrade and natural subgrade layer respectively. Stabilized subgrade moduli were calculated based on designed or effective stabilized subgrade thickness obtained from DCP profiles. Detailed assumptions of seed values and layer thickness are summarized in Appendix B. Deflections under the loading plate (D_0) were adjusted to a standard temperature of 20°C (68°F) using Equation (2). The temperature of middle depth of ACC pavement was measured as 9.8°C (49.3°F) prior to FWD testing. The key findings from the FWD testing are as follows: (1) the average uncorrected deflection was about 0.22 mm, and corrected deflection was about 0.34 under average applied load; (2) the average E_{FWD} was 138 MPa for natural subgrade and increased to 503 MPa for stabilized subgrade; (3) the average E_{FWD} of stabilized subgrade was about 3.7 times higher than the natural subgrade; (4) the values of FWD elastic modulus of subgrade varied significantly indicating non-uniform subgrade soil properties.

Figure 142 presents the stress-deflection relationships at test point 11. The value of E_{V1} and E_{V2} were calculated from the first load cycle and after reloading. The uncorrected modulus of soil reaction k′u was calculated using deflection under a load of 69.0 kPa as shown in Figure 143. The average E_{LWD} was increased 6.6 times from 18 MPa for natural subgrade to 118 MPa for stabilized subgrade. The average E_{LWD} of stabilized subgrade was equal to 0.9 E_{V1} and 0.4 E_{V2}.

Table 39 lists all LWD test results. Table 40 provides the elastic modulus ratio between stabilized and natural subgrade. The mean value, standard deviation, and coefficient of variation of in situ test results listed in Table 40. All in situ results are presented in Appendix C.

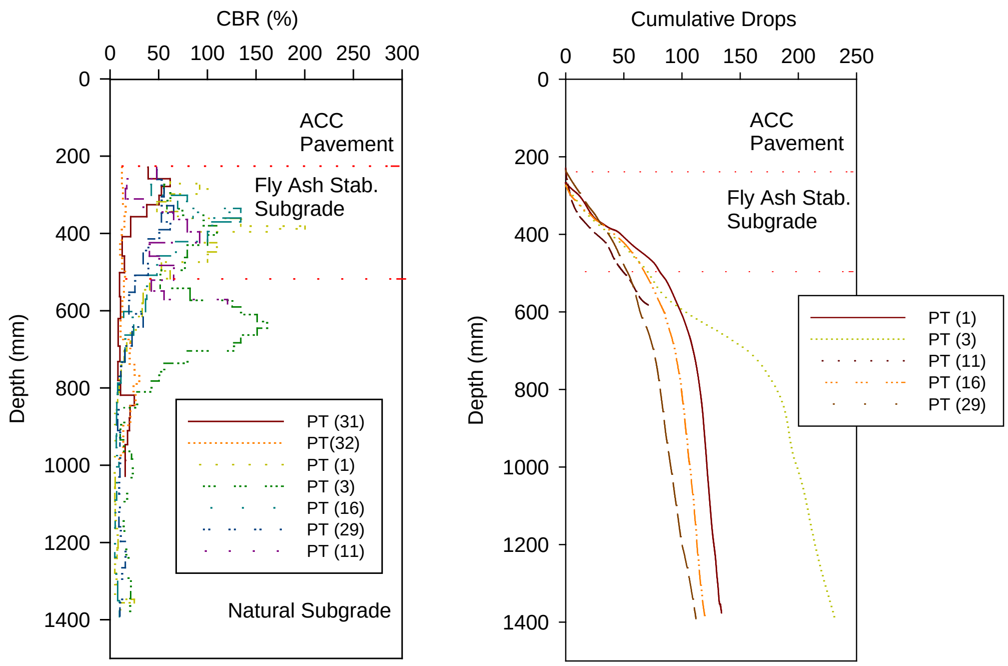


Figure 139. CBR – DCP profile and cumulative drops versus CBR of test points K 7 NB Doniphan County, Kansas

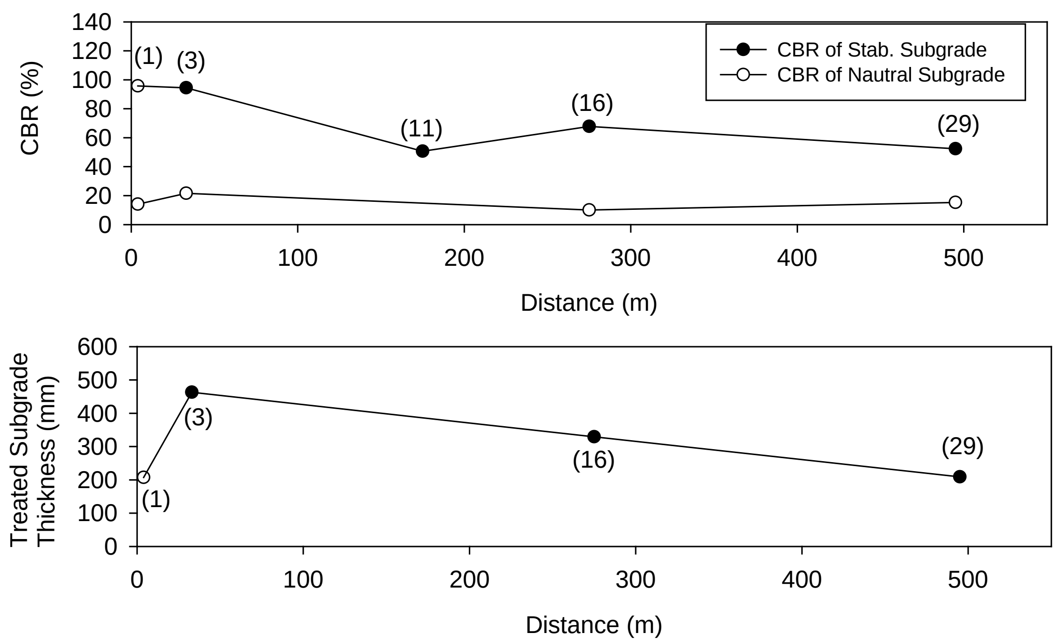


Figure 140. CBR and stabilized subgrade thickness from DCP profile K 7 NB Doniphan County, Kansas

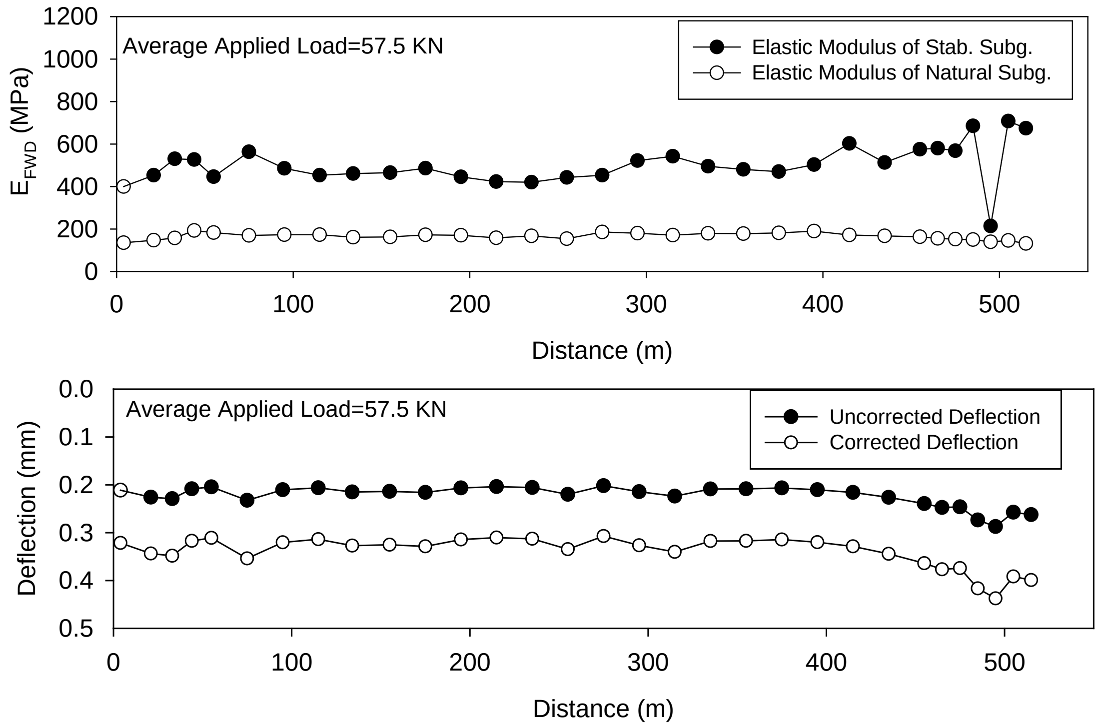


Figure 141. Back-calculated FWD elastic modulus of stabilized and natural subgrade, and deflections under the loading plate K 7 NB Doniphan County, Kansas

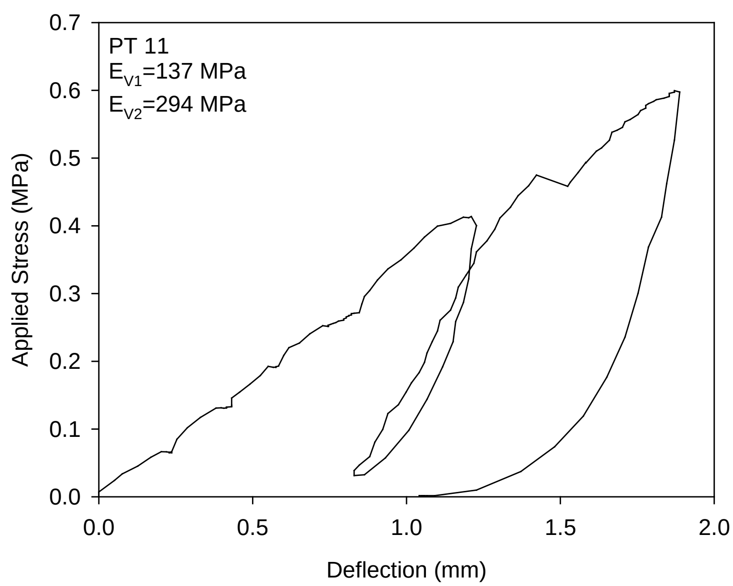


Figure 142. Stress – strain curves from plate load test at point 11 K 7 NB Doniphan County, Kansas

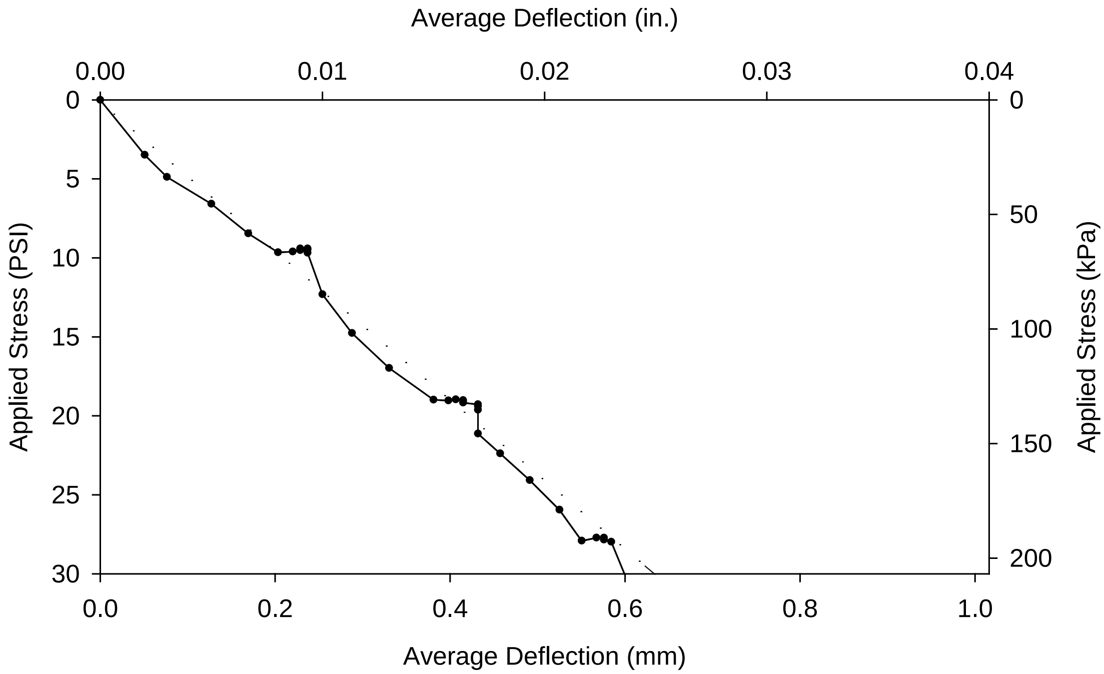


Figure 143. Stress – strain curves for obtaining K_U at point 11 K 7 NB Doniphan County, Kansas

Table 39. Summary of LWD test results K 7 NB Doniphan County, Kansas

Test Point	Material Type	Depth of Measurement	E_{LWD}	Average E_{LWD}
			MPa	MPa
11	Stabilized Subgrade	Top of stabilized subgrade	89	89
32	Natural Subgrade	Top of natural subgrade	12	11
33	Natural Subgrade	Top of natural subgrade	10	

Table 40. Summary of elastic modulus ratio between stabilized and natural subgrade K 7 NB Doniphan County, Kansas

Stab. Subg./Nat. Subg. Ratio		
CBR	E_{FWD}	E_{LWD}
5.3	3.7	10.8

Table 41. Summary statistics of test results from in situ testing K 7 NB Doniphan County, Kansas

Statistic	Stabilized Subgrade							Natural Subgrade			FWD Def
Measurement	CBR	E_{FWD}	E_{LWD}	E_{V1}	E_{V2}	k_U	Thi.	CBR	E_{FWD}	E_{LWD}	$D_{0\text{-}Cor.}$
	%	MPa	MPa	MPa	MPa	kPa/mm	mm	%	MPa	MPa	mm
Number of Measurement	5	31	1	1	1	1	5	6	31	2	31

(n)											
Mean Value (μ)	72	503	89	137	294	125	302	16	138	11	0.34
Standard Deviation (σ)	22	94	—	—	—	—	122	4	13	2	0.03
Coefficient of Variation COV (%)	30	19	—	—	—	—	40	27	10	13	10

4.10 SUMMARY

This chapter provides a brief summary and comparison of test results for all nine test location in Texas, Oklahoma, and Kansas. Table 42 lists the key information at each test site including material properties of subgrade include soil type, fine contents, plastic index and pH value. Based on DCP, FWD, and LWD test results, modulus ratios are compared between stabilized and natural subgrade.

Table 42. Summary of laboratory and in situ test results for all test sites

Location	Age (Yrs)	Subgrade Type	Fines Cont. (%)	Soil Type	Plastic Index (%)	pH Value	Ratio Between Stab. and Nat. Subg.		
							CBR	E_{FWD}	E_{LWD}
SH 121 TX	15	200 mm lime stab.	27.4	SM	N.P.	9.2	—	4.3	—
FM 1709 TX	16	150 mm lime stab.	40.6	SM	N.P.	9.6	2.2	3.1	—
US 287 TX	28	356 mm lime stab.	44.4	ML/SM	15.4	8.7	7.4	8.3	—
US 183 OK	11	Natural	84.3	ML	10.2	7.9-8.3	4.5	12.3	8.5
		203 mm lime stab.	35.0	SM	6.3	8.1-8.9			
SH 99 OK	11	Natural	37.7	SM	4.9	8.2	3.8	1.6	4.1
		203 mm fly ash stab.	44.7	SM	N.P.	9.2			
US 59 OK	10	Natural	65.9	ML	24.7	4.8	6.4	2.3	5.3
		203 mm fly ash stab.	35.7	SM	5.6	8.9			
US 75 SB KS	15	Natural	96.7	ML	33.1	6.5-8.0	2.7	2.2	—
		100 mm lime stab.	44.5	ML/SM	21.1	7.7-8.8			
US 75 NB KS	15	Natural	68.9	ML	34.3	7.9-8.1	2.9	—	6.1
		152 mm lime stab.	34.6	SM	7.8	8.7-9.4			
K 7 KS	5	Natural	94.3	ML	18.3	7.4	5.3	3.7	10.8
		300mm fly ash stab.	26.6	SM	5.1	8.3			

CHAPTER 5. SUMMARY AND CONCLUSIONS

Nine test sites were selected to access the long-term performance of lime or fly ash stabilized subgrades. Ages of these stabilized subgrades ranged from 5 to 28 years. In situ tests were conducted on eight ACC pavements and one PCC pavement. FWD moduli were back-calculated using the ERI data analysis program. Test results from the nine site studies led to the following conclusions:

- FWD testing conducted at 8 to 50 test locations at each site showed non-uniform conditions at each site with coefficient of variation of surface deflections varying from about 10 to 30%, and E_{FWD} value for stabilized subgrade from about 20 to 70%. The FWD plate deflections on top of the flexible pavements are strongly influenced by the CBR profile of the underlying stabilized subgrade layer.

- The in situ elastic modulus of chemical stabilized subgrades determined from the static PLT varied from 7 MPa to 317 MPa at the nine test sites. The MEPDG recommended typical modulus value for lime stabilized soils is 310 MPa with a range of 207 MPa to 414 MPa, and a deteriorated modulus value for lime stabilized soil is 103 MPa. Two out of the six lime stabilized subgrade sites tested showed modulus < 103 MPa (note: MEDPG does not provide typical values for fly ash stabilized subgrades).

- Field results indicated that the elastic modulus value determined in the field is dependent on the test method used. On average, LWD and the back-calculated FWD modulus were about 0.7 times and 8.3 times the static PLT modulus, respectively. This is divergence in calculated modulus values is an important aspect to considered when selecting design values are establishing QC/QA target values.

- The ratio of LWD modulus of stabilized subgrade and natural subgrade varied from about 4 to 11. Similarly, CBR ratios between stabilized and natural subgrade ranged from about 2.2 to 7.4. Results indicated that these ratios are influenced by the thickness of the stabilized layers (lower the thickness, lower the ratio).

- The improved soil strength and stiffness due to chemical stabilization remained after many years of construction, but testing revealed that the in situ stiffness is highly non-uniform in the longitudinal direction (i.e., along the road alignment) and vertical direction (i.e., with depth).

- Scanning electron microscopy (SEM) analysis of treated subgrade samples showed that cementitious reaction products formed and remained in lime stabilized subgrade samples even after several years after construction.

- This study identified that the top of the stabilized layer is often weaker than near the center of the stabilized layer. Additional research is needed to understand why this is occurring (e.g., construction issue, environmental factors, etc.).

- Pavement performance was good at all of the test sites.

- Based on DCP-CBR profiles, the effective stabilized thickness was obtained and differed from designed stabilized subgrade thickness.

- CBR ratios between stabilized and natural subgrade ranged from 2.2 to 7.4. The ratios were smaller than the value of 3 at the FM 1709, US 75 SB, and US 75 NB test sites. Those pavements contain 152 mm (6 in.) thick lime, 100 mm (4 in.) thick lime, and 152 mm (6 in.) thick fly ash stabilized subgrade, respectively. Generally, the 152 mm (6 in.) stabilized subgrades have lower CBR values compared to the thicker sections.

- Based on the field observations at all three sites in TX, the top 50 mm (2 in.) stabilized subgrades had higher moisture contents compared to the middle and bottom of the stabilized subgrades. DCP-CBR profiles show that CBR values at the top and bottom of the stabilized subgrades are lower than the middle of the stabilized subgrades.

- FWD modulus ratios between stabilized and natural subgrade ranged from 1.6 to 12.3. Two out of three fly ash stabilized subgrades have average FWD modulus smaller than 3. Variations in the FWD modulus at each test point indicated non-uniform stabilized subgrade.

- The average E_{V1} ranged from 7 to 317 MPa, and the average E_{V2} ranged 15 from 202 MPa.

- Modulus of subgrade reaction values ranged from 31 to 202 kPa/mm (multiply by 3.625 to convert to pci).

Overall, the old stabilized subgrades performed well based on the test results and analysis. The improved soil strength and stiffness remained after many years of construction. However, new information reveals that the tops of the stabilized layers are generally weaker and warrants further research to determine the cause (e.g., construction, curing, environmental factors, etc.).

CHAPTER 6. RECOMMENDATIONS

The following recommendations should guide future research to establish case studies of long-term performance of stabilized subgrades.

- Conduct life cycle cost analysis for using stabilized subgrade in structural pavement design.
- Back-calculate the subbase layer coefficient to determine the structural benefit provided by stabilized subgrades. In the back-calculation, consider treating the stabilized subgrade as a subbase layer.
- Conduct resilient modulus tests on undisturbed stabilized subgrade samples and compare these resilient modulus values with back-calculated FWD modulus values.
- Conduct x-ray diffraction (XRD) and x-ray fluorescence (XRF) tests to quantitatively analyze chemical reaction byproducts in stabilized subgrades.
- Compare other stabilization technologies (e.g., mechanical stabilization using geosynthetics, fiber reinforcement) with chemical stabilization of subgrade.
- Document the long-term performance of stabilized subgrade with other stabilizers such as cement or combined stabilizers.
- In the field, it is important to follow a QC/QA program that improves construction quality to uniformly mix and compact chemical stabilized subgrade. Additional research is warranted to investigate construction methods to ensure quality construction of chemically stabilized subgrades.

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APPENDIX A. SEM IMAGES OF SUBGRADES NOT SHOWN IN CHAPTER 4

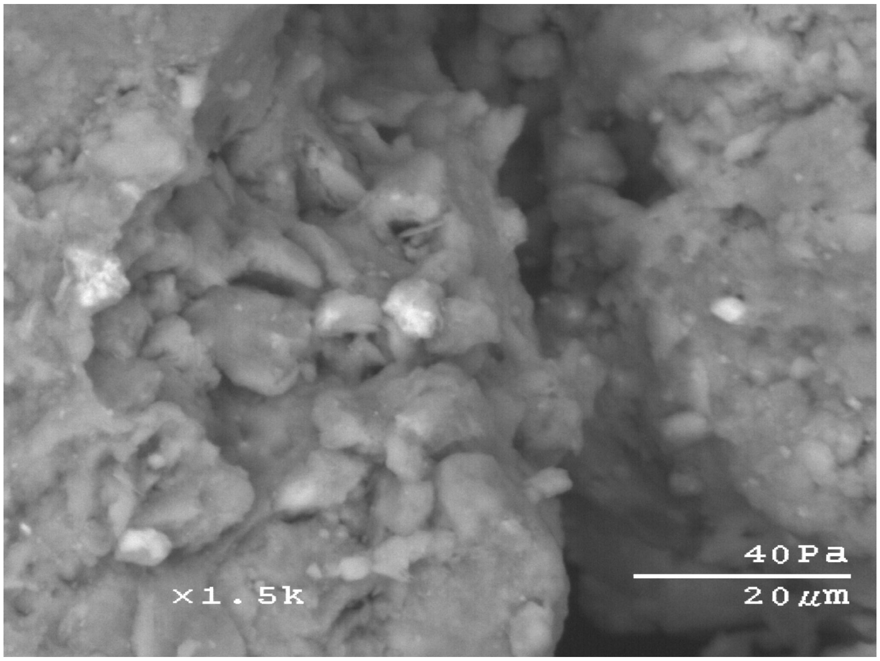

Figure 144. SEM image of stabilized subgrade in area b (1500 ×) – SH 121

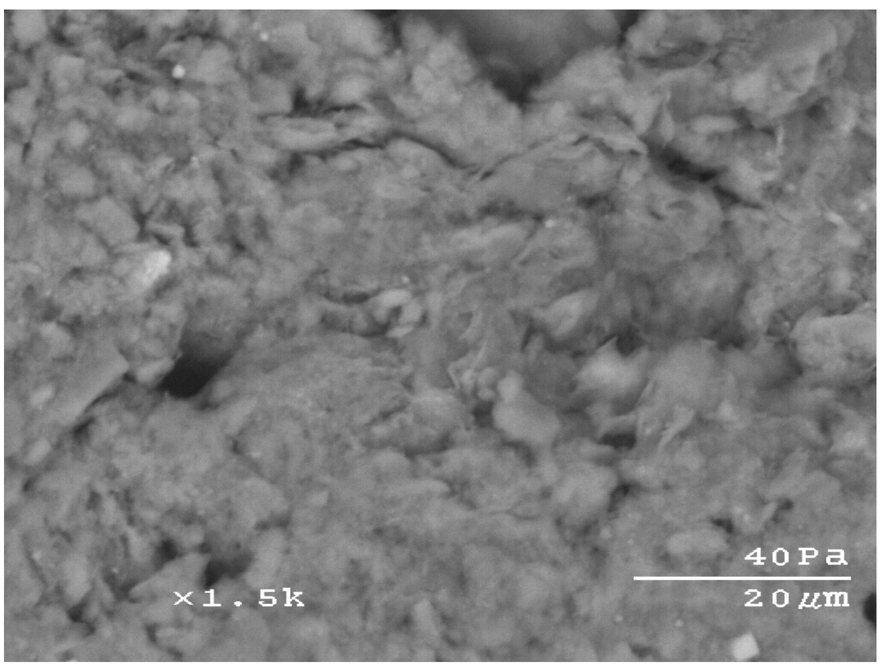

Figure 145. SEM image of stabilized subgrade (1500 ×) – FM 1709

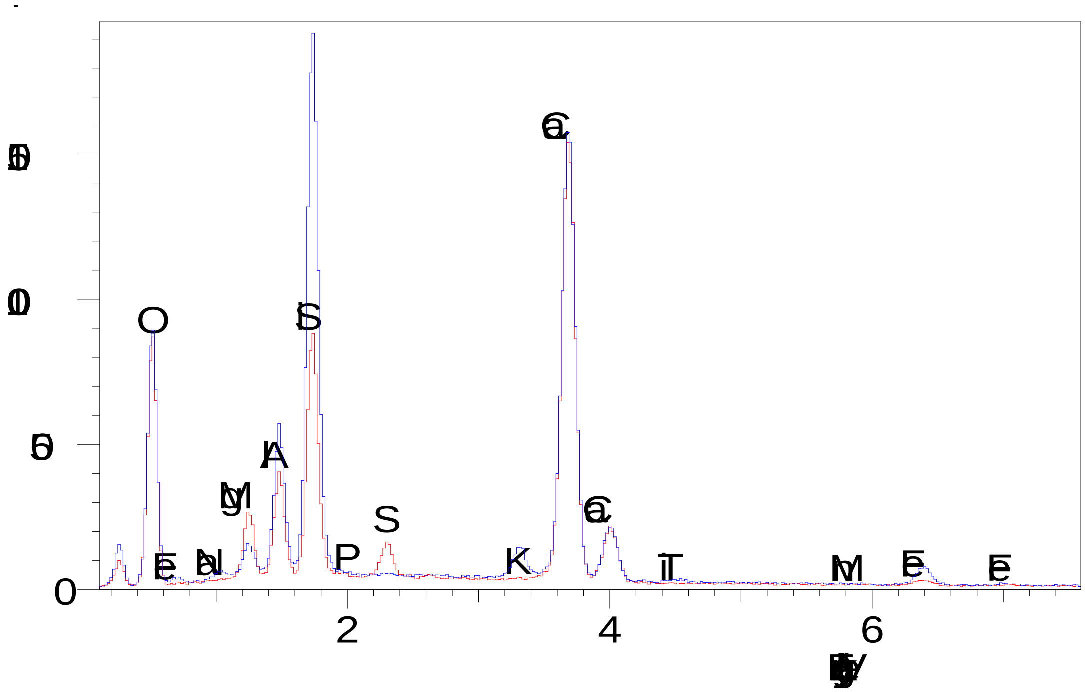

Figure 146. EDS intensity counts for stabilized subgrade sample in area a and stabilized subgrade sample in area b (red line 500×; blue line 500×) – US 183

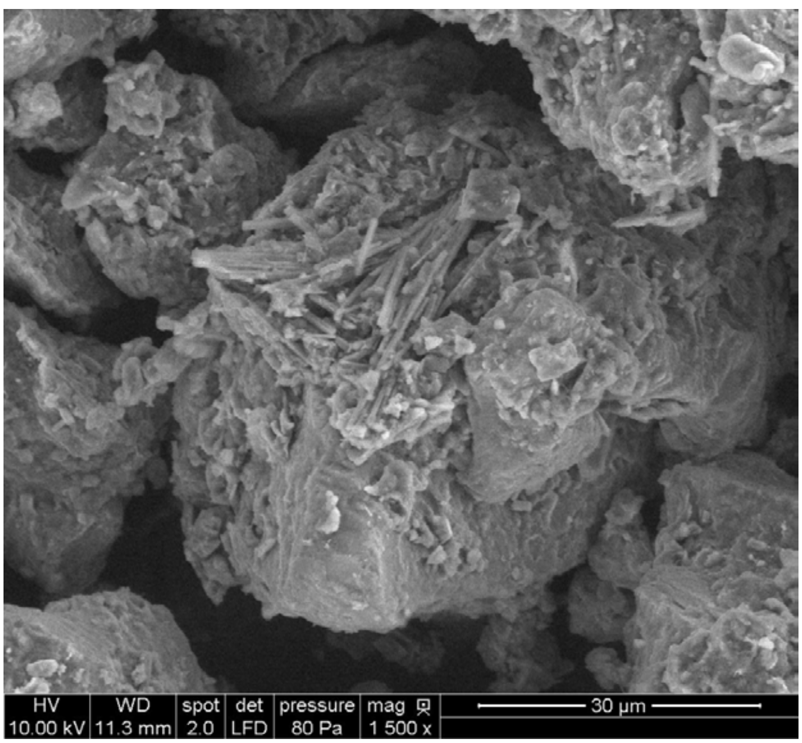

Figure 147. SEM image of natural subgrade (1500 ×) – US 183

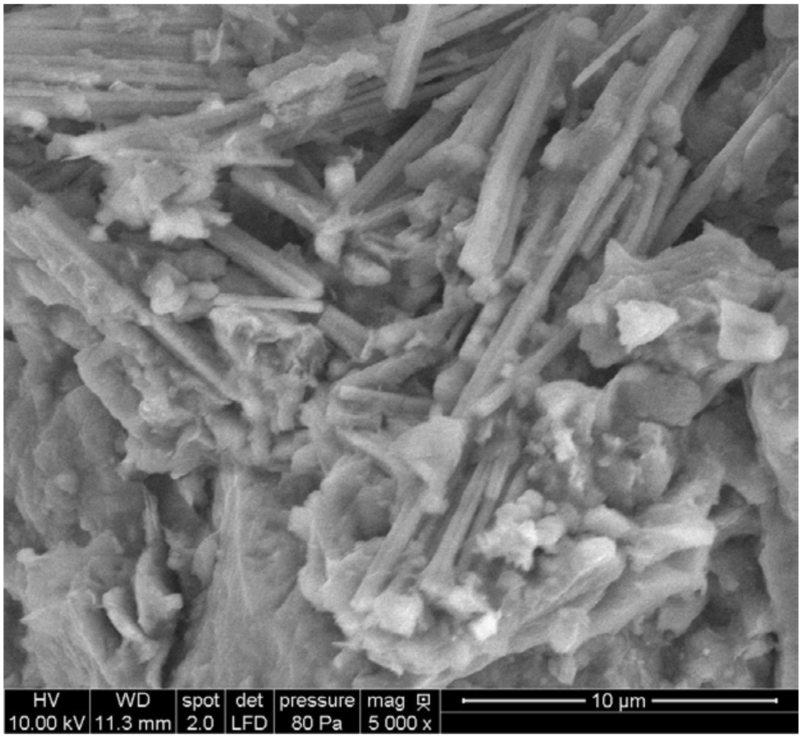

Figure 148. SEM image of natural subgrade (5000 ×) – US 183

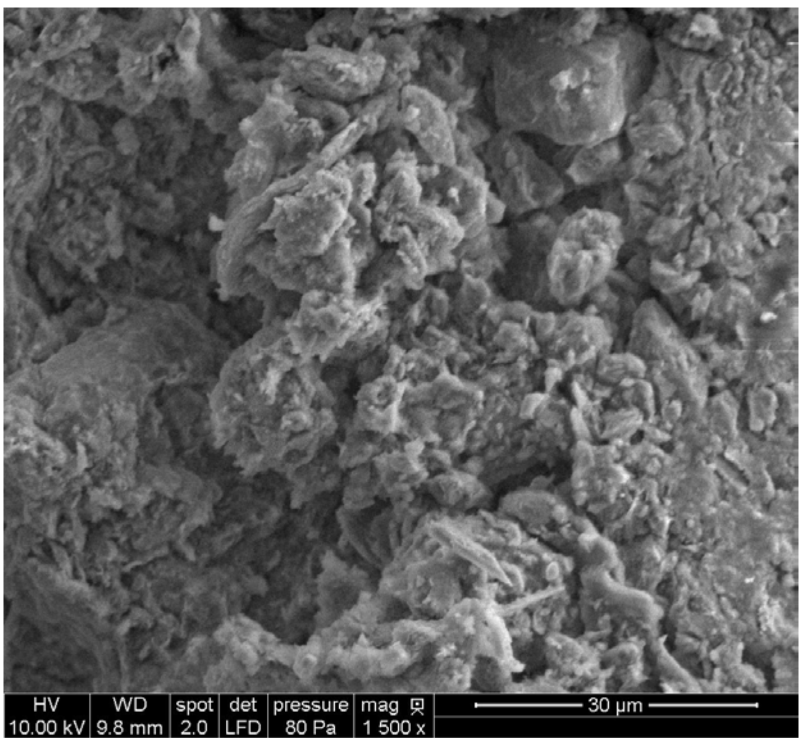

Figure 149. SEM image of stabilized subgrade (1500 ×) – US 183

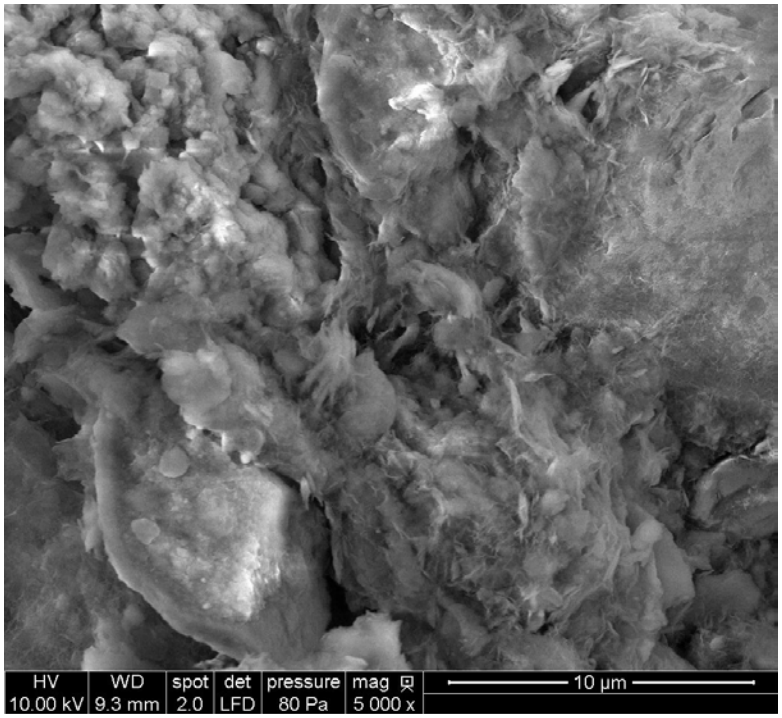

Figure 150. SEM image of stabilized subgrade (5000 ×) – US 183

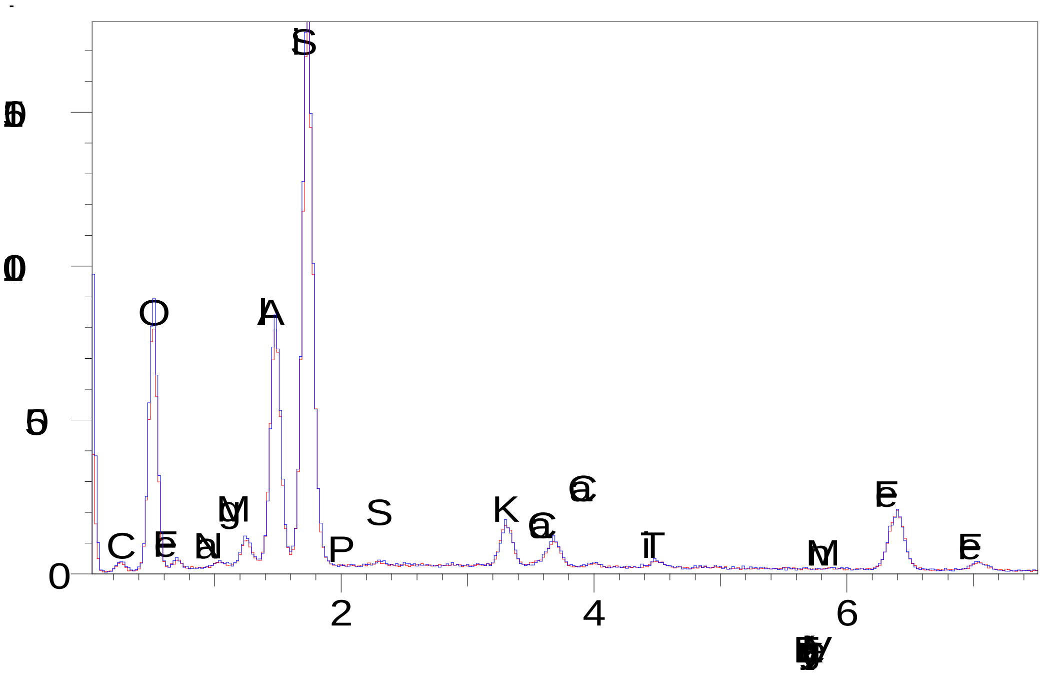

Figure 151. EDS intensity counts for stabilized subgrade sample (red line 150x; blue line 25x) – SH 99

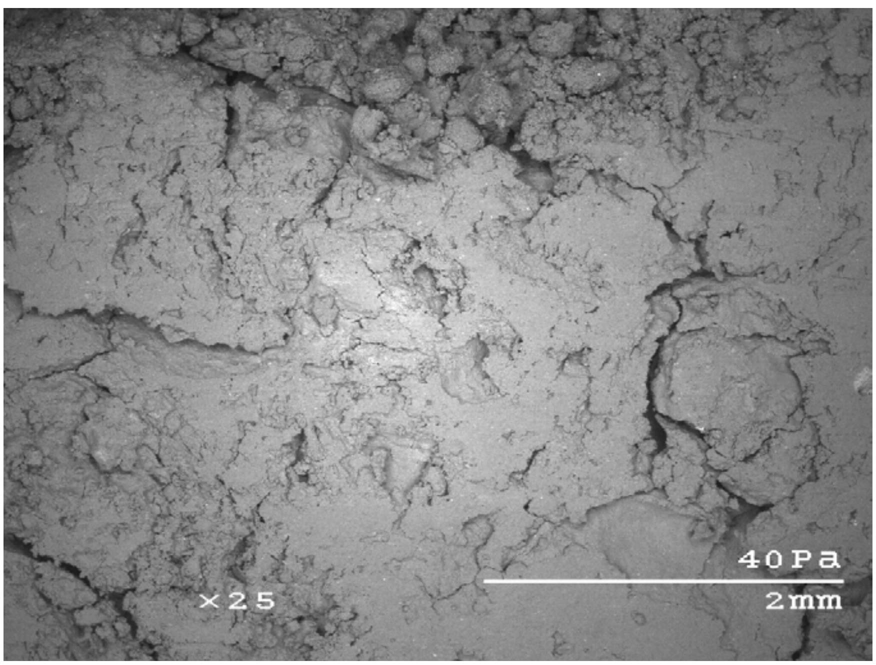

Figure 152. SEM image of stabilized subgrade (25 ×) in area a – SH 99

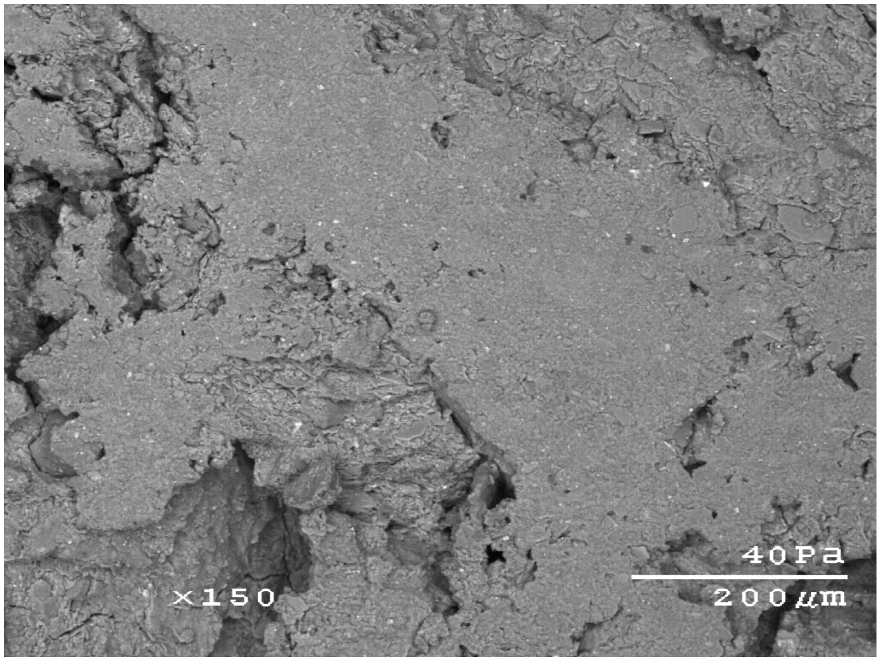

Figure 153. SEM image of stabilized subgrade (150 ×) in area a – SH 99

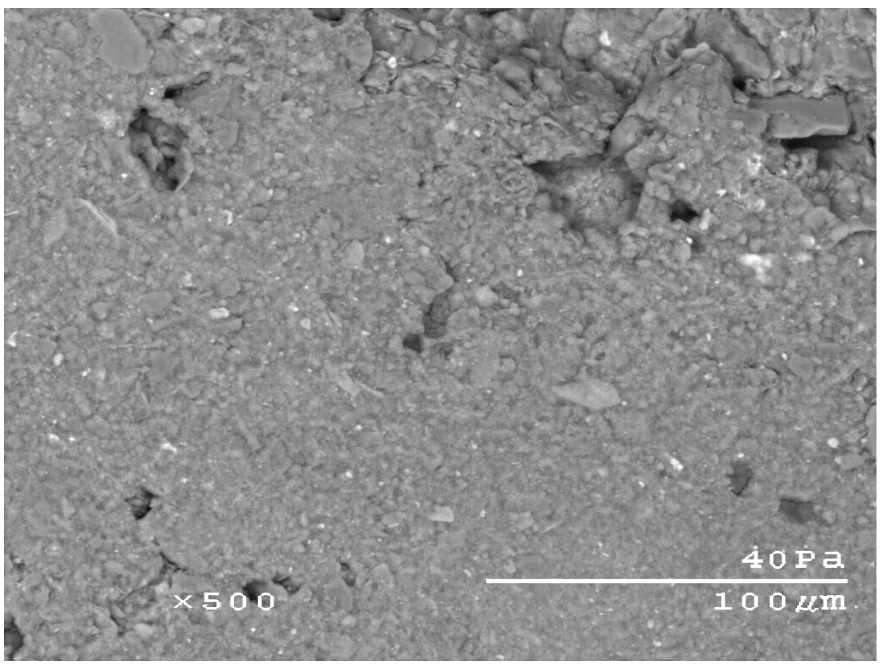

Figure 154. SEM image of stabilized subgrade (500 ×) in area a – SH 99

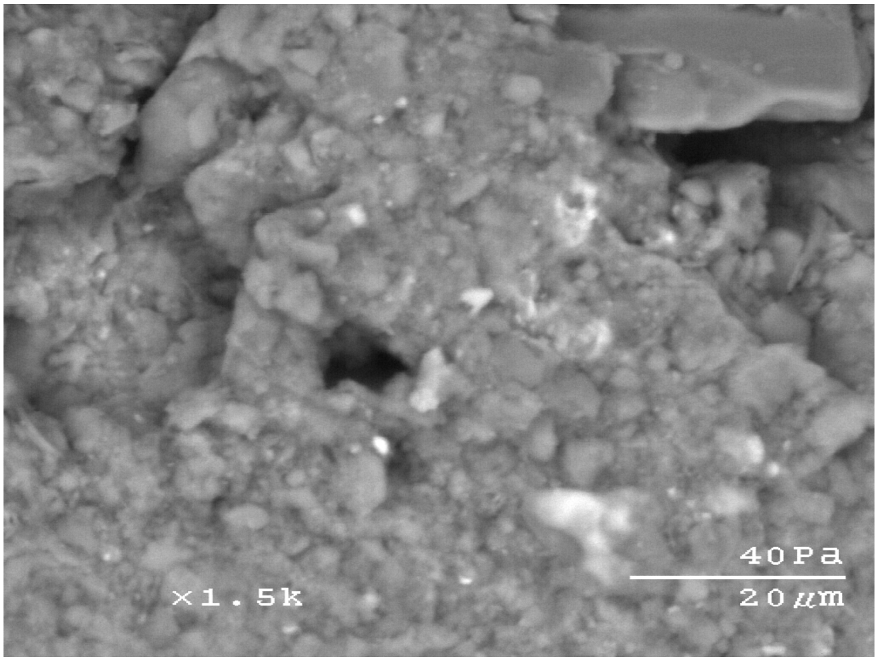

Figure 155. SEM image of stabilized subgrade (1500 ×) in area a – SH 99

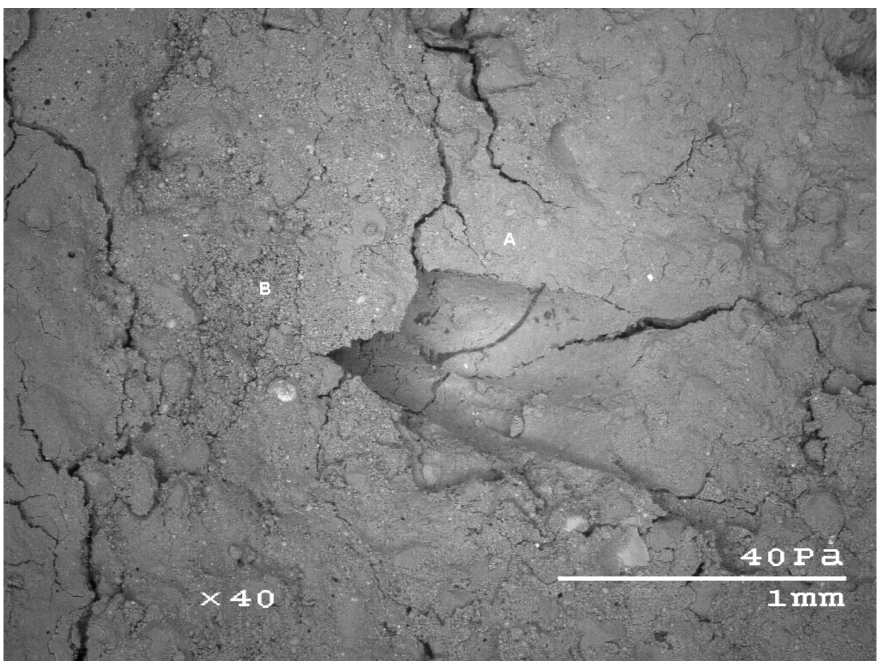

Figure 156. SEM image of stabilized subgrade (40 ×) in area b – SH 99

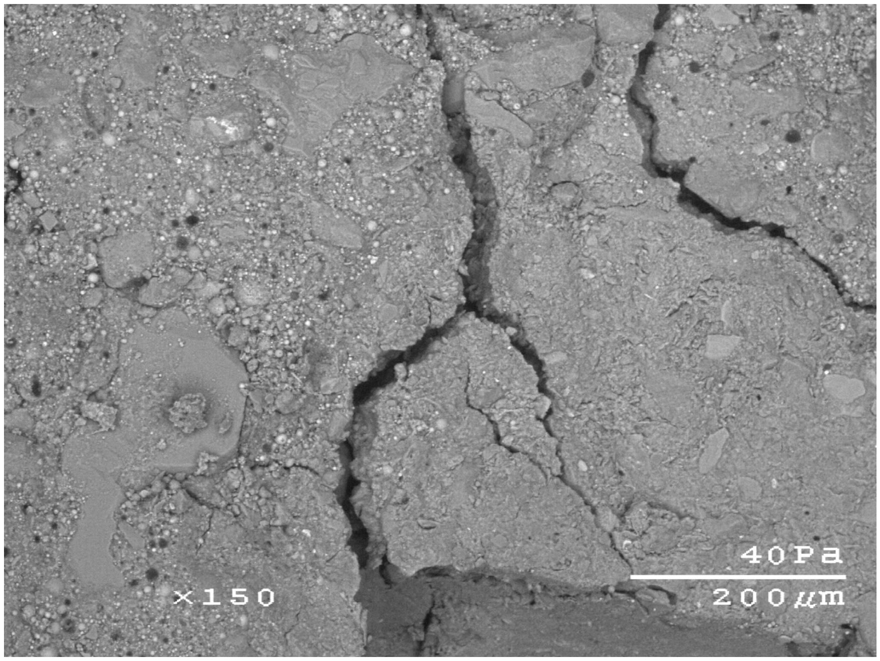

Figure 157. SEM image of stabilized subgrade (150 ×) in area b – SH 99

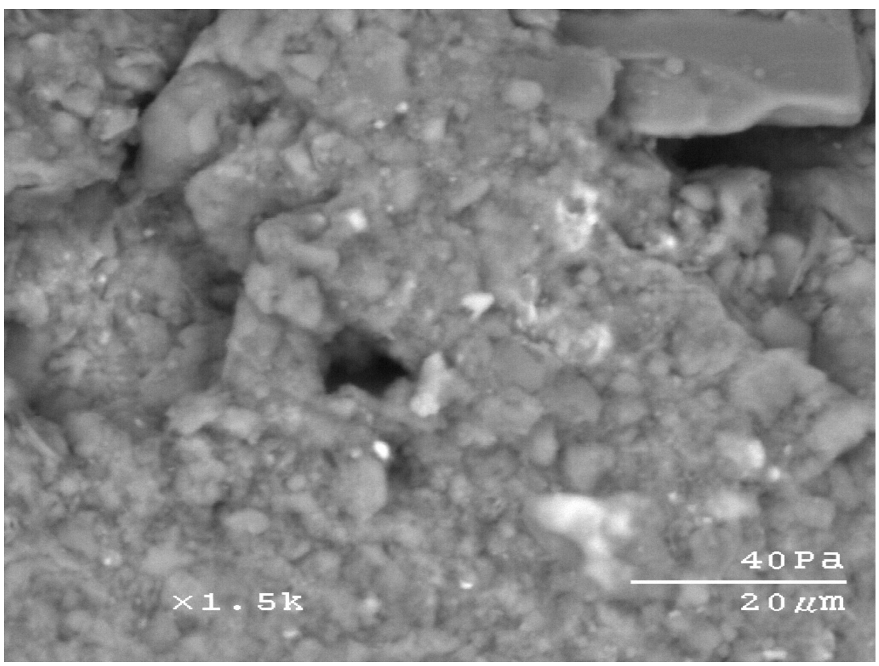

Figure 158. SEM image of stabilized subgrade (1500 ×) in area b – SH 99

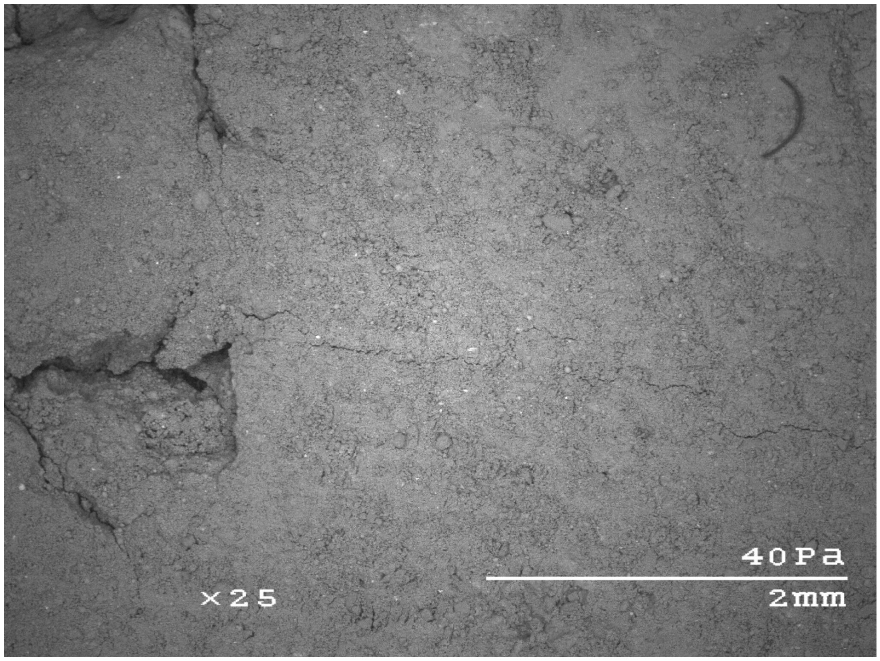

Figure 159. SEM image of stabilized subgrade (25 ×) -US 59

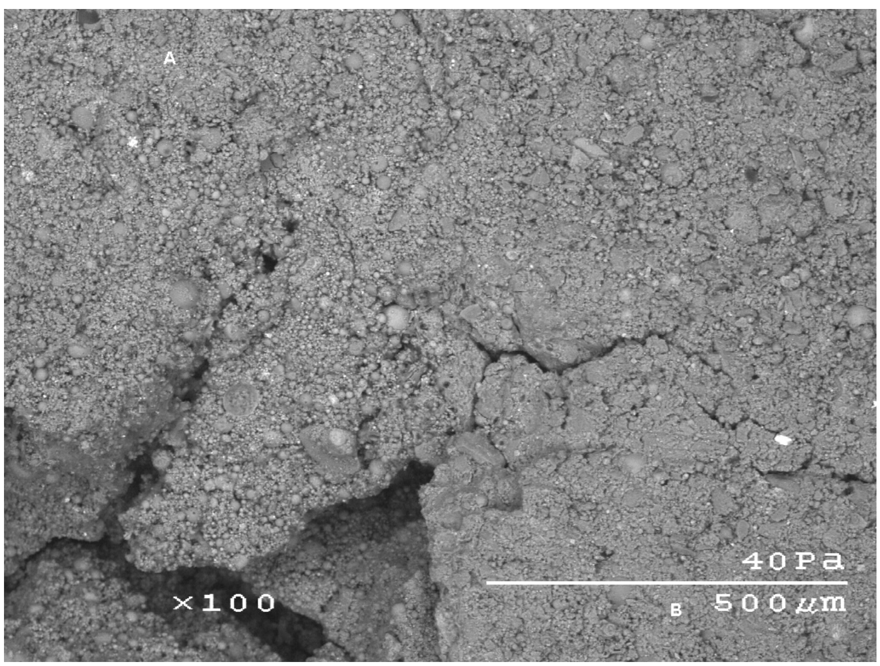

Figure 160. SEM image of stabilized subgrade (100 ×) – US 59

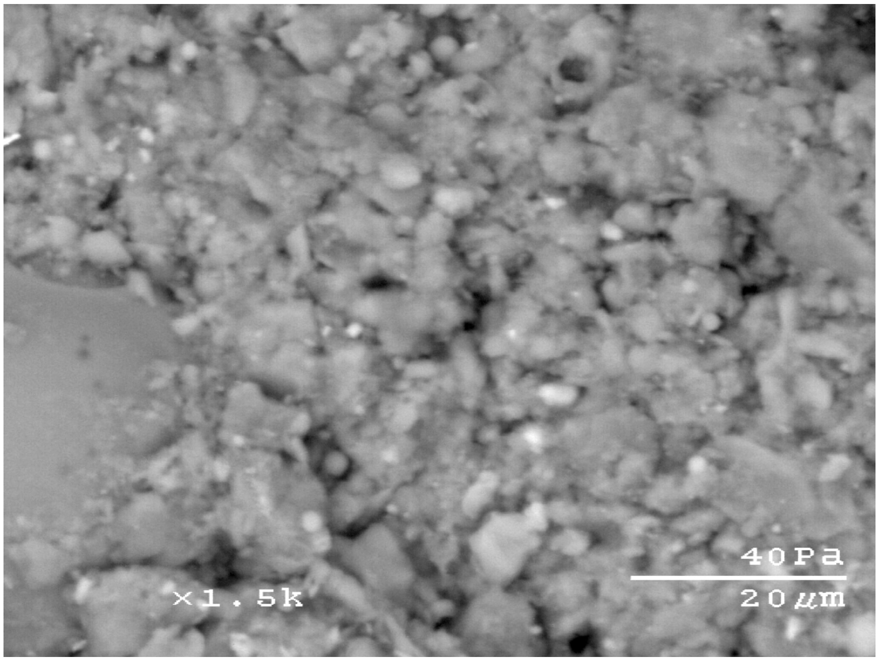

Figure 161. SEM image of stabilized subgrade (1500 ×) – US 59

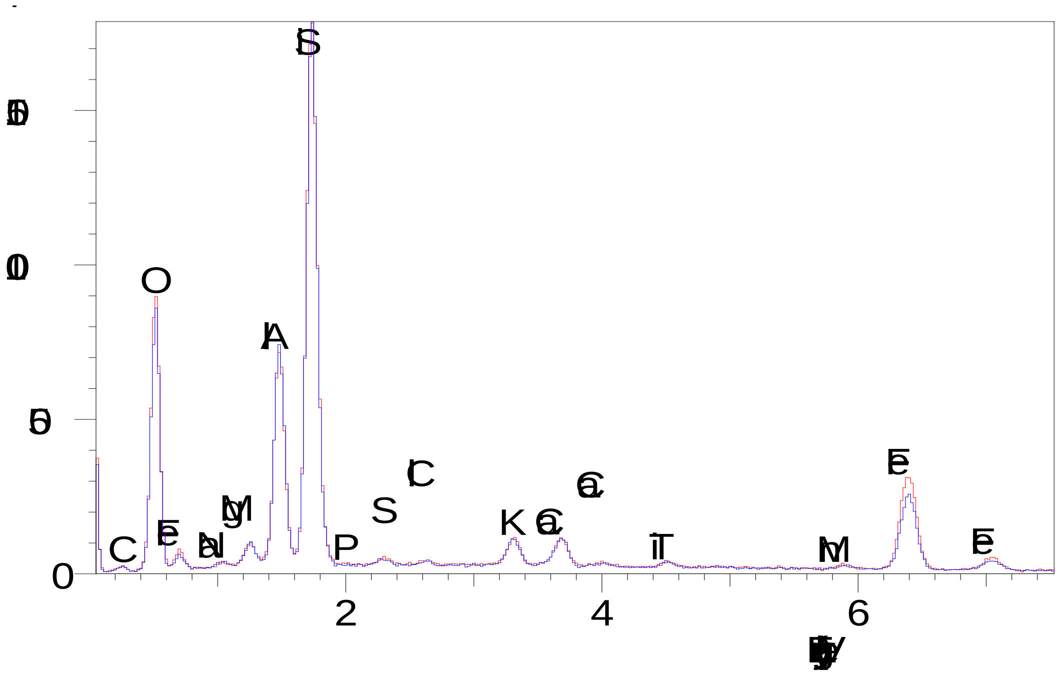

Figure 162. EDS intensity counts for stabilized subgrade sample (red line: 1500×, blue line: 500 ×) – US 75 NB

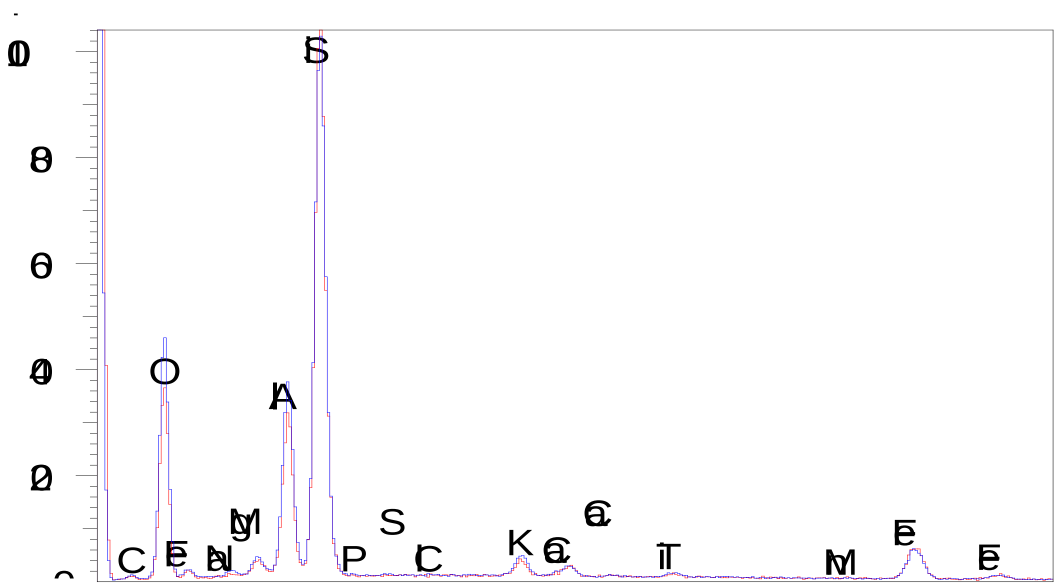

Figure 163. EDS intensity counts for stabilized subgrade sample (red line: 1500×, blue line: 150 ×) – US 75 NB

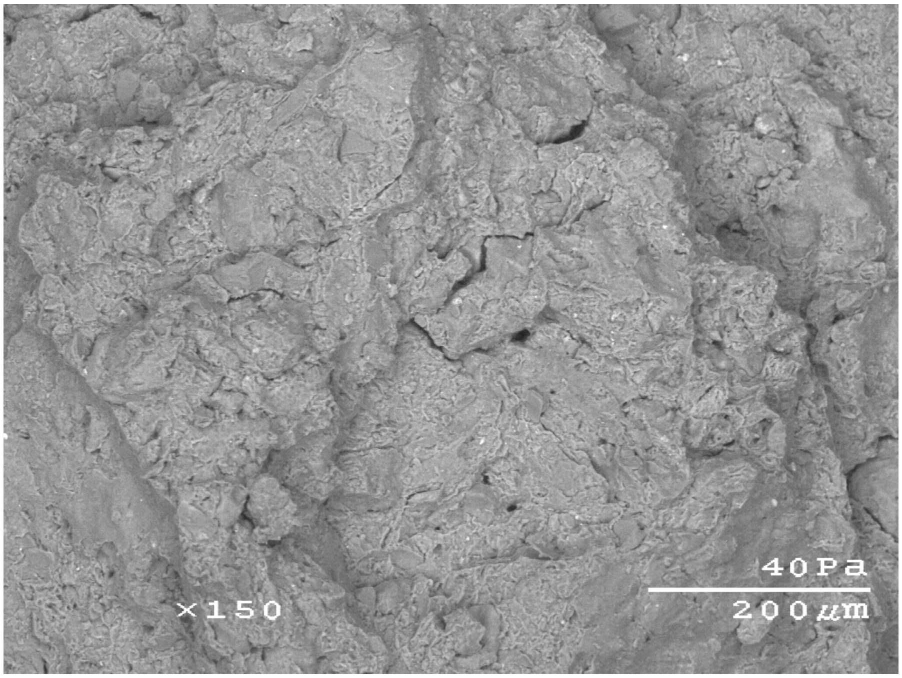

Figure 164. SEM image of natural subgrade in area b (150×) – US 75 SB

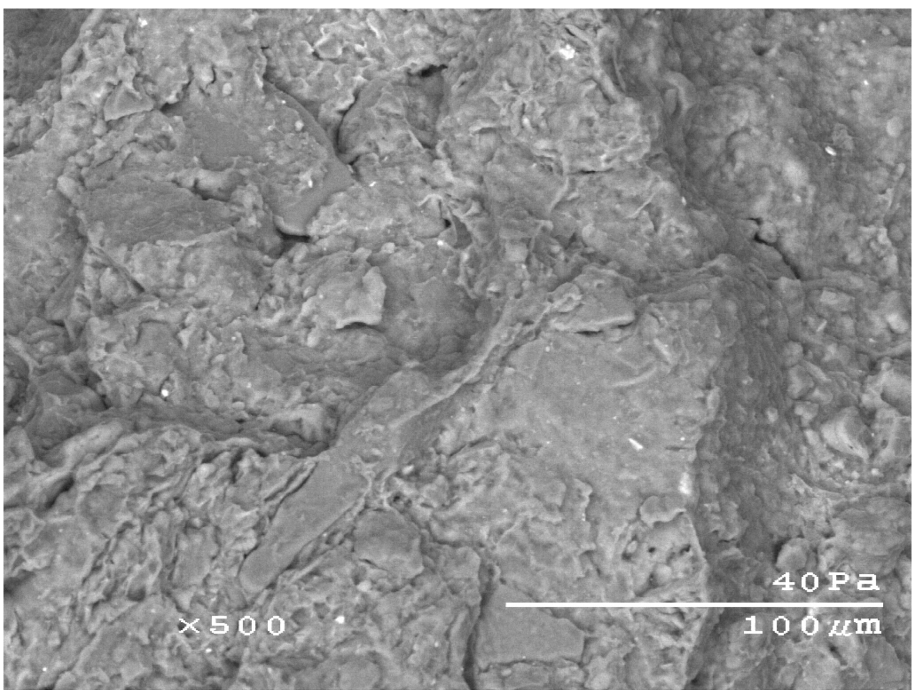

Figure 165. SEM image of natural subgrade in area b (500×) – US 75 SB

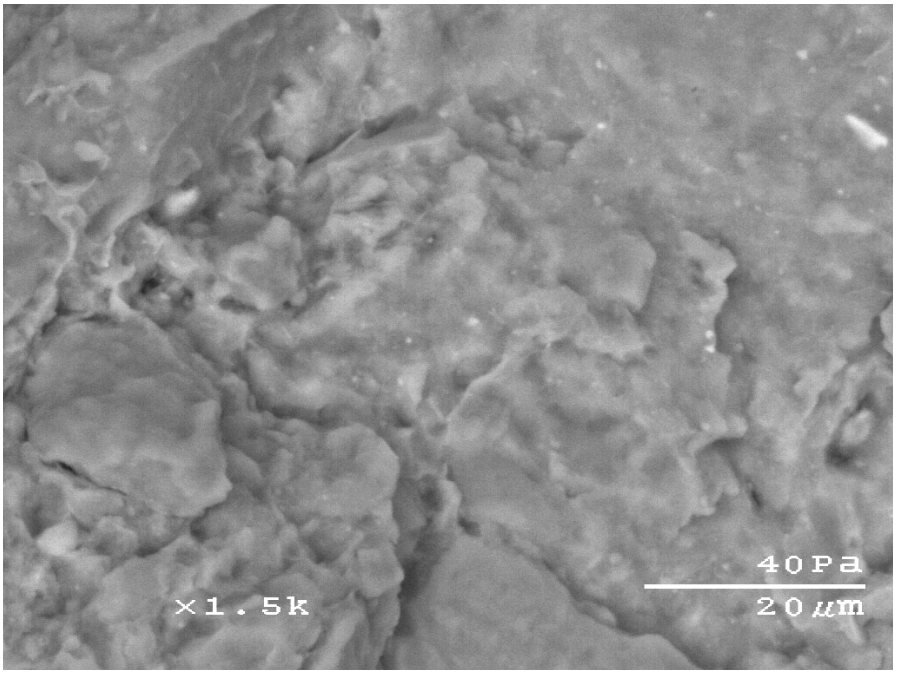

Figure 166. SEM image of natural subgrade in area b (1500×) – US 75 SB

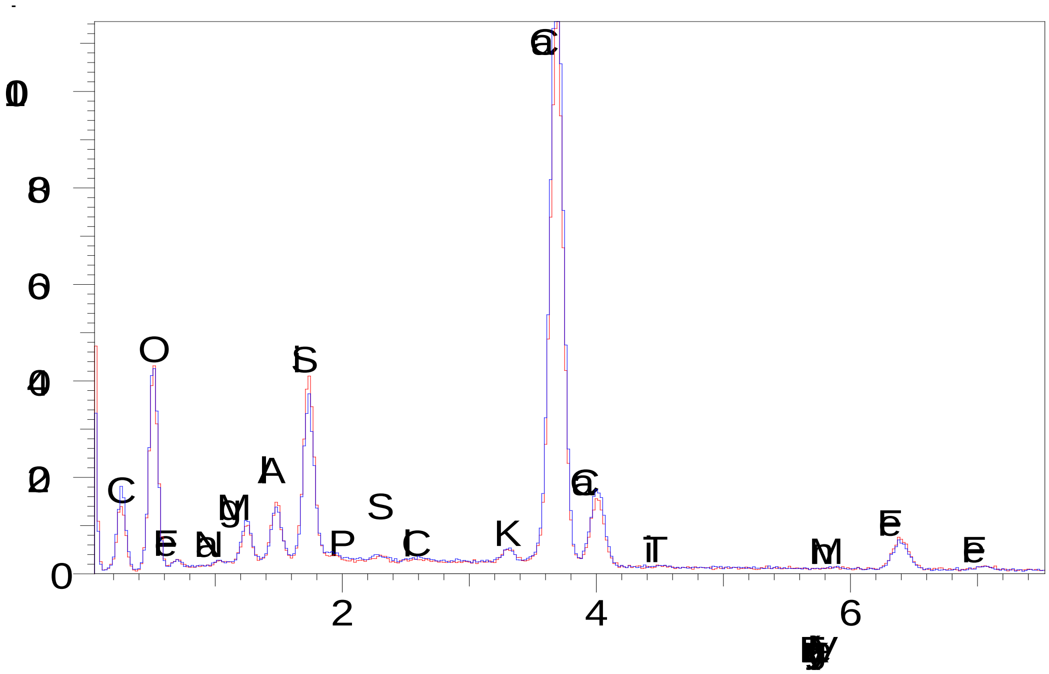

Figure 167. EDS intensity counts for stabilized subgrade sample (red line: 30×, blue line: 150 ×) – K 7

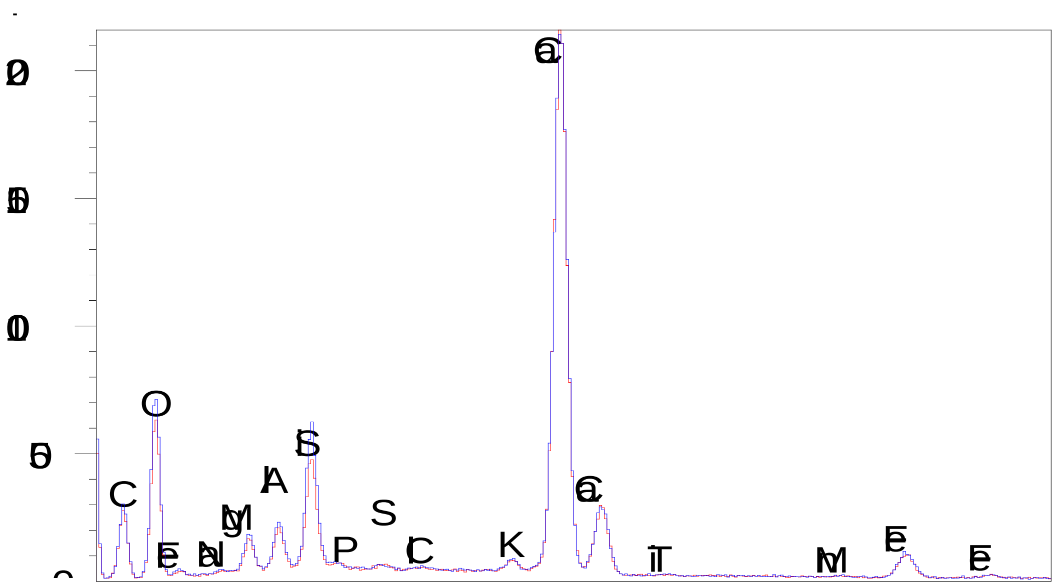

Figure 168. EDS intensity counts for stabilized subgrade sample (red line: 500×, blue line: 150 ×) – K 7

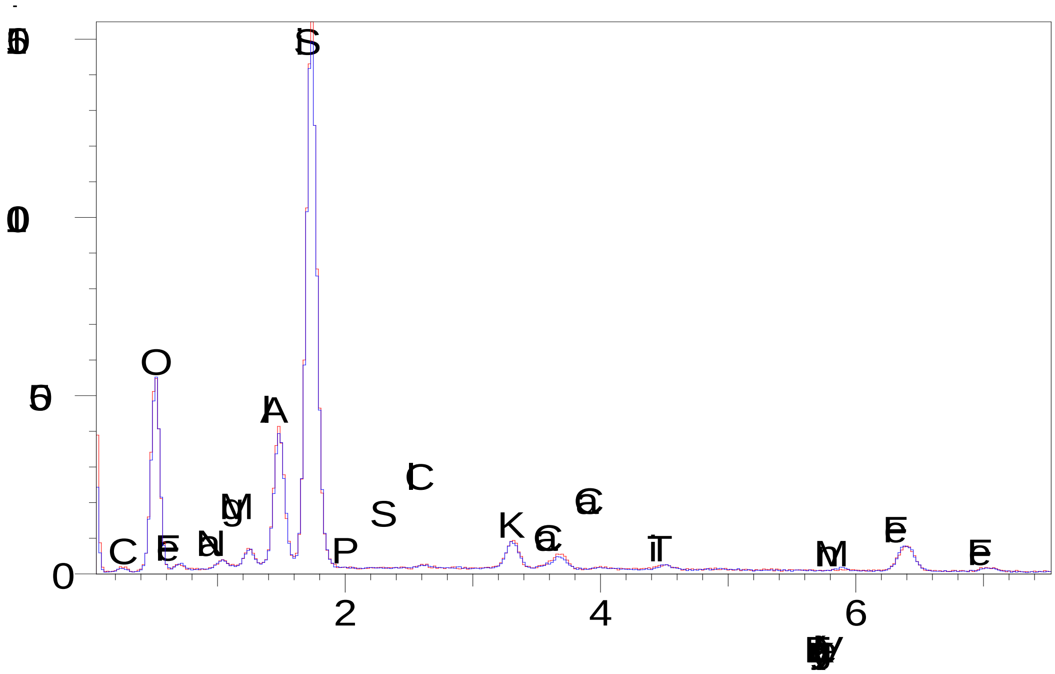

Figure 169. EDS intensity counts for natural subgrade sample (red line: 30×, blue line: 150 ×) – K 7

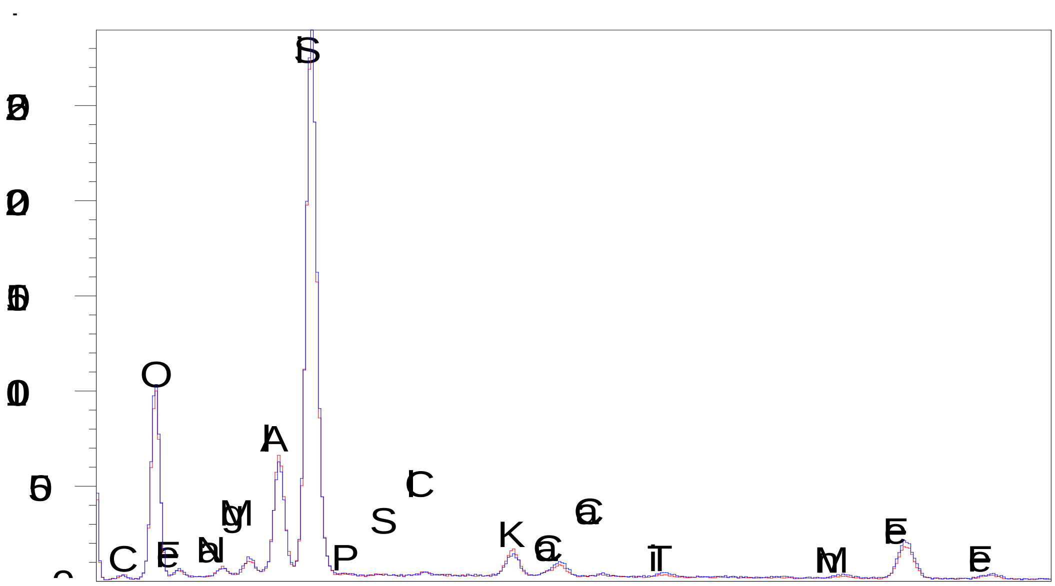

Figure 170. EDS intensity counts for natural subgrade sample (red line: 1500×; blue line: 500 ×) – K 7

APPENDIX B. PARAMETER VALUE ASSUMPTIONS FOR FWD ANALYSIS

Table 43. Parameter value assumptions for E_{FWD} analysis – SH 121

	ACC pavement (Layer 1)				**Aggregate Base (Layer 2)**				**Treated Subgrade (Layer 3)**				**Natural Sub.**
	Seed	Minimum	Maximum	Thi.	Seed	Minimum	Maximum	Thi.	Seed	Minimum	Maximum	Thi.	Thi.
PT	psi	psi	psi	in.	psi	psi	psi	in.	psi	psi	psi	in.	in.
1	1500000	60000	6000000	2	150000	10000	600000	8	60000	10000	500000	12	400
2	1500000	60000	6000000	2	150000	10000	600000	8	60000	10000	500000	10	400
3	1500000	60000	6000000	2	150000	10000	600000	8	60000	10000	500000	10	400
4	1000000	60000	6000000	2	100000	10000	600000	8	40000	10000	500000	8	340
5	1000000	60000	6000000	2	100000	10000	600000	8	40000	10000	500000	8	340
6	1000000	60000	6000000	2	100000	10000	600000	8	40000	10000	500000	10	400
7	1000000	60000	6000000	2	100000	10000	600000	8	90000	10000	500000	8	240
8	1500000	60000	6000000	2	150000	10000	600000	8	60000	10000	500000	14	400
9	1000000	60000	6000000	3	40000	10000	600000	8	90000	10000	500000	8	500
10	1000000	60000	6000000	3	40000	10000	600000	8	90000	10000	500000	8	175
11	1000000	60000	6000000	3	40000	10000	600000	8	90000	10000	500000	8	480
12	1000000	60000	6000000	3	40000	10000	600000	8	90000	10000	500000	8	480
13	600000	60000	6000000	2	40000	10000	600000	8	60000	10000	500000	8	380
14	600000	60000	6000000	2	40000	10000	600000	8	60000	10000	500000	8	380

Table 44. Parameter value assumptions for E_{FWD} analysis – FM 1709

	ACC pavement (Layer 1)				Aggregate Base (Layer 2)				Treated Subgrade (Layer 3)				Natural Sub.
	Seed	Minimum	Maximum	Thi.	Seed	Minimum	Maximum	Thi.	Seed	Minimum	Maximum	Thi	Thi
PT	psi	psi	psi	in.	psi	psi	psi	in.	psi	psi	psi	in.	in.
1	100000	50000	3000000	6.5	50000	10000	500000	7.5	30000	10000	80000	6	120
2	100000	50000	3000000	6.5	50000	10000	500000	7.5	30000	10000	80000	6	350
3	100000	50000	3000000	6.5	50000	10000	500000	7.5	30000	10000	80000	6	350
4	100000	50000	3000000	6.5	50000	10000	500000	7.5	30000	10000	80000	6	350
5	100000	50000	3000000	6.5	50000	10000	1000000	7.5	30000	10000	200000	6	350
6	100000	50000	3000000	6.5	50000	10000	5000000	7.5	30000	10000	800000	6	150
7	100000	50000	3000000	6.5	50000	10000	1000000	7.5	40000	15000	300000	6	400
8	100000	50000	3000000	6.5	50000	10000	1000000	7	20000	15000	300000	4	120

Table 45. Parameter value assumptions for E_{FWD} analysis – US 287

	ACC pavement (Layer 1)				Aggregate Base (Layer 2)				Treated Subgrade (Layer 3)				Nat.Sub.
	Seed	Minimum	Maximum	Thi.	Seed	Minimum	Maximum	Thi.	Seed	Minimum	Maximum	Thi.	Thi.
PT	psi	psi	psi	in	psi	psi	psi	in	psi	psi	psi	in	in
1	1000000	500000	5000000	4	10000	1000	300000	8	60000	10000	200000	14	180
2	40000	200000	5000000	4	1000	1000	300000	11	60000	10000	200000	14	260
3	800000	100000	5000000	4	80000	1000	300000	8	80000	10000	500000	14	300
4	800000	100000	5000000	4	80000	1000	300000	11	80000	10000	500000	14	280
5	800000	100000	5000000	4	80000	1000	300000	11	80000	10000	500000	14	200
6	500000	10000	5000000	4	100000	1000	300000	8	120000	10000	500000	14	260
7	500000	10000	5000000	4	100000	1000	300000	8	120000	10000	500000	14	300
8	500000	10000	5000000	4	100000	1000	300000	8	120000	10000	500000	18	250
9	500000	10000	5000000	4	100000	1000	300000	11	120000	10000	500000	14	250
10	500000	10000	5000000	4	100000	1000	300000	11	120000	10000	500000	18	250
11	800000	200000	5000000	4	60000	1000	300000	11	50000	10000	200000	14	250
12	800000	200000	5000000	4	60000	1000	300000	11	50000	10000	200000	14	300
13	500000	10000	5000000	4	100000	1000	300000	11	120000	10000	500000	16	350
14	500000	10000	5000000	2	100000	1000	300000	11	120000	10000	500000	16	350
15	800000	200000	5000000	4	80000	1000	300000	11	50000	10000	200000	14	350
16	800000	200000	5000000	4	80000	1000	300000	11	50000	10000	200000	14	310
17	800000	200000	5000000	4	80000	1000	300000	11	50000	10000	200000	14	310
18	800000	200000	5000000	4	80000	1000	300000	11	50000	10000	200000	14	310
19	800000	200000	5000000	4	80000	1000	300000	6	50000	10000	200000	14	310

Table 46. Parameter value assumptions for E_{FWD} analysis – US 183

	ACC pavement (Layer 1)				**Treated Subgrade (Layer 2)**				**Natural Subgrade**
	Seed	Minimum	Maximum	Thickness	Seed	Minimum	Maximum	Thickness	Thickness
PT	psi	psi	psi	in.	psi	psi	psi	in.	in.
1	850000	300000	2000000	12	390000	10000	700000	8.7	240
2	750000	300000	2000000	12	160000	10000	700000	8.5	240
3	1200000	600000	5000000	12	220000	10000	3000000	8.4	240
4	1200000	600000	5000000	12	120000	10000	3000000	8.3	240
5	650000	300000	3000000	12	300000	10000	380000	8.1	240
6	700000	300000	3000000	12	240000	10000	380000	8	240
7	700000	300000	3000000	12	240000	10000	380000	7.8	240
8	700000	300000	3000000	12	240000	10000	380000	7.7	240
9	460000	300000	9000000	12	120000	10000	280000	3.4	240
10	1000000	100000	3000000	12	260000	50000	580000	8	240
11	1000000	100000	3000000	12	260000	50000	580000	8	240
12	700000	100000	3000000	12	240000	50000	580000	6	240
13	700000	100000	3000000	12	340000	50000	580000	8	240
14	700000	100000	3000000	12	340000	50000	580000	8	240
15	700000	100000	3000000	12	340000	50000	580000	8	240
16	700000	100000	3000000	12	340000	50000	580000	8	240
17	700000	100000	3000000	12	200000	50000	580000	8	240
18	700000	100000	3000000	12	230000	50000	580000	6	240
19	800000	100000	3000000	12	220000	50000	580000	8	240
20	900000	100000	3000000	12	220000	50000	580000	8	240
21	600000	100000	3000000	12	220000	50000	580000	8	240
22	850000	100000	3000000	12	340000	50000	580000	8	240
23	900000	100000	3000000	12	280000	50000	580000	8	240
24	400000	100000	1000000	12	330000	50000	580000	8	240
25	1000000	100000	3000000	12	270000	50000	580000	8	240

Table 47. Parameter value assumptions for E_{FWD} analysis – SH 99

	ACC pavement (Layer 1)				**Aggregate Base (Layer 2)**				**Treated Subgrade (Layer 3)**				**Natural Subgrade**
	Seed	Minimum	Maximum	Thi.	Seed	Minimum	Maximum	Thi.	Seed	Minimum	Maximum	Thi.	Thickness
PT	psi	psi	psi	in	psi	psi	psi	In.	psi	psi	psi	in	in
1	3000000	100000	5000000	10	30000	1000	250000	4	50000	10000	100000	11	210
2	3000000	100000	5000000	10	10000	2000	250000	6	50000	10000	100000	8	300
3	3000000	100000	5000000	10	10000	2000	250000	6	50000	10000	100000	10	300
4	3000000	100000	5000000	10	10000	2000	250000	6	50000	10000	100000	8	300
5	3000000	100000	5000000	10	10000	2000	250000	6	50000	10000	100000	8	300
6	3000000	100000	5000000	10	10000	2000	250000	6	50000	10000	100000	8	300
7	1000000	100000	5000000	10	10000	2000	550000	6	50000	10000	100000	8	300
8	3000000	100000	5000000	10	40000	10000	550000	8	50000	10000	100000	12	280
9	3000000	100000	5000000	10	10000	2000	550000	6	50000	10000	100000	8	280
10	3000000	100000	5000000	10	10000	2000	550000	6	50000	10000	100000	8	340
11	3000000	100000	5000000	10	100000	10000	550000	6	50000	10000	100000	8	340
12	3000000	100000	5000000	8	40000	2000	550000	6	50000	10000	100000	10	400
13	3000000	100000	5000000	8	40000	2000	550000	6	50000	10000	100000	10	340
14	3000000	100000	5000000	10	40000	2000	550000	6	50000	10000	100000	8	300
15	3000000	100000	5000000	10	40000	2000	550000	6	50000	10000	100000	8	300
16	2500000	100000	5000000	8	10000	2000	550000	5	80000	10000	100000	7	240
17	2500000	100000	5000000	8	10000	2000	550000	5	80000	10000	300000	7	360
18	2500000	100000	5000000	9	10000	2000	550000	6	80000	10000	300000	8	360
19	2500000	100000	5000000	10	10000	2000	550000	6	80000	10000	300000	8	300
20	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	280
21	900000	100000	5000000	9	10000	2000	550000	6	80000	10000	300000	8	300
22	900000	100000	5000000	10	10000	2000	550000	6	80000	10000	300000	8	340
23	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	340
24	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
25	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360

	ACC pavement (Layer 1)				Aggregate Base (Layer 2)				Treated Subgrade (Layer 3)				Natural Subgrade
26	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
27	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
28	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
29	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
30	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
31	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
32	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
33	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
34	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
35	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
36	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
37	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
38	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
39	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
40	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
41	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
42	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
43	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	100000	10	340
44	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360
45	900000	100000	5000000	10	10000	2000	550000	6	50000	10000	300000	8	360

Table 48. Parameter value assumptions for E_{FWD} analysis – US 59

	ACC pavement (Layer 1)				**Aggregate Base (Layer 2)**				**Stabilized Subgrade (Layer 3)**				**Natural Sub.**
	Seed	Minimum	Maximum	Thi.	Seed	Minimum	Maximum	Thi.	Seed	Minimum	Maximum	Thi.	Thi.
PT	psi	psi	psi	in.	psi	psi	psi	in.	psi	psi	psi	in.	in.
1	800000	200000	3000000	10	50000	10000	1000000	10	100000	30000	200000	8	240
2	750000	200000	2000000	10	20000	4000	500000	10	100000	10000	800000	8	240
3	550000	100000	5000000	10	30000	10000	200000	10	80000	10000	300000	8	240
4	450000	200000	2000000	10	20000	4000	500000	10	70000	10000	800000	8	240
5	850000	100000	5000000	10	20000	10000	200000	10	90000	10000	300000	8	300
6	800000	200000	2000000	10	30000	4000	500000	10	120000	10000	800000	8	240
7	850000	100000	5000000	10	20000	10000	200000	10	90000	10000	300000	8	300
8	800000	200000	2000000	10	20000	4000	500000	10	90000	10000	800000	8	240
9	800000	200000	2000000	10	20000	4000	500000	10	90000	10000	800000	8	240
10	800000	100000	5000000	10	20000	4000	90000	10	40000	10000	500000	12	120
11	800000	200000	3000000	10	20000	4000	500000	10	90000	30000	200000	8	140
12	800000	100000	5000000	10	30000	10000	500000	10	90000	10000	300000	8.3	95
13	800000	100000	5000000	10	40000	10000	500000	10	90000	10000	300000	8.6	110
14	700000	100000	5000000	10	20000	4000	100000	6	80000	10000	300000	6	160
15	800000	200000	2000000	10	40000	4000	300000	10	50000	10000	300000	8	110
16	800000	100000	5000000	10	40000	10000	500000	8	80000	10000	750000	8	110
17	700000	400000	6000000	10	30000	10000	80000	10	100000	50000	800000	8	80
18	600000	100000	5000000	10	30000	10000	200000	8	80000	10000	200000	10	110
19	800000	100000	5000000	10	30000	10000	500000	10	80000	10000	750000	9.5	80
20	700000	200000	2000000	10	35000	10000	500000	10	200000	40000	500000	8	100
21	800000	100000	5000000	10	30000	10000	500000	10	80000	10000	750000	9	180
22	700000	100000	2000000	12	20000	10000	200000	8	80000	10000	200000	6	100
23	700000	400000	6000000	10	20000	4000	80000	10	110000	40000	250000	8	240
24	700000	100000	5000000	12	20000	4000	200000	8	80000	10000	200000	10	110
25	560000	400000	6000000	10	20000	4000	80000	10	95000	40000	450000	8	240

	ACC pavement (Layer 1)				Aggregate Base (Layer 2)				Stabilized Subgrade (Layer 3)				Natural Sub.
26	560000	400000	6000000	10	20000	4000	80000	10	95000	40000	450000	8	240
27	560000	400000	6000000	10	50000	4000	80000	10	95000	40000	450000	8	240
28	560000	400000	6000000	10	40000	4000	80000	10	95000	40000	450000	8	240
29	560000	400000	6000000	10	40000	4000	80000	10	95000	40000	450000	8	240
30	560000	400000	6000000	10	30000	4000	80000	10	95000	40000	450000	8	240
31	560000	400000	6000000	10	20000	4000	80000	10	95000	40000	450000	8	240

Table 49. Parameter value assumptions for E_{FWD} analysis – US 75 SB

	ACC pavement (Layer 1)				**Aggregate Base (Layer 2)**				**Treated Subgrade (Layer 3)**				**Natural Sub.**
	Seed	Minimum	Maximum	Thi.	Seed	Minimum	Maximum	Thi.	Seed	Minimum	Maximum	Thi.	Thi.
PT	psi	psi	psi	in	psi	psi	psi	in	psi	psi	psi	in	in
1	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	210
2	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	210
3	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	210
4	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	250
5	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	250
6	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	250
7	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	250
8	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	250
9	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	250
10	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	250
11	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	250
12	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	250
13	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	250
14	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	200
15	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	200
16	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	200
17	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	200
18	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	250
19	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	250
20	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	150
21	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	150
22	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	180
23	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	150
24	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	150
25	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	150

	ACC pavement (Layer 1)				Aggregate Base (Layer 2)				Treated Subgrade (Layer 3)				Natural Sub.
26	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	150
27	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	150
28	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	150
29	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	150
30	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	150
31	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	160
32	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	160
33	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	160
34	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	200
35	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	200
36	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	200
37	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	200
38	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	200
39	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	200
40	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	260
41	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	200
42	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	200
43	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	250
44	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	200
45	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	200
46	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	200
47	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	260
48	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	260
49	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	260
50	1000000	500000	5000000	14	10000	1000	250000	2	80000	10000	300000	4	260

Table 50. Parameter value assumptions for E_{FWD} analysis – K 7

	ACC pavement (Layer 1)				**Treated Subgrade (Layer 2)**				**Natural Sub.**
	Seed	Minimum	Maximum	Thi.	Seed	Minimum	Maximum	Thi.	Thi.
PT	psi	psi	psi	in.	psi	psi	psi	in.	in.
1	1000000	100000	5000000	8	100000	10000	200000	10	290
2	1000000	100000	5000000	8	100000	10000	200000	10	290
3	1000000	100000	5000000	8	100000	10000	200000	10	290
4	1000000	100000	5000000	8	100000	10000	200000	10	290
5	1000000	100000	5000000	8	100000	10000	200000	10	390
6	1000000	100000	5000000	8	100000	10000	200000	10	390
7	1000000	100000	5000000	8	100000	10000	200000	10	390
8	1000000	100000	5000000	8	100000	10000	200000	10	390
9	1000000	100000	5000000	8	100000	10000	200000	10	390
10	1000000	100000	5000000	8	100000	10000	200000	10	390
11	1000000	100000	5000000	8	100000	10000	200000	10	390
12	1000000	100000	5000000	8	100000	10000	200000	10	390
13	1000000	100000	5000000	8	100000	10000	200000	10	390
14	1000000	100000	5000000	8	100000	10000	200000	10	390
15	1000000	100000	5000000	8	100000	10000	200000	10	390
16	1000000	100000	5000000	8	100000	10000	200000	10	390
17	1000000	100000	5000000	8	100000	10000	200000	10	390
18	1000000	100000	5000000	8	100000	10000	200000	10	390
19	1000000	100000	5000000	8	100000	10000	200000	10	390
20	1000000	100000	5000000	8	100000	10000	200000	10	390
21	1000000	100000	5000000	8	100000	10000	200000	10	390
22	1000000	100000	5000000	8	100000	10000	200000	10	390
23	1000000	100000	5000000	9	100000	10000	200000	10	390

	ACC pavement (Layer 1)				Treated Subgrade (Layer 2)				Natural Sub.
24	1000000	100000	5000000	8	100000	10000	200000	10	390
25	1000000	100000	5000000	8	100000	10000	200000	10	390
26	1000000	100000	5000000	8	100000	10000	200000	10	390
27	1000000	100000	5000000	8	100000	10000	200000	10	390
28	1000000	100000	5000000	8	100000	10000	200000	10	390
29	1000000	100000	5000000	7	50000	10000	200000	10	200
30	1000000	100000	5000000	9	100000	10000	200000	10	490
31	1000000	100000	5000000	9	100000	10000	200000	10	490

APPENDIX C. SUMMARY OF FIELD TEST RESULTS

Table 51. Summary of test results from in situ testing – SH 121

	Flex Base	**Stabilized Subgrade**					**Natural Sub.**	**FWD Def.**
	E_{LWD}	CBR	E_{FWD}	E_{LWD}	E_{V1}	E_{V2}	E_{FWD}	D_0
PT	MPa	%	MPa	MPa	MPa	MPa	MPa	mm
1	—	—	1112	—	—	—	262	0.31
2	—	—	1313	—	—	—	198	0.34
3	—	—	1298	—	—	—	218	0.28
4	83	119	1620	51	140	360	169	0.35
5	—	—	2022	—	—	—	265	0.31
6	—	—	1124	—	—	—	204	0.42
7	140	—	297	87	282	338	152	0.63
8	—	—	2419	—	—	—	285	0.30
9	—	—	575	—	—	—	245	0.36
10	—	—	779	—	—	—	406	0.27
11	125	—	582	70	—	—	356	0.20
12	—	—	728	—	—	—	274	0.29
13	—	—	867	—	—	—	290	0.15
14	—	—	1077	—	—	—	340	0.30

Table 52. Summary of test results from in situ testing – FM 1709

	Stabilized Subgrade						**Natural Subgrade**		**FWD Def.**
	CBR	E_{V1}	E_{V2}	E_{FWD}	E_{LWD}	Thi.	E_{FWD}	CBR	D_0
PT	%	MPa	MPa	MPa	MPa	mm	MPa	%	mm
1	53	129	184	129	240	100	74	24	0.63
2	—	—	—	385	—	—	121	—	0.45
3	—	—	—	237	—	—	103	—	0.49
4	—	—	—	287	—	—	186	—	0.32
5	—	—	—	609	—	—	112	—	0.50
6	—	—	—	171	—	—	95	—	0.50
7	—	—	—	550	—	—	120	—	0.34
8	—	—	—	802	—	—	208	—	0.36

Table 53. Summary of test results from in situ testing – US 287

	Base		**Stabilized Subgrade**						**Natural Subgrade**		**FWD Def.**
	CBR	E_{LWD}	CBR	E_{FWD}	E_{LWD}	E_{V1}	E_{V2}	Thi.	E_{FWD}	CBR	D_0
PT	MPa	MPa	%	MPa	MPa	MPa	MPa	mm	MPa	%	mm
1	—	—	—	125	—	—	—	—	84	—	0.50
2	—	—	—	346	—	—	—	—	108	—	0.50
3	—	—	—	1223	—	—	—	—	122	—	0.24
4	—	—	—	437	—	—	—	—	133	—	0.27
5	—	—	—	1330	—	—	—	—	120	—	0.28
6	—	—	—	2063	—	—	—	—	137	—	0.17
7	—	—	—	1327	—	—	—	—	131	—	0.27
8	—	—	—	1849	—	—	—	—	121	—	0.25
9	—	—	—	276	—	—	—	—	119	—	0.29
10	—	—	—	1643	—	—	—	—	131	—	0.23
11	—	—	—	375	—	—	—	—	94	—	0.39
12		107	150	842	65	150	235	400	99	22	0.38
13	—	—	—	1997	—	—	—	—	123	—	0.35
14	—	—	—	1807	—	—	—	—	106	—	0.25
15	60	—	—	570	—	—	—	—	99	—	0.27
16	133	—	175	353	—	—	—	—	105	—	0.51
17	—	—	—	372	—	—	—	—	93	—	0.46
18	—	—	—	183	—	—	—	—	88	—	0.36
19	—	—	—	481	—	—	—	—	95	—	0.52

Table 54. Summary of test results from in situ testing – US 183

	Stabilized Subgrade						Natural Subgrade			FWD Def.	
	CBR	E_{FWD}	E_{LWD}	E_{V1}	E_{V2}	Thi.	E_{FWD}	E_{LWD}	CBR	$D_{0\text{-Cor.}}$	D_0
PT	%	MPa	MPa	MPa	MPa	mm	MPa	MPa	%	mm	mm
1	214	2606	—	—	—	237	167	—	34	0.12	0.15
2	—	1089	—	—	—	—	139	—	—	0.15	0.20
3	—	1475	—	—	—	—	131	—	—	0.13	0.17
4	—	815	—	—	—	—	109	—	—	0.16	0.20
5	—	2076	—	—	—	—	140	—	—	0.15	0.20
6	—	1614	—	—	—	—	131	—	—	0.15	0.19
7	—	1610	—	—	—	—	140	—	—	0.15	0.19
8	147	1670	164	317	592	213	137	—	36	0.15	0.20
9	57	841	—	—	—	104	107	—	21	0.24	0.32
10	—	2000	—	—	—	—	120	—	—	0.17	0.22
11	—	1928	—	—	—	—	141	—	—	0.15	0.20
12	115	1706	—	—	—	149	139	—	23	0.17	0.22
13	—	2306	—	—	—	—	166	—	—	0.13	0.17
14	—	2347	—	—	—	—	150	—	—	0.13	0.18
15	—	2321	—	—	—	—	182	—	—	0.12	0.16
16	—	2399	—	—	—	—	160	—	—	0.13	0.18
17	—	1372	—	—	—	—	154	—	—	0.14	0.19
18	—	1581	—	—	—	—	137	—	—	0.16	0.21
19	—	1621	—	—	—	—	140	—	—	0.15	0.20
20	—	1505	—	—	—	—	141	—	—	0.13	0.18
21	—	1552	—	—	—	—	146	—	—	0.16	0.21
22	—	2361	—	—	—	—	146	—	—	0.13	0.18
23	—	1947	—	—	—	—	161	—	—	0.12	0.16
24	—	2256	—	—	—	—	171	—	—	0.15	0.20
25	—	1858	—	—	—	—	146	—	—	0.13	0.17
26	—	—	—	—	—	—	—	25	—	—	—
27	—	—	—	—	—	—	—	17	—	—	—
28	—	—	—	—	—	—	—	16	—	—	—

Table 55. Summary of test results from in situ testing – SH 99

	Stabilized Subgrade						Natural Subgrade			FWD Deflection	
	CBR	E_{FWD}	E_{LWD}	E_{V1}	E_{V2}	Thi.	E_{FWD}	E_{LWD}	CBR	D_0	$D_{0\text{-Cor.}}$
PT	%	MPa	MPa	MPa	MPa	mm	MPa	MPa	%	mm	mm
1	175	337	—	—	—	211	244	—	24	0.13	0.18
2	—	366	—	—	—	—	312	—	—	0.11	0.16
3	—	390	—	—	—	—	251	—	—	0.12	0.18
4	—	324	—	—	—	—	239	—	—	0.13	0.19
5	—	433	—	—	—	—	267	—	—	0.12	0.17
6	—	330	—	—	—	—	270	—	—	0.11	0.16
7	—	276	—	—	—	—	245	—	—	0.16	0.23
8	—	289	—	—	—	—	241	—	—	0.12	0.18
9	—	417	—	—	—	—	270	—	—	0.12	0.17
10	—	348	—	—	—	—	191	—	—	0.18	0.25
11	—	412	—	—	—	—	222	—	—	0.13	0.19
12	—	323	—	—	—	—	197	—	—	0.19	0.27
13	—	273	—	—	—	—	220	—	—	0.14	0.20
14	—	308	—	—	—	—	185	—	—	0.15	0.21
15	—	273	—	—	—	—	251	—	—	0.11	0.16
16	—	290	—	—	—	—	258	—	—	0.12	0.17
17	—	458	—	—	—	—	249	—	—	0.13	0.19
18	—	496	—	—	—	—	342	—	—	0.13	0.19
19	—	637	—	—	—	—	233	—	—	0.14	0.20
20	—	1000	—	—	—	—	268	—	—	0.11	0.16
21	—	268	—	—	—	—	234	—	—	0.17	0.24
22	—	328	—	—	—	—	261	—	—	0.14	0.20
23	—	320	—	—	—	—	252	—	—	0.14	0.20
24	—	389	—	—	—	—	254	—	—	0.14	0.20
25	—	419	—	—	—	—	240	—	—	0.15	0.21
26	—	273	—	—	—	—	195	—	—	0.16	0.23
27	—	533	—	—	—	—	232	—	—	0.17	0.24
28	—	454	—	—	—	—	208	—	—	0.16	0.23
29	—	245	—	—	—	—	163	—	—	0.18	0.26
30	—	224	—	—	—	—	200	—	—	0.15	0.22
31	—	297	—	—	—	—	205	—	—	0.16	0.23
32	—	459	—	—	—	—	232	—	—	0.16	0.22
33	—	296	—	—	—	—	214	—	—	0.17	0.24
34	—	554	—	—	—	—	234	—	—	0.16	0.23
35	—	324	—	—	—	—	249	—	—	0.15	0.22
36	—	317	—	—	—	—	264	—	—	0.15	0.21
37	—	390	—	—	—	—	238	—	—	0.14	0.21
38	—	268	—	—	—	—	243	—	—	0.15	0.22

	Stabilized Subgrade						Natural Subgrade			FWD Deflection	
39	—	413	—	—	—	—	245	—	—	0.15	0.21
40	—	312	—	—	—	—	219	—	—	0.18	0.25
41	77	260	—	—	—	176	229	—	36	0.17	0.24
42		271	—	—	—	—	232	—	—	0.16	0.23
43	156	314	—	—	—	246	198	—	—	0.18	0.26
44	79	425	—	—	—	256	277	—	52	0.13	0.19
45	30	263	107	63	149	213	233	—	23	0.15	0.21
46	—	—	—	—	—	—	—	16	29	—	—

Table 56. Summary of test results from in situ testing – US 59

	Base	Stabilized Subgrade						Natural Subgrade			FWD Deflection	
	E_{LWD}	CBR	E_{FWD}	E_{LWD}	E_{V1}	E_{V2}	Thi.	E_{FWD}	E_{LWD}	CBR	$D_{0\text{-Cor.}}$	D_0
PT	MPa	%	MPa	MPa	MPa	MPa	mm	MPa	MPa	%	mm	mm
1	—	—	994	—	—	—	—	339	—	—	0.21	0.20
2	—	—	646	—	—	—	—	339	—	—	0.22	0.20
3	—	—	1400	—	—	—	—	430	—	—	0.30	0.28
4	—	141	1054	—	—	—	96	373	—	30	0.27	0.25
5	—	—	700	—	—	—	—	425	—	—	0.20	0.18
6	—	—	978	—	—	—	—	265	—	—	0.21	0.19
7	—	—	586	—	—	—	—	206	—	—	0.23	0.21
8	—	—	640	—	—	—	—	244	—	—	0.24	0.22
9	—	—	655	—	—	—	—	563	—	—	0.23	0.22
10	—	—	562	—	—	—	—	489	—	—	0.19	0.18
11	—	—	776	—	—	—	—	420	—	—	0.19	0.18
12	—	105	1782	—	—	—	113	536	—	19	0.15	0.14
13	—	—	1411	—	—	—	—	525	—	—	0.15	0.14
14	—	—	731	—	—	—	—	523	—	—	0.13	0.12
15	—	—	658	—	—	—	—	382	—	—	0.18	0.16
16	—	166	572	—	—	—	180	409	—	19	0.15	0.14
17	—	—	642	—	—	—	—	509	—	—	0.15	0.14
18	—	—	649	—	—	—	—	398	—	—	0.17	0.16
19	—	—	1230	—	—	—	—	531	—	—	0.17	0.15
20	—	196	1365	—	—	—	251	447		32	0.15	0.14
21	—	—	575	—	—	—	—	543	—	—	0.15	0.14
22	—	—	627	—	—	—	—	392	—	—	0.17	0.16
23	—	—	689	—	—	—	—	351	—	—	0.19	0.18
24	126	106	933	105	177	261	124	210	20	14	0.23	0.21
25	—	—	879	—	—	—	—	280	—	—	0.27	0.25
26	—	—	567	—	—	—	—	244	—	—	0.23	0.22
27	—	—	489	—	—	—	—	244	—	—	0.20	0.19
28	—	119	613	—	—	—	136	311	—	21	0.21	0.19
29	—	—	644	—	—	—	—	344	—	—	0.20	0.19
30	—	—	692	—	—	—	—	343	—	—	0.22	0.21
31	—	—	664	—	—	—	—	216	—	—	0.29	0.27
32	—	—	—	—	—	—	—	—	33.7	27	—	—
33	—	—	—	—	—	—	—	—	17.1	—	—	—
34	—	—	—	—	—	—	—	—	25.1	—	—	—

Table 57. Summary of test results from in situ testing – US 75 SB

	Stabilized Subgrade						Natural Subgrade		FWD Deflection	
	CBR	E_{FWD}	E_{LWD}	E_{V1}	E_{V2}	Thi	E_{FWD}	CBR	D_0	$D_{0\text{-Cor.}}$
PT	%	MPa	MPa	MPa	MPa	mm	MPa	%	mm	mm
1	—	926	—	—	—	—	436	—	0.10	0.16
2	—	818	—	—	—	—	376	—	0.11	0.16
3	—	945	—	—	—	—	437	—	0.11	0.16
4	82	921	—	—	—	95	460	29	0.08	0.12
5	—	879	—	—	—	—	427	—	0.08	0.13
6	—	1260	—	—	—	—	420	—	0.09	0.14
7	—	1084	—	—	—	—	452	—	0.08	0.13
8	—	1072	—	—	—	—	387	—	0.09	0.14
9	—	871	—	—	—	—	467	—	0.09	0.13
10	—	1168	—	—	—	—	369	—	0.10	0.16
11	9	453	—	—	—	—	297	13	0.14	0.22
12	—	1047	—	—	—	—	319	—	0.12	0.18
13	—	1019	—	—	—	—	308	—	0.12	0.18
14	—	396	—	—	—	—	292	—	0.14	0.20
15	—	472	—	—	—	—	279	—	0.14	0.21
16	—	428	—	—	—	—	266	—	0.14	0.21
17	—	1604	—	—	—	—	298	—	0.12	0.19
18	14	945	31	7	15	120	350	6	0.11	0.17
19		949	—	—	—		313		0.14	0.21
20	12	573	—	—	—	140	270	6	0.16	0.25
21	—	487	—	—	—	—	275	—	0.15	0.22
22	—	511	—	—	—	—	288	—	0.15	0.23
23	—	588	—	—	—	—	299	—	0.15	0.23
24	—	462	—	—	—	—	317	—	0.12	0.19
25	—	476	—	—	—	—	218	—	0.14	0.22
26	—	488	—	—	—	—	218	—	0.16	0.24
27		531	—	—	—	—	229	—	0.16	0.24
28	18	572	—	—	—	—	226	6	0.17	0.26
29	—	508	—	—	—	—	268	—	0.15	0.22
30	—	510	—	—	—	—	271	—	0.13	0.20
31	—	545	—	—	—	—	311	—	0.14	0.22
32	—	482	—	—	—	—	337	—	0.11	0.17
33	—	534	—	—	—	—	351	—	0.12	0.18
34	54	1640	—	—	—	—	287	9	0.12	0.18
35		461	—	—	—	—	266		0.14	0.21
36	—	434	—	—	—	—	297	—	0.13	0.20
37	—	452	—	—	—	—	301	—	0.13	0.20
38	—	422	—	—	—	—	291	—	0.13	0.19

	Stabilized Subgrade						Natural Subgrade		FWD Deflection	
39	—	459	—	—	—	—	272	—	0.14	0.21
40	—	779	—	—	—	—	372	—	0.10	0.16
41	—	475	—	—	—	—	247	—	0.15	0.23
42	—	415	—	—	—	—	262	—	0.14	0.22
43	—	681	—	—	—	—	297	—	0.12	0.18
44	—	742	—	—	—	—	359	—	0.12	0.18
45	19	400	—	—	—	—	259	7	0.14	0.21
46	—	451	—	—	—	—	293	—	0.14	0.21
47	—	835	—	—	—	—	363	—	0.10	0.15
48	—	671	—	—	—	—	307	—	0.13	0.20
49	—	835	—	—	—	—	420	—	0.10	0.16
50	—	896	—	—	—	—	428	—	0.11	0.16

Table 58. Summary of test results from in situ testing – US 75 NB

	Base	Stabilized Subgrade					Natural Subgrade		FWD Def.	FWD Modulus
	E_{LWD}	CBR	E_{V1}	E_{V2}	E_{LWD}	Thi.	E_{LWD}	CBR	D_0	E_{sg}
PT	MPa	%	MPa	MPa	MPa	mm	MPa	%	mm	MPa
1	—	—	—	—	—	—	—	—	0.13	156
2	—	—	—	—	—	—	—	—	0.20	162
3	—	13	—	—	—	132	—	7	0.14	154
4	—	—	—	—	—	—	—	—	0.20	161
5	—	—	—	—	—	—	—	—	0.12	180
6	—	—	—	—	—	—	—	—	0.16	191
7	—	—	—	—	—	—	—	—	0.11	187
8	—	—	—	—	—	—	—	—	0.14	225
9	—	—	—	—	—	—	—	—	0.12	172
10	—	—	—	—	—	—	—	—	0.17	191
11	—	17	—	—	—	115	—	9	0.11	192
12	—	—	—	—	—	—	—	—	0.15	208
13	—	—	—	—	—	—	—	—	0.13	156
14	—	—	—	—	—	—	—	—	0.17	177
15	—	—	—	—	—	—	—	—	0.13	163
16	—	—	—	—	—	—	—	—	0.17	167
17	—	—	—	—	—	—	—	—	0.12	158
18	—	—	—	—	—	—	—	—	0.18	175
19	—	—	—	—	—	—	—	—	0.12	165
20	—	—	—	—	—	—	—	—	0.16	196
21	—	—	—	—	—	—	—	—	0.13	147
22	—	—	—	—	—	—	—	—	0.17	177
23	—	—	—	—	—	—	—	—	0.13	153
24	—	—	—	—	—	—	—	—	0.19	175
25	81	—	81	119	91	—	—	—	0.12	176
26	—	—	—	—	—	—	—	—	0.17	189
27	—	—	—	—	—	—	—	—	0.13	158
28	—	—	—	—	—	—	—	—	0.16	181
29	—	—	—	—	—	—	—	—	0.12	164
30	—	—	—	—	—	—	—	—	0.18	180
31	—	26	—	—	—	125	—	9	0.14	150
32	—	—	—	—	—	—	—	—	0.18	170
33	—	—	—	—	—	—	—	—	0.13	157
34	—	—	—	—	—	—	—	—	0.17	172
35	—	—	—	—	—	—	—	—	0.11	143
36	—	—	—	—	—	—	—	—	0.18	167
37	—	—	—	—	—	—	—	—	0.13	153
38	—	—	—	—	—	—	—	—	0.17	168

	Base	Stabilized Subgrade					Natural Subgrade		FWD Def.	FWD Modulus
39	—	—	—	—	—	—	—	—	0.13	152
40	—	—	—	—	—	—	—	—	0.18	164
41	—	—	—	—	—	—	—	—	0.11	180
42	—	—	—	—	—	—	—	—	0.16	187
43	—	26	—	—	—	153	—	6	0.10	178
44	—	—	—	—	—	—	—	—	0.14	210
45	—	—	—	—	—	—	—	—	0.12	164
46	—	—	—	—	—	—	—	—	0.16	180
47	—	—	—	—	—	—	—	—	0.11	159
48	—	—	—	—	—	—	—	—	0.17	179
49	—	19	—	—	—	116	—	8	0.11	159
50	—	—	—	—	—	—	—	5	0.16	182

Table 59. Summary of test results from in situ testing – K 7

	Stabilized Subgrade						Natural Subgrade			FWD Deflection	
	CBR	E_{FWD}	E_{LWD}	E_{V1}	E_{V2}	Thi.	E_{FWD}	E_{LWD}	CBR	D_0	$D_{0\text{-Cor.}}$
PT	%	MPa	MPa	MPa	MPa	mm	MPa	MPa	%	mm	mm
1	96	399	—	—	—	207	113	—	14	0.21	0.32
2	—	453	—	—	—	—	123	—	—	0.23	0.34
3	—	530	—	—	—	—	131	—	—	0.23	0.35
4	94	527	—	—	—	463	160	—	22	0.21	0.32
5	—	446	—	—	—	—	152	—	—	0.20	0.31
6	—	563	—	—	—	—	141	—	—	0.23	0.35
7	—	485	—	—	—	—	144	—	—	0.21	0.32
8	—	453	—	—	—	—	144	—	—	0.21	0.31
9	—	461	—	—	—	—	134	—	—	0.22	0.33
10	—	465	—	—	—	—	136	—	—	0.21	0.33
11	51	486	89	137	294	—	143	—	—	0.22	0.33
12	—	445	—	—	—	—	142	—	—	0.21	0.31
13	—	423	—	—	—	—	132	—	—	0.20	0.31
14	—	420	—	—	—	—	139	—	—	0.21	0.31
15	—	442	—	—	—	—	128	—	—	0.22	0.33
16	68	453	—	—	—	329	155	—	10	0.20	0.31
17	—	522	—	—	—	—	150	—	—	0.21	0.33
18	—	542	—	—	—	—	143	—	—	0.22	0.34
19	—	495	—	—	—	—	149	—	—	0.21	0.32
20	—	480	—	—	—	—	148	—	—	0.21	0.32
21	—	470	—	—	—	—	151	—	—	0.21	0.31
22	—	503	—	—	—	—	158	—	—	0.21	0.32
23	—	602	—	—	—	—	143	—	—	0.22	0.33
24	—	512	—	—	—	—	140	—	—	0.23	0.34
25	—	575	—	—	—	—	136	—	—	0.24	0.36
26	—	580	—	—	—	—	130	—	—	0.25	0.37
27	—	568	—	—	—	—	127	—	—	0.25	0.42
28	—	685	—	—	—	—	125	—	—	0.27	0.44
29	52	214	—	—	—	209	116	—	—	0.29	0.39
30	—	708	—	—	—	—	121	—	—	0.26	0.40
31	—	674	—	—	—	—	110	—	—	—	—
32	—	—	—	—	—	—	—	12	—	—	—
33	—	—	—	—	—	—	—	10	—	—	—

APPENDIX D. CONSTRUCTION RECORD

Table 60. Field nuclear density test at the US 183 site

SHRP-2

JOB

SHEET NO. OF

CALCULATED BY DATE

CHECKED BY DATE

SCALE

WASHITA CO. US-183 @ TEST LOCATION

			DEN. TEST			
					DD	WD
407+00	PROCTOR	OPT.%	% COMPAC	% M		
	103.6	21.3	102.6	19.02	106.3	126.4
414+00	103.3	21.1	101.4	17.22	108.8	127.[illegible]

8'

126.4 × .62 × 9 = 762.11

3m

Table 61. Compaction test results at the SH 99 site

DH Form 393-92

COMPACTION TEST #

Project No. NH-1-12N (005) County Sem. Div. 3

Date 2-15-2000 Tested By O.J. AASHTO T-99 Method A

COMPACTED DENSITY DETERMINATIONS

Water Added		64	64			
Gross Weight		3812.1	3872.2	3845.7		
Tare Weight		1925.3	1925.3	1925.3		
Net Weight		1886.8	1946.9	1920.4		
Wet Den. Lbs./Cu. Ft.		124.24	128.20	126.46		
Dry Den. Lbs./Cu. Ft.		107.87	109.84	106.19		109.11

MOISTURE DETERMINATIONS M-W

Dish No.		S	200	4		200
Dish - Wet		162.63	158.23	160.55		158.23
Dish - Dry		145.09	139.46	139.77		139.23
Water		17.54	18.17	20.76		19.00
Dish - Dry		145.09	139.46	139.77		139.23
Dish		29.50	30.76	31.00		30.76
Dry Soil		115.59	108.70	108.77		108.47
% Moist.		15.17	16.71	19.09		17.5

Material ______

Passing #4 sieve

Sample Wt. 7#

Mold Vol. 5.33 Kg/m 3

Mold Factor .06585

new mold.

Remarks: Soil + Flyash Final mix

Standard Dry Density 109.1 PCF

Optimum Moisture 17.5 %

DH Form 393-92

COMPACTION TEST #

Project No. NH-1-12N (005) County Sem. Div. 3

Date 1-18-2K Tested By O.J AASHTO T-99 Method A

COMPACTED DENSITY DETERMINATIONS

Water Added						
Gross Weight	3793.7	3814.2	3774.7			
Tare Weight	1815.8	1815.8	1815.8			
Net Weight	1977.9	1998.4	1958.9			
Wet Den. Lbs./Cu. Ft.	130.74	132.09	129.48			
Dry Den. Lbs./Cu. Ft.	114.70	113.75	109.40			

MOISTURE DETERMINATIONS

Dish No.	20	4	10	1		
Dish - Wet	171.48	160.25	159.50			
Dish - Dry	154.21	142.30	139.65			
Water	17.27	17.95	19.85			
Dish - Dry	154.21	142.30	139.65			
Dish	30.66	31.00	30.25	29.36		
Dry Soil	123.55	111.30	109.40			
% Moist.	13.98	16.12	18.14			

Material A/R dried

Passing #4 sieve

Sample Wt. 7#

Mold Vol. 5.33 Kg/m 3

Mold Factor .0661 Kg/m 3

Remarks: Flyash + Soil Final mix STA - 5108+00 - To 5127+00

Standard Dry Density 114.7 PCF

Optimum Moisture 13.9

Table 62. Field nuclear density test at the SH 99 site (1)

TROXLER LABORATORIES

NUCLEAR COMPACTION TEST DATA

PROJECT NHY12N(605)
JOB NUMBER
DATE
TAKEN BY A. Moore

Date	1-6-00		1-11-00	1-25-00		1-17-00	1-19-00		2-11-00	
TEST NUMBER	1	2	3	4	5	6	7	8	9	10
STATION	5092+00	5090+75	5096+75	Base line	5103+15	5103+75	5113+50	5119+33	5122+10	5126+00
OFFSET	14' Lt	Lt 24'	Lt 17'	℄	℄	2' Rt	5' Rt	4' Lt	16' Lt ℄	16' Lt ℄
ELEVATION	Sub F/Ash	Sub F/Sub	Sub F/Ash	Sub F/Ash	Sub F/Ash	Sub F/Ash	Sub F/Ash	Sub F/Ash	Sub/F Ash	Sub/F Ash
MODE & DEPTH	8"	8"	8"	8"	8"	8"	8"	8"	8"	8"
DENS. CNT.	1346	1388	1209	1240	1242	1298	1200	1170	1245	1545
WET DENS.	123.2	122.0	127.7	126.7	126.7	125.1	128.0	128.9	129.5	121.6
MSTRE. CNT.	220	218	197	198	185	182	142	153	178	173
MOISTURE	21.4	21.2	18.9	19.0	17.6	17.3	13.0	14.2	14.3	13.9
DRY DENS.	101.7	100.8	108.8	107.7	109.1	107.7	115.0	114.7	115.1	107.7
% MOISTURE	21.1	21.1	17.4	17.7	16.2	16.1	11.3	12.3	12.4	12.9
STD. DENS.	97.6	97.6	97.6	107.4	107.4	107.4	114.7	114.7	114.7	106.9
OPT. MSTRE.	24.6	24.6	24.6	19.3	19.3	19.3	13.9	13.9	13.9	11.5
% COMP.	104.2	103.3	101.3	100.3	100.5	100.3	100.2	100.0	100.4	100.7
MSTRE. CORR.	RETAKE	RETAKE								

Date	2-10-2000			2-16	3-6-00	3-10-00	3-14-00		4-7-00	4-8-00
TEST NUMBER	11	12	13	14	15	16	17	18	19	20
STATION	EW119	5160	EW Stpd	5130+75	5134+10	5042+00	5147+50	5151+08	5156+50	5160+35
OFFSET	10' Lt ℄	12 Rt ℄	.	℄	7' Lt	℄	5' Lt ℄	8' Lt ℄	5 Lt	℄
ELEVATION	Clay sub	Clay Sub	Clay Sub	Sub F/Ash	Sub F/Ash	Sub Fly/Ash	Fly/Ash Sub	Fly/Ash	Fly Ash	FLY ASH
MODE & DEPTH	6"	6"	8"	8"	8"	8"	8"	8"	8"	8"
DENS. CNT.	1821	1887	1279		1444	2185	1288	1443	1453	1456
WET DENS.	131.5	130.1	128.4	127.6	123.6	123.6	128.2	123.7	129.5	129.9
MSTRE. CNT.	179	151	162		249	241	165	223	189	184
MOISTURE	14.5	11.9	12.9	16.6	20.9	20.4	13.4	18.7	20.6	15.1
DRY DENS.	117.0	118.2	115.4	111.0	102.7	103.2	114.8	105.0	108.9	114.8
% MOISTURE	12.4	10.1	11.2	14.9	20.3	19.7	11.7	17.8	18.9	13.2
STD. DENS.	116.9	115.5	115.5	109.1	101.4	101.4	114.7	104.2	107.4	114.7
OPT. MSTRE.	12.4	12.4	12.4	17.5	21.2	21.2	13.9	19.6	17.3	13.9
% COMP.	100.1	102.3	99.9	101.8	101.3	101.8	100.1	100.8	101.4	100.4
MSTRE. CORR.										

DENSITY	MOISTURE

REMARKS:

Table 63. Field nuclear density test at the SH 99 site (2)

SUBGRADE

PROJECT: DSB-40A(424)

DATE: 5-7-99

TESTED BY: AKERS

588+86 – 594+86 57.08' x 0.67'

9 LOADS (240.13 TONS)

FLYASH/SOIL OPERATION

STANDARD COUNT

DENSITY COUNT: 2906

MOISTURE COUNT: 516

DENSITY COUNT								
TEST NO.	1	2	3					
DEPTH OF TEST	6"	6"	6"					
STATION	590+00	592+00	594+00					
LOCATION	S. BOUND LANES	S. BOUND LANES	S. BOUND LANES					
WET DENSITY	128.6	125.0	124.4					
DRY DENSITY	113.6	109.2	108.4					
% MOISTURE	13.1	14.5	14.7					
STANDARD DRY DENSITY	107.3	107.3	107.3					
OPTIMUN MOISTURE	15.9	15.9	15.9					
COMPACTION % PR	105.9	101.8	101.1					
COMPACTION REQUIRED	100	100	100					
REMARKS	COMPACTION PASSES	PASS	PASS					
	STA 589+00 TO STA 591+00	STA 591+00 STA 593+00	593+00 595+00					

Rec'd 5-[illegible]-99 Q.D.O

Table 64. Field nuclear density test at the SH 99 site (3)

NUCLEAR FIELD DENSITY WORK SHEET

SUBGRADE

S.B

594+86 – 599+05   57.08' wide x .67 deep

6 LOADS (167.21 TONS)

JECT: DS8-40A (424)

E: 5-13-99

STED BY: RK DARNeal

STANDARD COUNT

DENSITY COUNT: 2922

MOISTURE COUNT: 578

DENSITY COUNT	1014		870					
TEST NO.	1		2					
DEPTH OF TEST	BACK SCATTER		BACK SCATTER					
STATION	596+00		598+00					
LOCATION	CENTER OF INSIDE LT. LANE		CENTER OF OUTSIDE LT. LANE					
WET DENSITY	121.5		133.2					
DRY DENSITY	108.1		118.0					
% MOISTURE	12.4		12.9					
STANDARD DRY DENSITY	107.3		107.3					
OPTIMUN MOISTURE	15.9		15.9					
COMPACTION % PR	100.7%		110.00%					
COMPACTION REQUIRED	100 %		100%					
REMARKS	OK		OK					

Table 65. Field nuclear density test at the SH 99 site (4)

NUCLEAR FIELD DENSITY WORK SHEET

SUBGRADE

S.B.
599+05 - 605+10
(242.13 TONS) LOADS ?

JECT: DSB-90A (424)

TE: 5-14-99

STED BY: RK Dolmund

STANDARD COUNT

DENSITY COUNT: 2903

MOISTURE COUNT: 523

DENSITY COUNT	844		871		909		
TEST NO.	1		2		3		
DEPTH OF TEST	Back Scatter		Back Scatter		Back Scatter		
STATION	LT Lane's 600+00		602+00		604+00		
LOCATION	4' OFF ℄ OF Inside LT lane		2' 6 FF ℄ Outside RT. Lane		2' LT OF ℄ OF RT Lane, Inside Lane		
WET DENSITY	135.2		132.5		129.5		
DRY DENSITY	120.7		115.4		111.6 ~~129.5~~		
% MOISTURE	12.0		14.8		16.0		
STANDARD DRY DENSITY	107.3		107.3		107.3		
OPTIMUN MOISTURE	15.9		15.9		15.9		
COMPACTION % PR	112.0 %		107.6		104.0 %		
COMPACTION REQUIRED	100 %		100 %		100 %		
REMARKS	OK		OK		OK		

5-17-99
O.Ro.

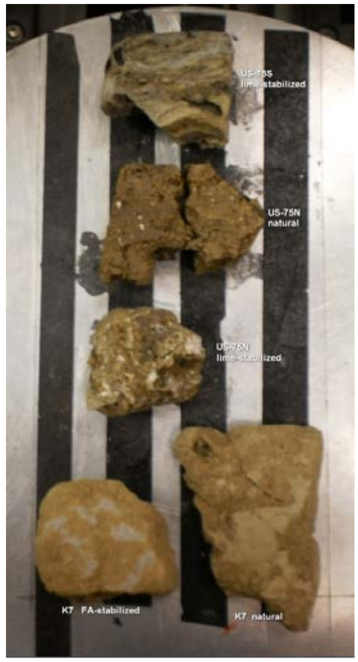

Figure 8. Prepared SEM samples from test sites in Kansas

Figure 9. White product presented in stabilized subgrade at test site of US 183

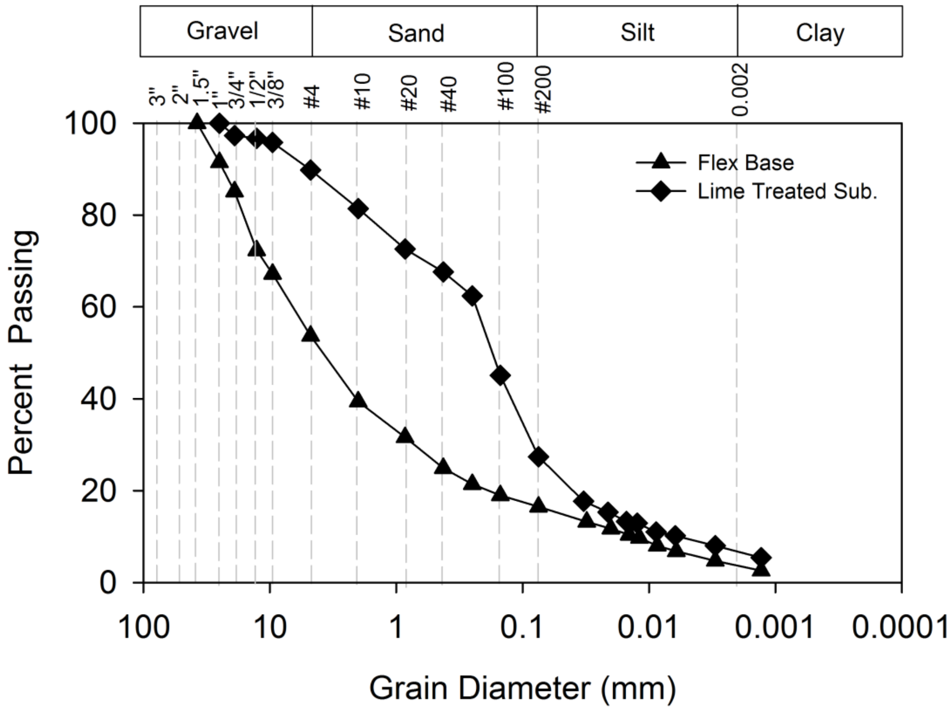

Figure 13. Particle size distribution curves for subgrade materials SH 121 Fort Worth, Texas

4.1.2.2 pH of Stabilized and Natural Subgrade

The pH value of stabilized subgrade was 9.2.

4.1.2.3 SEM Analysis

The energy dispersive spectrometry (EDS) map of stabilized subgrade is shown in Figure 14. The majority elements were silica (Si), alumina (Al), and oxygen (O). Calcium (Ca) is distributed in a few concentrated pockets. Additional elements identified include iron (Fe) and magnesium (Mg).

Figure 15 and Figure 16 compare element concentrations with different magnifications including Al, Si, O, S, Mg, Ca, K, and C for the stabilized subgrade. The sample at 30 × magnifications shows higher concentrations of Ca than that the sample at 150 × magnification. The sample at 500 × magnification shows higher concentrations of Al, O, and Si than the sample at 150× magnification. All SEM images are presented in Figure 17 and Appendix A.

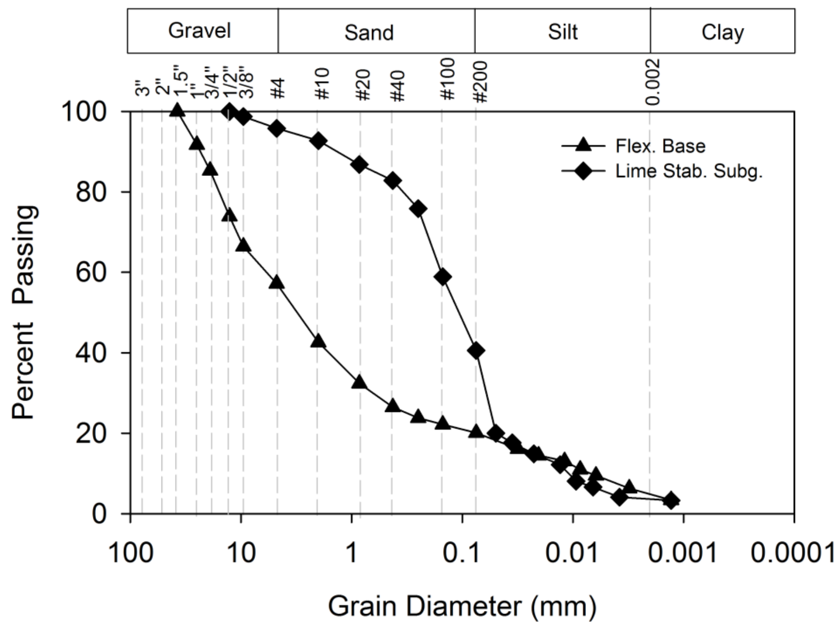

Figure 26. Particle size distribution curves for subgrade materials FM 1709 Fort Worth, Texas

4.2.2.2 pH of Stabilized and Natural Subgrade

The pH value of stabilized sample was 9.6.

4.2.2.3 SEM Analysis

The energy dispersive spectrometry (EDS) map of stabilized subgrade is shown in Figure 27 and Figure 28. The majority elements were calcium (Ca), silica (Si), alumina (Al), and oxygen (O). These elements commonly exist in lime stabilized subgrade. Additional elements were iron (Fe), potassium (K), and Sodium (Na).

Figure 29 shows element concentration in Al, Si, O, S, Mg, Ca, K, and C for stabilized subgrade. The stabilized subgrade sample has higher concentration of Si, Al, O, and Ca, and less concentration of C, Fe, and Mg. All SEM images are presented in Figure 30 and Appendix A.

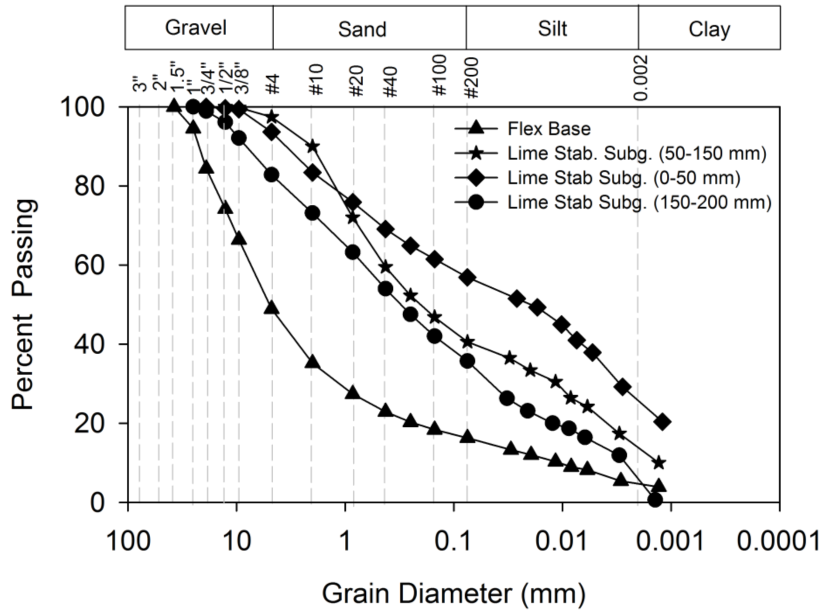

Figure 38. Particle size distribution curves for subgrade materials US 287 Mansfield, Texas

4.3.2.2 pH of Stabilized and Natural Subgrade

Table 12 shows pH values of stabilized subgrade from a depth of 0-200 mm (0-8 in.). It decreases gradually from the top to bottom of stabilized subgrade.

Table 12. Summary of pH value of subgrade US 287 Mansfield, Texas

Depth mm (in.)	pH
0-50 (0-2)	8.2
50-150 (2-6)	8.7
150-200 (6-8)	9.2

4.3.2.3 SEM Analysis

The energy dispersive spectrometry (EDS) map of stabilized subgrade is shown in Figure 39. The majority elements were calcium (Ca), silica (Si), alumina (Al), and oxygen (O). These elements commonly exist in lime stabilized subgrade. Additional elements were iron (Fe), potassium (K), and Sodium (Na).

Figure 40 and Figure 41 compares element concentration in Al, Si, O, S, Mg, Ca, K, and C for stabilized subgrade. The sample shows higher concentration of Ca, Si, Al, and O, and less concentration of Fe, S, and Mg. All SEM images are presented in Figure 42 to Figure 48 and Appendix A.

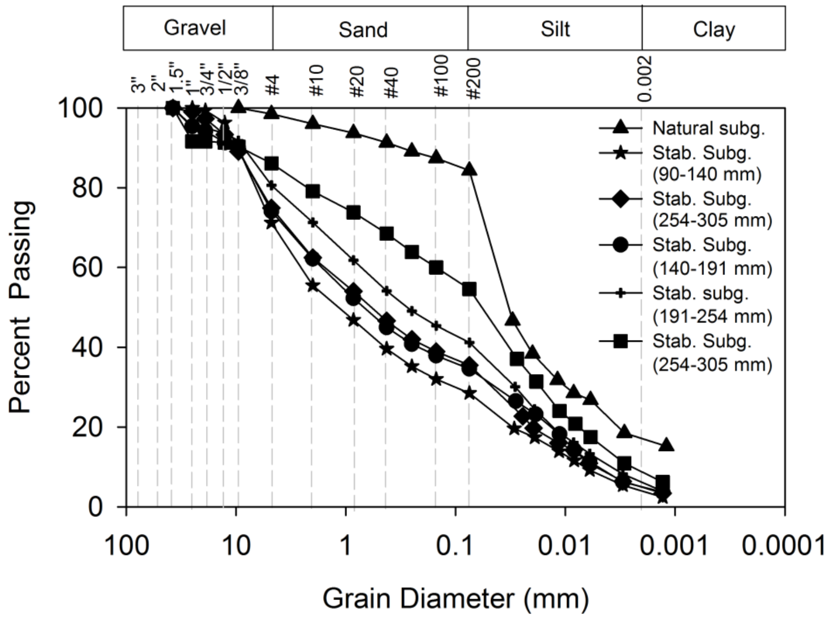

Figure 54. Particle size distribution curves for subgrade materials US 183 Clinton, Oklahoma

4.4.2.2 pH of Stabilized and Natural Subgrade

Figure 55 shows the pH profile of the subgrade layers at test point 8. The pH values of stabilized subgrade varied from 8.1 to 8.9 and for the natural subgrade from 7.9 to 8.3.

Table 20. Summary of material properties SH 99 Seminole County, Oklahoma

Parameter	Materials		
Material Description	Base	Stabilized Subgrade	Natural Subgrade
Depth mm (in.)	0-150 (0-6)	0-200 (0-8)	—
Gravel Content (%) (> 4.75mm)	64.7	6.7	3.9
Sand Content (%) (4.75mm – 75µm)	29.0	48.6	58.4
Silt Content (%) (75µm – 2µm)	5.1	35.4	27.2
Clay Content (%) (< 2µm)	1.2	9.3	10.5
Coefficient of Uniformity (Cu)	37.9	68.8	84.7
Coefficient of Curvature (Cc)	2.6	2.0	2.0
Liquid Limit, LL (%)	16.1	—	22.3
Plasticity Index, PI	4.5	N.P.	4.9
AASHTO	A-1-a	A-4-0	A-4-0
USCS	GW-GM	SM	SM
Water Content (%)	3.4	20.6	11.7

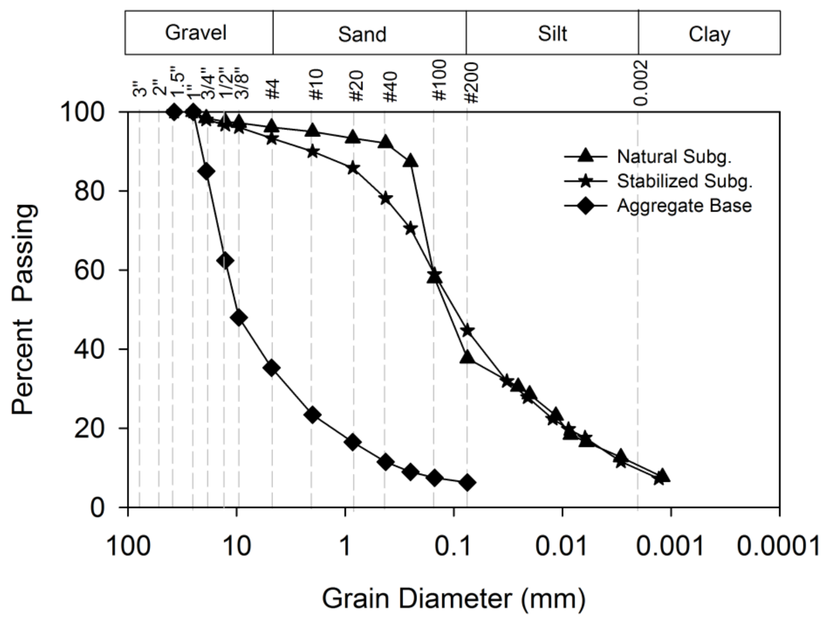

Figure 71. Particle size distribution curves for subgrade materials SH 99 Seminole County, Oklahoma

4.5.2.2 pH of Stabilized and Natural Subgrade

Table 21 shows pH values of natural and stabilized subgrade. Similar to the other test sites, the pH is higher in the stabilized layer.

Table 21. Summary of pH value of subgrade SH 99 Seminole County, Oklahoma

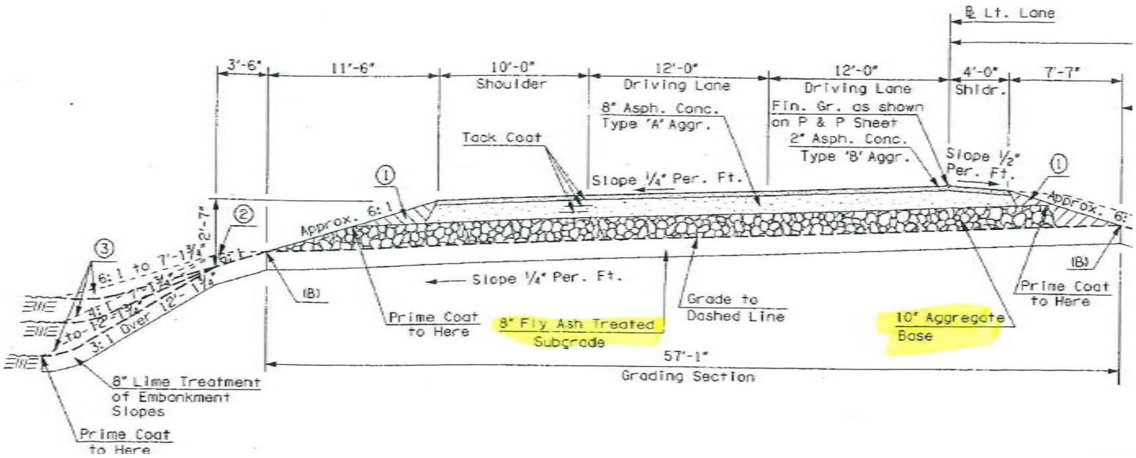

Figure 83. Typical cross section US 59 Le Flore County, Oklahoma

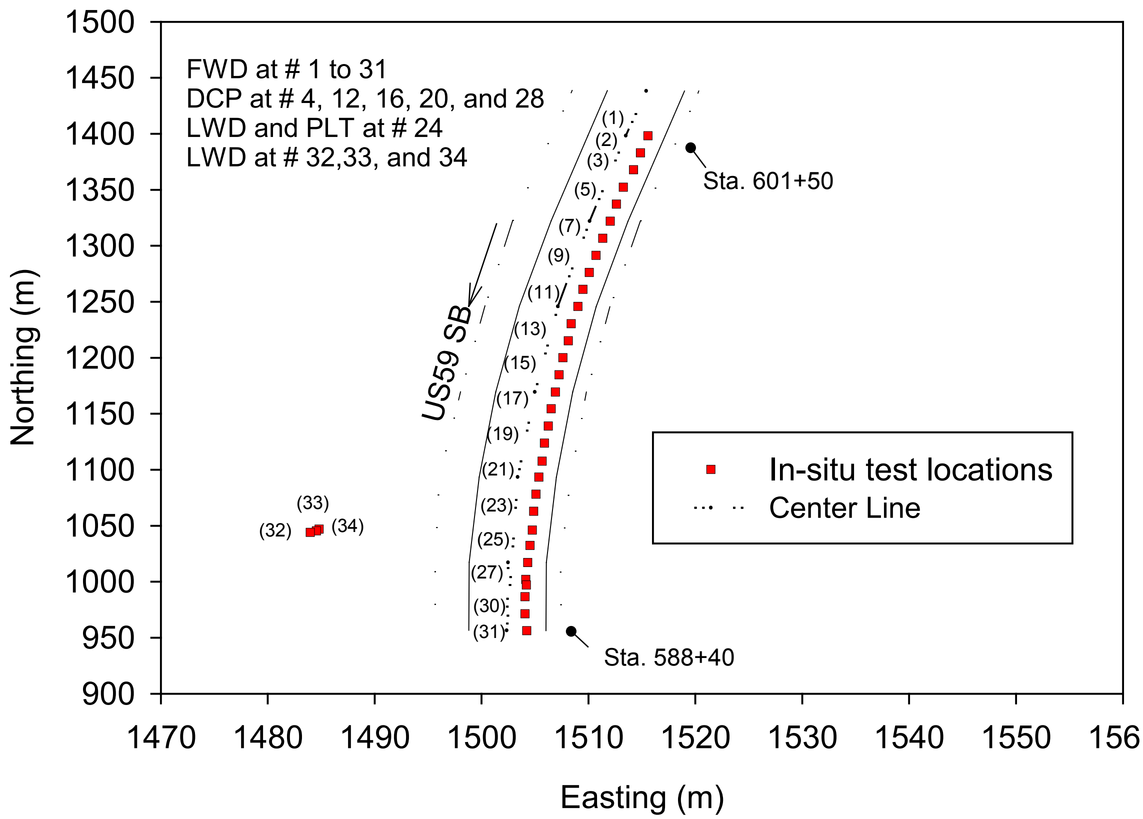

Figure 84. Test section plan layout with RTK GPS test points US 59 Le Flore County, Oklahoma

Table 25. Summary of material properties US 59 Le Flore County, Oklahoma

Parameters	Materials		
Material Description	Base	Stabilized Subgrade	Natural Subgrade
Depth mm (in.)	0-254 (0-10)	0-200 (0-8)	—
Gravel Content (%) (> 4.75mm)	49.7	16.1	3.6
Sand Content (%) (4.75mm – 75µm)	31.1	48.2	30.5
Silt Content (%) (75µm – 2µm)	15.2	31.5	37.7
Clay Content (%) (< 2µm)	4.0	4.2	28.2
Coefficient of Uniformity (C_u)	446.7	110.3	—
Coefficient of Curvature (C_c)	5.2	0.4	—
Liquid Limit, LL (%)	24.7	32.7	45.9
Plasticity Index, PI	9.7	5.6	24.7
AASHTO	A-1-b	A-4	A-4
USCS	GM	SM	ML
Water Content (%)	5.0	17.7	13.2

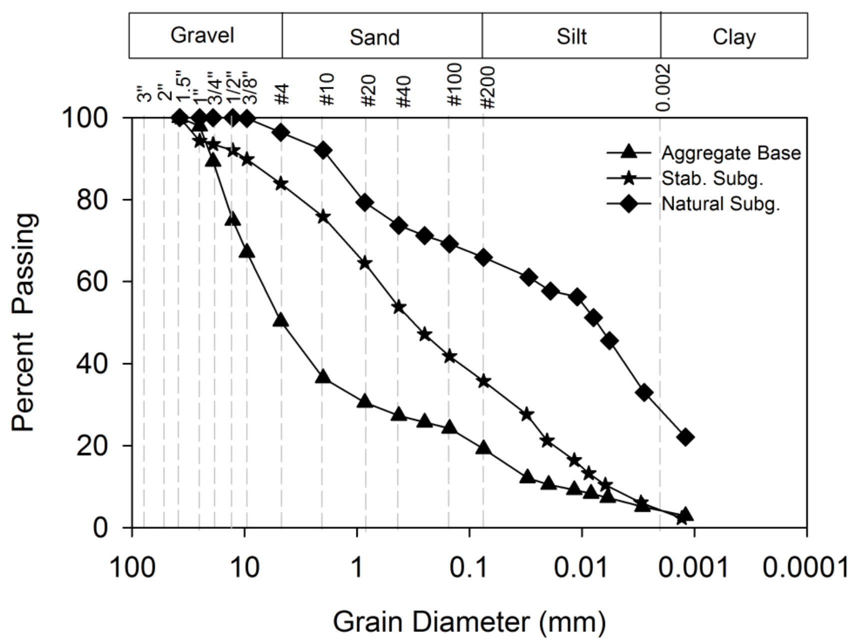

Figure 86. Particle size distribution curves for subgrade materials US 59 Le Flore County, Oklahoma

4.6.2.2 pH of Stabilized and Natural Subgrade

Table 26 provides pH values of natural and stabilized subgrade.

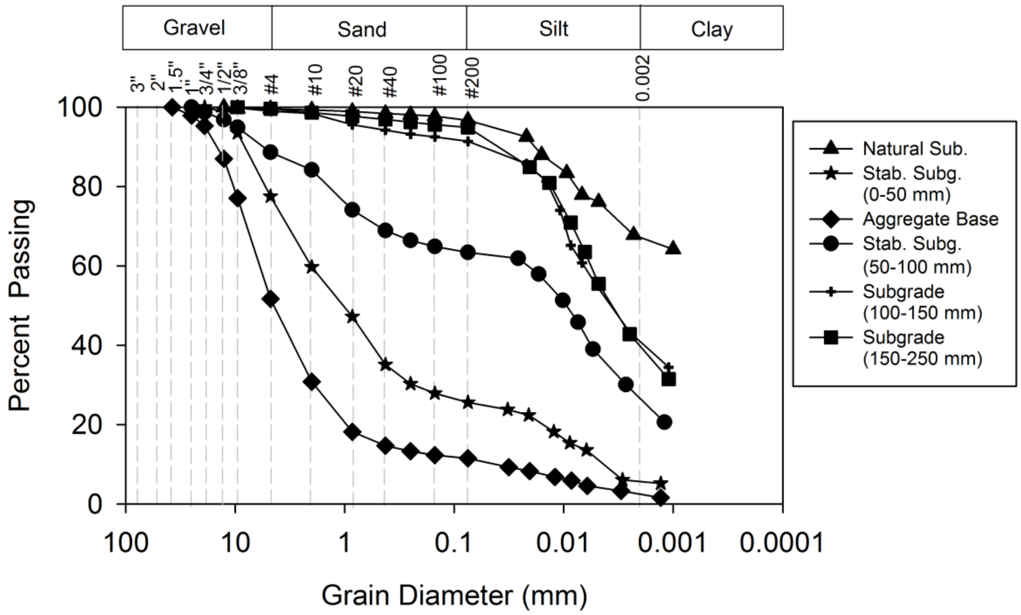

Figure 98. Particle size distribution curves for subgrade materials US 75 Osage County, Kansas

4.7.2.2 pH of Stabilized and Natural Subgrade

Figure 99 shows the pH profile of the subgrade layers at test point 8. The pH values of stabilized subgrade ranged from about 7.7 to 8.8. It gradually decreased from the top of stabilized subgrade to the bottom of stabilized subgrade. Below the stabilized subgrade, the pH values were relatively constantly to a depth of 400 mm. Then the pH value decrease from 7.5 to 6.5 to a depth of 1000 mm.

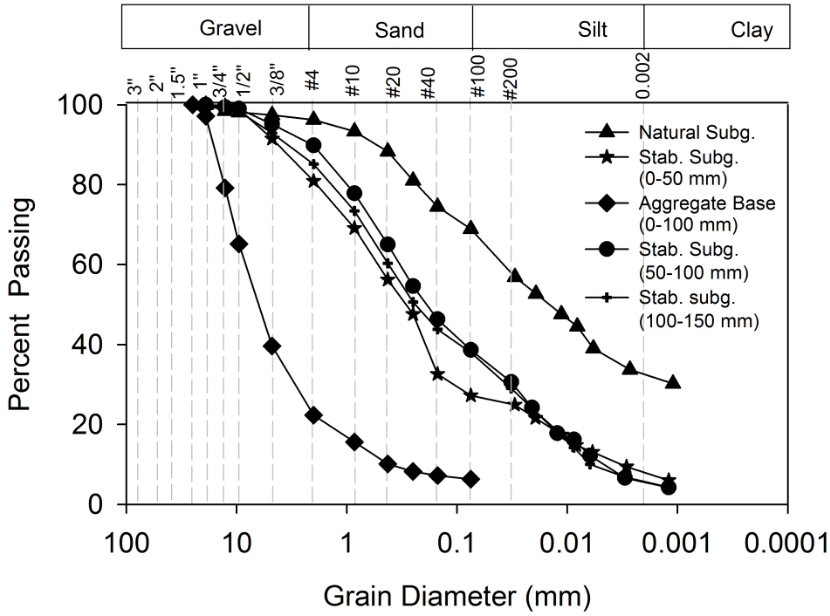

Figure 116. Particle size distribution curves for subgrade materials US 75 NB Jackson County, Kansas

4.8.2.2 pH of Stabilized and Natural Subgrade

Figure 117 shows the pH profile of subgrade materials at test point 25. The pH values of stabilized subgrade ranged from 8.7 to 9.4. pH gradually decreased from the top of the stabilized subgrade to the bottom of the stabilized subgrade. pH of natural subgrade ranged from 7.9 to 8.1.

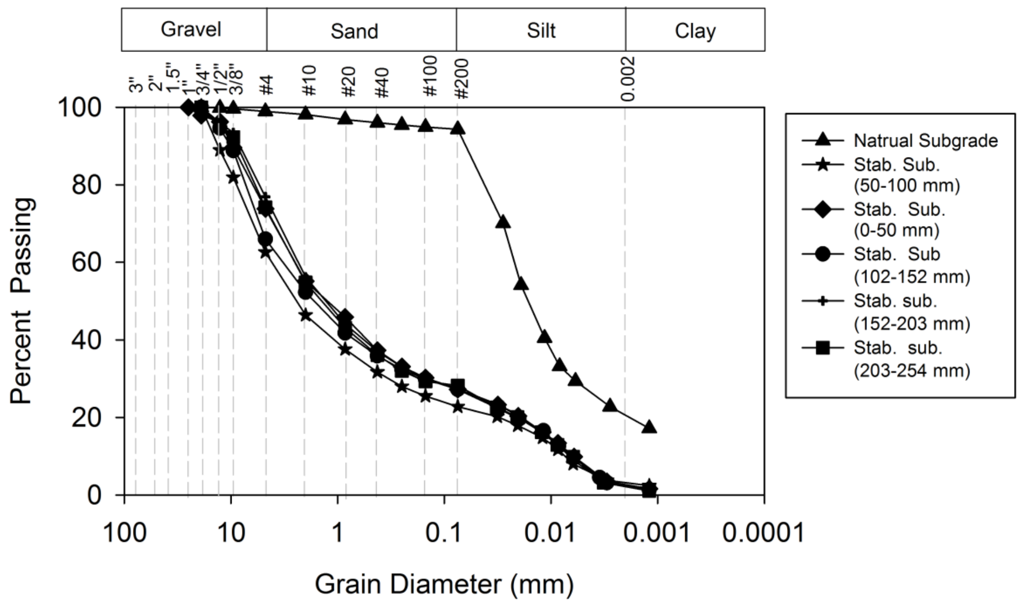

Figure 132. Particle size distribution curves for subgrade K 7 NB Doniphan County, Kansas

4.9.2.2 pH of Stabilized and Natural Subgrade

Figure 133 shows the pH profile of subgrade at test point 11. The pH profile of the stabilized subgrade increased gradually from the top to a depth of 300 mm subgrade. The pH of stabilized subgrade ranged from 7.8 to 8.3. pH of the natural subgrade was 7.4.

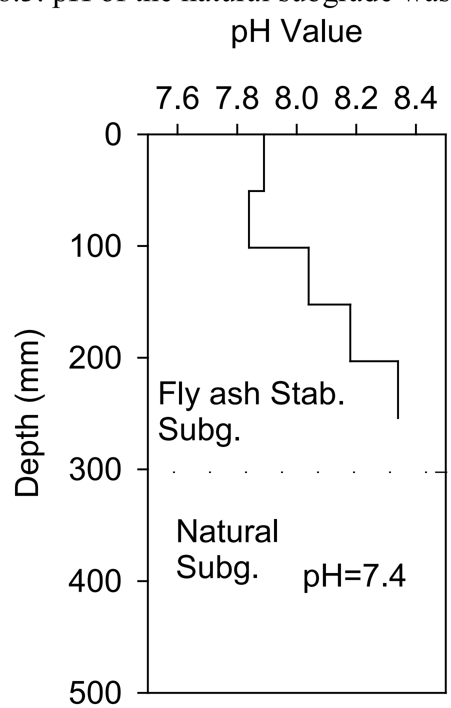

Figure 133. pH profile of subgrade K 7 NB Doniphan County, Kansas