Engineering Properties of Stabilized Subgrade Soils for Implementation of the AASHTO 2002 Pavement Design Guide

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16. ABSTRACT

A comprehensive laboratory study was undertaken to determine engineering properties of cementitiously stabilized common subgrade soils in Oklahoma for the design of roadway pavements in accordance with the AASHTO 2002 Mechanistic-Empirical Pavement Design Guide (MEPDG). These properties include resilient modulus (M_r), modulus of elasticity (M_E), unconfined compressive strength (UCS), moisture susceptibility and three-dimensional (3-D) swell. Four different types of soils encountered in Oklahoma, namely, Port Series (P-soil), Kingfisher Series (K-soil), Vernon Series (V-soil), and Carnasaw Series (C-soil) were used in this study. These soils were stabilized with three locally produced and economically viable stabilizers used in Oklahoma, namely, hydrated lime (or lime), class C fly ash (CFA), and cement kiln dust (CKD). Additionally, mineralogical studies such as scanning electron microscopy, energy dispersive spectroscopy and X-ray diffraction were used to verify the findings from the macro test results.

The percentage of stabilizer used (3%, 6% and 9% for lime; 5%, 10% and 15% for CFA and CKD) was selected on the basis of pH test and literature review. Cylindrical specimens of stabilized soil were compacted and cured for 28 days in a moist room having a constant temperature (23.0 ± 1.7^oC) and controlled humidity (>95%). The 28-day curing period is consistent with the new MEPDG for evaluation of design M_r, M_E and UCS. After curing, specimens were tested for resilient modulus (M_r), modulus of elasticity (M_E) and unconfined compressive strength (UCS). Selective specimens were also tested for moisture susceptibility (tube suction test) and three-dimensional swell during 60 days of capillary soaking.

Results for the tested stabilized soil specimens showed that all three stabilizers improved the strength/stiffness properties, namely, M_r, UCS and M_E values, of P-, K-, V- and C-soil specimens. At lower application rates (3% to 6%), the lime-stabilized soil specimens showed the highest improvement in the strength/stiffness. At higher application rates, however, P-, K, V- and C-soil specimens stabilized with 15% CKD showed the highest improvement. The P-soil specimens, however, showed more improvement in strength due to lower PI, as compared to K-, V- and C-soil. The SEM analysis showed formation of crystals with soil matrix as a result of stabilization. It is reasoned that the crystals within the matrix provide better interlocking between the particles and possible higher resistance to shear deformation and also reduce void within the matrix resulting in overall strength gain. The results of the analysis conforms to the results of the M_r, M_E and UCS tests.

The tube suction test (TST) results revealed that lime- and CFA-treatment is helpful because it reduces the moisture susceptibility. CKD-stabilization, however, makes stabilized specimens more susceptible to moisture, as compared to raw soil specimens. Three-dimensional (3-D) swelling test showed increase in volume for lime- and CKD-stabilized specimens while reduction in volume for CFA-stabilized specimen, as compared to raw soil. This increase in volume is attributed to sulfate-induced heaving which results in the formation of expansive mineral ettringite. Further, presence of ettringite was verified using SEM/EDS tests in conjunction with XRD analyses.

This study generated useful information that would enrich the database pertaining to M_r, M_E, UCS, 3-D swell and moisture susceptibility of selected soils in Oklahoma. An enriched database would benefit highway agencies, specifically pavement engineers, when dealing with construction of new pavements or rehabilitation of existing pavements. It will also facilitate the implementation of the new AASHTO 2002 pavement design guide.

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SI (METRIC) CONVERSION FACTORS

Approximate Conversions to SI Units

Symbol	*When you know*	*Multiply by*	*To Find*	*Symbol*
		LENGTH		
in	*inches*	*25.40*	*millimeters*	*mm*
ft	*feet*	*0.3048*	*meters*	*m*
yd	*yards*	*0.9144*	*meters*	*m*
mi	*miles*	*1.609*	*kilometers*	*km*
		AREA		
in^2	*square inches*	*645.2*	*square millimeters*	mm^2
ft^2	*square feet*	*0.0929*	*square meters*	m^2
yd^2	*square yards*	*0.8361*	*square meters*	m^2
ac	*acres*	*0.4047*	*hectacres*	*ha*
mi^2	*square miles*	*2.590*	*square kilometers*	km^2
		VOLUME		
fl oz	*fluid ounces*	*29.57*	*milliliters*	*mL*
gal	*gallon*	*3.785*	*liters*	*L*
ft^3	*cubic feet*	*0.0283*	*cubic meters*	m^3
yd^3	*cubic yards*	*0.7645*	*cubic meters*	m^3
		MASS		
oz	*ounces*	*28.35*	*grams*	*g*
lb	*pounds*	*0.4536*	*kilograms*	*kg*
T	*short tons (2000 lb)*	*0.907*	*megagrams*	*Mg*
		TEMPERATURE (exact)		
°F	*degrees Fahrenheit*	*(°F-32)/1.8*	*degrees Celsius*	*°C*
		FORCE and PRESSURE or STRESS		
lbf	*poundforce*	*4.448*	*Newtons*	*N*
lbf/in^2	*poundforce per square inch*	*6.895*	*kilopascals*	*kPa*

Approximate Conversions from SI Units

Symbol	*When you know*	*Multiply by*	*To Find*	*Symbol*
		LENGTH		
mm	*millimeters*	*0.0394*	*inches*	*in*
m	*meters*	*3.281*	*feet*	*ft*
m	*meters*	*1.094*	*yards*	*yds*
km	*kilometers*	*0.6214*	*miles*	*mi*
		AREA		
mm^2	*square millimeters*	*0.00155*	*square inches*	in^2
m^2	*square meters*	*10.764*	*square feet*	ft^2
m^2	*square meters*	*1.196*	*square yards*	yd^2
ha	*hectacres*	*2.471*	*acres*	*ac*
km^2	*square kilometers*	*0.3861*	*square miles*	mi^2
		VOLUME		
mL	*milliliters*	*0.0338*	*fluid ounces*	*fl oz*
L	*liters*	*0.2642*	*gallon*	*gal*
m^3	*cubic meters*	*35.315*	*cubic feet*	ft^3
m^3	*cubic meters*	*1.308*	*cubic yards*	yd^3
		MASS		
g	*grams*	*0.0353*	*ounces*	*oz*
kg	*kilograms*	*2.205*	*pounds*	*lb*
Mg	*megagrams*	*1.1023*	*short tons (2000 lb)*	*T*
		TEMPERATURE (exact)		
°C	*degrees Fahrenheit*	*9/5(°C)+32*	*degrees Celsius*	*°F*
		FORCE and PRESSURE or STRESS		
N	*Newtons*	*0.2248*	*poundforce*	*lbf*
kPa	*kilopascals*	*0.1450*	*poundforce per square inch*	lbf/in^2

The contents of this report reflect the views of the author(s) who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the views of the Oklahoma Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation. While trade names may be used in this report, it is not intended as an endorsement of any machine, contractor, process, or products.

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Final Report

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CHAPTER 1 INTRODUCTION

1.1 Background

Pavement conditions data for Oklahoma show that 46% of major roads in the state are in poor or mediocre condition due to weak subgrade soils, as one of the main factors (ODOT, 2007). Driving roads in need of repairs threaten public safety and cost Oklahoma motorists over $1 Billion annually in extra vehicle repairs (OAPA, 2005). In the last few decades, pavement engineers have been challenged to build, repair and maintain pavement systems with enhanced longevity and reduced costs. Specifically, efforts have been made to improve the design methodology (AASHTO, 2004) and to establish techniques for modification of highway pavement materials. Cementitious stabilization is considered one of these techniques; it enhances the engineering properties of subgrade layers, which produces structurally sound pavements.

Cementitious stabilization is widely used in Oklahoma and elsewhere as a remedial method to ameliorate subgrade soil properties (e.g., strength, stiffness, swell potential, workability and durability) through the addition of cementitious additives. It consists of mixing stabilizing agents such as lime, class C fly ash (CFA) and cement kiln dust (CKD) with soil. In the presence of water, these agents react with soil particles to form cementing compounds that are responsible for the improvement in engineering properties such as strength and stiffness. However, the degree of enhancement is influenced by many factors such as stabilizing agent type, the type of soil to be stabilized, curing time, the required strength, the required durability, cost, and environmental conditions (AFJMAN, 1994; Parsons et al., 2004).

With the movement toward implementation of the new Mechanistic-Empirical Pavement Design Guide (MEPDG) (AASHTO, 2004), new material properties required for critical performance prediction of cementitiously stabilized layers are recommended. These properties include resilient (M_r) or elastic (M_E) modulus, unconfined compressive strength (UCS), and moisture susceptibility. The evaluation of these inputs is required to pursue a Level-1 (most accurate) design under the hierarchical scheme. For a Level-2 (intermediate) design, however, design inputs are user selected possibly from an agency database or from a limited testing program or could be estimated through correlations (AASHTO, 2004). Level-3, which is the least accurate, requires only the default values and is generally not recommended.

1.2 Previous Studies

Cementitious stabilization using lime, CFA and CKD stabilization have been studied extensively by many researchers (McManis and Arman 1989; Baghdadi 1990; Zaman et al. 1992; Misra 1998; Little 2000; Miller and Zaman 2000; Qubain et al. 2000; Parsons and Kneebone 2004; Kim and Siddiki 2004). Chang (1995) investigated the resilient properties and microstructure of a fine grained soil (Lateritic soil) stabilized with CFA and lime. Strength was evaluated after a 7-day curing period by performing the UCS tests. Specimens were compacted at near OMC in a mold with a diameter of 38 mm and a height of 100 mm. The resilient modulus tests were performed in accordance with the AASHTO T 274-82 test method. Results showed that the M_r values varied between 125 to 250 MPa. But, no attempt was made to study the moisture susceptibility of specimens.

Little (2000) reported that the long-term effect of lime stabilization induces a 1,000 percent or more increase in M_r over that of the untreated soil. The AASHTO T 294 method was used to determine the resilient modulus values. Values of back calculated (from field falling weight deflectometer testing) M_r typically falls within a range of 210 MPa and 3,500 MPa. The strength values determined for lime-stabilized soil was reported as high as 7,000 to 10,000 kPa. TST was also performed to evaluate the moisture susceptibility on 7-day cured specimens. The study by Little (2000) addressed most of the properties that were evaluated in the present study. Also, that study addressed the design inputs for the MEPDG (Mechanistic-Empirical Pavement Design Guide). However, it was carried out on predominantly fine grained soils, encountered in Texas. In addition, that study was limited to lime-stabilized subgrade soils and no attempt was made to compare with other additives.

Further, Parsons and Milburn (2003) conducted a series of tests, namely UCS, modulus, freeze-thaw, wet-dry and swell to evaluate the relative performance of lime, cement, CFA and an enzymatic stabilizer. These stabilizers were combined with a total of seven different soils having USCS classifications of CH, CL, ML and SM. Lime- and cement-stabilized soils showed the most improvement in performance for multiple soils, with CFA-stabilized soils showing substantial improvement. The results also showed that for many soils, more than one stabilization options may be effective for the construction of subgrade. No attempt was made to examine the moisture susceptibility.

In another study by Parsons and Kneebone (2004), eight different soils with classifications of CH, CL, ML, SM and SP were tested for strength, swell and durability (freeze-thaw, wet-dry, and leach test) to evaluate the relative performance of CKD as a stabilizing agent. Results were compared with previous findings for the same soils stabilized with lime, cement, and fly ash. Substantial increase in strength and decrease in swell were found with the addition of CKD. It was also reported that the CKD treated soil samples' performance in wet-dry testing was similar to that for lime, fly ash and cement treated soils. However, CKD-stabilized samples were not as durable in freeze-thaw testing as lime, fly ash and cement treated soil samples. However, no attempt was made to evaluate and compare the resilient modulus, which is one of the critical pavement performance parameters (AASHTO 2004).

In a recent study, Khoury and Zaman (2007) evaluated the laboratory performance of three different aggregates namely, Meridian, Richard Spur and Sawyer stabilized with CKD, CFA and fluidized bed ash (FBA). Cylindrical specimens of stabilized aggregates were subjected to 0, 8, 16 and 30 freeze-thaw (FT) cycles after 28 days of curing. All the specimens were also tested for resilient modulus after FT cycles. It was found that the CKD-stabilized Meridian and Richard Spur aggregates exhibited a higher reduction in M_r values than the corresponding values of CFA- and FBA-stabilized specimens. The CFA-stabilized Sawyer specimens performed better than their CKD- and FBA-stabilized counterparts.

As noted in the preceding paragraphs, several pertinent studies have been conducted in the past to evaluate the engineering properties of soils stabilized using different cementitious additives. A summary of different studies is presented in Table 1.1. A limited number of studies (e.g., Little 2000, Arora and Aydilek 2005), however, attempted to address all the required design inputs for the MEPDG, namely, M_r, M_E, UCS and long term performance parameters namely, moisture susceptibility (durability) and three-dimensional (3-D) swell. Although some of the aforementioned studies are relevant to the present study, it is important to note that the mineralogical and textural characteristics of soils in Oklahoma are different than those in other regions, and thus results from other studies may not be directly used for

1.3 Objectives

The primary objective of this study is to determine engineering properties of cementitiously stabilized common subgrade soils in Oklahoma for the design of roadway pavements in accordance with the AASHTO 2002 PDG. These properties include resilient modulus (M_r), modulus of elasticity (M_E), moisture susceptibility and permeability. To this end, four different types of soils, namely, Port Series (silty clay), Kingfisher Series (lean clay), Vernon Series (lean clay), and Carnasaw Series (fat clay) were stabilized with hydrated lime, Class C Fly Ash (CFA), and Cement Kiln Dust (CKD). Stabilized soil specimens were cured for 28 days and tested for different properties. The more specific tasks include the following:

(1) Develop moisture-density relationships for different percentages of soil-additive mixtures.

(2) Determine M_r and M_E values of 28-day cured stabilized specimens.

(3) Evaluate the coefficient of permeability of selective stabilized specimens.

(4) Conduct suction tests on selective specimens using filter paper technique.

1.4 Organization of Report

A description of properties of soil and stabilizers is first presented in Chapter 2. Chapter 3 provides detailed information on the laboratory experiments used in this study, followed by the sample preparation method. The final results are presented and discussed in Chapter 4. This includes the pH, M_r, M_E, UCS, moisture susceptibility and 3-D swell values.

Additional results including soil suction, permeability and mineralogical studies are presented in Chapter 5. And lastly, the conclusions and recommendations are given in the final chapter – Chapter 6.

Table 1.1 A Summary of Relevant Laboratory Studies on Soils Stabilized with Different Additives

Reference	**Type of soil[a]**	**Type of additive**	**Parameters/Tests[b] (Statistical Analysis for M_r: Yes/No)**
Haston and Wohlgemuth (1985)	CL	Lime	UCS (No)
McManis and Arman (1989)	A-3, A-2-4	FA	UCS, Durability (F-T), R (No)
Baghdadi (1990)	Kaolinite clay	CKD	UCS (No)
Zaman et al. (1992)	Clays	CKD	UCS (No)
Chang (1995)	Lateritic soil	FA, Lime	UCS, M_r (No)
Achampong (1996)	CL, CH	PC, Lime	UCS, M_r (**Yes**)
Misra (1998)	Clays	FA	UCS (No)
Prusinski et al. (1999)	Clays	PC, Lime	UCS, CBR, Shrinkage, Durability (W-D, F-T, Leaching) (No)
Prusinski and Bhattacharja (1999)	Clays	Lime	UCS (No)
Little (2000)	Fine grained soils	Lime	UCS, M_r, TST, Swell (No)
Miller and Azad (2000)	CH, CL, ML	CKD	UCS (No)
Miller and Zaman (2000)	Shale, Sand	CKD	CBR, UCS, Durability (F-T, W-D) (No)
Qubain et al. (2000)	CL	Lime	UCS, M_r (No)
Zia And Fox (2000)	Loess	FA	UCS, CBR, Swell potential (No)
Senol et al. (2002)	Clays	FA	UCS, CBR, M_r (No)
Parsons and Milburn (2003)	CH, CL, ML, SM	Lime, PC, CFA, Enzymatic stabilizer	UCS, Modulus, Durability (F-T, W-D), Swell (No)
Kim and Siddiki (2004)	A-4, A-6, A-7-6	Lime, LKD	UCS, CBR, volume stability, M_r (**Yes**)
Prabakar et al. (2004)	CL, OL, MH	FA	UCS, CBR, Shear strength parameters, Free swelling (No)
Arora and Aydilek (2005)	SM	FA	UCS, CBR, M_r, Durability (F-T) (**Yes**)
Barbu and McManis (2005)	CL, ML	Lime, PC	UCS, Cyclic Triaxial test, TST (No)
Hillbrich and Scullion (2006)	A-3	PC	M_r, Seismic Modulus (**Yes**)
Osinubi and Nwaiwu (2006)	CL	Lime	UCS (No)
Puppala et al. (2006)	CH	Lime with polypropylene fiber	UCS, free swell, linear shrinkage strain (No)

[a] Soils according to USCS and AASHTO classification; [b] pH, Compaction and Atterberg limit tests are not included in the list
M_r: Resilient Modulus test; TST: Tube Suction Test; CBR: California Bearing Ratio; F-T: Freeze-Thaw; W-D: Wet-Dry
R: Soil support resistance value FA: Fly Ash; PC: Portland Cement; CKD: Cement Kiln Dust; LKD: Lime Kiln Dust
R: Soil support resistance value FA: Fly Ash; PC: Portland Cement; CKD: Cement Kiln Dust; LKD: Lime Kiln Dust

2.1 General

This chapter is devoted to presenting the sources of materials that were used in this study. The subgrade soils were collected from different counties in Oklahoma and the stabilizing agents were shipped to our laboratory from different agencies. The moisture-density tests were conducted on raw and stabilized soils to determine the optimum moisture content (OMC) and maximum dry density (MDD). These results are presented in this chapter.

2.2 Soil Types and Properties

As noted earlier, four different soils were used in this study: (1) Port series (P-soil); (2) Kingfisher series (K-soil); (3) Vernon series (V-soil); and (4) Carnasaw series (C-soil). Bulk soil samples were collected from different counties located in Oklahoma. More than 40 plastic bags, each having a weight of approximately 20 kgs (44 lbs), were transported to the Broce Laboratory and stored for processing and testing. After collection, these soils were air dried in the laboratory and processed by passing through the U.S. standard sieve #4. Figures 2.1, 2.2 and 2.3 photographically depict the field sampling, processing and storage of these soils, respectively. A summary of the soil properties determined in the laboratory and the corresponding standard testing identification are presented in Table 2.1. The chemical properties of the soils determined using X-ray Fluorescence analysis are given in Table 2.2.

2.2.1 Port Series

Port series soil (P-soil) is found in 33 counties and it covers about one million acres in central Oklahoma. Bulk samples were collected from a location in Norman (Cleveland County), Oklahoma. According to the Unified Soil Classification System (USCS), P-soil is classified as CL-ML (silty clay with sand) with a liquid limit of approximately 27 and a plasticity index (PI) of approximately 5. The soil is inactive with an activity of approximately 0.24 and a pH of 8.91. Particle size analysis showed the percentage passing U.S. Standard No. 200 sieve as 83%. For comparison, P-soil was also tested at Oklahoma DOT materials division soils laboratory. A liquid limit of 26 and plastic limit of 19 (PI = 6) was reported.

2.2.2 Kingfisher Series

Kingfisher series soil (K-soil) belongs to the Cleveland County, Oklahoma. It is classified as CL (lean clay), according to the Unified Soil Classification System (USCS) with an average liquid limit of approximately 39% and a PI of approximately 21. The soil is inactive with an activity of approximately 0.47 and a pH of 8.82. Using the Oklahoma DOT Specification number OHD L-49 (ODOT, 2006) no soluble sulfates were detected within a detection range of greater than 40 ppm. Particle size analysis showed the percentage passing U.S. Standard No. 200 sieve as 97%.

2.2.3 Vernon Series

Vernon series soil (V-soil) was collected from Glass Mountains slope on US 412 in Major County (northwestern Oklahoma). Selection of this soil was based on the soluble sulfate content measured in this soil. Soluble sulfate content in the soil was measured using the Oklahoma Department of Transportation procedure for determining soluble sulfate content: OHD L-49 (ODOT, 2006). This soil has a sulfate content of 15,400 ppm (>10,000 ppm). Physical properties of this soil were determined from Atterberg limit test, hydrometer tests, and standard Proctor compaction tests. As per the Unified Soil Classification System (USCS), this soil is classified as lean clay (CL), with an average liquid limit of approximately

37 and a PI of approximately 11. The soil is inactive with an activity of approximately 0.28 and a pH of 8.14. Particle size analysis showed the percentage passing U.S. Standard No. 200 sieve as approximately 100%. For comparison, V-soil was also tested at Oklahoma DOT materials division soils laboratory. A liquid limit of 39 and plastic limit of 25 (PI = 15) was reported.

2.2.3 Carnasaw Series

Carnasaw series soil (C-soil) with a high PI value of 29 was sampled from on-ramp junction of SH 52 and NE 1130th Avenue in Latimer County. This soil is classified as fat clay (CH) according to USCS with a liquid limit of approximately 58. C-soil is acidic in nature with a very low pH value of 4.17. In addition, this soil is also having sulfate content of 267 ppm which is lower than 2,000 ppm; Petry (1995) suggested that soils containing sulfate contents greater than 2,000 ppm have the potential to cause swelling due to calcium-based stabilizer. The soil is having an activity of approximately 0.69 and a low pH of 4.17. Particle size analysis showed the percentage passing U.S. Standard No. 200 sieve as approximately 94%.

2.3 Additive Types and Properties

In this study, hydrated lime, class C fly ash (CFA), and cement kiln dust (CKD) were the main additives, also called as stabilizers or stabilizing agents (Figure 2.4). Many properties of soils and stabilizing agents are related with the silica/sesquioxide ratio (*SSR*) (Winterkorn and Baver 1934; Fang 1997) as:

$$SSR = \frac{\frac{x}{A}}{\frac{y}{B} + \frac{z}{C}} \tag{2.1}$$

where, x is the percent of SiO_2, y is the percent of Al_2O_3, z is the percent of Fe_2O_3, A is the molecular weight of SiO_2 (60.1), B is the molecular weight of Al_2O_3 (102.0), and C is the molecular weight of Fe_2O_3 (159.6). Hydrated lime (or lime), class C fly ash (CFA), and cement kiln dust (CKD) were used. Hydrated lime was supplied by the Texas Lime Company, Cleburne, Texas. It is a dry powder manufactured by treating quicklime (calcium oxide) with sufficient water to satisfy its chemical affinity with water, thereby converting the oxides to hydroxides. CFA from Lafarge North America (Tulsa, Oklahoma) was brought into well-sealed plastic buckets. It was produced in a coal-fired electric utility plant. CKD used was provided by Lafarge North America located in Tulsa, Oklahoma. Sealed buckets were shipped to our laboratory from Tulsa, Oklahoma. It is an industrial waste collected during the production of Portland cement. The chemical properties of the stabilizing agents are presented in Table 2.3. From the aforementioned chemical properties (Table 2.3), differences between the chemical composition and physical properties among the selected additives are clearly evident. These differences will lead to different performance of stabilized soil specimens as reported by Chang (1995), Parsons and Milburn (2003), Kim and Siddiki (2004) and Khoury and Zaman (2007).

2.4 Moisture-Density Test

In the laboratory soil was mixed manually with stabilizer for determining moisture-density relationship of soil-additive mixtures. The procedure consists of adding specific amount of additive, namely, lime (3%, 6% or 9%) or CFA (5%, 10% or 15%) or CKD (5%, 10% or 15%) to the processed soil. The amount of additive was added based on the dry weight of soil. The additive and soil were mixed manually to uniformity, and tested for moisture-density relationships by conducting Proctor test in accordance with ASTM D 698 test method.

2.4.1 P-soil and Additive Mixtures

The moisture-density test results (i.e., OMCs and MDDs) for P-soil are presented in Table 2.4. The moisture content was determined by oven-drying the soil-additive mixture. The OMC and MDD of raw soil was found to be 13.1% and 17.8 kN/m^3 (108.7 pcf), respectively. In the present study, laboratory experiments showed an increase in OMC with increasing percentage of lime and CKD. On the other hand, a decrease in the MDDs with increasing percent of lime and CKD is observed from Table 2.4. Other researchers (e.g., Haston and Wohlegemuth, 1985; Zaman et al., 1992; Miller and Azad, 2000; Sreekrishnavilasam et al., 2007) also observed effects similar to those in the current study. One of the reasons for such behavior can be attributed to the increased number of fines in the mix due to the addition of lime and CKD.

A higher MDD was obtained by increasing the CFA content. However, the MDD increase diminished with the increase in CFA beyond 10%. Conversely, the OMC showed an increase for 5% CFA and then it generally decreased with increasing CFA content. These observations were similar to those reported by McManis and Arman (1989) for sandy silty soil and by Misra (1998) for clays.

2.4.2 K-soil and Additive Mixtures

The moisture-density test results for K-soil are presented in Table 2.5. The OMC and MDD of raw soil was found to be 16.5% and 17.4 kN/m^3 (110.6 pcf), respectively. In the present study, laboratory experiments showed an increase in OMC with increasing percentage of lime. On the other hand, a decrease in the MDDs with increasing percent of lime is observed from Table 3. This is consistent with the results reported by Nagaraj (1964), Haston and Wohlegemuth (1985), Ali (1992) and Little (1996). Little (1996) believed that OMC increased with increasing lime content because more water was needed for the soil-lime chemical reactions. Nagaraj (1964) suggested that the decrease in MDD of the lime-treated soil is reflective of increased resistance offered by the flocculated soil structure to the compactive effort.

For CFA stabilization, MDD increased with increase in CFA content. On the other hand, OMC decreased for 5 percent CFA mix and then increased for 10 and decreased again for 15 percent of fly ash mix. A similar observation was reported by McManis and Arman (1989), Misra (1998) and Solanki et al. (2007a). Misra (1998) reported that the increase in MDD can be attributed to the packing of finer fly ash particles (smaller than a no. 200 sieve) in voids between larger soil particles. This behavior of OMC was attributed to progressive hydration of soil and fly ash mixtures and increased number of finer particles (specific surface) in the soil-fly ash mixtures.

CKD-stabilized soil showed the same trends like lime-stabilized soil. An increase in OMC and a decrease in MDD with increase in the percentage of additive was observed. Other researchers (e.g., Zaman et al. 1992; Miller and Azad 2000; Solanki et al. 2007b) also observed effects similar to those in the current study. Similar statements as mentioned for lime-stabilization can be used to rationalize the compaction behavior of CKD-stabilized soils.

2.4.3 V-soil and Additive Mixtures

The moisture-density test results for V-soil mixed with different percentages of additives are summarized in Table 2.6. The Proctor tests conducted on raw V-soil showed an

OMC and MDD value of 23.0% and 16.0 kN/m^3 (101.9 pcf), respectively. Similar to P- and K-soil-lime/CKD mixtures, OMC-MDD essentially showed the same trend. Hence, reasons as mentioned in the preceding section can be used to justify the observed trends in OMC and MDD values.

For CFA stabilization, the moisture-density results were more complex. Laboratory experiments showed that MDD decreased with 5 percent CFA, and then increased with increase in the percentage of additive. On the other hand, OMC decreased with the increase in the amount of CFA, as evident from Table 2.6.

2.4.4 C-soil and Additive Mixtures

The OMC was found to be 20.3% for the raw C-soil. For lime- and CKD-stabilized soil samples, it was evident that OMC increased and MDD decreased with increasing percentage of lime as illustrated in Table 2.7. For CFA stabilization, Proctor results showed that MDD decreases for 5 percent of CFA, increases for 10 percent and then again decreases for 15 percent CFA as shown in Table 2.7. On the other hand, OMC decreased with the increase in the percentage of CFA. Since moisture-density results of C-soil and additive mixtures showed similar trends to other soil-additive mixtures used in this study, similar reasons as mentioned in the preceding section 2.4.1 can be used to justify the observed OMC-MDD trends.

Table 2.1 Testing Designation and Soil Properties

Method	Parameter/Units	P-soil	K-soil	V-soil	C-soil
ASTM D 2487	USCS Symbol	CL-ML	CL	CL	CH
AASHTO M 145	AASHTO Designation	A-4	A-6	A-6	A-7-6
ASTM D 2487	USCS Name	Silty clay with sand	Lean clay	Lean clay	Fat clay
ASTM D 2487	% finer than 0.075 mm	83	97	100	94
ASTM D 4318	Liquid limit	27	39	37	58
ASTM D 4318	Plastic limit	21	18	26	29
ASTM D 4318	Plasticity index	5	21	11	29
…	Activity	0.24	0.47	0.28	0.69
ASTM D 854	Specific gravity	2.65	2.71	2.61	2.64
ASTM D 698	Optimum moisture content (%)	13.1	16.5	23.0	20.3
ASTM D 698	Max. dry unit weight (pcf)	113.4	110.6	101.9	103.7
ASTM D 6276	pH	8.91	8.82	8.14	4.17
OHD L-49	Sulfate content (ppm)	<40	<40	15,400	267

USCS: Unified Soil Classification System; OHD: Oklahoma Highway Department

Table 2.2 Chemical Properties of Soils used in this Study

Chemical Compound	Percentage by weight, (%)			
	P-soil	K-soil	V-soil	C-soil
Silica (SiO_2)[a]	73.7	60.7	50.2	47.5
Alumina (Al_2O_3)[a]	7.0	11.9	16.4	16.1
Ferric oxide (Fe_2O_3)[a]	2.2	4.4	6.7	6.8
Silica/Sesquioxide ratio (SSR) $SiO_2/(Al_2O_3+Fe_2O_3)$	14.9	7.0	4.1	3.9
Calcium oxide (CaO)[a]	2.9	3.3	3.5	0.1
Magnesium oxide (MgO)[a]	1.8	3.2	4.7	0.9
Sodium oxide (Na_2O)[a]	0.8	0.8	1.0	0.2
Potassium oxide (K_2O)[a]	1.4	2.1	4.4	2.1
Sulfur trioxide (SO_3)[a]	0.0	0.0	1.7	0.0
Loss on Ignition	5.1	7.8	7.1	25.1
Percentage passing No. 325	54.0	88.8	94.8	87.2
UCS (psi)	31.9	27.6	29.0	30.5

[a]X-ray Fluorescence analysis

Table 2.3 Chemical Properties of Stabilizers used in this Study

Chemical Compound	Percentage by weight, (%)		
	Lime	CFA[c]	CKD[d]
Silica (SiO_2)[a]	0.6	37.7	14.1
Alumina (Al_2O_3)[a]	0.4	17.3	3.1
Ferric oxide (Fe_2O_3)[a]	0.3	5.8	1.4
$SiO_2 + Al_2O_3 + Fe_2O_3$ (SAF)	1.3	60.8	18.6
Silica/Sesquioxide ratio (SSR) $SiO_2/(Al_2O_3+Fe_2O_3)$	1.7	3.0	6.0
Calcium oxide (CaO)[a]	68.6	24.4	47.0
Magnesium oxide (MgO)[a]	0.7	5.1	1.7
Sulfur trioxide (SO_3)[a]	0.1	1.2	4.4
Calcium hydroxide ($Ca(OH)_2$)[a]	94.5	…	…
Free lime[a]	94.5	0.4	8.5
Loss on Ignition[b]	28.4	1.1	25.8
Percentage passing No. 325[c]	98.4	85.8	94.2
pH[c]	12.58	11.83	12.55
Sulfate Content (ppm)[c]	< 40	3,280	28,133
28-day UCS[c] (psu)	…	4,876.6	464.4

[a]X-ray Fluorescence analysis; [c]Determined independently
[b]ASTM C 575;[c]CFA: Class C Fly Ash; [d]CKD: Cement Kiln Dust

Table 2.4 A Summary of OMC-MDD of Lime-, CFA- and CKD-P-soil Mixtures

Type of additive	Percentage of additive	OMC (%)	Maximum dry density	
			kN/m^3	pcf
Raw	0	13.1	17.8	113.4
Lime	3	14.7	17.1	108.7
	6	15.9	16.9	107.2
	9	16.5	16.6	105.9
CFA	5	14.0	17.8	113.5
	10	12.8	18.1	114.9
	15	11.7	18.0	114.7
CKD	5	14.8	17.4	110.5
	10	15.2	17.2	109.3
	15	15.3	17.1	108.6

1 pcf = 0.1572 kN/m^3; OMC: optimum moisture content; MDD: maximum dry density; CFA: class C fly ash; CKD: cement kiln dust

Table 2.5 A Summary of OMC-MDD of Lime-, CFA- and CKD-K-soil Mixtures

Type of additive	Percentage of additive	OMC (%)	Maximum dry density kN/m^3	Maximum dry density pcf
Raw	0	16.5	17.4	110.6
Lime	3	16.1	17.0	108.4
	6	16.5	16.8	106.6
	9	18.5	16.3	103.8
CFA	5	13.0	17.4	110.8
	10	15.3	17.4	111.0
	15	15.1	17.5	111.5
CKD	5	16.9	17.3	110.2
	10	17.3	17.1	108.6
	15	17.6	16.9	107.8

1 pcf = 0.1572 kN/m^3; OMC: optimum moisture content; MDD: maximum dry density; CFA: class C fly ash; CKD: cement kiln dust

Table 2.6 A Summary of OMC-MDD of Lime-, CFA- and CKD-V-soil Mixtures

Type of additive	Percentage of additive	OMC (%)	Maximum dry density kN/m^3	Maximum dry density pcf
Raw	0	23.0	16.0	101.9
Lime	3	25.4	15.6	99.5
	6	25.9	15.3	97.4
	9	26.8	14.9	95.0
CFA	5	22.6	16.0	101.6
	10	21.7	16.1	102.5
	15	21.2	16.2	102.9
CKD	5	24.1	15.7	100.1
	10	23.5	15.8	100.3
	15	23.1	15.8	100.7

1 pcf = 0.1572 kN/m^3; OMC: optimum moisture content; MDD: maximum dry density; CFA: class C fly ash; CKD: cement kiln dust

Table 2.7 A Summary of OMC-MDD of Lime-, CFA- and CKD-C-soil Mixtures

Type of additive	Percentage of additive	OMC (%)	Maximum dry density	
			kN/m^3	pcf
Raw	0	20.3	16.3	103.7
Lime	3	22.0	16.0	101.5
	6	22.7	15.6	99.0
	9	23.8	15.3	97.3
CFA	5	20.0	16.3	103.5
	10	18.6	16.6	105.3
	15	16.6	16.4	104.1
CKD	5	21.6	16.1	102.3
	10	21.7	16.0	101.8
	15	21.9	15.9	101.4

1 pcf = 0.1572 kN/m^3; OMC: optimum moisture content; MDD: maximum dry density; CFA: class C fly ash; CKD: cement kiln dust

Figure 2.1 Sampling of C-soil from Latimer County

Figure 2.2 Processing of Soil Samples

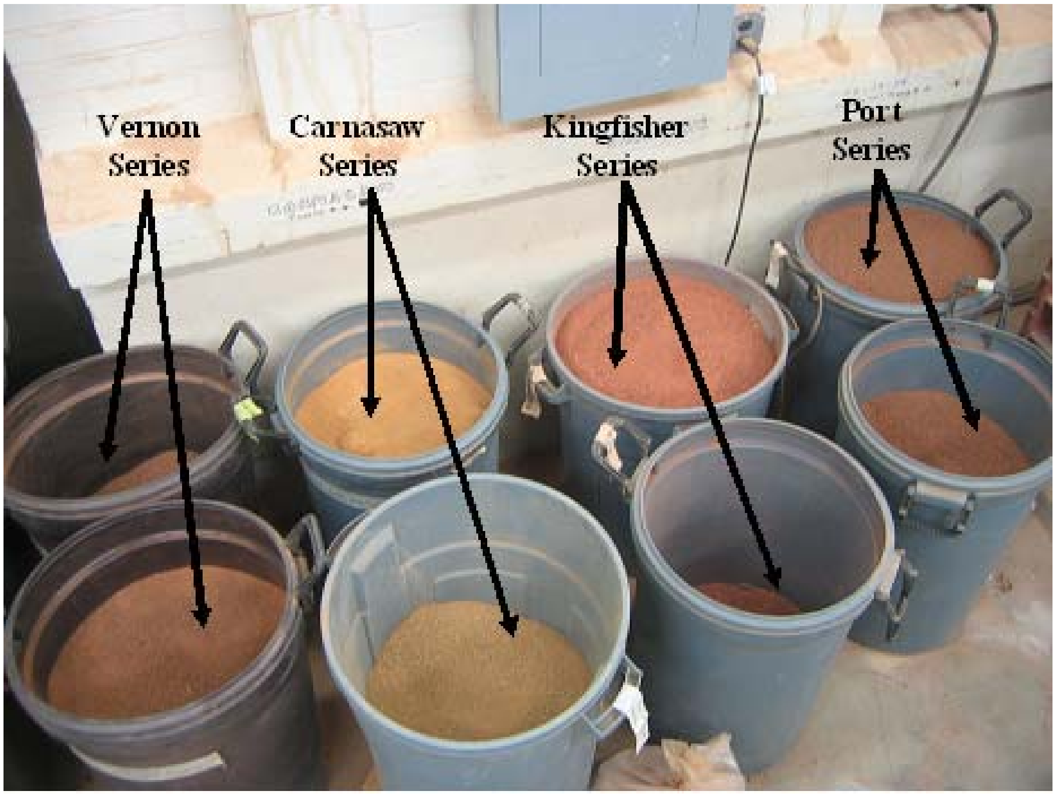


Figure 2.3 Storage of Processed Soils

(a) Hydrated lime (b) Class C Fly Ash (c) Cement Kiln Dust

Figure 2.4 Photograph of Different Additives used in this Study

3.1 General

This chapter describes the experimental methodology that was followed to evaluate the effects of different additives on the engineering properties of stabilized soils. The laboratory tests performed in this study placed emphasis on pH, resilient modulus (M_r), unconfined compressive strength (UCS), moisture susceptibility, and three-dimensional swell. These tests are described in this chapter. Also, a description of sample preparation and compaction method is included.

3.2 pH Test

The pH of soil-additve mixtures was determined using the method recommended by ASTM D 6276, which involves mixing the solids with de-ionized (DI) water, periodically shaking samples, and then testing with a pH meter after 1 h (Figure 3.1). This procedure was developed to determine the lime requirements of soil. If the soil-lime-water mixture is elevated to a point where it approaches the pH of a lime-water mixture then it is assumed that sufficient lime is available to satisfy ion-exchange and other reactions. Since elevated pH levels are important for promoting chemical activity, tests were conducted with each of the soils mixed with various amounts of lime as well as CFA and CKD to investigate whether pH would reflect the performance of stabilized soil specimens.

3.3 Resilient Modulus Test

The resilient modulus (M_r) tests were performed in accordance with the AASHTO T 307 test method. The test procedure consisted of applying 15 stress sequences using a cyclic haversine shaped load with duration of 0.1 seconds and rest period of 0.9 seconds. A haversine load pulse, having the form of ([1-cos (θ)]/ 2), is shown in Figure 3.2 and is recognized as the best pulse shape to simulate the induced load shape in pavement layers by a moving vehicle (NCHRP, 1997). The sample was loaded following the sequences shown in Table 3.1. For each sequence, the applied load and the vertical displacement for the last five cycles were recorded and used to determine the M_r. A 2.23 kN (500-lb) load cell was used to apply the load. Two linear variable differential transformers (LVDTs) were used to measure the resilient vertical deformation. These LVDTs were attached to two aluminum clamps that were mounted on the specimen at a distance of approximately 50.8 mm (2.0 in) from both ends of the specimen. The LVDTs had a maximum stroke length of 5.08 mm (0.2 in). Figure 3.3 shows a photographic view of the LVDTs mounted on a sample. A power supply was used to excite and amplify the LVDT signals. This is consistent with Barksdale et al. (1997) that measuring relative displacement between two points on the specimen eliminates the extraneous deformations occurring past the ends of the specimens. A complete setup of M_r testing on stabilized subgrade soil specimen is shown in Figure 3.4.

To generate the desired haversine-shaped load and to read the load and displacement signals, a program was written using MTS Flex Test SE Automation software, as shown in Figure 3.5. All the data were collected and stored in an MS Excel file and a macro program in Excel was written to process these data and evaluate the resilient modulus. Further, details of the apparatus and the noise reduction method used are given by Khoury et al. (2003).

3.4 Modulus of Elasticity and Unconfined Compressive Strength

The new MEPDG recommends the use of Mixture Design and Testing Protocol (MDTP) developed by Little (2000) in conjunction with the AASHTO T 307 test protocol for determining the M_r of soils stabilized with lime. The PDG also requires M_E as one of the design inputs for soil-cement, cement-treated materials, lime-cement-fly ash mixtures and lean concrete. Since no specific parameters were recommended for CFA and CKD stabilization, it was decided to evaluate the M_E and UCS as an additional indicator of the mechanical behavior of CFA- and CKD-stabilized specimens.

Modulus of elasticity (M_E) and unconfined compression test (UCS) tests were conducted in accordance with the ASTM D 1633 test method. Specimens were loaded in an MTS frame (Figure 3.6) at a displacement rate of 1 mm/min (0.05 in/min). Deformation values were recorded during the test using two LVDTs fixed to opposite sides of and equidistant from piston the rod with a maximum stroke length of ±12.7 mm (0.5 in). The load values were obtained from a load cell having a capacity of 97.9 kN (22 kips).

Each specimen was subjected to two unloading-reloading cycles and loaded up to failure in the third sequence of reloading to determine the UCS. Figure 3.7 shows a typical stress-strain curve obtained from the UCS test using unloading-reloading cycles. A straight line "AB" is drawn through the first unloading-reloading curve, (see Figure 3.7). Similarly, line "CD" is drawn through the other unloading-reloading curve, as shown. The average slope of these lines is treated as the modulus of elasticity (M_E) of the stabilized specimen.

3.5 Moisture Susceptibility Test

In the current study, moisture susceptibility of stabilized specimens was evaluated by conducting Tube Suction Test (TST). The TST was developed by the Finnish National Road Administration and the Texas Transportation Institute to evaluate the moisture susceptibility or the amount of "free" water present within a soil system (Syed et al., 1999). In this test the evolution of the moisture conditions is evaluated in terms of *dielectric constant* using a dielectric probe. The dielectric constant of dry soil is about 5, and the dielectric constant of air is about 1. The dielectric constant of "free" water is about 81. The Adek Percometer^{TM} (Figure 3.8) is a surface probe that operates at a central frequency of 50MHz, and is used to measure the dielectric constants on a surface of material samples by measuring the change in capacitance of the probe (Syed et al., 2003).

According to Syed et al. (1999) and Zhang and Tao (2006), TST is a time-efficient procedure to determine the optimum additive amount in stabilized materials. Several other researchers also recommended the use of TST to study the behavior of stabilized materials (see e.g., PCA, 1992; Little, 2000; Syed et al., 2000; Guthrie, 2003; Saeed et al., 2003; Syed et al., 2003; Barbu et al., 2004 and Solanki et al., 2008). The TST involves measurement of capillary rise and surface dielectric values (DV) of the test specimens. In this test, the capillary rise is monitored with a dielectric probe, which measures the dielectric properties at the surface of the sample. The DV is a measure of the unbound or "free" moisture within the sample. High surface dielectric readings indicate suction of water by capillary forces and can be an indicator of a non-durable material that will not perform well under saturated or repeated freeze-thaw conditions (Scullion and Saarenketo, 1997).

The TST procedure used in this study consists of placing M_r tested specimens in an oven at 35 ± 5^{o}C until no more significant weight changes are observed. After drying, specimens were allowed to cool down at room temperature. Specimens were then placed on a porous plate and covered with a membrane in an ice chest containing approximately 12.7 mm (0.5 in.) of de-ionized (DI) water under controlled temperature (23.0 ± 1.7^o C) and humidity (>96%). During wetting of specimens in DI water, the DV increased with time due to capillary soaking of water in the specimens. Four measurements were taken along the circumference of the sample in separate quadrants and the fifth reading was taken at the center of specimen and an average of all five readings was reported. Measurements were taken daily, until the DV became constant. Figure 3.9 shows photographic view of setup used for TST.

Guthrie and Scullion (2003) suggested the following interpretation of DV for aggregate base material:

Lower DV	**Upper DV**	**Interpretation of Aggregate Base Moisture Susceptibility**
NA	10	Good
10	16	Marginal
16	NA	Poor

NA: Not Applicable

It is clear from the above mentioned values that a decrease of DV from 16 to 10 makes aggregate base material from good to poor in terms of moisture susceptibility. Thus, a decrease in DV indicates a reduction in the moisture susceptibility of specimens. To the authors' knowledge, however, there are no recommended lower and upper DV values for stabilized soil specimens.

3.6 Three-Dimensional Swell Test

Comparison of field and laboratory data obtained from oedometer tests revealed that the laboratory results from 1-D swell tests overestimate the *in-situ* heave by a factor of about 3 (see e.g., Johnson and Snethen, 1979; Erol, 1992). Hence, to investigate the swelling potential of specimens, three-dimensional (3-D) swell tests were conducted in accordance with a procedure described by Harris et al. (2004) (Figure 3.10). The 3-D swell values were measured by determining the height to the nearest 0.025 mm (0.001 in.) in 3 places that are 120^o apart during capillary soaking of specimens for Tube Suction Test. The diameter was measured to the nearest 0.025 mm near the top, in the middle, and near the base of each sample using a digital vernier caliper. The three height and diameter measurements were

3.7 Sample Preparation

In this study, both raw and stabilized soil specimens were compacted in accordance with AASHTO T-307 test method (Figure 3.12). Figure 3.13 shows a photographic view of sample preparation method. The procedure consists of adding a specific amount of additive to the raw soil. The amount of additive (3%, 6%, or 9% for lime and 5%, 10%, or 15% for CFA and CKD) was added based on the dry weight of the soil. The additive and soil were mixed manually for uniformity. After the blending process, a desired amount of water was added based on the OMC, as discussed in Chapter-2. Then, the mixture was compacted in a mold having a diameter of 101.6 mm (4.0 in) and a height of 203.2 mm (8.0 in) to reach a dry density of between 95%-100% of the MDD. After compaction, specimens were cured at a temperature of 23.0 ± 1.7^{o} C and a relative humidity of approximately 98% for 28 days; 28-day curing period is recommended by the new MEPDG (AASHTO, 2004).

A total of four replicates were prepared for each additive content, of which two specimens were tested for M_r and then followed by TST and three-dimensional (3-D) swell test by subjecting samples to 60 days of capillary soaking under controlled temperature (23.0 ± 1.7^{o} C) and humidity (>96%) in an ice chest. After 60 days of capillary soaking, selected specimens were again tested for M_r and then followed by M_E and UCS tests. The other two specimens were tested for M_r and then followed by M_E and UCS tests, without capillary soaking. After UCS test broken specimens were air dried for approximately 2 days, and then pulverized and passed through a No. 40 sieve. The finer material was reconstituted with moisture for 1 day, and then tested for liquid limit and plastic limit in accordance with ASTM D 4318.

Table 3.1 Testing Sequence used for Resilient Modulus Test

Sequence Number	**Confining Pressure (psi)**	**Maximum Axial Stress (psi)**	**Cyclic Stress (psi)**	**Constant Stress (psi)**	**No. of Load Applications**
Conditioning	6	4	3.6	0.4	500
1	6	2	1.8	0.2	100
2	6	4	3.6	0.4	100
3	6	6	5.4	0.6	100
4	6	8	7.2	0.8	100
5	6	10	9	1	100
6	4	2	1.8	0.2	100
7	4	4	3.6	0.4	100
8	4	6	5.4	0.6	100
9	4	8	7.2	0.8	100
10	4	10	9	1	100
11	2	2	1.8	0.2	100
12	2	4	3.6	0.4	100
13	2	6	5.4	0.6	100
14	2	8	7.2	0.8	100
15	2	10	9	1	100

Figure 3.1 Setup for pH Test

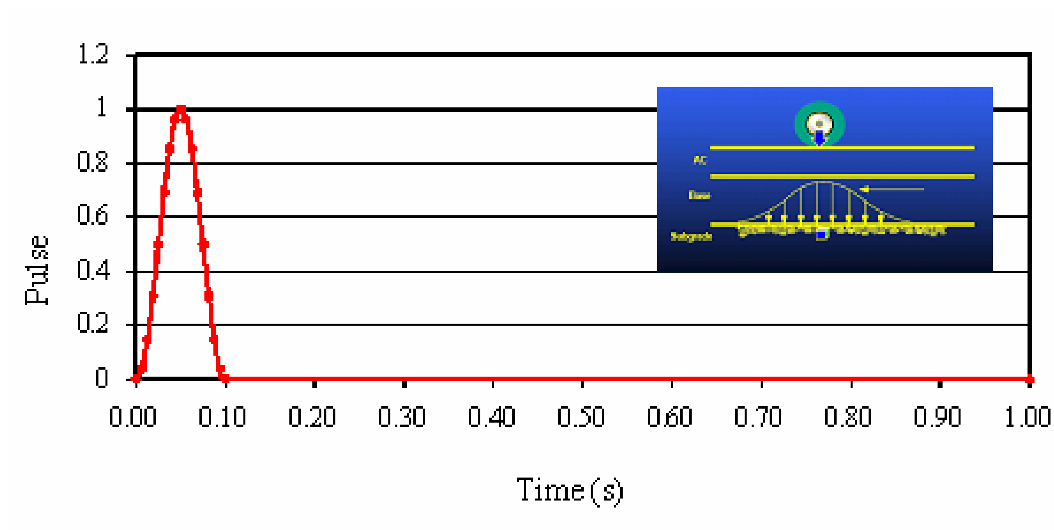


Figure 3.2 Cyclic Load used in Resilient Modulus Test

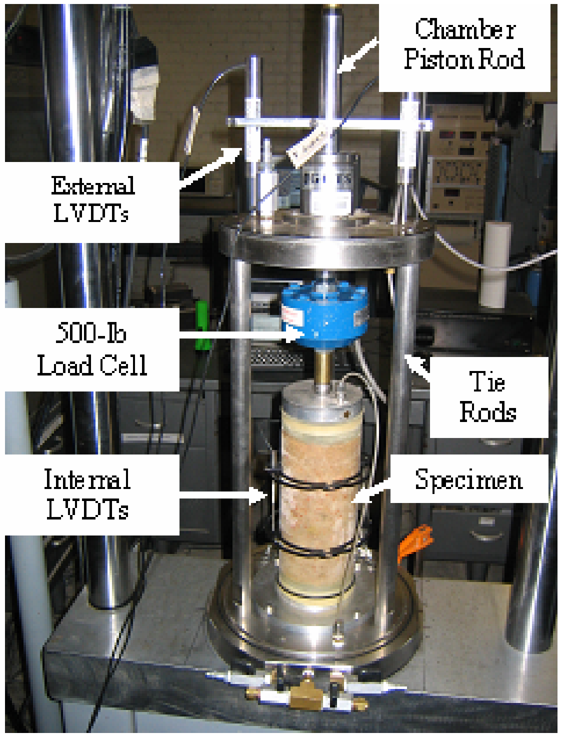


Figure 3.3 Setup for Resilient Modulus Test (without pressure chamber)

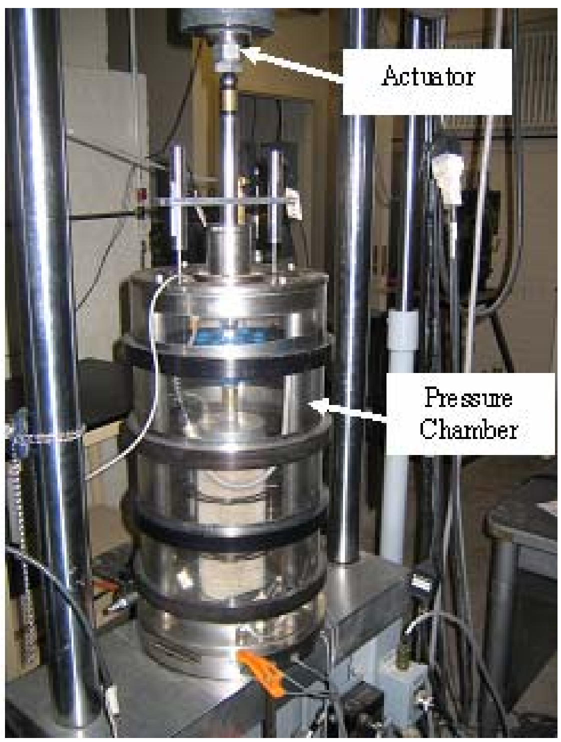


Figure 3.4 Setup for Resilient Modulus Test (with pressure chamber)

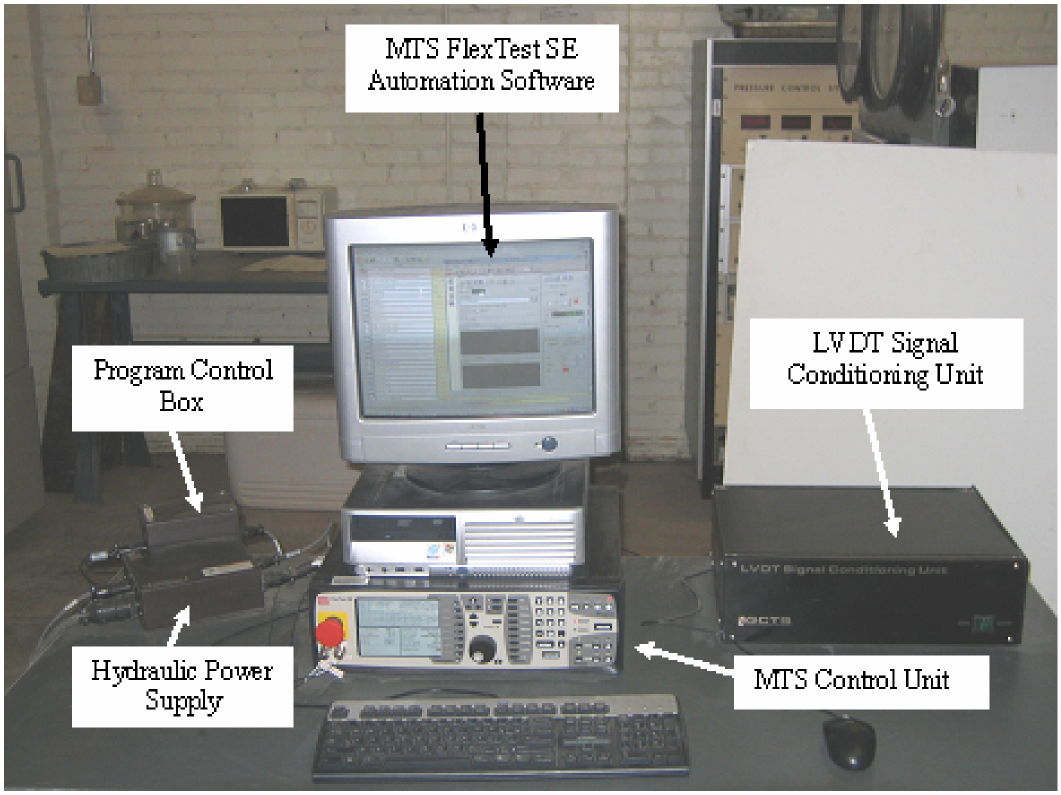


Figure 3.5 MTS Digital Control System and Computer

Figure 3.6 Specimen in MTS Frame for UCS Test

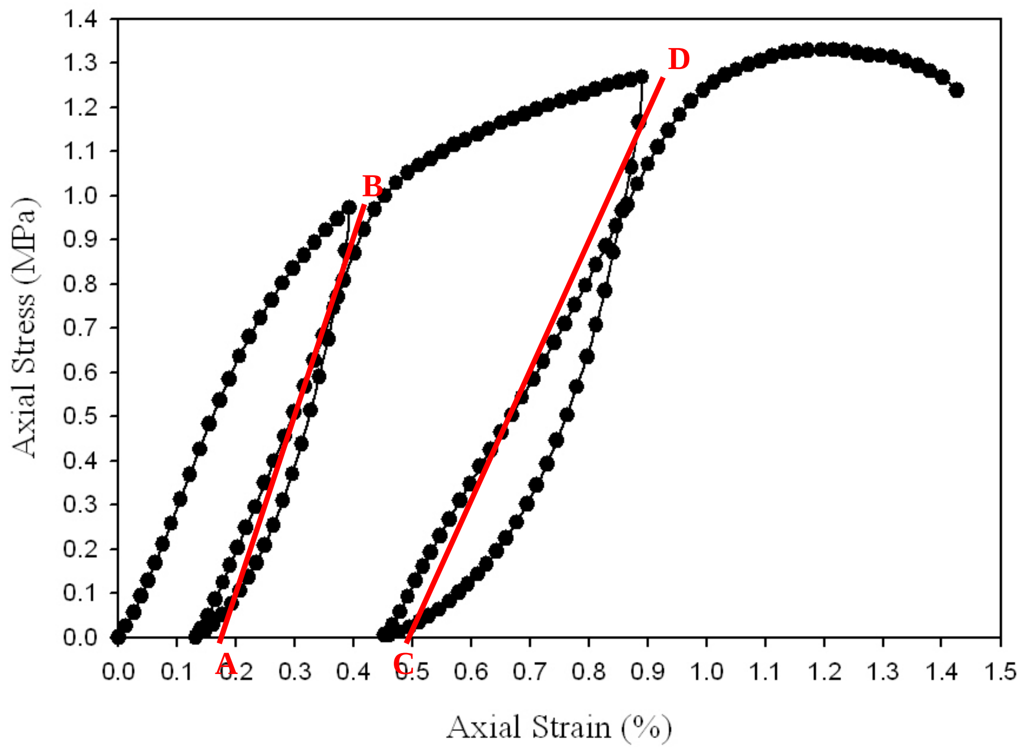


Figure 3.7 Determination of Modulus of Elasticity from Stress-Strain Curve

Figure 3.8 Adek Percometer™

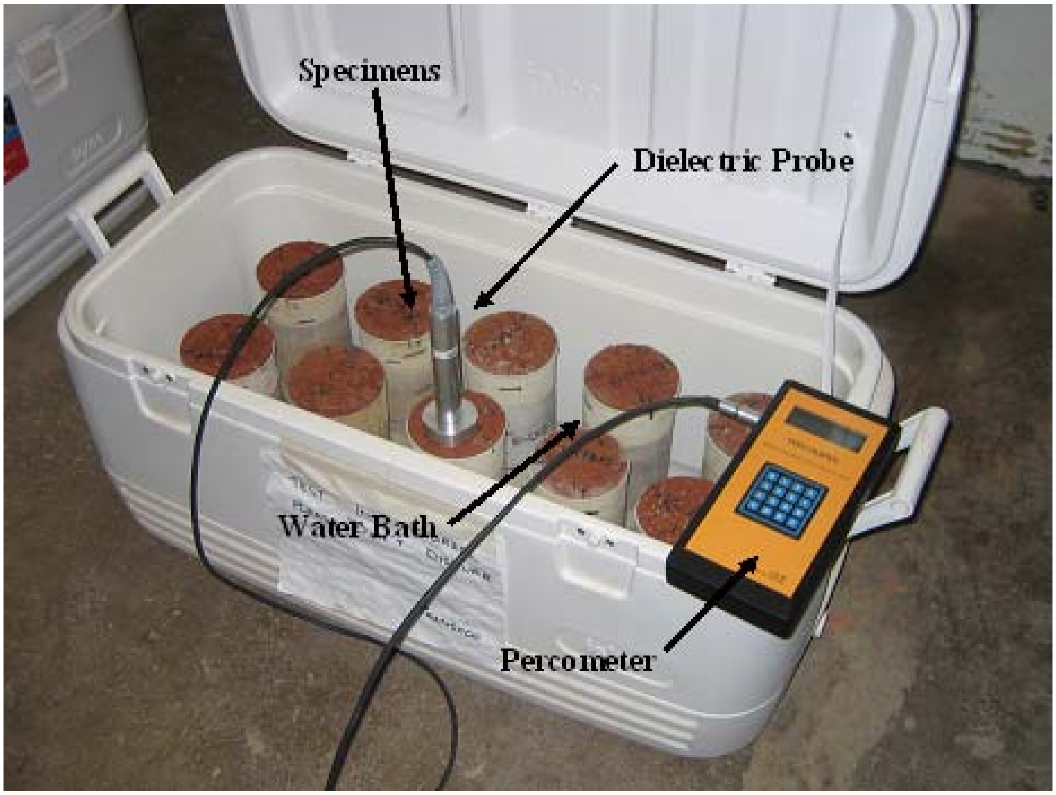
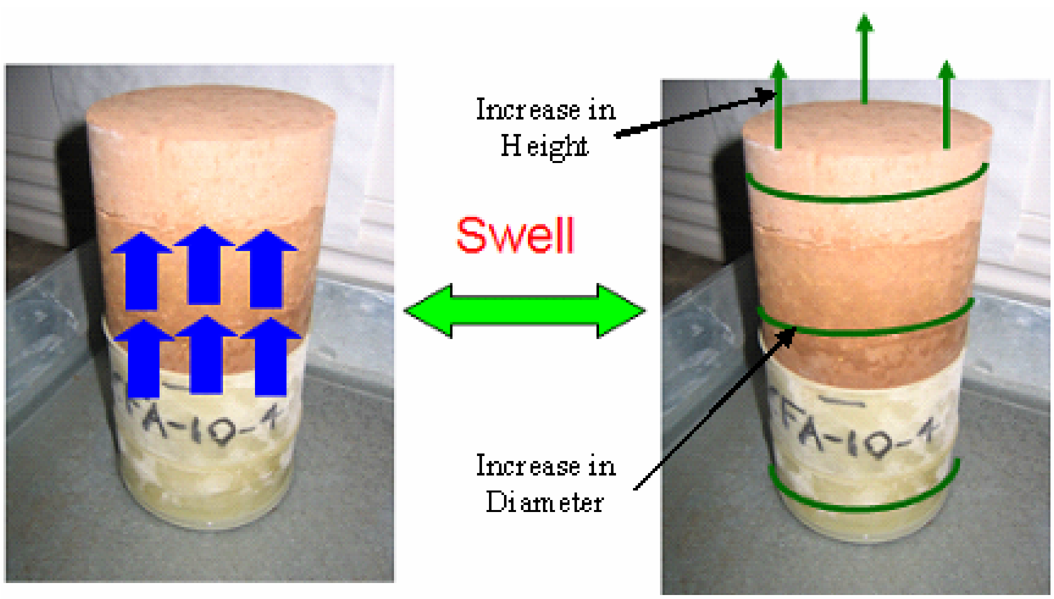


Figure 3.9 Setup for Tube Suction Test



(a) (b)

Figure 3.10 Three-Dimensional Swelling of Specimens: (a) Capillary Rise of Water (b) Swelling

(a) (b)

Figure 3.11 Three-Dimensional Swelling Measurements (a) Diameter (b) Height

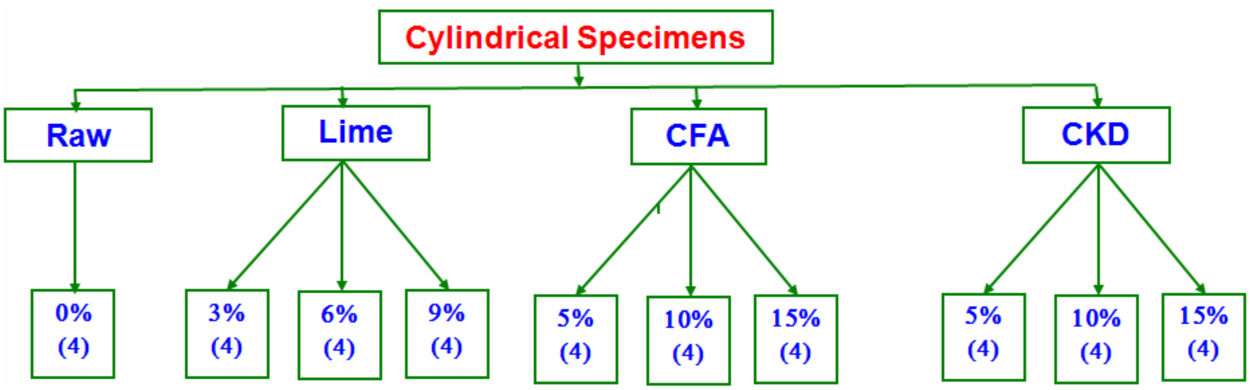


Figure 3.12 Test Matrix

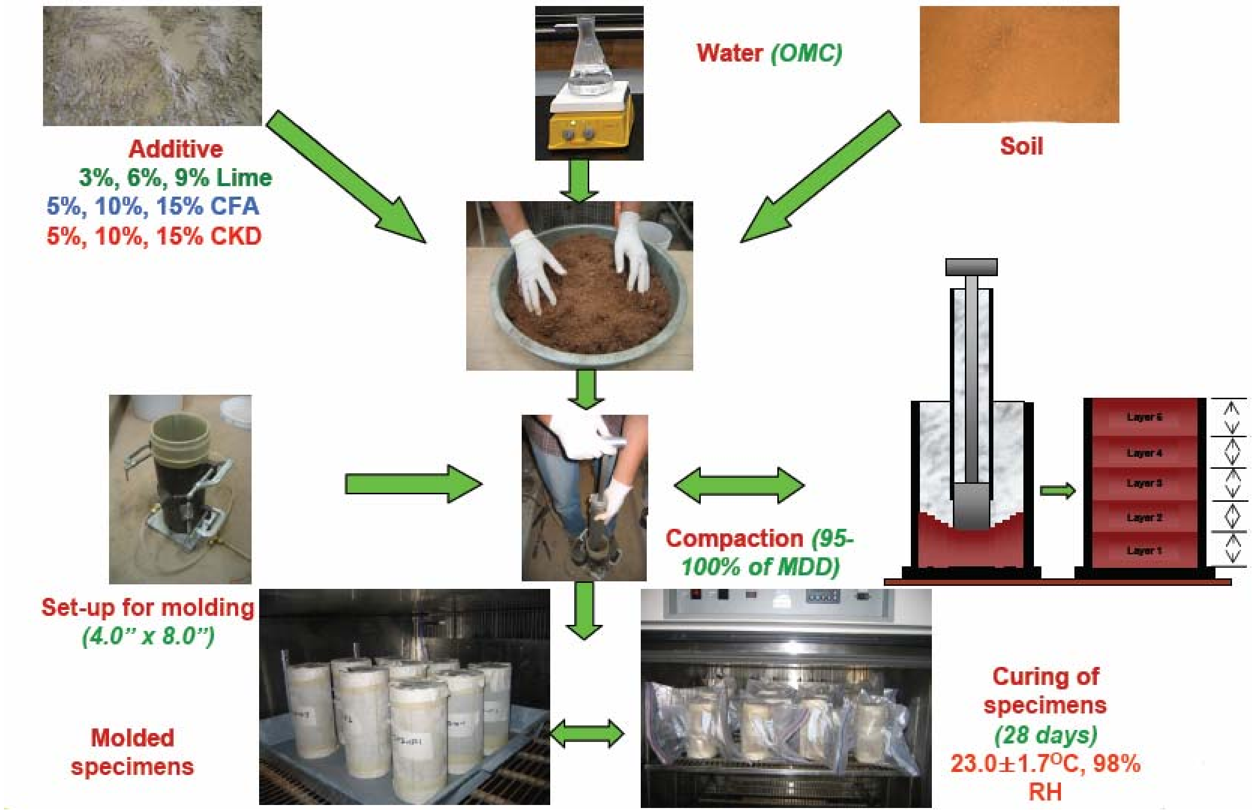


Figure 3.13 Sample Preparation

4.1 General

This chapter is devoted to presenting and discussing the results of pH, resilient modulus (M_r), modulus of elasticity (M_E), unconfined compressive strength (UCS), moisture susceptibility and three-dimensional (3-D) swelling tests. Emphasis is placed on evaluating the effect of lime, CFA and CKD on the aforementioned properties of stabilized specimens.

4.2 pH Test

The pH values of soil-additive mixtures were determined to investigate whether pH would reflect the performance of stabilized soil specimens. Results are presented in Table 4.1 and are used as the primary guide for determining the amount of additive required to stabilize each soil, as recommended by ASTM D 6276. The amount of additive selected for use in treatment was based on the percentage required to reach an asymptotic pH value in a soil-additive mixture. It is noteworthy that an elevated pH level is important to promote cementitious/chemical activity (Little, 1999).

4.2.1 Effect of Lime Content

For lime, eight different percentages (i.e., 0%, 1%, 3%, 5%, 6%, 7%, 9% and 100%) of soil-lime mix were selected for the pH test. It was found that raw lime had a pH value of 12.58. As shown in Figure 4.1, all the pH values increase with the increase in the percentage of lime and show an asymptotic behavior after a certain percentage. In the current study, an increase of less than 1% in pH values is assumed as starting point of asymptotic behavior. As evident from Table 4.1, pH values started showing an asymptotic behavior with 3% lime for P-, K- and V-soil. However, C-soil, due to acidic nature, attained asymptotic behavior at a higher lime content of 5%. These lime contents also provided a minimum pH value of 12.4, as recommended by ASTM D 6276.

4.2.2 Effect of CFA Content

For CFA, nine different percentages (i.e., 0%, 2.5%, 5%, 7.5%, 10%, 12.5%, 15%, 17.5% and 100%) of soil-CFA mixes were selected for the pH test. Raw CFA gives a pH of about 11.83, which is consistent with the results reported by Sear (2001). The results of pH tests for the four selected soils mixed with different percentages of CFA are presented in Figure 4.2. It is evident that the pH values of P-soil, and K-soil increased as the percentage of CFA increased and attained an asymptotic behavior at 10% of CFA. On the other hand, V-soil attained asymptotic behavior at a higher CFA percentage (12.5%). C-soil, having a pH value of 4.17 never attained an asymptotic behavior with CFA contents up to 17.5%. This can be attributed to the acidic behavior of C-soil which requires higher amount of moderately basic CFA for neutralization.

4.2.3 Effect of CKD Content

The pH values of specimens prepared with various CKD contents are presented in Figure 4.3. Figure 4.3 shows that raw CKD specimens have a pH of 12.55, which is similar to the results reported by other researchers (e.g., Miller and Azad, 2000 and Parsons et al., 2004). It is also evident that the pH values of P-, K-, V- and C-soil exhibited the same trends as CFA-soil mixtures. The mixture of P-, K- and V-soil with CKD attained asymptotic behavior at 10%, 10% and 12.5% of CKD, respectively. The pH values of C-soil never attained asymptotic behavior with CKD contents upto 17.5%. Hence similar reasons, as mentioned in the preceding section, can be used to justify this performance.

4.3 Resilient Modulus Test

The M_r test results of the selected soils stabilized with lime, CFA and CKD are shown in Tables 4.2 to 4.4. Each M_r value listed in Tables 4.2 to 4.4 is an average of M_r tests conducted on four specimens. One way to observe the effect of different percentages of additives on the resilient properties is to compare the average M_r at a particular stress level (Drumm et al. 1997). A simple and commonly model used ODOT was chosen in this study for this purpose.

$$M_r = k_1 \ x \ S_d^{k_2}$$

In this model, the M_r is expressed as a function of cyclic axial stress (S_d). The M_r values were calculated at a S_d of 6 psi and a confining pressure (S_3) of 4 psi, as suggested by ODOT (Dean 2009). The results are presented in the form of bar chart in Figure 4.4.

4.3.1 Effect of Lime Content

It is clear that M_r values increased due to stabilization. This increase, however, depends on the type of soil. For example, 3% lime provided an increase of approximately 435%, 1,647%, 914% and 123% with P-, K-, V- and C-soil, respectively. This improvement is maximum with K-soil, however, a reduction in M_r values was observed beyond a certain percent (Figure 4.4). For example, K-soil specimens stabilized with 9% lime showed 28 percent decrease in M_r values as compared to specimens stabilized with 6% lime. This is consistent with other studies (Haston and Wohlgemuth, 1985; Petry and Wohlgemuth, 1988; Osinubi and Nwaiwu, 2006) that an increase in lime beyond 5% results in lower strength values. One explanation is that excess lime behaved as low strength filler, effectively weakening the lime-soil mixture (Osinubi and Nwaiwu, 2006).

4.3.2 Effect of CFA Content

From Table 4.3 and Figure 4.4, one can see that the average M_r value increased with the increase in the percentage of CFA. The increase in M_r values with increased amount of CFA is consistent with the studies conducted by other researchers such as McManis and Arman (1989), Chang (1995), Misra (1998), Senol et al. (2002), Mir (2004), and Arora and Aydilek (2005). It is evident from Figure 4.4 that for the percentages used in this study, 15% CFA-stabilized specimens showed a maximum increase in M_r values of approximately 983%, 1,449%, 1,203%, and 215% for P-, K-, V- and C-soil specimens, respectively, as compared to raw soil. For 5% and 10% CFA, K-soil specimens showed highest improvements of approximately 553% and 1319%, respectively. Hence, K-soil showed the highest improvements with CFA stabilization.

4.3.3 Effect of CKD Content

Figure 4.4 summarized the effect of CKD on M_r. Results showed that M_r increased with the increased percentage of additive; this is consistent with Zhu (1998), Miller and Azad (2000), Parsons et al. (2004), Khoury (2005), Solanki et al. (2007a). For example, the M_r values of 15% CKD-stabilized specimens increased as much as 1,963%, 2,998%, 2,001%, and 691% for P-, K-, V- and C-soil, respectively. As depicted in Figure 4.4, a large increase in average M_r can be observed when the CKD content is increased from 0 to 5%, 5 to 10% and 10 to 15%. This rate of increase in M_r values is the highest between 5% and 10% CKD. For example, this increase is 341%, 262%, 352% and 103% for P-, K-, V- and C-soil, respectively. This finding indicates that CKD showed best performance with K-soil. In the present study, CKD treatment (>10%) resulted in the highest M_r values (Figure 4.4).

To study the comparative effectiveness of lime, CFA and CKD on the four soils, graphs of percent improvement in M_r values vs percentage of additive were plotted (Figure 4.5 – 4.8). For all the four soils used in this study, it is clear, in general, that at lower application rates (3% to 6%), the lime-stabilized soil specimens showed the highest improvement in the M_r values. At higher application rates (10% to 15%), however, the CKD treatment provided the maximum enhancements. Overall, 15% CKD-stabilized specimens showed the highest improvement for all the four soils. In addition, stabilization of K-soil resulted in the maximum enhancement in M_r values (Figure 4.6). On the other hand, C-soil specimens showed much lower improvements in M_r values, as shown in Figure 4.8. One of the explanations could be differences in the pH values. For example, raw K- and C-soil are having the highest (pH = 9.07) and the lowest (pH = 4.17) pH values, respectively, among all the four soils used in this study (pH = 4.17) of C-soil.

It is believed that the difference in M_r values are attributed to the differences in physical and chemical properties of the additives presented in Tables 2.3, which leads to various pozzolanic reactions. The pozzolanic reactivity of a cementitious additive depends on the following four properties: (1) silica/sesquioxide ratio (SSR); (2) percentage of additive passing No. 325 sieve; (3) loss on ignition or carbon content; and (4) alkali contents or the free lime content that will eventually contribute to the alkali content (NCHRP 1976; Bhatty and Todres 1996; Zaman et al. 1998; Parsons et al. 2004; Khoury 2005). In this study, the highest M_r value of 15% CKD-stabilized specimen after 28-day curing can be attributed to the CKD characteristics such as high SSR and high free lime content (as shown in Table 2.3).

4.4 Modulus of Elasticity and Unconfined Compressive Strength

The variation of modulus of elasticity (M_E) and UCS values with the additive content is shown graphically in Figures 4.9 and 4.10, respectively. The UCS values were found to be 33, 28, 24 and 30 psi for the raw P-, K-, V- and C-soil, respectively. In general, the trend of the behavior of M_E and UCS values for different percentages of additives is the same as that observed for M_r values. Hence, similar reasons, as mentioned in the preceding sections, can be used to justify this performance.

4.4.1 Effect of Lime Content

As depicted in Figure 4.9, in lime-stabilized specimens an increase of approximately 186%, 516%, 436% and 72% in M_E values was observed for 3% lime-stabilized P-, K-, V- and C-soil specimens, respectively. Similarly, addition of 3% lime increased the UCS values by 64%, 136%, 304% and 20% for P-, K-, V- and C-soil, respectively. It is clear that K- and V-soil showed the highest improvement with lime. On the other hand, C-soil with the lowest pH value showed the lowest enhancements in both M_E and UCS values.

4.4.2 Effect of CFA Content

It is evident from Figures 4.9 and 4.10 that there is a significant increase in M_E and UCS with increasing CFA content in the treated soils. A maximum increase of 367%, 586%, 616%, and 95% was observed in M_E values for 15% CFA stabilized P-, K-, V- and C-soil, respectively. Correspondingly, these different stabilized soil specimens showed an increase in UCS values by 273%, 246%, 404%, and 100%. Clearly, V-soil specimens stabilized with CFA showed better performance, as compared to other soils used in this study.

4.4.3 Effect of CKD Content

It is evident that there is significant increase in the M_E with increasing amount of CKD content in the stabilized soils (Figure 4.9). The M_E values in all soils exhibited an increase with the amount of CKD. As depicted in Figure 4.9, in P-soil specimens the maximum increase (about 638%) in M_E values was observed by adding 15% CKD. Similarly, 15% CKD-stabilized K-, V- and C-soil specimens exhibited the maximum increase of approximately 1061%, 1042% and 194%, respectively, compared to the raw soil. This trend in M_E values for different CKD-stabilized clays is similar to that observed for M_r values. Hence, similar reasons, as mentioned in the preceding section, can be used to justify this performance. The variation of UCS values with the CKD content is illustrated in Figure 4.10. It is observed that UCS values of all the soils used in this study increases as the amount of CKD increases. For example, the UCS values increased by 521%, 500%, 717%, and 153% for the P-, K-, V-, and C-soil specimens, respectively, when stabilized with 15% CKD. This observation is consistent with that of Miller and Azad (2000), Sreekrishnavilasam et al. (2007), and Peethamparan and Olek (2008).

4.5 Stress-Strain Behavior

The stress-strain behavior of the four raw soils, 3% lime-, 10% CFA- and 10% CKD-stabilized specimens are presented in Figures 4.11 to 4.14, respectively. A summary of failure strain of all the raw and stabilized specimens is presented in Figure 4.15. It is evident from Figure 4.11 to 4.15 that the addition of additives (lime or CFA or CKD) increased the peak stress (or UCS) and reduced the peak strain (or failure strain) considerably. Figure 4.16 shows the failure patterns of raw and stabilized C-soil specimens. As evident from Figure 4.16, specimens failed with an inclined failure plane or cylindrical shape/splitting.

According to OHD L-50 (ODOT 2006), percentage of CFA/CKD that gives a minimum strength of 50 psi but not more than 150 psi should be selected. Hence, for all the four soils, only those percentages of CFA or CKD fulfilling the above mentioned criteria were selected (Figure 4.17). For lime-stabilized soil specimens, amount of lime providing a minimum pH of 12.3 were selected following the ASTM D 6276 requirements. A summary of UCS of selected stabilized specimens is presented in Table 4.5. For P-soil (A-4), OHD L-50 (ODOT 2006) recommends use of 10% CKD or 12% CFA. For K-soil (A-6), OHD L-50 (ODOT 2006) recommends use of 12% CFA or 4% lime. For V-soil (A-6), OHD L-50 (ODOT 2006) recommends use of 12% CFA or 4% lime. For C-soil (A-7-6), OHD L-50 (ODOT 2006) recommends use of 5% lime. The OHD L-50 recommendations are comparable to the results obtained in this study (Table 4.5).

4.6 Moisture Susceptibility

A summary of the final dielectric constants values (DV) for the P-, K-, V- and C-soil stabilized specimens with different percentages of additives is summarized in Figure 4.18. The raw P-, K-, V- and C-soil specimens showed an average DV of approximately 38, 38, 31 and 34, respectively.

4.6.1 Effect of Lime Content

It is clear that lime is most effective additive in reducing the moisture susceptibility of the P-, K-, V-, and C-soil specimens. For example, 9% lime reduced DV of raw P-, V-, K- and C-soil by 47%, 24%, 6%, and 26%, respectively. These results are consistent with the observations made by Little (2000) and Barbu and McManis (2005). Little (2000) conducted TST on low, moderate and high plasticity soils stabilized with lime. He found a decrease of

DV for low plasticity soils from 6.5 to 4.7 (27.7% decrease) and suggested this as no significant difference. However, a decrease of DV from 31.2 to 10 (67.9% decrease) for moderate plasticity soil and 26.5 to 9 (64.1% decrease) for high plasticity soils was reported as significant reduction. In contrast to the above observation by Little (2000), the present study showed maximum improvement with soil having the lowest PI value (P-soil).

4.6.2 Effect of CFA Content

Figure 4.18 shows the effect of CFA-stabilization on the DV of P-, K-, V-, and C-soil stabilized specimens. The same qualitative trends as lime-stabilized specimens were observed. The DV decreased as the percentages of CFA increased up to 15%. The percentage decrease in DV due to 15% CFA was found to be approximately 8%, 11%, 16% and 9% for P-, K-, V- and C-soil specimens, respectively. It is an indication that CFA stabilization has more or less same degree of effectiveness in reducing the moisture susceptibility for all the soils. It is also worth noticing that CFA-stabilized specimens with P-soil showed a decrease in DV for 5% CFA-stabilized specimens, while 10% and 15% CFA stabilized specimens exhibited only a slight increase in the values (Figure 4.18). This may be attributed to the presence of extra CFA in the specimen which is not reacting with the host material; hence it absorbs water increasing the dielectric constant.

4.6.3 Effect of CKD Content

The variation of moisture susceptibility of P-, K-, V- and C-soil stabilized specimens with the percentages of CKD is shown in Figure 4.18. The DV of K- and C-soil specimens exhibited an increase with the percentages of CKD, an opposite trends as compared to lime- and CFA-stabilized specimens. For example, K- and C-soil specimens prepared with 15%

CKD showed an average increase of approximately 11% and 18% as compared to raw specimens. On the other hand, CKD-stabilization in P- and V-soil specimens helped by reducing DV values by 53% and 13%, respectively. Hence, CKD was found to be most effective with P-soil specimens.

4.7 Three-Dimensional Swell Behavior

Figures 4.19 to 4.21 show final 60-day three-dimensional (3-D) swell values for selected raw soils (K-, V- and C-soil) and stabilized specimens. Negative swells are a result of drying of the specimens before placing them in water bath for the swell test. A summary of final 60-day 3-D swell values is presented in Figure 4.22. Further, the effect of different additives on 3-D swell values is discussed in the following section.

4.7.1 Non-sulfate Bearing Soil (K- and C-soil)

The effects of different additives on the percentage reduction in 3-D swell values of K- and C-soil are presented in Figures 4.23 and 4.24, respectively.

4.7.1.1 Effect of Lime Content

For K- and C-soil stabilized specimens, the 3-D swell values decreased as the percentage of lime increased up to 9 percent. For example, the K-soil specimens prepared with 9% lime had an average 3-D swell value of -2.8% compared to 6.1% for raw specimens. From Figures 4.23 and 4.24, it can also be concluded that the 3-D swell values of stabilized materials varied with the type of soil. For example, 3% lime in K-soil stabilized specimens reduces approximately 95% of raw soil swelling, whereas the same percent of lime in C-soil reduces almost 1C00% swelling of raw soil specimens (Figure 4.23 and 4.24).

4.7.1.2 Effect of CFA Content

Figure 4.22 shows the 3-D swell values of specimens stabilized with various percentages of CFA. Similar to lime-stabilized specimens, 3-D swell values decreased with the percentages of CFA. It is also obvious from Figures 4.23 and 4.24 that CFA stabilization is more effective in reducing swelling of K-soil specimens as compared to C-soil specimens. For example, 15% CFA reduced approximately 54% and 32% 3-D swell in K- and C-soil specimens, respectively. CFA-stabilization, however, is less effective in reducing swelling, as compared to lime.

4.7.1.3 Effect of CKD Content

In contrast to lime- and CFA-stabilized specimens, CKD-stabilized specimens showed an increase in 3-D swell values as the percentages of CKD increased up to 15. The specimens of K- and C-soil stabilized with 15% CKD showed an increase in 3-D swell by 98% and 113%, respectively. This issue has been further discussed in the following sections.

4.7.2 Sulfate Bearing Soil (V-soil)

The effect of different additives on the percentage reduction in 3-D swell values of V-soil is presented in Figure 4.25.

4.7.2.1 Effect of Lime Content

The V-soil (sulfate content ≈ 15,400 ppm) specimens stabilized with lime showed an increase in 3-D volume. Addition of 3 percent lime increased the swelling of raw soil by 1237%. Swelling of lime-stabilized specimens can be attributed to the presence of high soluble sulfate content in the V-soil, which could lead to the formation of an expansive mineral ettringite, known as primary sulfate attack (see e.g., Mitchell, 1986; Mitchell and

Dermatas, 1990; Rao and Shivananda, 2005). As discussed in Chapter-5, formation of ettringite was verified by conducting mineralogical studies such as SEM/EDS and XRD analysis.

4.7.2.2 Effect of CFA Content

It is evident from Figure 4.25 that V-soil stabilized specimens showed a reduction in 3-D swell values with CFA. For example, 5%, 10% and 15% CFA reduced 3-D swell of raw V-soil specimens by approximately 40%, 75% and 145%, respectively.

4.7.2.3 Effect of CKD Content

Similar to non-sulfate bearing soils (K- and C-soil), the specimens of V-soil stabilized CKD showed an increase in 3-D swell. For example, 15% CKD increased 3-D swell of raw specimens by 593%.

4.7.3 Swell Assessment

The increase in swell of CKD-stabilized specimens can be attributed to the presence of high soluble sulfate content in CKD, which will correspond to high soluble sulfate content in the soil-CKD mix. In order to explain such a behavior, sulfate tests were performed on CKD-stabilized specimens. Result showed that high soluble sulfate content (> 2,000 ppm) existed in stabilized specimens, as shown in Figure 4.26. According to a study by Kota et al. (1996), sulfate levels of greater than 2,000 ppm in soil-additive mix could potentially result in sulfate-induced heaving due to the formation of expansive mineral ettringite. To confirm the ettringite formation, SEM/EDS study was also conducted on representative tiny pieces of 15% CKD-stabilized K-, V- and C-soil specimens, after 60 days of swelling, as will be discussed later in Chapter-5. Sulfate present in additives, water, and spilled chemicals constitute the “secondary” sulfate source (Rao and Shivananda, 2005). Although there has been significant research on the “primary” sulfate-induced heaving of stabilized subgrade soils (Hunter 1988; Mitchell and Dermatas 1990; Petry and Little 1992; Rajendran and Lytton 1997; Rollings et al. 1999; Puppala et al. 2004), only a few studies have identified and addressed the “secondary” sulfate-induced heaving problems. For example, Hopkins and Beckham (1999) observed swelling of highway subgrade stabilized with an additive (residue of atmospheric fluidized bed combustion, AFBC). Mineralogical studies such as scanning electron microscopy (SEM) and X-ray diffraction (XRD) analysis showed the presence of ettringite, thaumasite, and gypsum throughout the AFBC-stabilized subgrades. Using chemical analyses technique, the presence of high concentration (10%) of calcium sulfate in AFBC was also found.

In a laboratory study, Miller and Azad (2000) observed soluble sulfate levels varying from 2,270 to 25,800 ppm in CKDs from three different manufacturers. Their study, however, focused on determining pH, Atterberg limits and UCS of soil stabilized with low sulfate content CKD (6,450 ppm). In another laboratory study, Rao and Shivananda (2005) examined “secondary” sulfate-induced heaving from spillage of sulfate rich chemicals. The objective was realized by infiltrating laboratory prepared sodium sulfate solutions (sulfate concentrations ranged from 13,500 to 27,000 ppm) on the heave characteristics of lime-stabilized specimens that were practically free of natural sulfate. Experimental results illustrated that lime-stabilized expansive soils experiencing sulfate contamination are susceptible to sulfate-induced heave.

Mohamed (2002) observed one-dimensional swelling of specimens stabilized with CKD. For example, raw and 10% CKD-stabilized specimen showed a 7-day swell value of

0.1% and 0.4%, respectively. Swelling of CKD-stabilized specimens was attributed to the formation of ettringite in the soil-CKD mix. It was also reported that formation of ettringite depends on pH value greater than 11.7. In the current study, it is expected to have higher swelling values because the ratio of 1-D swell to the 3-D swell is approximately 0.55 (Al-Shamrani and Al-Mhaidib 2000). In addition, 60-day swell values should be higher compared to 7-day values reported by Mohamed (2002).

In a recent combined laboratory and field study, Si and Herrera (2007) identified CKD as a potential sulfate source. It was also found that the addition of more CKD increased sulfate content in the pavement material. Further, increase in dielectric constant and conductivity was also noticed for specimens stabilized with 2% CKD. But, no attempt was made to evaluate and compare the Mr, one of the critical pavement performance parameters (AASHTO 2004).

As noted from limited available literature, most of the studies identified a “secondary” sulfate-induced heaving problem, but only few addressed this issue. It is also worth noticing that properties of CKD can vary significantly from plant to plant depending on the raw materials and type of collection process used (Miller and Zaman 2000). Similarly, fly ash properties may be unique to same source while it may differ from ashes obtained from other sources (Ferguson and Levorson 1999). These differences in physical and chemical properties can lead to different performance of stabilized soil specimens. In the present study, for example, CKD showed swelling of specimens due to high sulfate content (28,133 ppm), while CFA with lower sulfate content (3,280 ppm) exhibited reduced swelling.

4.8 Atterberg Limits

A summary of the Atterberg limits (after 28-day curing) for selected soil types, namely, K-, V-, and C-soil, and percentage of additives are presented in Table 4.6 and Figure 4.27. It is observed that lime is the most effective additive in reducing plasticity index (PI). As evident from Figures 4.28 to 4.30, all the three soil series used in the current study showed similar trend of reductions in PI properties with lime. Reduction in PI values for lime-stabilized soil specimens are well known and are attributed to chemical reactions between lime and soils including ion exchange and associated flocculation reactions (see e.g., Prusinski and Bhattacharia, 1999; IRC, 2000).

Adding CFA and CKD to the soils also produced changes in the plasticity. The percentage of reduction in PI was observed maximum with K-soil among all the three soils (K-, V- and C-soil). For example, 15% CKD reduced PI values of K-, V- and C-soil by 67%, 18% and 21%, respectively. This could also be one of the reasons for highest improvement in M_r values of stabilized K-soil specimens, as discussed in section 4.3. However, effectiveness of CFA or CKD in reducing the plasticity of soil is low as compared to lime (Figure 4.19). One of the explanations could be less alkalinity (or pH) of CFA and CKD, as compared to lime. Similar observations of unproductive effect of CKD on PI were reported by other researchers (Parsons et al., 2004; Miller and Azad, 2000).

4.9 Parameter Ranking and Identification of Best Additives

An attempt is made here to rank the additives based on their contributions to enhancements to soil properties of PI, UCS, M_r, final 3-D swell and DV values. The recommendations made by Nelson and Miler (1992), Wattanasanticharoen (2000) and Chavva et al. (2005) were followed in the ranking analysis. It should be noted that the ranges for final dielectric constant were arbitrarily chosen. However, the arbitrary selection would not influence the overall rank since the same ranking was used to characterize all stabilized soil specimens.

The established ranking systems characterize the transformation of each soil property from problematic to non-problematic levels. If the soil-additive mix condition is poor; it is assigned a rank of zero. If the condition of the soil-additive mix is the best, the rank is given as four. The rankings of 1 to 3 are given for the middle ranges of soil-additive mix properties between severe and non-severe conditions. Table 4.7 shows the ranking scale for all the parameters used in this analysis. The values for PI, UCS and M_r were developed from the documented literature information (Wattanasanticharoen 2000; Chavva 2005). The recommendations made by Nelson and Miller (1992) were used for formulating vertical swell values. The ratio of swell in the vertical direction to the volumetric swell was assumed to be 0.55 for converting vertical swell to 3-D swell (Al-Shamrani and Al-Mhaidib 2000).

All the ranks of each additive for various test results were compiled, averaged and then presented as an overall rank (OR). Tables 4.8, 4.9 and 4.10 provide ranking scores for stabilized K-, V- and C-soil specimens used in the present study. The OR value was used to identify and select the best, medium and low performers among different additives. The following discussion is presented for each additive based on the results observed on all three soil types.

For lime, the OR ranged from 1.8 – 3.0. A maximum rank increase of 2.2 was observed by this stabilization method. This finding indicates significant improvements in soil properties were obtained with the lime-stabilization method. The best performance was obtained when 9% lime was used to stabilize K-soil.

For CFA, the OR of stabilized soils ranged from 1.2 – 3.0. A maximum rank increase of 1.4 was observed with this stabilization method. This finding indicates that only moderate improvements in soil properties were obtained with the CFA-stabilization. The best performance was obtained with the sulfate-bearing soil (V-soil).

For CKD, the OR ranged from 0.8 – 2.8. A maximum rank increase of 1.2 was observed with this stabilization method. This finding indicates that moderate improvements in soil properties were obtained with the CKD-stabilization. The best performance was obtained when 15% of CKD was used to stabilize P-soil.

When all additive results are grouped and compared with respect to the effectiveness with different soils, 9% lime showed the best performance with the non-sulfate soils (K- and C-soil). On the other hand, 15% CFA showed the best performance with the sulfate-bearing soil (V-soil).

4.10 Statistical Analysis

As noted in Table 1.1, only a few number of models and correlations are available in the literature for predicting M_r, but those models are either limited to one type of additive (e.g. Achampong, 1996 and Arora and Aydilek, 2005) or applicable only for a particular stress level (e.g., Thompson 1966; Boyce 1980; Chen 1994; AASHTO 2004; and Hillbrich and Scullion 2006). Only a few studies for e.g. Khoury and Zaman (2007) conducted statistical analysis for predicting M_r values by considering effect of different additive properties and specimen properties at different stress levels. However, no studies, to the authors' knowledge, have addressed the statistical model for stabilized soil specimens correlating soil-additive mix properties with M_r values at different stress levels.

4.10.1 Model Development

Literature review revealed several models to correlate the resilient modulus of pavement materials with stresses. For example, Witczak (2000) reported that 14 models are available for predicting the M_r values of unbound pavement materials. In the present study, the cyclic axial stress and confining pressure were used in the following form to predict the resilient modulus:

Model-1(AASHTO 2004): This log-log model was selected because it is recommended by the new MEPDG for unbound materials.

$$M_r = k_1 p_a \left(\frac{\theta}{p_a} \right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1 \right)^{k_3} \qquad (k_1 \geq 0,\ k_2 \geq 0,\ k_3 \leq 0) \qquad (4.1)$$

where, p_a = atmospheric pressure (14.7 psi), θ = bulk stress (sum of three principal stresses), τ_{oct} = octahedral shear stress acting on the material, and k_1, k_2 and k_3 are the model constants. Using the stepwise method of linear regression option in SAS 9.1, these model constants are correlated with the soil-additive mix properties (e.g., dry density, moisture content, UCS, M_E) and additive properties (percent of free lime, SAF, percent passing #325 and percent loss on ignition).

$$k_1 = D_d^{A_1} \times M_E^{A_2} \times UCS^{A_3} \times SAF^{A_4} \times P325^{A_5} \times LOI^{A_6}; \quad k_2 = -0.053; \quad k_3 = -0.720 \qquad (4.2)$$

(R^2 = 0.914; Pr<0.0001)

where, D_d is ratio of molded dry density of specimen to maximum dry density, M_E is the modulus of elasticity, UCS is the unconfined compressive strength and SAF, P325 and LOI are the final product of the percentage and amount of SAF, passing No. 325 and loss on ignition value for the additive used. A_1 through A_6 are model constants with the following values obtained from the regression analysis:, A_1 = 2.912, A_2 = 1.368, A_3 = -0.233, A_4 = 0.133, A_5 = -0.323 and A_6 = 0.150. The F test for the multiple regressions was conducted using statistical analysis software (SAS 2004) to validate the significance of the relationship between M_r and independent variable included in the model. The associated probability is designated as p-value. A small p-value implies that the model is significant in explaining the variation in the dependent variable.

Model-2 (Witczak 2000): This semi-log model was selected on the basis of previous studies conducted on bound materials (Solanki et al. 2008)

$$M_r = k_1 \times k_2^{S_d} \times k_3^{S_3} \tag{4.3}$$

In a logarithmic form, the model can be written as:

$$\log(M_r) = \log(k_1) + S_d \log(k_2) + S_3 \log(k_3) \tag{4.4}$$

This model is similar to semi-log k_1, k_2, k_3 (S_3, S_d) model reported by Witczak (2000) and Andrei et al. (2004). Also, Khoury and Zaman (2007) used the same model to assess the durability effect on the resilient modulus of stabilized aggregate bases. One of the advantages of using the aforementioned semi-log model is that it is valid for either $S_3 = 0$ or $S_d = 0$.

This model correlating the variation of actual M_r test values with the aforementioned mix properties (stabilized soil specimen and additive) and stress levels was developed using the stepwise method at a 0.15 level. The stepwise method consists of entering variables in the final model at a certain significant level (0.15 significant level was used in this study). It was statistically found that the final model of predicting M_r is a function of additive properties (i.e., amount of SAF, percent passing No. 325 and loss on ignition value), mechanical properties of the mixture (i.e. M_E and UCS), and stress levels. The model is given in the following equation:

$$M_r = A_1 \times M_E^{A_2} \times UCS^{A_3} \times SAF^{A_4} \times P325^{A_5} \times LOI^{A_6} \times A_7^{S_d} \times A_8^{S_3} \tag{4.5}$$

($R^2 = 0.927$; Pr<0.0001)

where, M_E is the modulus of elasticity, UCS is the unconfined compressive strength, S_d is deviator stress, and S_3 is confining pressure. SAF, P325 and LOI are the final product of the percentage and amount of SAF, passing No. 325 and loss on ignition value for the additive used, respectively. The regression analysis yields the following coefficients: A_1 = 0.253, A_2 = 1.462, A_3 = -0.313, A_4 = 0.141, A_5 = -0.279, A_6 = -0.139, A_7 = 0.995 and A_8 = 1.002. The relative effects of mechanical properties of mixture and chemical properties of additive are summarized in Table 4.11. The analysis of variance (ANOVA) results show that the effects of M_E, UCS, SAF, P325, LOI and stress levels is statistically significant at $\alpha = 0.05$ (i.e., $p<0.05$). The corresponding R^2 value is 0.927 and the F-value is 543 with a $Pr<0.0001$, which indicates that the model is considered statistically significant in predicting the variation of M_r values with stress level and type of additive. Since Model-1 and Modle-2 yielded very similar R^2 values, Model-2 was selected for validation because of added advantage of validity of this model at $S_3 = 0$ or $S_d = 0$.

4.10.2 Validation of Model

The selected model-2 was validated using resilient modulus data of P-soil, as mentioned previously. This provides different views on the prediction quality and the importance of datasets on statistical analysis (Myers et al. 2001; Montgomery et al. 2006). A comparison between the predicted M_r values and the actual M_r values is illustrated in Figure 4.31. From this figure, it is evident that the scatters for stabilized K- as well as P-soil are around the 45^o line. It is also evident that the predicted values are closer to the equality line when the M_r values are less than 2,500 MPa for both K- and P-soil. This observation may be due to the distribution of dataset. For K-soil, only 44 M_r values out of 313 M_r values (approximately 14%) are in the upper range of 2,500 MPa. Similarly, 59 M_r values out of 326

M_r values (approximately 18%) are in the upper range of 2,500 MPa. The remaining 86% and 82% of the M_r values for this study are in the lower range of the development dataset for K- and P-soil, respectively. Furthermore, a frequency histogram was plotted to compare the predicted M_r values for both stabilized K- as well as P-soil, as illustrated in Figure 4.32. The trend clearly shows the similar kind of trend and magnitude of error for both K- (development) and P- (validation) soil. This discussion indicates that such a model could be a good indicator in making performance predictions of resilient modulus of stabilized soil specimens.

Table 4.1 Variation of pH Values with Soil and Additive Type

Type of Additive	Additive Content (%)	P-soil		K-soil		V-soil		C-soil	
		pH value	% Increase	pH value	% Increase	pH value	% Increase	pH value	% Increase
Lime	0	8.91	0.0	9.07	0.0	8.14	0.0	4.17	0.0
	1	12.24	37.4	12.04	32.7	11.67	43.4	9.22	121.1
	3	***12.43***	***39.5***	***12.49***	***37.7***	***12.41***	***52.5***	12.23	193.3
	5	12.45	39.7	12.5	37.8	12.49	53.4	***12.54***	***200.7***
	6	12.45	39.7	12.54	38.3	12.52	53.8	12.58	201.7
	7	12.46	39.8	12.57	38.6	12.52	53.8	12.61	202.4
	9	12.47	40.0	12.57	38.6	12.52	53.8	12.63	202.9
	100	12.58	41.2	12.58	38.7	12.58	54.5	12.58	201.7
CFA	0	8.91	0.0	9.07	0.0	8.14	0.0	4.17	0.0
	2.5	10.97	23.1	10.03	10.6	10.4	27.8	5.19	24.5
	5	11.3	26.8	10.83	19.4	10.85	33.3	5.93	42.2
	7.5	11.39	27.8	11.28	24.4	11.05	35.7	6.55	57.1
	10	***11.5***	***29.1***	***11.42***	***25.9***	11	35.1	7.79	86.8
	12.5	11.59	30.1	11.5	26.8	***11.15***	***37.0***	8.32	99.5
	15	11.6	30.2	11.57	27.6	11.19	37.5	8.86	112.5
	17.5	11.62	30.4	11.61	28.0	11.38	39.8	9.47	127.1
	100	11.83	32.8	11.83	30.4	11.83	45.3	11.83	183.7
CKD	0	8.91	0.0	9.07	0.0	8.14	0.0	4.17	0.0
	2.5	11.35	27.4	11.11	22.5	10.99	35.0	7.05	69.1
	5	11.88	33.3	11.73	29.3	11.59	42.4	8.8	111.0
	7.5	12.09	35.7	12	32.3	11.79	44.8	10.11	142.4
	10	***12.22***	***37.1***	***12.15***	***34.0***	12.04	47.9	10.88	160.9
	12.5	12.31	38.2	12.23	34.8	***12.24***	***50.4***	11.28	170.5
	15	12.36	38.7	12.3	35.6	12.32	51.4	11.62	178.7
	17.5	12.38	38.9	12.36	36.3	12.38	52.1	11.98	187.3
	100	12.55	40.9	12.55	38.4	12.55	54.2	12.55	201.0

Table 4.2 M_r Values of Different Soils Stabilized with Different Percentage of Lime

σ_3	σ_d	P-soil (M_r psi)				K-soil (M_r psi)				V-soil (M_r psi)				C-soil (M_r psi)			
(psi)	(psi)	Raw	3% Lime	6% Lime	9% Lime	Raw	3% Lime	6% Lime	9% Lime	Raw	3% Lime	6% Lime	9% Lime	Raw	3% Lime	6% Lime	9% Lime
6	1.8	26,205	247,386	115,777	201,503	14,150	170,374	200,546	134,826	21,014	138,982	119,171	108,191	19,944	42,525	83,150	61,074
6	3.6	22,193	124,287	158,401	162,555	12,093	163,979	175,077	118,028	18,333	136,614	112,024	102,399	18,787	40,260	72,800	56,275
6	5.4	19,834	100,768	118,249	124,860	9,875	159,822	154,282	109,582	15,640	132,967	107,387	101,573	16,981	37,268	68,515	52,273
6	7.2	18,404	89,278	102,798	106,957	8,322	155,180	143,319	105,192	13,598	132,233	103,045	99,012	15,395	34,458	64,650	48,479
6	9.0	17,482	79,894	92,464	94,406	7,321	149,086	134,439	98,675	12,068	127,069	104,116	94,543	13,966	31,996	61,196	45,031
4	1.8	23,392	170,719	117,212	182,101	13,371	160,702	182,071	120,221	18,362	152,295	122,294	104,383	18,822	41,809	80,498	59,338
4	3.6	19,298	116,019	135,557	127,976	11,235	162,140	160,861	111,717	15,019	133,859	108,797	101,283	17,884	37,849	71,299	54,364
4	5.4	17,447	93,940	98,547	102,505	9,420	153,101	143,626	106,865	13,121	131,918	105,801	98,069	16,651	35,181	66,874	49,144
4	7.2	16,483	82,754	90,666	93,226	8,204	149,349	139,052	101,526	12,046	130,015	101,971	96,101	15,158	32,843	63,152	47,254
4	9.0	15,912	77,330	85,930	88,965	7,338	150,056	133,772	98,321	11,195	126,162	98,405	93,611	13,887	31,423	61,377	44,581
2	1.8	21,146	158,533	*	182,979	13,052	171,073	175,519	120,752	17,289	146,600	119,052	102,980	16,976	41,029	75,269	58,053
2	3.6	16,888	107,358	143,690	122,513	10,901	158,252	161,842	112,039	13,848	136,275	109,720	99,139	16,475	37,481	68,592	52,164
2	5.4	15,210	86,669	100,849	101,357	9,134	154,139	145,995	104,909	11,997	130,401	105,115	96,970	15,455	34,362	65,085	48,434
2	7.2	14,400	77,246	91,320	89,165	7,985	150,966	136,615	101,493	10,952	128,134	101,441	94,397	14,305	32,382	62,550	46,274
2	9.0	13,946	73,176	90,998	84,905	7,195	151,225	133,229	96,865	10,291	125,934	99,324	93,321	13,241	30,909	60,723	43,231

1 psi = 6.89 kPa; 1 ksi = 6.89 Mpa; CFA: class C fly ash; CKD: cement kiln dust

σ_d : cyclic axial stress; σ_3 : confining pressure; Mr : resilient modulus (using internal LVDTs)

* Deformations are out of the measuring range of LVDTs

Table 4.3 M_r Values of Different Soils Stabilized with Different Percentage of CFA

σ_3	σ_d	P-soil (M_r psi)				K-soil (M_r psi)				V-soil (M_r psi)				C-soil (M_r psi)			
(psi)	(psi)	Raw	5% CFA	10% CFA	15% CFA	Raw	5% CFA	10% CFA	15% CFA	Raw	5% CFA	10% CFA	15% CFA	Raw	5% CFA	10% CFA	15% CFA
6	1.8	26,205	40,661	118,709	208,261	14,150	87,798	145,528	143,083	21,014	108,925	158,502	182,177	19,944	34,969	42,150	52,066
6	3.6	22,193	37,012	93,212	312,030	12,093	71,960	130,413	141,942	18,333	91,324	147,075	181,926	18,787	32,621	39,698	51,315
6	5.4	19,834	33,333	85,138	202,387	9,875	65,307	127,180	138,412	15,640	85,120	144,362	169,699	16,981	29,802	37,223	47,969
6	7.2	18,404	30,704	80,199	167,586	8,322	60,370	123,528	137,670	13,598	79,706	138,968	171,254	15,395	27,579	34,823	45,378
6	9.0	17,482	28,566	75,531	150,722	7,321	56,506	122,729	134,106	12,068	76,449	135,679	166,147	13,966	25,506	33,047	42,834
4	1.8	23,392	35,765	108,223	*	13,371	78,302	128,962	150,138	18,362	104,369	160,199	175,920	18,822	33,218	40,077	48,070
4	3.6	19,298	30,283	84,182	254,926	11,235	62,574	127,970	141,457	15,019	85,366	145,688	169,468	17,884	29,949	36,520	45,142
4	5.4	17,447	27,511	78,227	195,703	9,420	57,651	123,997	138,200	13,121	80,972	139,983	166,608	16,651	27,422	34,303	42,760
4	7.2	16,483	26,329	75,501	166,580	8,204	54,070	122,991	133,075	12,046	77,160	135,491	165,793	15,158	25,699	32,403	41,395
4	9.0	15,912	25,784	73,011	150,694	7,338	53,121	122,232	134,075	11,195	74,629	135,066	167,112	13,887	24,375	31,268	40,014
2	1.8	21,146	31,047	105,956	241,096	13,052	72,267	134,376	140,356	17,289	95,897	171,731	174,602	16,976	31,656	38,728	44,581
2	3.6	16,888	25,688	82,231	279,146	10,901	59,758	130,367	140,799	13,848	83,041	144,549	166,180	16,475	28,348	34,471	42,528
2	5.4	15,210	23,535	75,001	197,253	9,134	54,135	127,630	135,052	11,997	79,037	138,879	166,937	15,455	26,128	32,255	39,882
2	7.2	14,400	22,527	71,583	164,081	7,985	51,045	124,334	133,314	10,952	76,155	136,417	165,533	14,305	24,368	30,772	38,224
2	9.0	13,946	22,419	70,295	150,483	7,195	49,413	122,340	137,210	10,291	73,999	133,545	164,855	13,241	23,160	29,721	37,508

1 psi = 6.89 kPa; 1 ksi = 6.89 MPa; CFA: class C fly ash; CKD: cement kiln dust

σ_d : cyclic axial stress; σ_3 : confining pressure; Mr : resilient modulus (using internal LVDTs)

* Deformations are out of the measuring range of LVDTs

Table 4.4 M_r Values of Different Soils Stabilized with Different Percentage of CKD

σ_3 (psi)	σ_d (psi)	P-soil (M_r MPa)				K-soil (M_r MPa)				V-soil (M_r MPa)				C-soil (M_r MPa)			
		Raw	5% CKD	10% CKD	15% CKD	Raw	5% CKD	10% CKD	15% CKD	Raw	5% CKD	10% CKD	15% CKD	Raw	5% CKD	10% CKD	15% CKD
6	1.8	26,205	96,697	*	*	14,150	51,593	*	*	21,014	67,009	*	*	19,944	36,402	62,939	130,832
6	3.6	22,193	106,532	347,739	371,661	12,093	49,947	159,138	276,259	18,333	62,044	240,987	287,591	18,787	33,372	60,527	124,526
6	5.4	19,834	77,125	323,679	360,388	9,875	46,737	162,216	270,209	15,640	56,412	237,049	284,748	16,981	29,806	57,295	122,450
6	7.2	18,404	66,135	295,132	352,350	8,322	43,906	158,881	277,864	13,598	51,675	225,087	280,764	15,395	27,016	54,564	118,115
6	9.0	17,482	59,421	268,414	346,721	7,321	41,391	156,232	273,540	12,068	47,337	224,460	273,022	13,966	24,557	51,483	115,807
4	1.8	23,392	97,668	*	*	13,371	49,447	*	*	18,362	59,968	*	*	18,822	34,519	59,749	135,350
4	3.6	19,298	87,300	357,292	369,947	11,235	48,229	164,325	282,081	15,019	54,883	237,368	279,421	17,884	30,175	56,815	130,236
4	5.4	17,447	69,283	333,812	364,393	9,420	44,779	161,308	265,623	13,121	51,344	225,572	266,879	16,651	27,393	54,554	122,088
4	7.2	16,483	61,716	293,644	348,392	8,204	42,511	155,292	279,253	12,046	48,558	227,714	264,941	15,158	25,142	52,382	119,914
4	9.0	15,912	58,087	270,947	337,816	7,338	41,045	156,746	265,394	11,195	46,098	225,278	261,227	13,887	23,520	50,525	116,667
2	1.8	21,146	96,033	*	*	13,052	49,199	*	*	17,289	58,077	*	*	16,976	32,855	59,885	134,562
2	3.6	16,888	89,577	364,652	379,201	10,901	46,939	171,277	285,635	13,848	54,510	233,433	274,587	16,475	29,166	56,077	130,480
2	5.4	15,210	67,518	316,868	363,077	9,134	44,368	157,324	284,711	11,997	50,161	228,904	271,308	15,455	25,981	53,658	126,914
2	7.2	14,400	60,717	279,897	339,219	7,985	42,124	155,703	267,784	10,952	47,003	228,099	264,292	14,305	23,876	51,517	123,081
2	9.0	13,946	56,731	269,917	338,669	7,195	40,770	157,691	265,135	10,291	44,964	224,219	260,282	13,241	22,342	49,890	119,306

1 psi = 6.89 kPa; 1 ksi = 6.89 Mpa; CFA: class C fly ash; CKD: cement kiln dust

σ_d : cyclic axial stress; σ_3 : confining pressure; Mr : resilient modulus (using internal LVDTs)

* Deformations are out of the measuring range of LVDTs

Table 4.5 Summary of Failure Strength of Stabilized Specimens Fulfilling ASTM D 6276 Requirements for Lime-Stabilization and OHD L-50 Criteria for CFA- and CKD-Stabilization

P-soil									
Additive	*Lime*			*CFA*			*CKD*		
Percentage	*3*	*6*	*9*	*5*	*10*	*15*	*5*	*10*	*15*
UCS (psi)	*54*	*57*	*67*	---	---	*123*	---	*142*	---
K-soil									
UCS (psi)	*66*	*76*	*68*	---	---	*97*	---	*113*	*168*
V-soil									
UCS (psi)	*97*	*75*	*82*	---	*94*	*121*	---	*131*	---
C-soil									
UCS (psi)	---	*57*	*47*	---	---	---	---	---	---

Table 4.6 Summary of Atterberg Limits of 28-Day Cured Stabilized Soil Specimens

Percentage of Additive	K-soil			V-soil			C-soil		
	LL	PL	PI	LL	PL	PI	LL	PL	PI
Raw Soil									
0	39	18	21	37	26	11	58	29	29
Lime									
3	37	27	10	49	44	5	51	26	25
6	NP	NP	NP	51	NP	NP	51	NP	NP
9	NP	NP	NP	48	NP	NP	43	NP	NP
CFA									
5	35	17	18	40	30	10	50	23	27
10	36	25	11	37	30	7	46	25	21
15	34	22	12	39	33	6	43	24	19
CKD									
5	38	21	17	38	27	11	52	25	27
10	37	24	13	40	28	12	52	24	28
15	42	35	7	43	34	9	52	29	23

LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index

Table 4.7 Ranking Scale of Soil-Additive Mix

Plasticity Index[a]	UCS[a] (psi)	Resilient Modulus[b] (psi)	3-D Swell[c, d] (%)	Dielectric Constant	Rank
0 -5	> 232	> 87,083	0 – 0.90	< 21	4
6 – 15	174 – 232	58,055 – 87,083	0.91 – 2.70	21 – 25	3
16 – 25	102 – 174	29,028 – 58,055	2.71 – 7.30	26 – 30	2
> 25	58 – 102	14,514 – 29,028	> 7.30	31 - 35	1
> 50	< 58	< 14,514	> 14.50	> 35	0

[a]Wattanasanticharoen (2000); [b]Chavva (2005); [c]Nelson and Miller (1992); [d]Al-Shamrani and Al-Mhaidib (2000)

Table 4.8 Individual Rank and Overall Rank of K-soil Stabilized with Different Additives

Additive	PI	UCS	M_r	3-DS	DV	OR
None	2	0	0	2	0	0.8
3% Lime	3	1	4	4	0	2.4
6% Lime	4	1	4	4	1	2.8
9% Lime#	***4***	***1***	***4***	***4***	***2***	***3.0***
5% CFA	2	1	3	2	1	1.8
10% CFA	3	1	4	2	1	2.2
15% CFA	3	1	4	2	1	2.2
5% CKD	2	0	2	1	0	1.0
10% CKD	3	2	4	1	0	2.0
15% CKD	3	2	4	1	0	2.0

PI: Plasticity Index; UCS: Unconfined Compressive Strength; M_r: Resilient Modulus; 3-DS: Three-Dimensional Swell; DV: Dielectric Value;
OR: Overall Rank = (Ranks of PI + UCS + FS + M_r + 3-DS + DV)/4
#Additive content providing maximum OR value

Table 4.9 Individual Rank and Overall Rank of V-soil Stabilized with Different Additives

Additive	PI	UCS	M_r	3-DS	DV	OR
None	3	0	1	4	1	1.8
3% Lime	4	1	4	1	1	2.2
6% Lime	4	1	4	1	2	2.4
9% Lime	4	1	4	1	2	2.4
5% CFA	3	0	4	4	2	2.6
10% CFA	3	1	4	4	2	2.8
15% CFA#	***3***	***2***	***4***	***4***	***2***	***3.0***
5% CKD	3	0	3	4	2	2.4
10% CKD	3	2	4	3	2	2.8
15% CKD	3	3	4	2	2	2.8

PI: Plasticity Index; UCS: Unconfined Compressive Strength; M_r: Resilient Modulus; 3-DS: Three-Dimensional Swell; DV: Dielectric Value;
OR: Overall Rank = (Ranks of PI + UCS + FS + M_r + 3-DS + DV)/4
#Additive content providing maximum OR value

Table 4.10 Individual Rank and Overall Rank of C-soil Stabilized with Different Additives

Additive	PI	UCS	M_r	3-DS	DV	OR
None	1	0	1	2	1	1.00
3% Lime	2	0	2	4	1	1.80
6% Lime[#]	***4***	***0***	***3***	***4***	***2***	***2.60***
9% Lime[#]	***4***	***0***	***2***	***4***	***3***	***2.60***
5% CFA	1	0	2	2	1	1.20
10% CFA	2	0	2	2	1	1.40
15% CFA	2	1	2	2	1	1.60
5% CKD	1	0	2	1	0	0.80
10% CKD	1	0	3	1	0	1.00
15% CKD	2	1	4	0	0	1.40

PI: Plasticity Index; UCS: Unconfined Compressive Strength; M_r: Resilient Modulus; 3-DS: Three-Dimensional Swell; DV: Dielectric Value;
OR: Overall Rank = (Ranks of PI + UCS + FS + M_r + 3-DS + DV)/4
[#]Additive content providing maximum OR value

Table 4.11 A Summary of the Statistical Analyses of K-soil Stabilized with Lime, CFA and CKD

Type of additive	% of additive	$M_r = k_1 \times (k_2)^{S_d} \times (k_3)^{S_3}$ k1	k2	k3	R^2	F-value	Pr	Significant[a]	Calculated [b]M_r (psi)
		k_1	k_2	k_3					
None	0	102,135	0.986	1.002	0.990	30.12	<0.0001	Yes	8,779
Lime	3	1,158,283	0.998	1.001	0.820	27.26	<0.0001	Yes	155,752
	6	1,271,743	0.993	1.002	0.929	79.00	<0.0001	Yes	149,500
	9	851,970	0.995	1.002	0.928	77.89	<0.0001	Yes	106,957
CFA	5	475,174	0.992	1.006	0.913	62.60	<0.0001	Yes	59,695
	10	935,800	0.998	1.001	0.687	13.18	<0.0001	Yes	126,500
	15	996,051	0.999	1.000	0.692	13.48	<0.0001	Yes	137,750
CKD	5	640,294	0.989	1.003	0.879	43.57	<0.0001	Yes	64,211
	10	2,965,888	0.991	1.000	0.910	45.35	<0.0001	Yes	297,104
	15	2,716,952	0.997	1.000	0.891	36.74	<0.0001	Yes	357,877

[a]Significant at probability level (alpha) =0.05; [b]M_r values calculated at S_d = 6 psi, S_3 = 4 psi

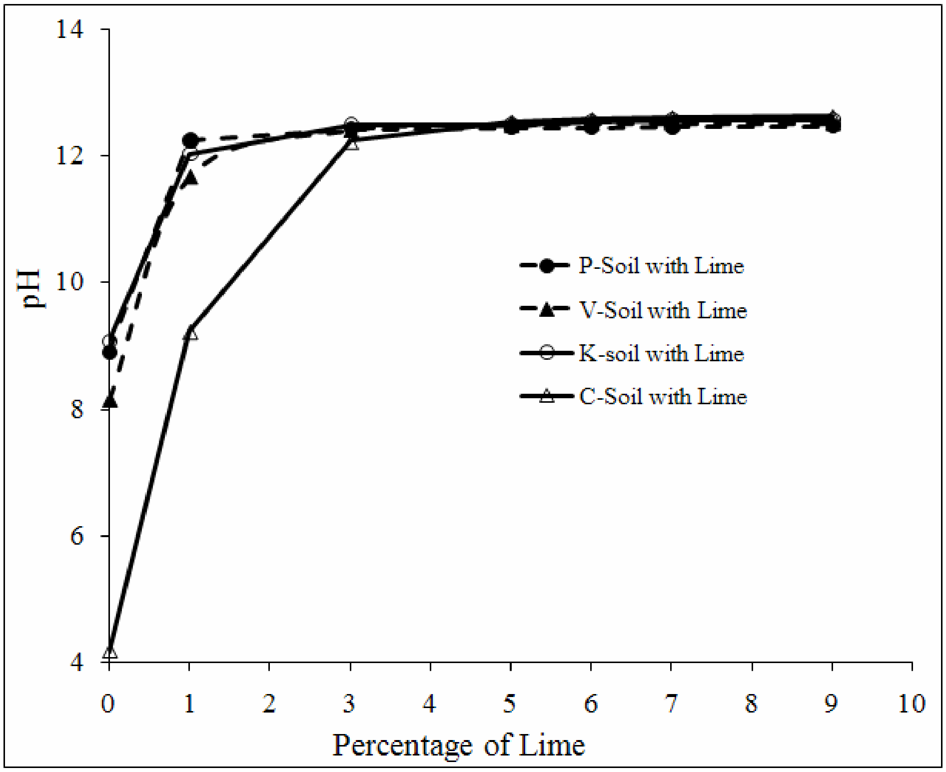


Figure 4.1 Variation of pH Values with Lime Content

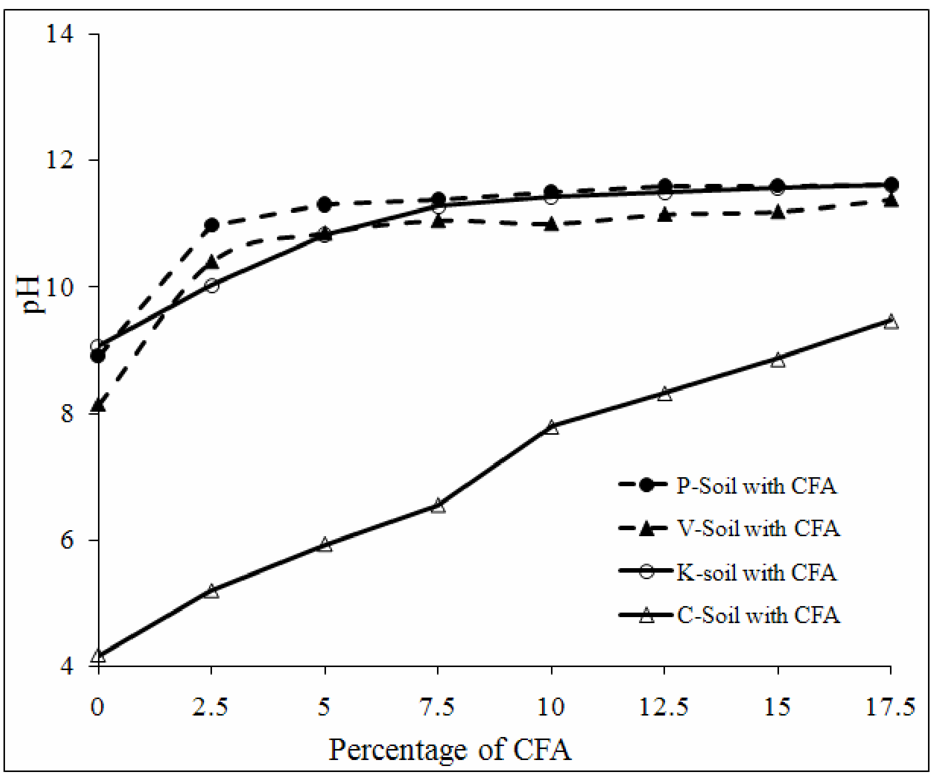


Figure 4.2 Variation of pH Values with CFA Content

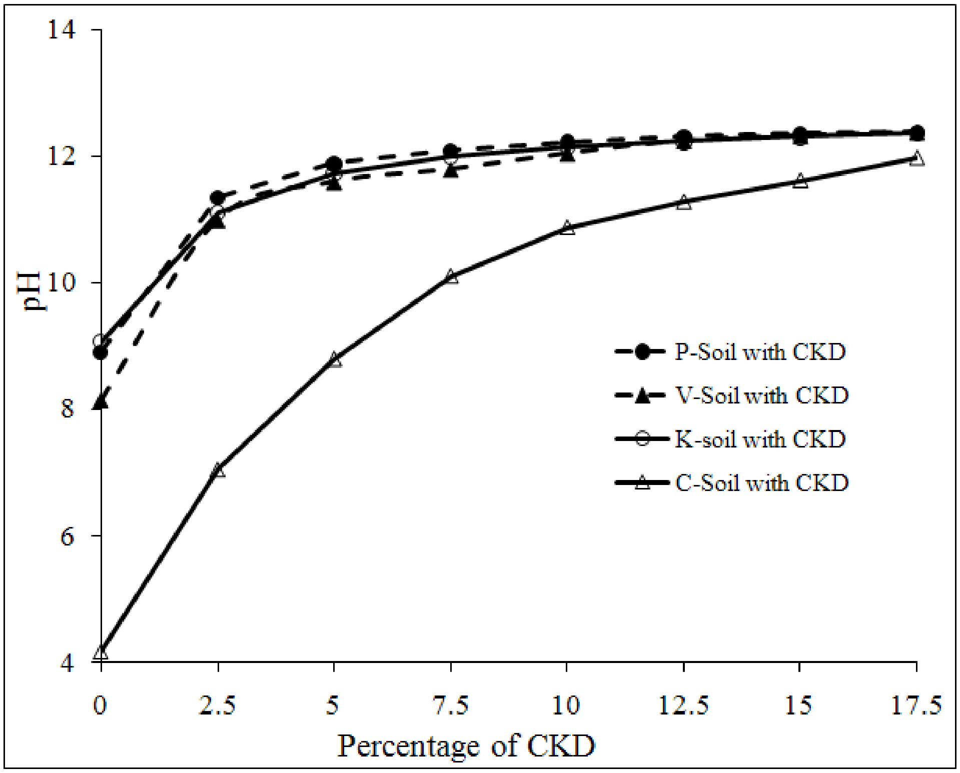


Figure 4.3 Variation of pH Values with CKD Content

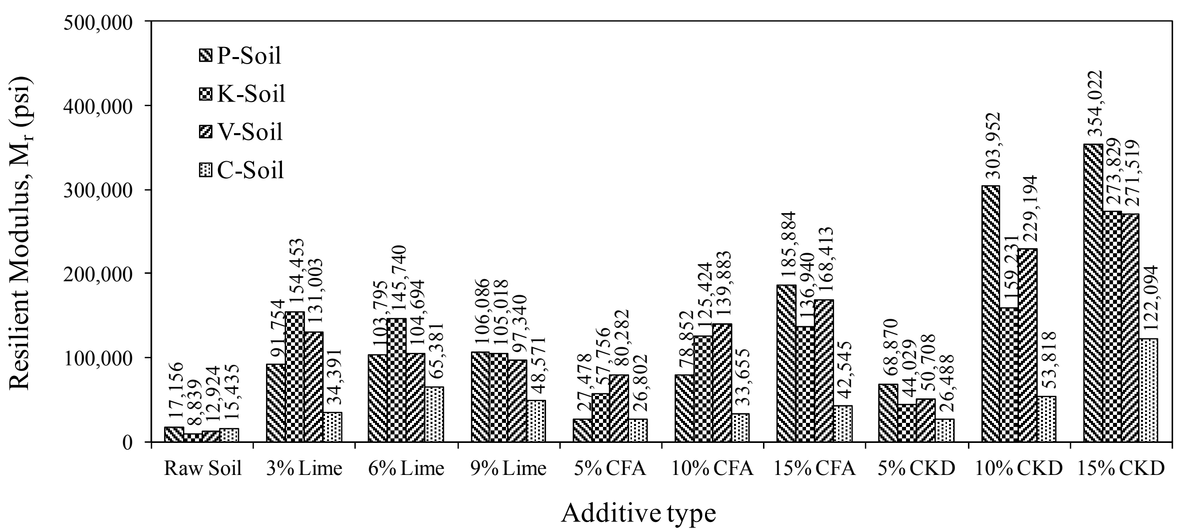


Figure 4.4 Variation of M_r Values with Soil and Additive Type (S_d = 6 psi, S_3 = 4 psi)

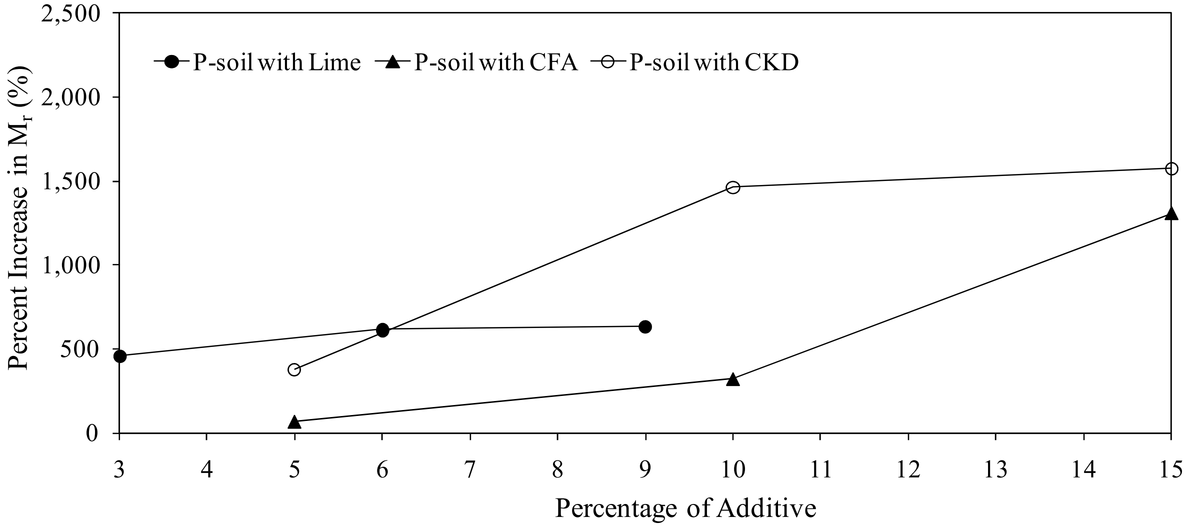


Figure 4.5 Improvement of M_r Values for P-soil

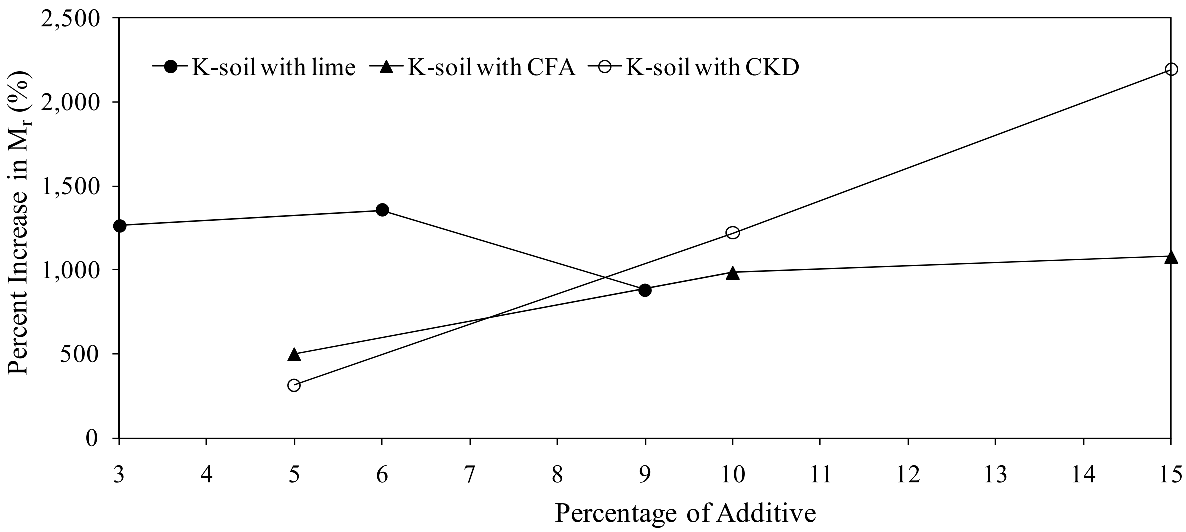


Figure 4.6 Improvement of M_r Values for K-soil

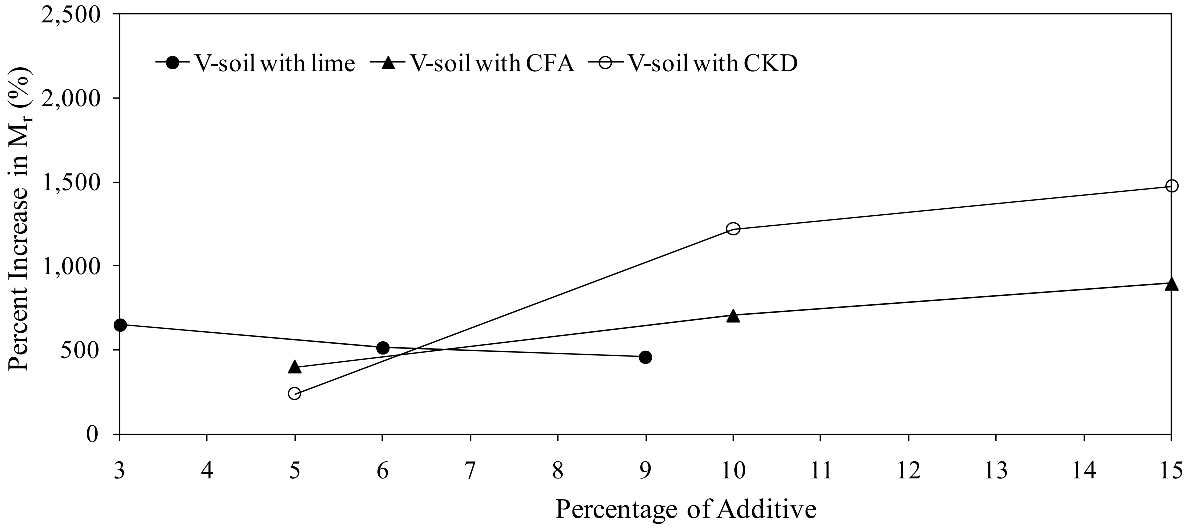


Figure 4.7 Improvement of M_r Values for V-soil

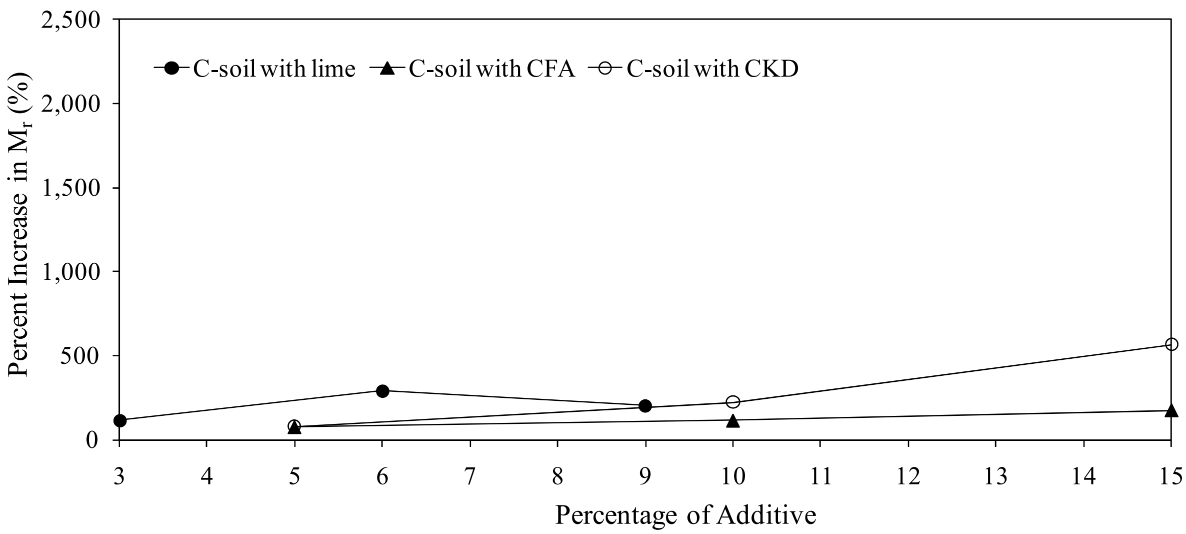


Figure 4.8 Improvement of M_r Values for C-soil

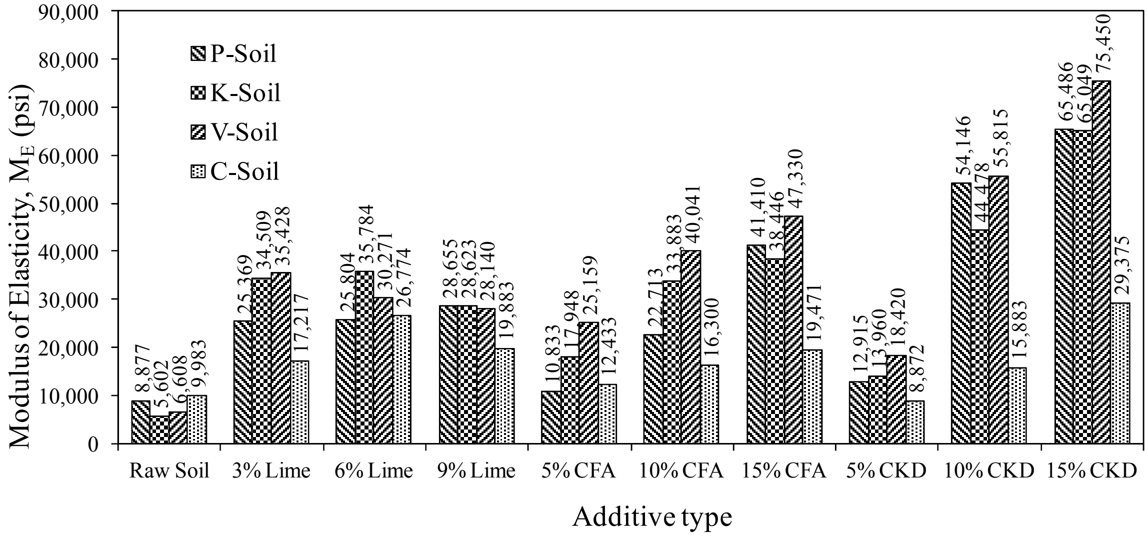


Figure 4.9 Variation of M_E Values with Soil and Additive Type

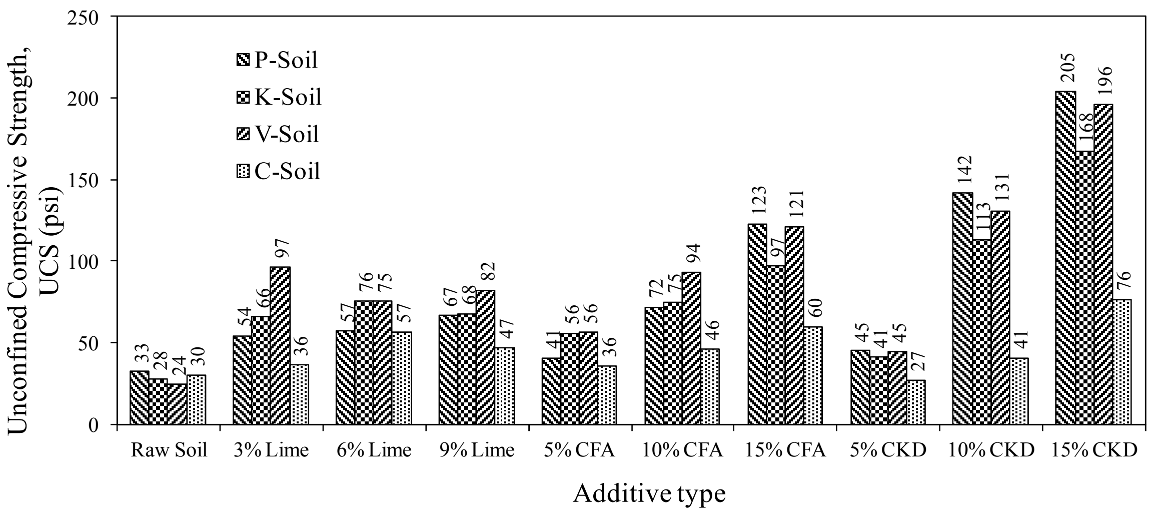


Figure 4.10 Variation of UCS Values with Soil and Additive Type

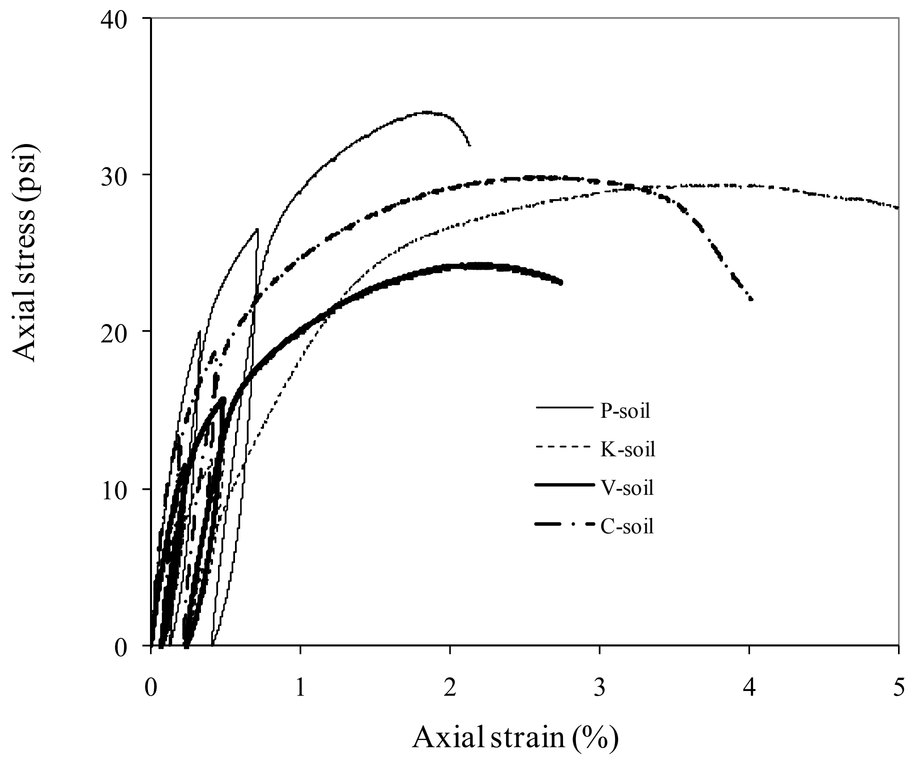


Figure 4.11 Stress-Strain Behavior of Different Raw Soils

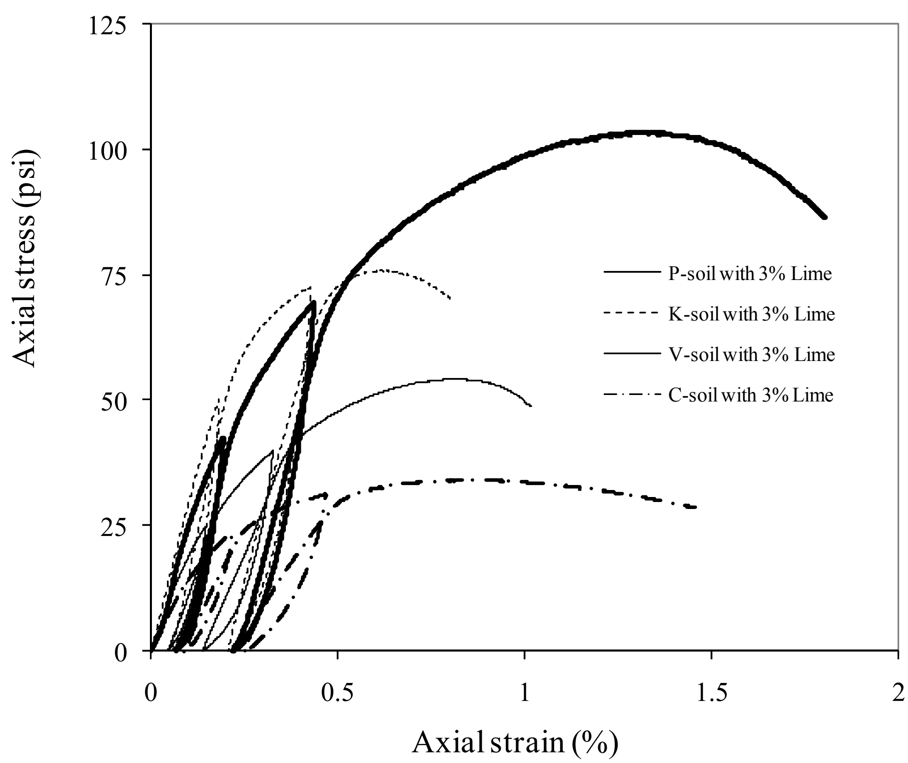


Figure 4.12 Stress-Strain Behavior of Different Raw Soils Stabilized with 3% Lime

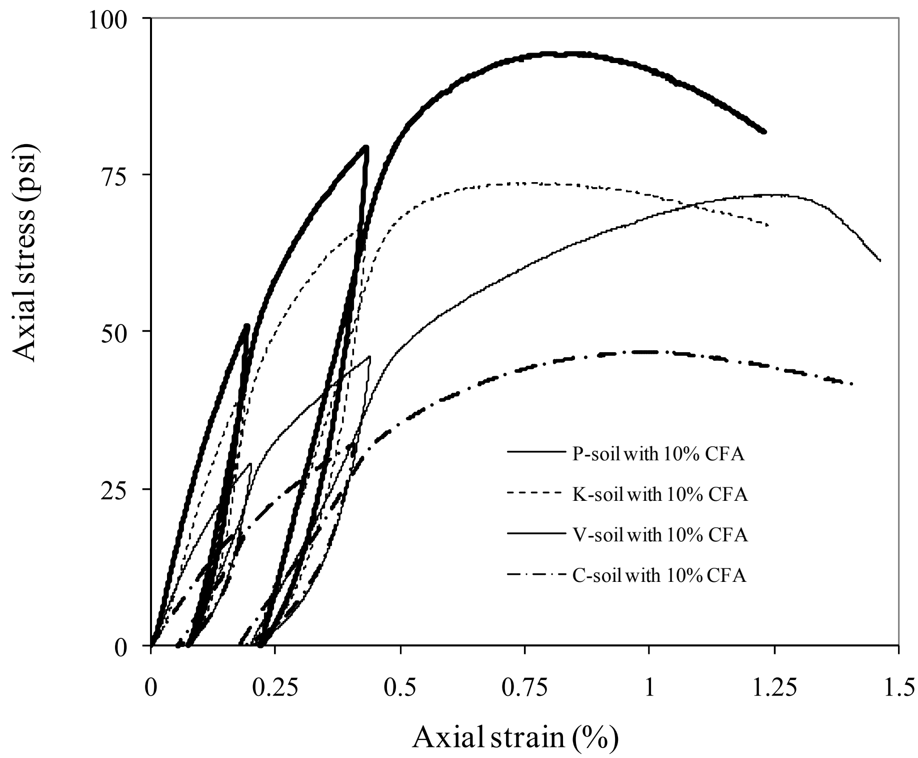


Figure 4.13 Stress-Strain Behavior of Different Raw Soils Stabilized with 10% CFA

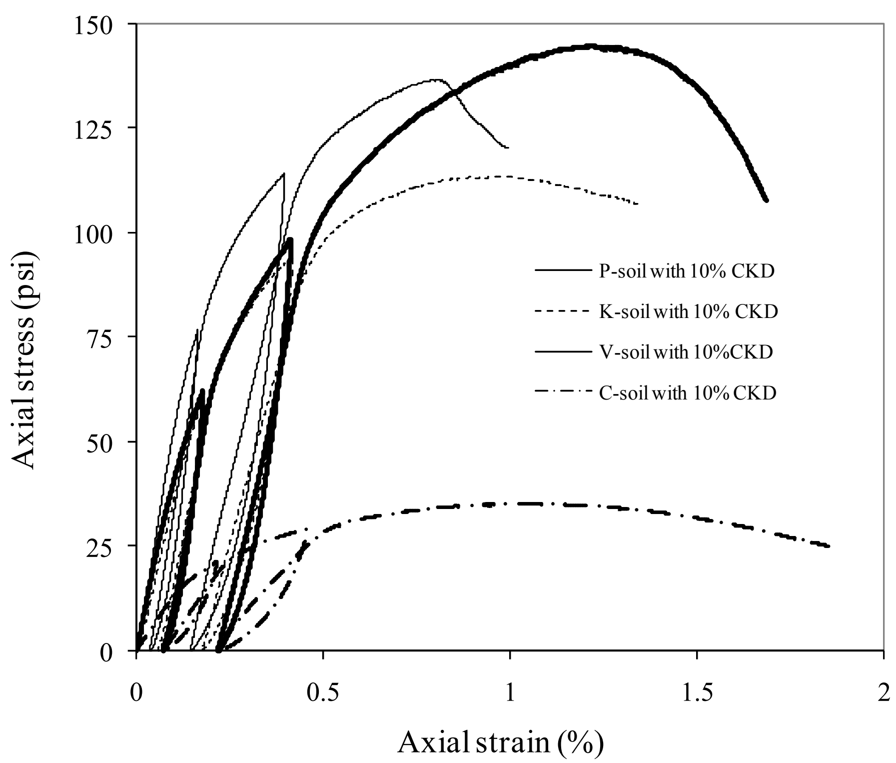


Figure 4.14 Stress-Strain Behavior of Different Raw Soils Stabilized with 10% CKD

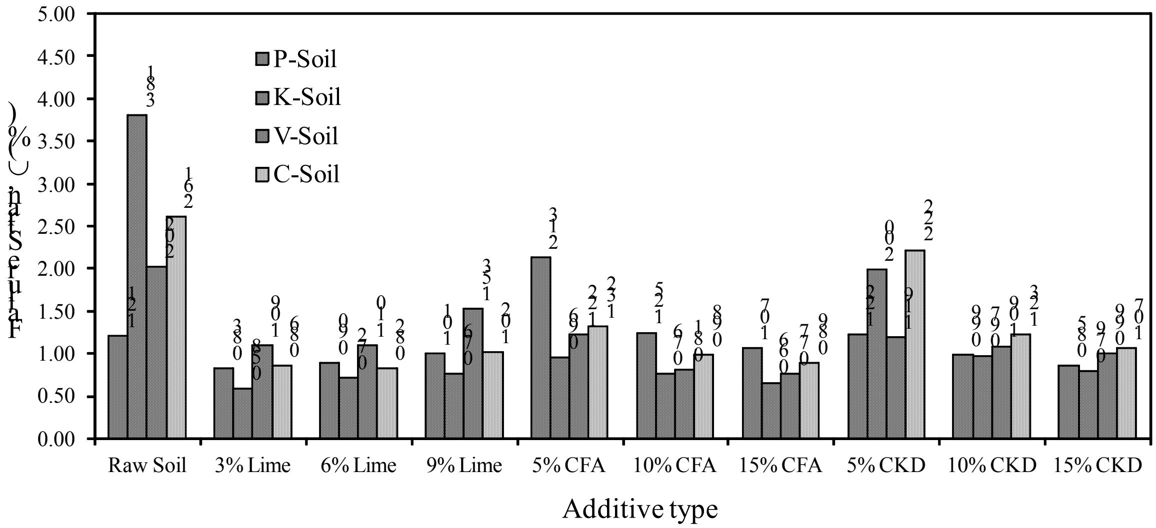


Figure 4.15 Variation of Failure Strain Values with Soil and Additive Type

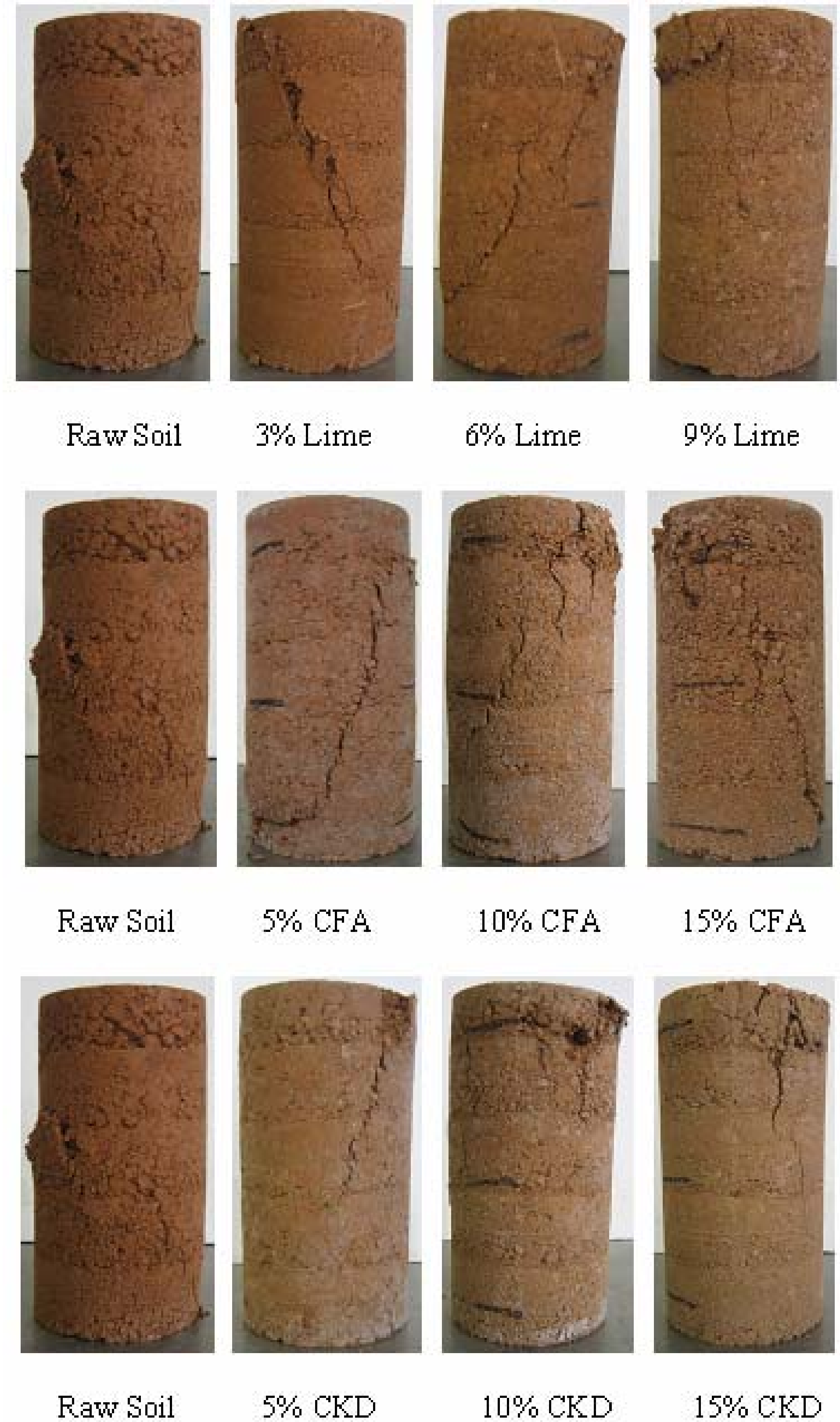


Figure 4.16 Failure Patterns of Raw and Stabilized C-soil Specimens

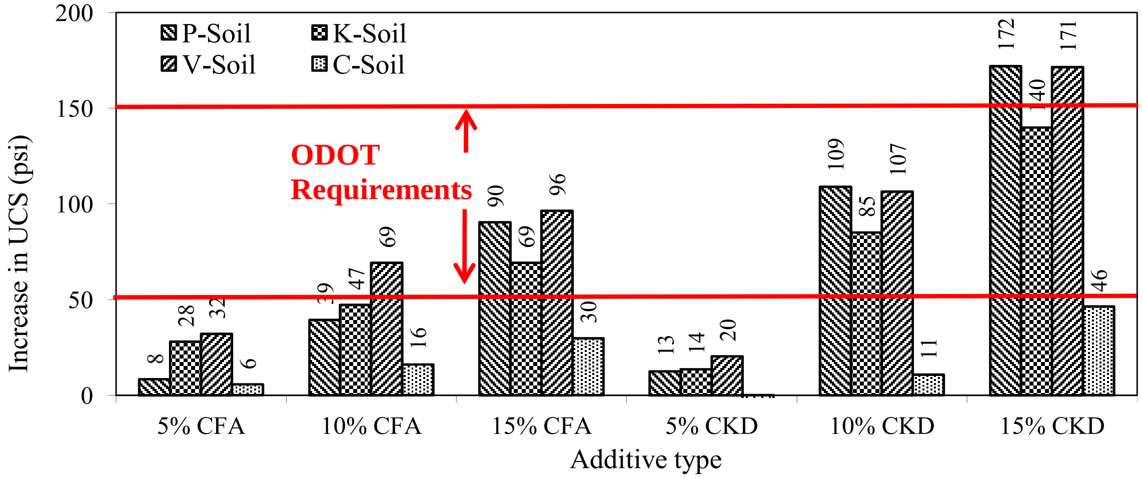


Figure 4.17 Variation of Increase in UCS Values with CFA- and CKD-Stabilized Soil Specimens

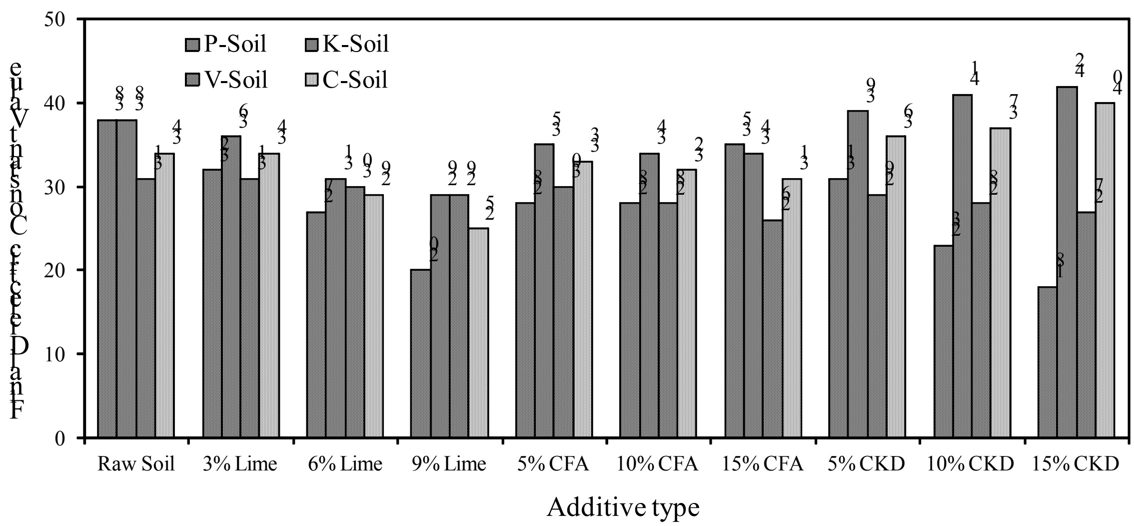


Figure 4.18 Variation of Final Dielectric Constant Values with Soil and Additive Type

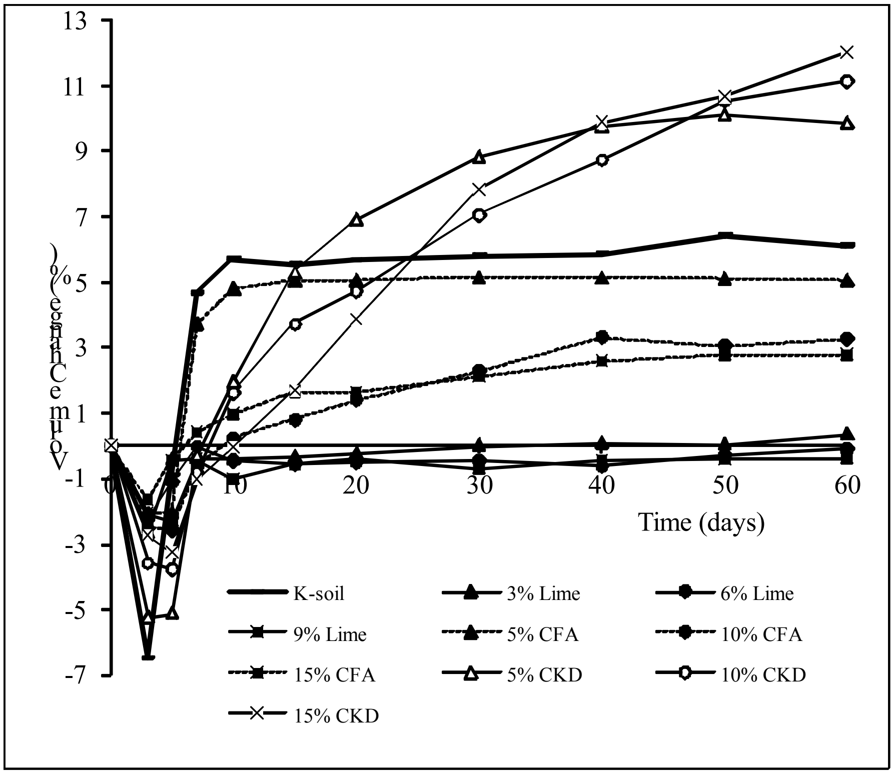


Figure 4.19 Variation of 3-D Swell Values of Stabilized K-soil Specimens with Time

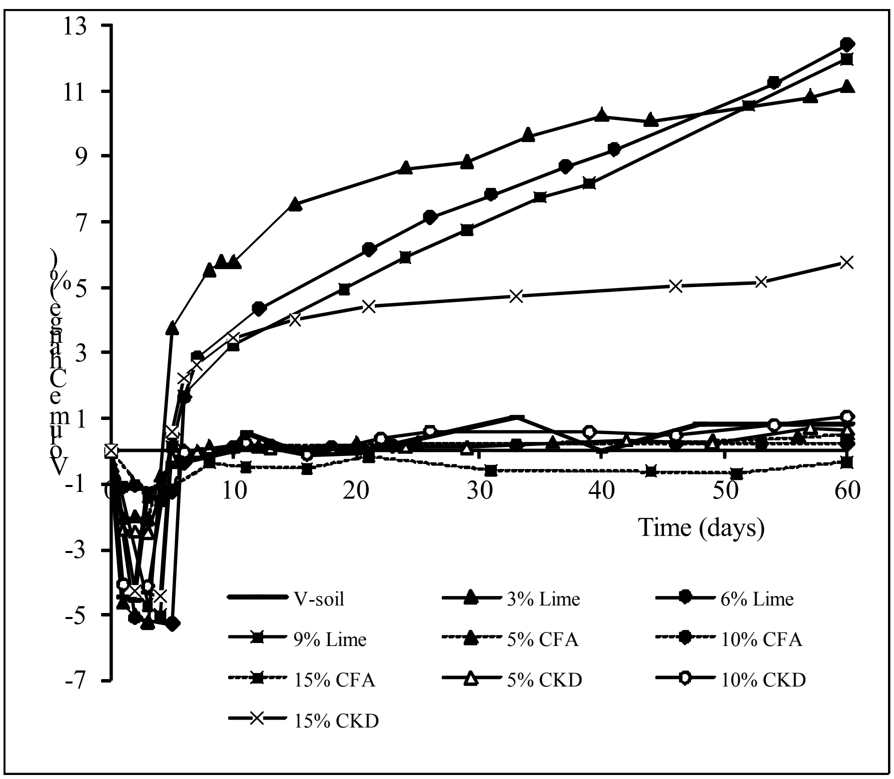


Figure 4.20 Variation of 3-D Swell Values of Stabilized V-soil Specimens with Time

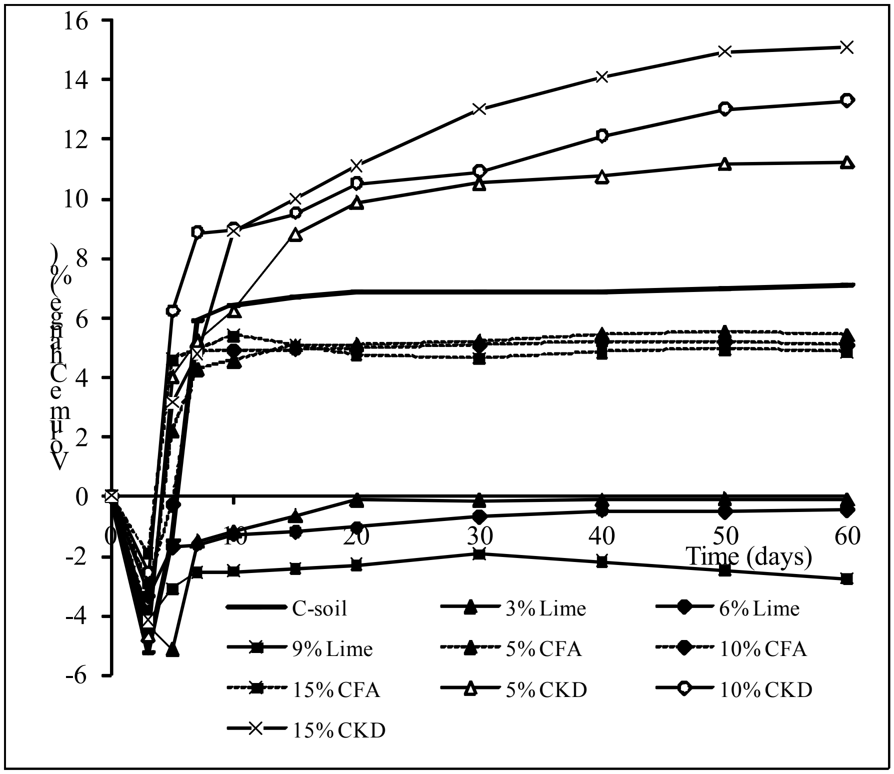


Figure 4.21 Variation of 3-D Swell Values of Stabilized C-soil Specimens with Time

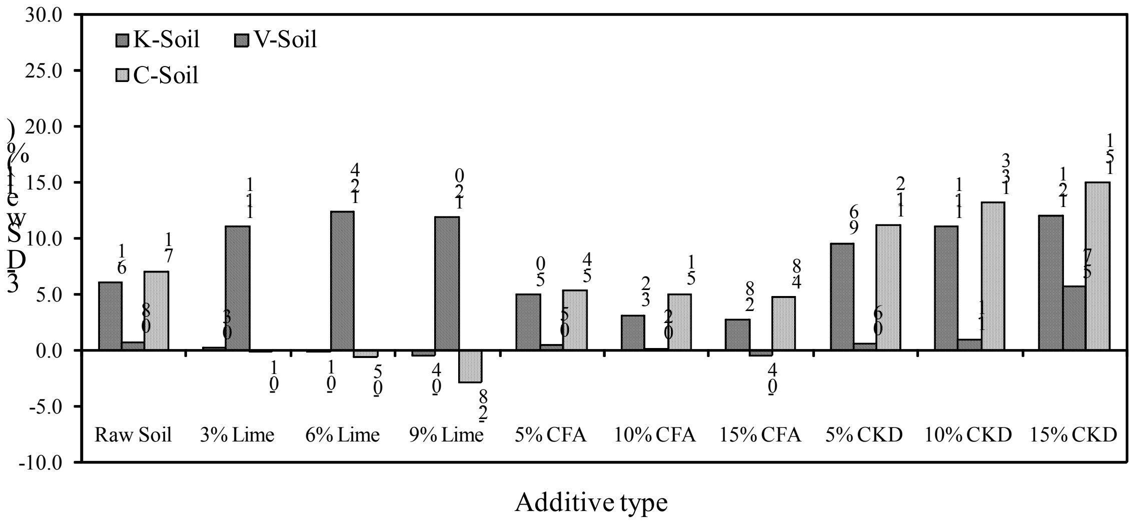


Figure 4.22 Variation of Final 60-Day 3-D Swell Values with Soil and Additive Type

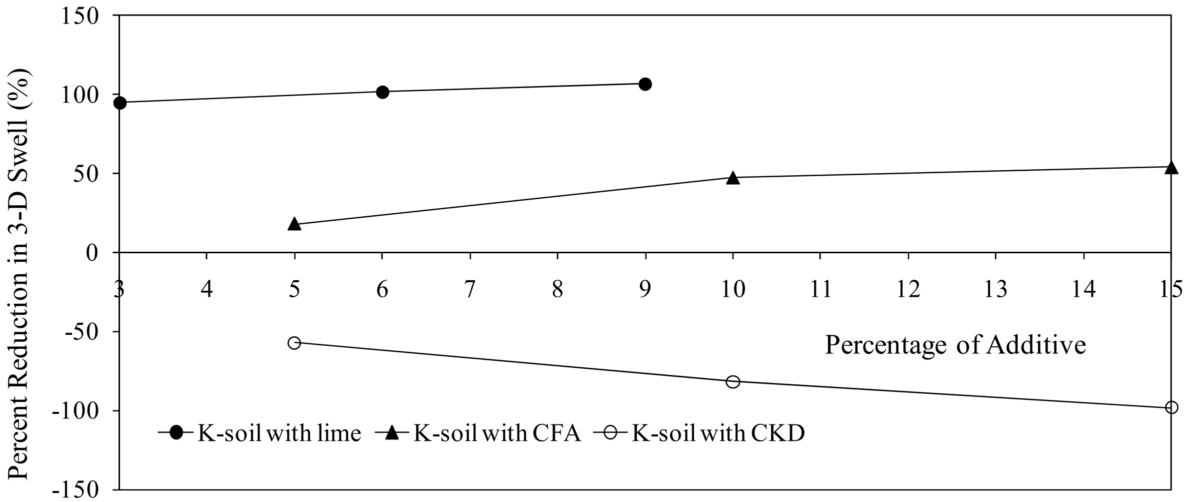


Figure 4.23 Variation of Reduction in 3-D Swell with Percentage of Additives for K-soil

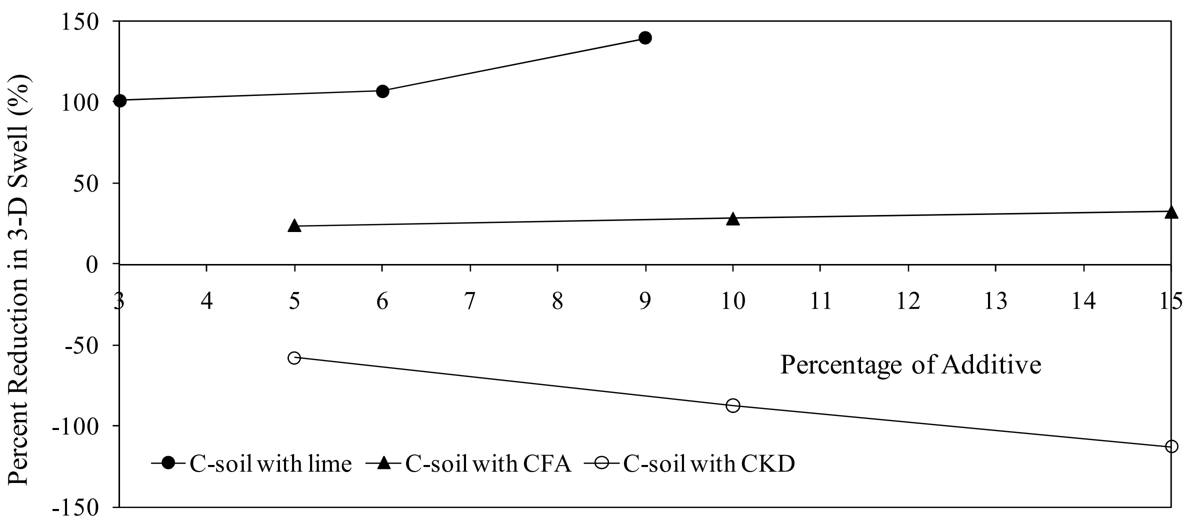


Figure 4.24 Variation of Reduction in 3-D Swell with Percentage of Additives for C-soil

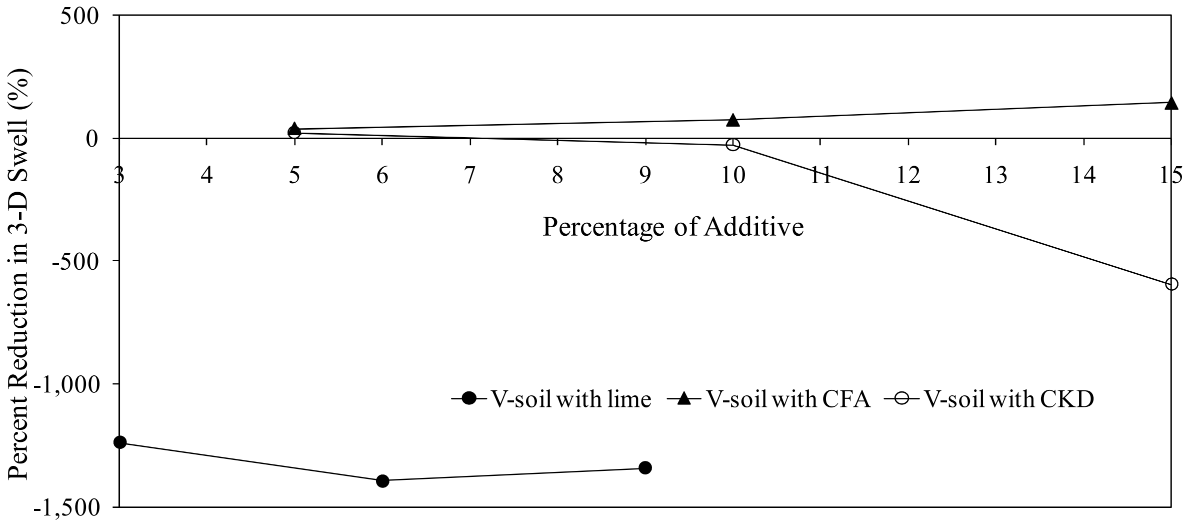


Figure 4.25 Variation of Reduction in 3-D Swell with Percentage of Additives for V-soil

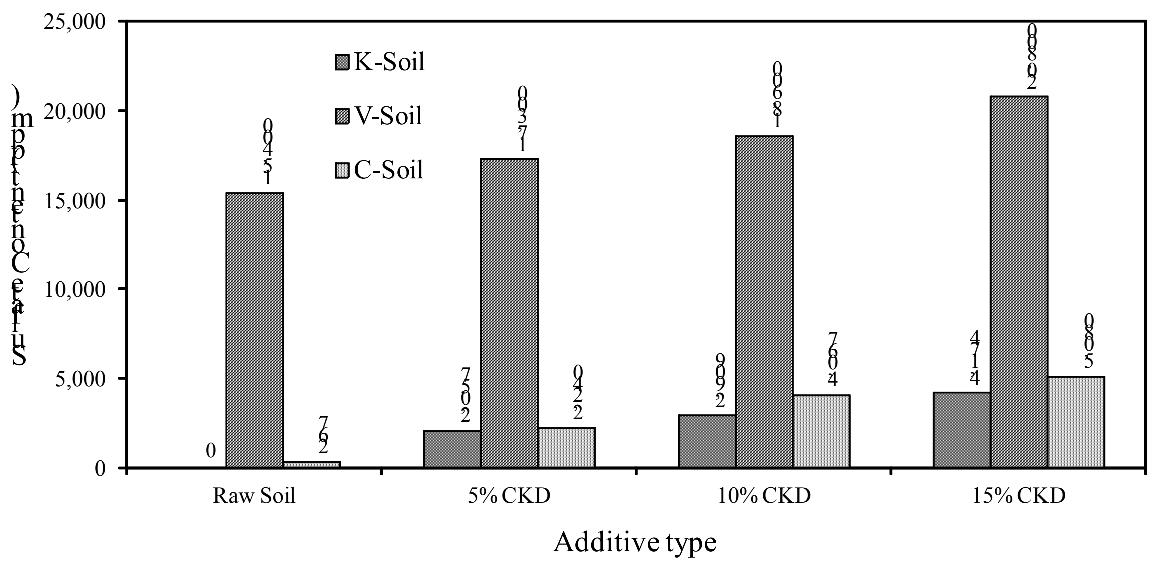


Figure 4.26 Variation of Sulfate Content with Type of Soil and Amount of CKD

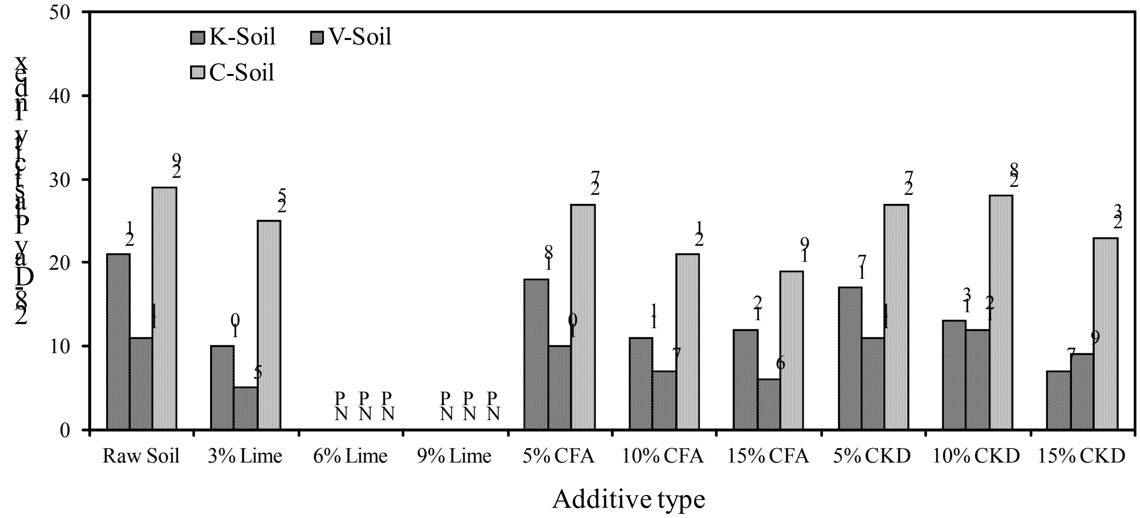


Figure 4.27 Variation of 28-Day Plasticity Index with Type of Soil and Additive

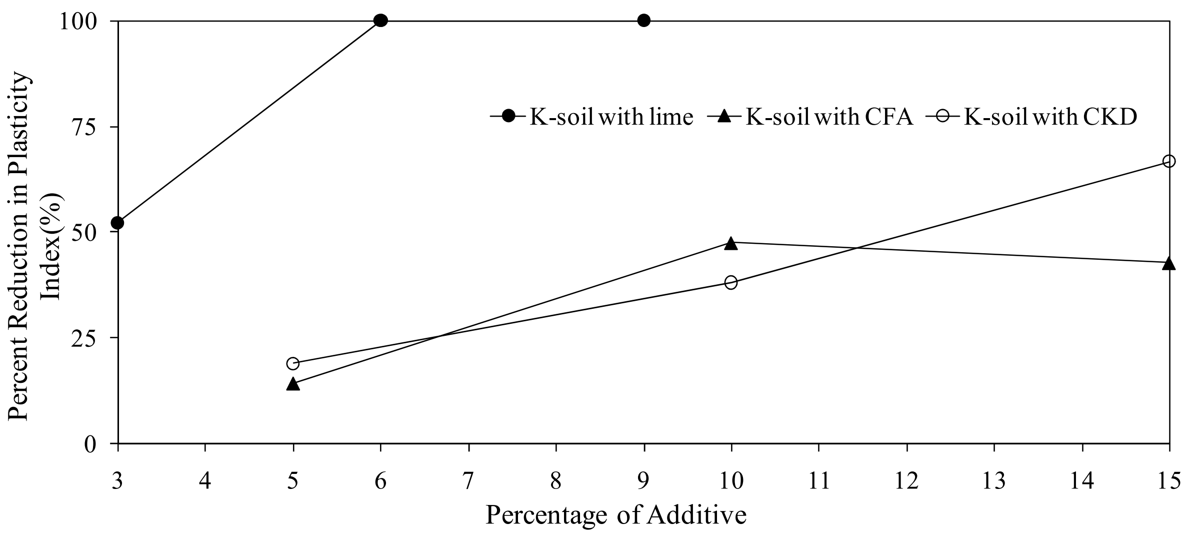


Figure 4.28 Variation of Reduction in 28-Day Plasticity Index with Percentage of Additives for K-soil

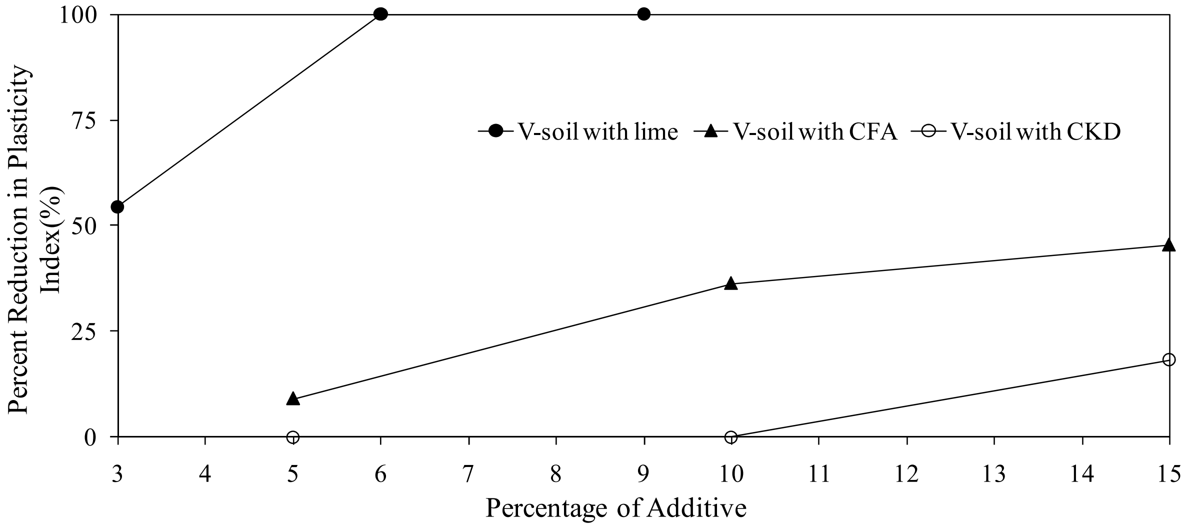


Figure 4.29 Variation of Reduction in 28-Day Plasticity Index with Percentage of Additives for V-soil

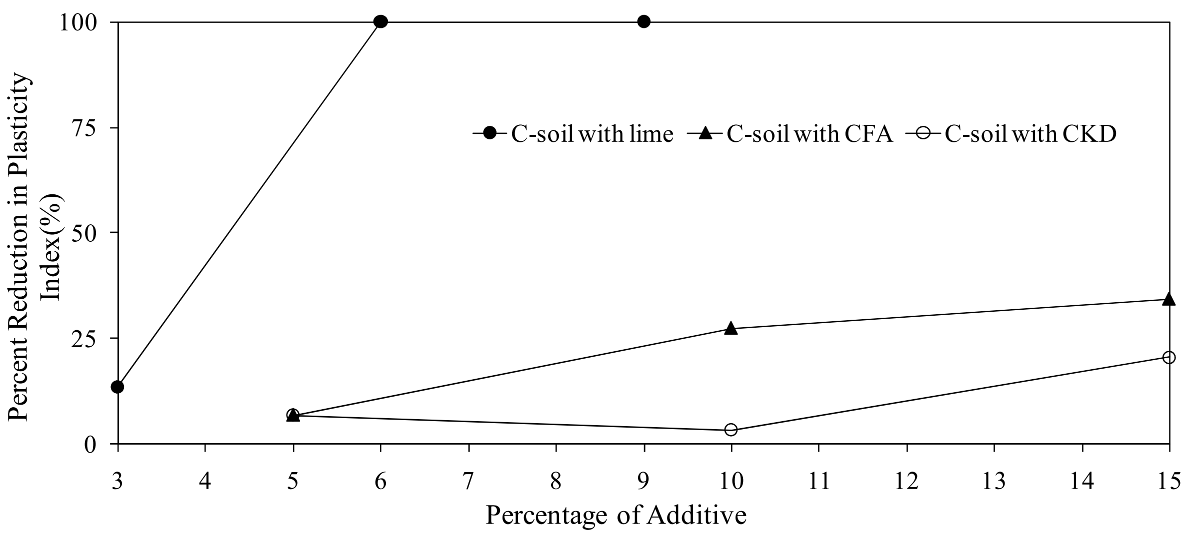


Figure 4.30 Variation of Reduction in 28-Day Plasticity Index with Percentage of Additives for C-soil

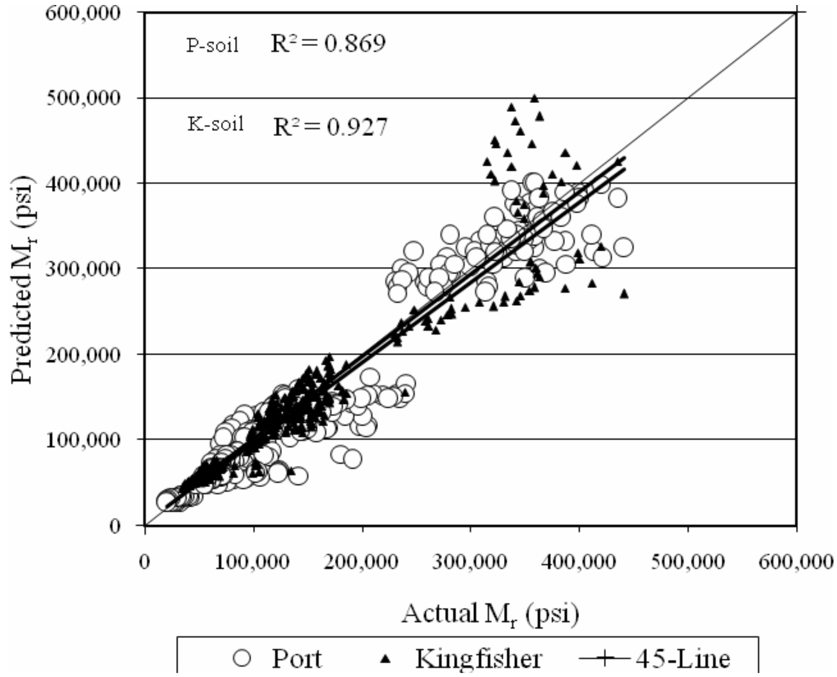


Figure 4.31 Predicted M_r versus Measured M_r for K-soil (Development Dataset) and P-soil (Validation dataset)

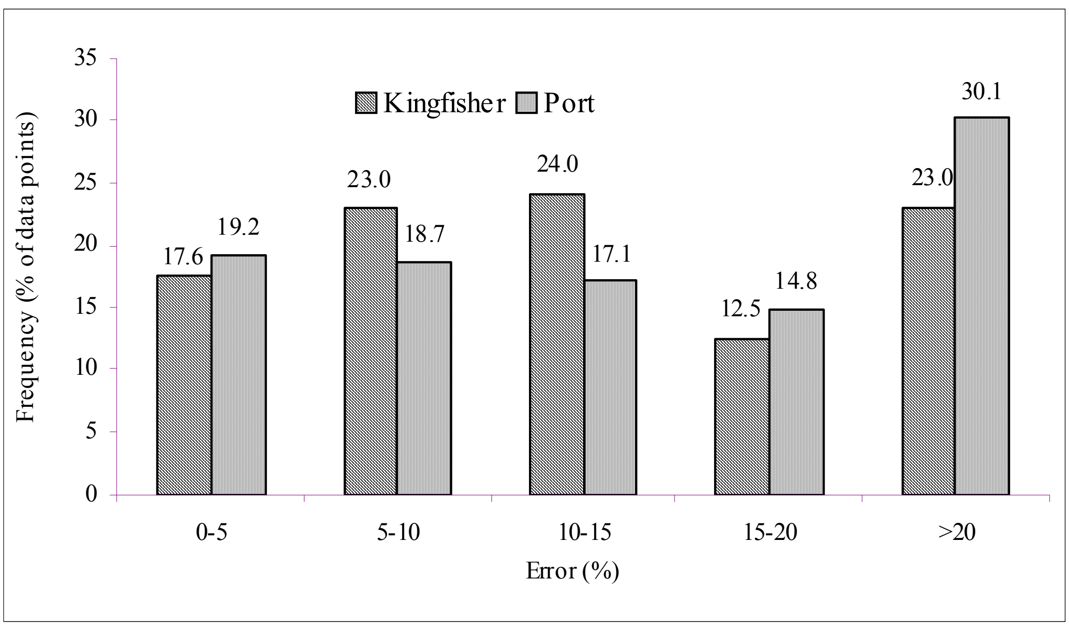


Figure 4.32 Percentage Error in the Predicted and Measured M_r Values of K- and P-soil

CHAPTER 5 SOIL SUCTION, PERMEABILITY AND MINERALOGICAL STUDIES

5.1 General

This chapter presents the efforts that were made to determine the soil suction, and permeability of selected stabilized soil specimens. An overview of the results is presented and problems that were faced are discussed. In addition, results of mineralogical studies such as X-ray diffraction (XRD), energy dispersive spectroscopy (EDS) and scanning electron microscopy (SEM) performed on selective stabilized specimens are also presented.

5.2 Soil Suction

Only few studies were conducted for determining soil suction parameters (total, matric, and osmotic suction) of stabilized soil specimens. For example, Puppala et al. (2006) used pressure plate apparatus for determining suction parameters of soil stabilized with fly ash. In a recent study, Petry and Jiang (2007) used a Dewpoint Potentiometer for evaluating suction parameters of soil stabilized with hydrated lime and KIS (solution containing potash and ammonium lignosulfonate). They also correlated soil suction with soil properties.

In this study, soil suction tests were conducted on the P- and K-soil specimens already tested for resilient modulus (M_r) and/or tube suction test (TST). At the conclusion of each resilient modulus test, specimens were sliced into five layers. Each layer was divided into five parts. Four of these parts were used to determine the moisture content, and one part for suction. Soil suction tests were performed using the *filter paper technique* according to the ASTM D 5298 test method. The filter paper moisture contents were converted to matric suction using the calibration curves in ASTM D 5298.

The average results for P-soil specimens are presented in Table 5.1. It is evident that stabilization of P- and K-soil with different additives, namely, lime, CFA and CKD, influences the soil suction parameters. For example, raw P-soil specimen and 6% lime-stabilized P-soil specimen compacted at similar moisture content showed an increase in total suction value by approximately 292%. Table 5.2 shows average suction test results conducted on K-soil specimens compacted at OMC and MDD, as discussed in Chapter-2. It is clear that all the additives used in this study influence suction parameters. Specimens stabilized with 3% lime showed an average total suction value of approximately 1928 kPa. However, specimens stabilized with 5% CFA and 5% CKD showed an average total suction value of approximately 2950 and 1164 kPa, respectively.

5.3 Permeability

In this study, efforts were made to conduct permeability on raw and stabilized soil specimens. A literature review was conducted for deciding the type of device needed for this study. Table 5.3 shows the summary of literature review of permeability test on stabilized soil specimens. Since no standard device or method was available for permeability test on stabilized specimens, it was decided to manufacture own permeability device. Hence, a new device shown in Figure 5.1 was manufactured at the University of Oklahoma to perform these tests.

The mixture for each permeability specimen, consist of raw soil blended with a specific amount of stabilizer. The amount of stabilizer was added based on the dry weight of the soil. After the blending process, a desired amount of water was added based on the optimum moisture content (OMC). Then, the mixture was compacted in a standard Proctor mold having a diameter of 101.6 mm (4.0 in) and a height of 115.8 mm (4.6 in) to reach a dry density between 95%-100% of the maximum dry density (MDD). After compaction, specimens were cured at a temperature of 23.0 ± 1.7^{o} C and a relative humidity of approximately 96% for 1-day. A total of two replicates were prepared for each combination. After curing, the mold was inverted and placed between two platens and sealed with gasket to avoid any leak, as shown in Figure 5.1. A water pressure was applied until a uniform water flow was obtained. After that, the flow and the time were recorded to determine the permeability. Two different water pressure heads of 213 cm (7.0-ft.) and 274 cm (9.0-ft.) were applied in this study. The permeability results of raw and stabilized P-soil specimens are presented in Table 5.4. The results for a water head of 274 cm are plotted in Figure 5.2, which show an increase in permeability with the increase in the percentage of lime. This is consistent with the observations made by Nalbantoglu and Tuncer (2001). They explained the increase in permeability with an increase in lime content due to pozzolanic reactions. The formation of lime particle aggregates results in the soil becoming more granular in nature and results in higher resistance to compression at similar stress levels. This produces a soil with a more open fabric and results in an increase in permeability. As evident from Figure 5.2, CKD-stabilized specimens exhibited higher permeability as compared to lime-stabilized specimens. This can be further attributed to the formation of cementitious reaction products during pozzolanic reactions.

Table 5.5 shows permeability test results of selective K-soil stabilized specimens. Results were in the range of 10^{-6} to 10^{-7} cm/s for stabilized specimens. Permeability of raw and stabilized specimens and the effect of different additives types, and additive content is a significant study by itself, and hence only selective specimens were tested.

5.4 Mineralogical Studies

To facilitate macro-behavior comparison and explanation, the mineralogical study techniques such as Scanning Electron Microscopy (SEM), Energy Dispersive Spectroscopy

(EDS) and X-Ray Diffraction (XRD) were employed to qualitatively identify the micro-structural developments in the matrix of the stabilized soil specimens.

5.4.1 Test Procedure

The Scanning Electron Microscopy (SEM) technique was employed using a JEOL JSM 880 microscope to qualitatively identify the micro-structural developments in the matrix of the stabilized soil specimens (Figure 5.3). After the UCS/TST test on specimens, broken mix was air-dried for approximately four days. Three representative tiny pieces were mounted on stubs (1 cm wide discs that have a pin-mount on the base of the disc) as shown in Figure 5.4. Then, pieces were coated with a thin layer (≈ 5 nm) of Iridium by sputter coating technique to provide surface conductivity. A JEOL JSM 880 scanning electron microscope operating at 15 kV was used to visually observe the coated specimens. The JEOL JSM 880 was fitted with an energy-dispersive X-ray spectrometer (EDS). The EDS was used to analyze chemical compositions of the specimen. In this technique, electrons are bombarded in the area of desired elemental composition; the elements present will emit characteristic X-rays, which are then recorded on a detector. The micrographs were taken using EDS2000 software.

To confirm the SEM results, X-ray diffraction (XRD) tests were performed using a Rigaku D/Max X-Ray diffractometer (Figure 5.5). Four-day air dried mix was pulverized with a mortar and pestle, sieved through a U.S. standard No. 325 sieve (45 μm) and the powder of less than 45 μm was collected and placed on a glass specimen holder prior to testing as evident from Figure 5.6. This holder was then mounted on a Rigaku D/Max X-ray diffractometer for analysis. This diffractometer is equipped with bragg-brentano parafocusing geometry, a diffracted beam monochromator, and a conventional copper target X-ray tube set to 40 kV and 30 mA. The measurements were performed from 5^{o} to 70^{o} (2θ range), with 0.03^{o}

step size and 1 seconds count at each step. Data obtained by the diffractometer were analyzed with Jade 3.1, an X-ray powder diffraction analytical software, developed by the Materials Data, Inc. (Jade, 1999). Generated diffractograms (using the peaks versus 2θ and d-spacing) were used to determine the presence of ettringite.

5.4.2 Assessment of Strength/Stiffness

Figure 5.7 shows SEM micrographs of raw soil samples at high magnification (x1,000 and x10,000). It is clear that the raw soil has a discontinuous structure, where the voids are more visible because of the absence of hydration products. The raw additives used in this study were also studied using SEM/EDS methods. Figures 5.8, 5.9 and 5.10 show SEM/EDS of raw lime, CFA and CKD powder, respectively. As evident from Figure 5.8, raw lime is an amorphous powder consisting mainly of calcium compounds. This is in agreement with the XRF results reported in Chapter 2 (see Table 2.3). On the other hand, CFA and CKD are more complex compounds (Figures 5.9 and 5.10). EDS results indicated presence of calcium, aluminum, silicon, sulfur, phosphorous, titanium, iron, and magnesium minerals in CFA. Whereas EDS results of CKD indicated presence of calcium, silicon, magnesium, sulfur, and potassium minerals. The SEM micrographs of raw CFA showed that CFA is composed of different size spherical particles (or cenosphere); however, CKD micrographs showed particles with poorly defined shapes.

To study the comparative K- and C-soil strength/stiffness behavior, 28-day UCS tested specimens were studied using SEM micrographs (Figure 5.11). One common characteristic of all the stabilized soil specimens was the abundance of hydration products. As noted earlier, stabilized K-soil specimens exhibited higher strength and stiffness values (see Figure 4.5 and 4.6). It is an indication that the development of cementing products with various percentages is responsible for such a difference. It is expected that more cementing compounds are formed in K-soil specimens and hence higher strength/stiffness values are obtained compared to C-soil. This observation is visually evident from Figure 5.11 that more hydration coating and needle-like hydration products are formed in stabilized K-soil specimens compared to C-soil specimens, as expected.

5.4.3 Assessment of Sulfate-Induced Heave

As noted earlier, V-soil specimens stabilized with lime and all CKD-stabilized specimens showed higher swell values as compared to raw soil specimens (see Figure 4.16). This swell behavior can be attributed to the formation of ettringite $\{Ca_6[Al(OH)_6]_2.(SO_4)_3.26H_2O\}$ due the reaction of calcium ions of the stabilizer with free alumina and soluble sulfates in soils, causing expansion of up to 250 percent when completely formed (Hunter, 1988; Berger et al., 2001). To confirm the formation of ettringite, SEM/EDS and XRD studies were conducted on representative tiny pieces of specimens tested for TST/3-D swell. Figures 5.12, 5.13 and 5.14 show SEM/EDS test results for 15% CKD-stabilized K-soil, 9% lime-stabilized, and 15% CKD-stabilized V-soil specimens, after 60-days of swelling. Elemental composition of soil specimen was analyzed on needle-shaped crystals using EDS. This elemental analysis showed the presence of calcium (Ca), sulfur (S), aluminum (Al) and/or oxygen (O), which are the main components for the formation of ettringite mineral.

Further, to confirm the ettringite formation, XRD tests were also conducted on 9% lime- and 15% CKD-stabilized V-soil specimens. For comparison raw soil was also tested, as shown in Figures 5.15 (A-C). Figure 5.15 (A) indicate that no ettringite peaks were noticed in the raw V-soil. The ettringite peaks were observed for 9% lime- and 15% CKD-stabilized V-soil specimens. This substantiates that in-situ formation of ettringite resulted in heaving as noted in Figure 4.16. Furthermore, the ettringite traces detected in 9% lime-stabilized soil were of higher intensity level as compared to 15% CKD-stabilized specimen, as a result, 9% lime-stabilized V-soil undergo higher sulfate induced heaving, as indicated in Figure 4.16. Based on SEM, EDS and XRD studies, it can be concluded that the ettringite was formed in lime- and CKD-stabilized specimen which yielded 3-D swelling.

Table 5.1 Soil Suction Parameters of Stabilized P-soil specimens

Type of Additive	Percent of Additive	Moisture Content (%)	Matric Suction (psi)	Total Suction (psi)	Osmotic Suction (psi)
None	0	9.9	168.7	317.7	149.1
	0	17.3	1.5	95.9	94.5
Lime	3	14.3	103.3	274.6	171.3
	6	14.2	73.4	211.0	137.4
	6	17.3	4.1	376.5	372.4
	9	18.6	4.1	117.9	113.6
	9	20.5	2.0	214.9	212.8
CFA	5	14.3	2.2	128.9	126.9
	10	13.4	3.3	134.3	130.9
	15	14.6	2.0	83.2	81.1
CKD	5	13.9	41.9	323.1	281.1
	5	18.4	0.7	94.9	94.2
	10	14.3	38.6	221.9	183.3
	10	19.1	2.0	115.8	113.8
	15	14.3	16.3	109.1	92.9
	15	19.0	1.0	112.9	111.9

Table 5.2 Soil Suction Parameters of Stabilized K-soil specimens

Type of Additive	Percent of Additive	Moisture Content (%)	Matric Suction (kPa)	Total Suction (kPa)	Osmotic Suction (kPa)
None	0	16.1	13.1	234.5	221.5
Lime	3	15.6	19.6	279.8	260.4
	6	15.9	17.3	139.8	122.5
	9	17.9	10.0	46.4	36.6
CFA	5	12.7	4.5	428.2	423.7
	10	14.8	5.4	350.8	345.4
	15	14.9	11.2	269.7	258.5
CKD	5	17.2	5.8	168.9	163.1
	10	17.0	5.5	321.5	315.8
	15	17.3	6.8	341.8	335.1

Table 5.3 Summary of Literature Review of Permeability Test on Stabilized Subgrade Soils

Authors/PDG	Codes Cited/ Apparatus Used	Type of Soil (Additive)	Dosage of Additive	Method to select OAC	Curing period	Specimen Size	Permeant	Influent pressure	Effluent pressure	Cell Pressure
Alquasimi (1993)	Constant Head Permeameter	Silty sand soils (C)	3% (C)	NA	28 days	4.0 in x 4.6 in	Distilled water	_______	0 psi	_______
Parsons et al. (2000)	Flexible Wall Leaching Cell	Clayey soils (L/C/FA)	5*% & 16% (FA)	ASTM D6276(L), PCA Guidelines(C)	7 days	4.0 in x 4.6 in	Distilled water	2.4 psi	0 psi	_______
Nalbantoglu et al. (2001)	Measured indirectly from 1-D Consolidation test	Marine Clays (FA+L)	15% & 25% (FA) 0-7% (L)	NA	0, 7, 30 & 100 days	3.0 in x 0.8 in	NA	NA	NA	NA
Mohamed (2002)	ASTM D2434	Sand (CKD)	6% (CKD)	Stress-strain Curves	_______	4.0 in x 4.6 in	Deionized water/ (CaCl, $CaSO_4$, $CaCO_3$, Solution)	5.0 psi	0 psi	_______
Lee et al. (2004)	ASTM D5084	Scoria (C)	3.5% - 5.5% (C)	NA	7 & 28 days	4.0 in x 2.0 in	Deaired tap water	0.6 psi – 3.8 psi	0 psi	_______
Kalinski et al. (2005)	ASTM D5084	Without Soil (FA+C)	NA	NA	7 days	4.0 in x 4.6 in	Deaired tap water	7.0 psi	0 psi	8.6 psi
AASHTO PDG (2002)	AASHTO T215	Granular soil	NA	NA	28 days	4.0 in x 4.6 in	Distilled water	NA	NA	NA
AASHTO PDG (2002)	US Army Corps Manual (EM-1110-2-1906)	Fine grained soil	_______	NA	28 days	4.0 in x 4.6 in	Distilled water	NA	NA	NA

ASTM D5084, "Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials using a Flexible Wall Permeameter", American Society for Testing and Materials
ASTM D2434, "Permeability of Granular Soils (Constant Head)", American Society for Testing and Materials
AASHTO T215, "Permeability of Granular Soils (Constant Head)", Standard Specifications for Transportation Materials and Methods of Sampling and Testing
US Army Corps of Engineers, "Engineering Manual (EM-1110-2-1906)", Procedure for permeability determination of fine grained soils (Falling Head) recorded in Appendix VII
ASTM D6276, "Standard Test Method for using pH to estimate the Soil-Lime proportion requirement for soil stabilization", American Society for Testing and Materials
Abbreviations: FA- Fly Ash, L-Lime, C-Cement, CKD-Cement Kiln Dust, PCA-Portland Cement Association, OAC-Optimum Additive Content, NA-Not Applicable

Table 5.4 Permeability Values of P-soil Stabilized Specimens

Type of Additive	Percentage of Additive	Water Head (cm)	Permeability (cm/s)
None	0	213	*
	0	274	*
Lime	3	213	*
	3	274	2.064×10^{-7}
	6	213	8.850×10^{-7}
	6	274	1.065×10^{-6}
	9	213	6.050×10^{-7}
	9	274	1.060×10^{-6}
CFA	5	213	*
	5	274	*
	10	213	*
	10	274	*
	15	213	*
	15	274	*
CKD	5	213	7.210×10^{-6}
	5	274	7.585×10^{-6}
	10	213	3.566×10^{-6}
	10	274	5.147×10^{-6}
	15	213	1.978×10^{-5}
	15	274	2.060×10^{-5}

*Samples were tested at a head > 600 cm, but no permeability was observed in 48 hours. Hence, samples were discarded.

Table 5.5 Permeability Values of K-soil Stabilized Specimens

Type of Additive	Percentage of Additive	Water Head (cm)	Permeability (cm/s)
None	0	213	*
	0	274	*
Lime	3	213	2.459×10^{-6}
	3	274	5.860×10^{-7}
CFA	5	213	1.022×10^{-6}
	5	274	2.973×10^{-7}
CKD	5	213	*
	5	274	*

*Samples were tested at a head > 600 cm, but no permeability was observed in 48 hours. Hence, samples were discarded.

Figure 5.1 Photographic View of Permeability Device used in this Study

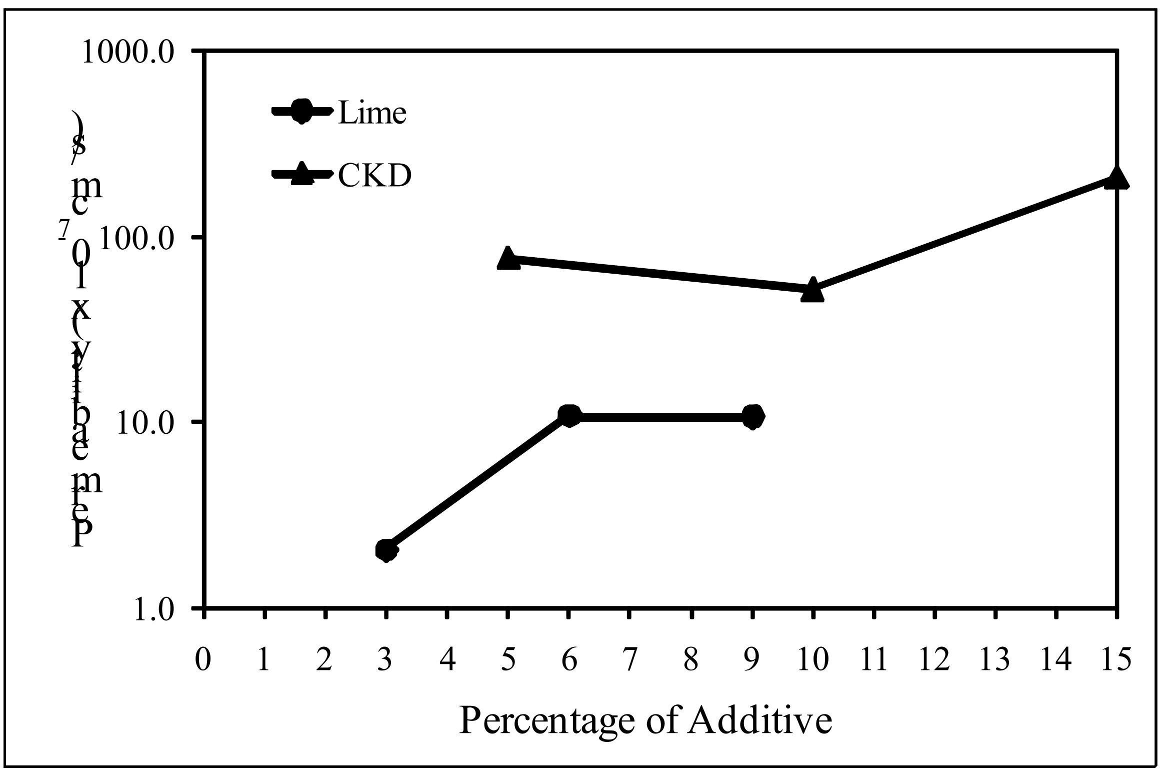


Figure 5.2 Variation of Permeability of P-soil with Percentage of Additives

Figure 5.3 JEOL JSM 880 used for Scanning Electron Microscopy

Figure 5.4 Specimen Mounted on Stubs for SEM

Figure 5.5 Rigaku D/Max X-ray Diffractometer

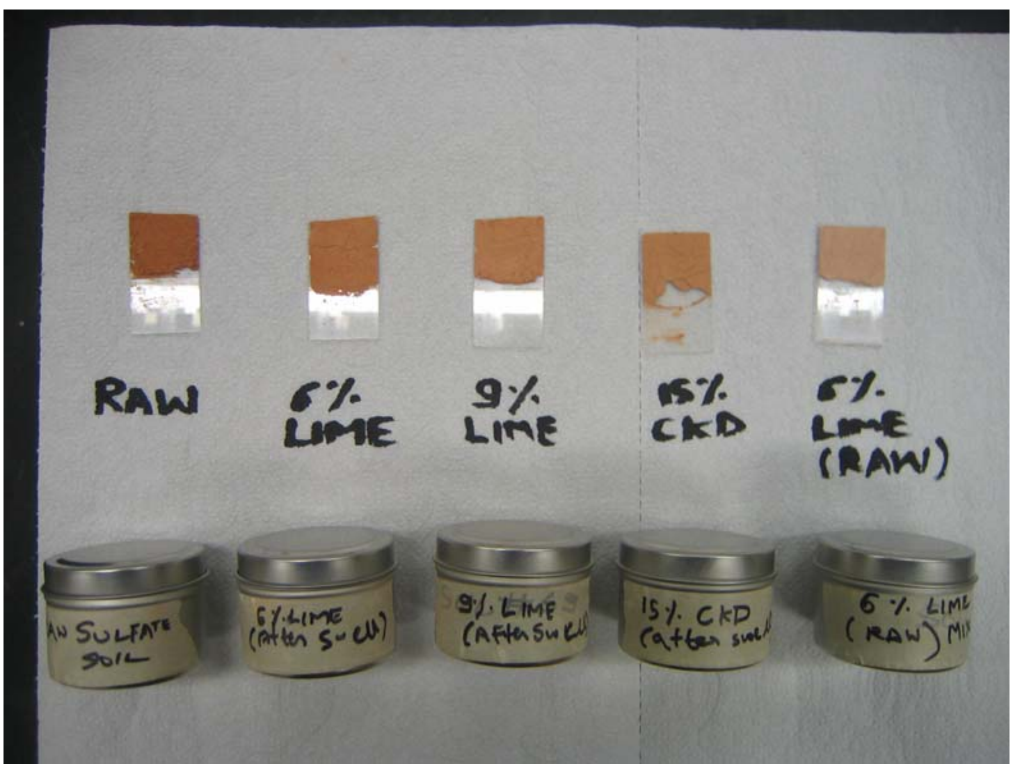


Figure 5.6 Specimen Powder Glued on Glass Plates for XRD

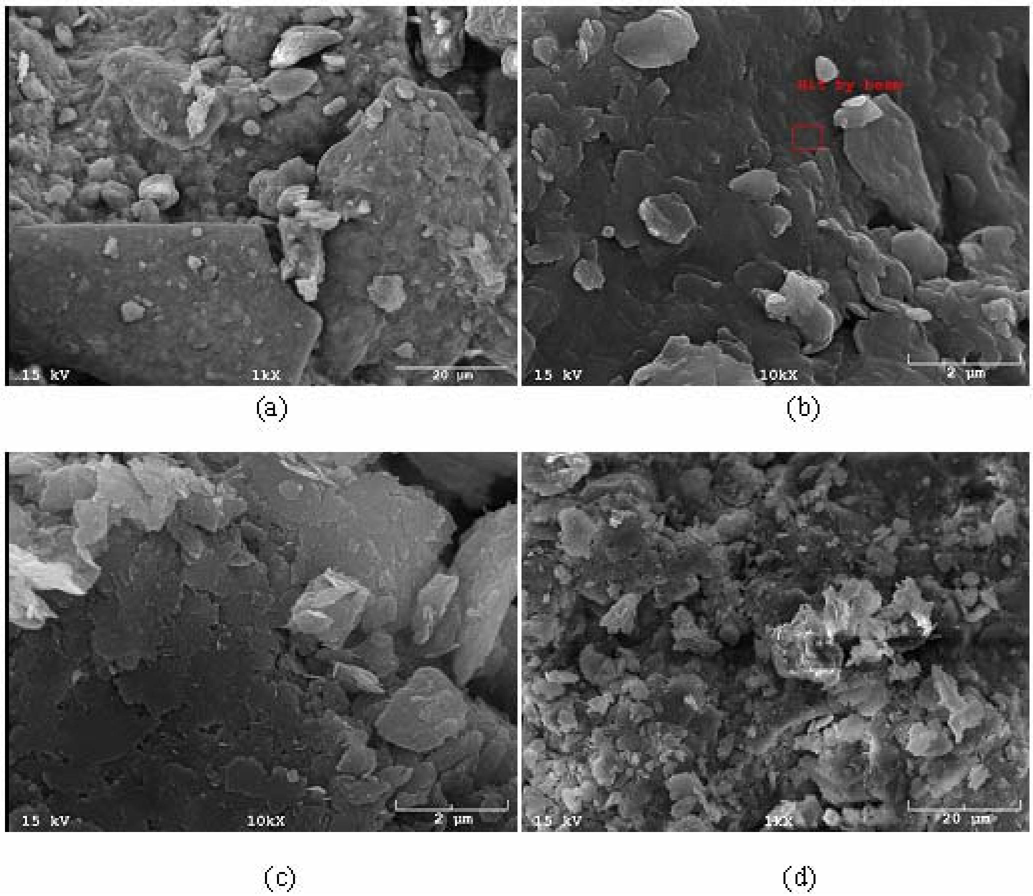


Figure 5.7 SEM Micrographs of Raw (a) P-, (b) K-, (c) V-, and (d) C-soil Specimens

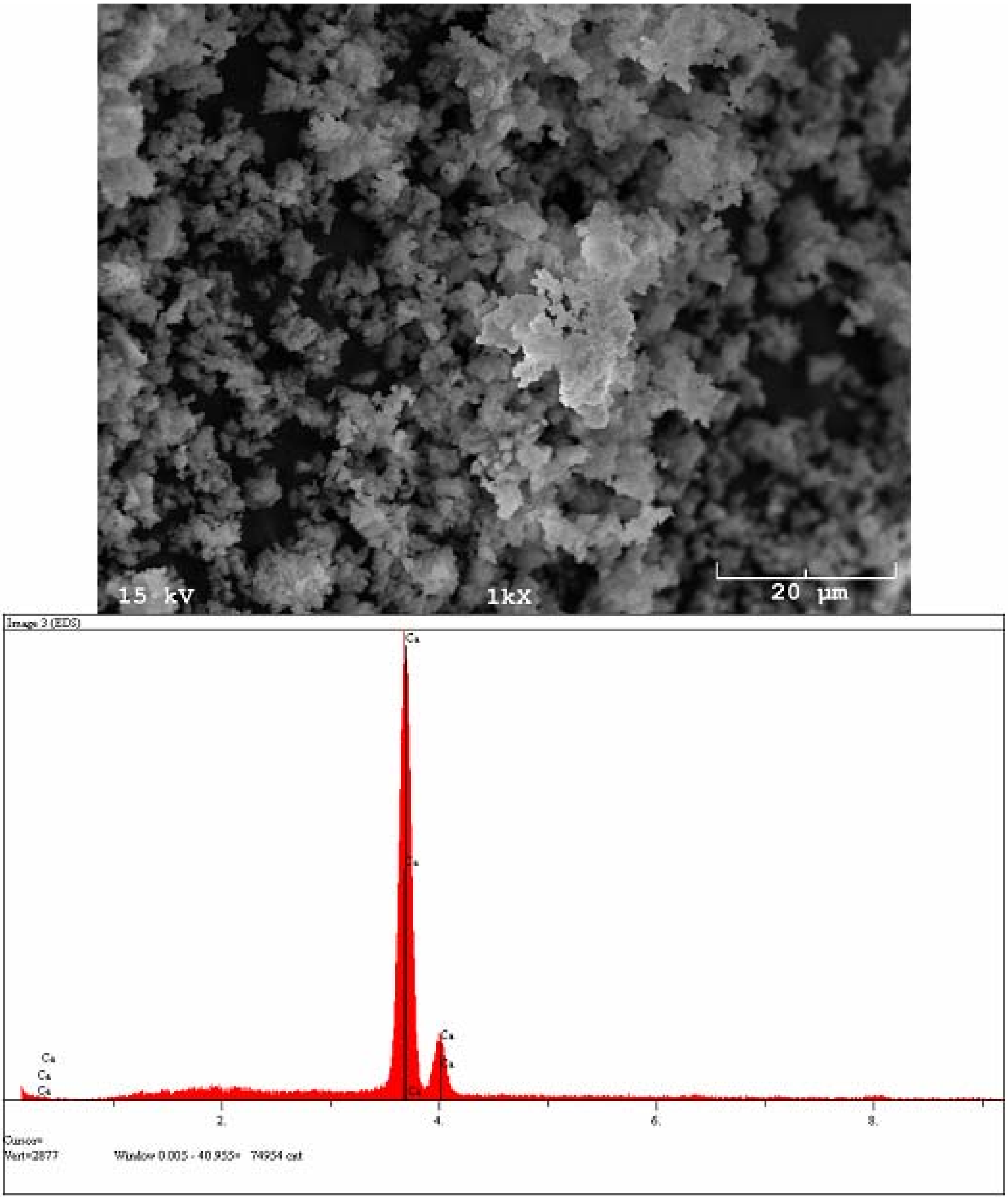


Figure 5.8 SEM/EDS of Raw Lime Powder

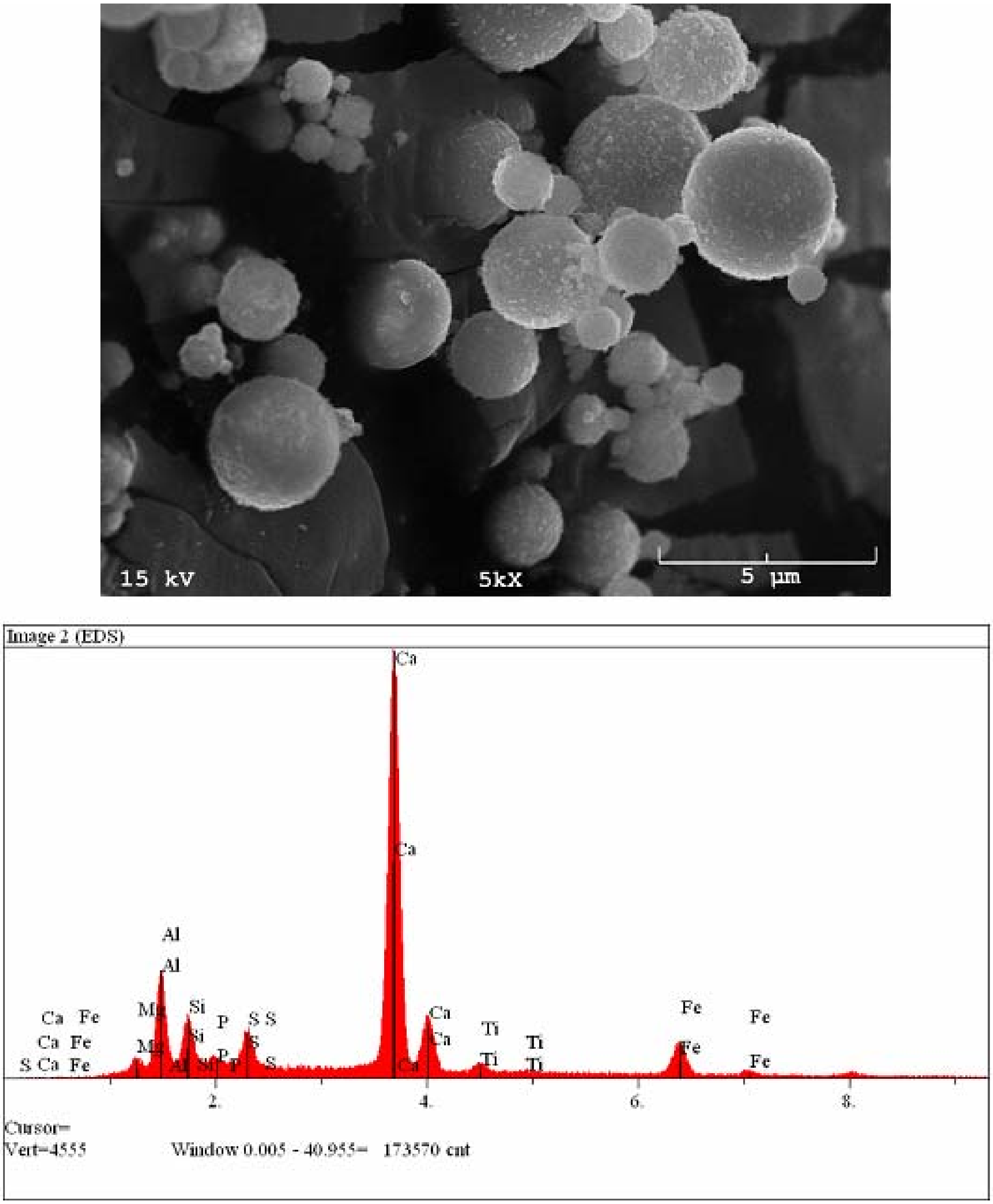
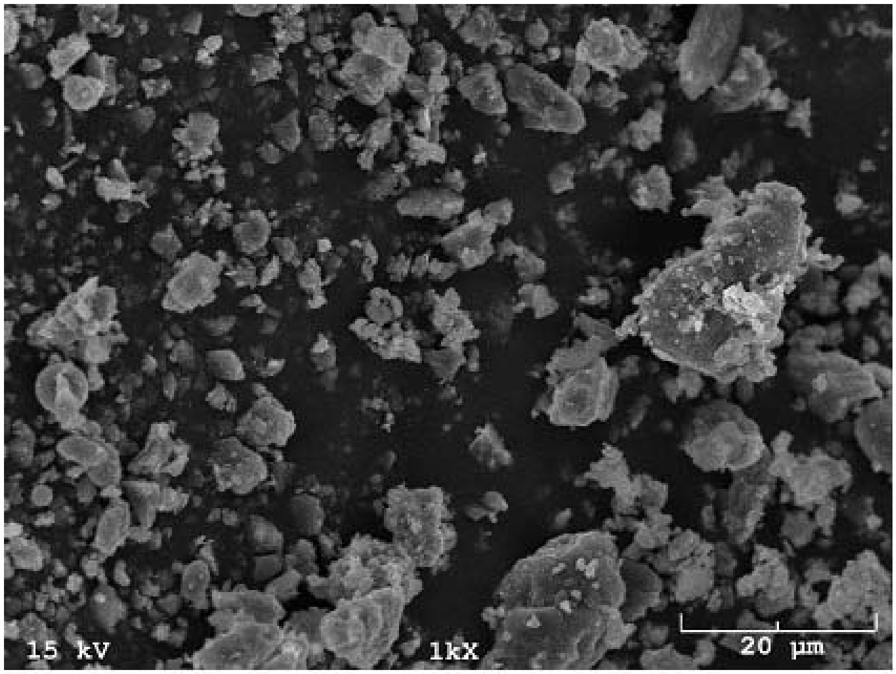


Figure 5.9 SEM/EDS of Raw CFA Powder



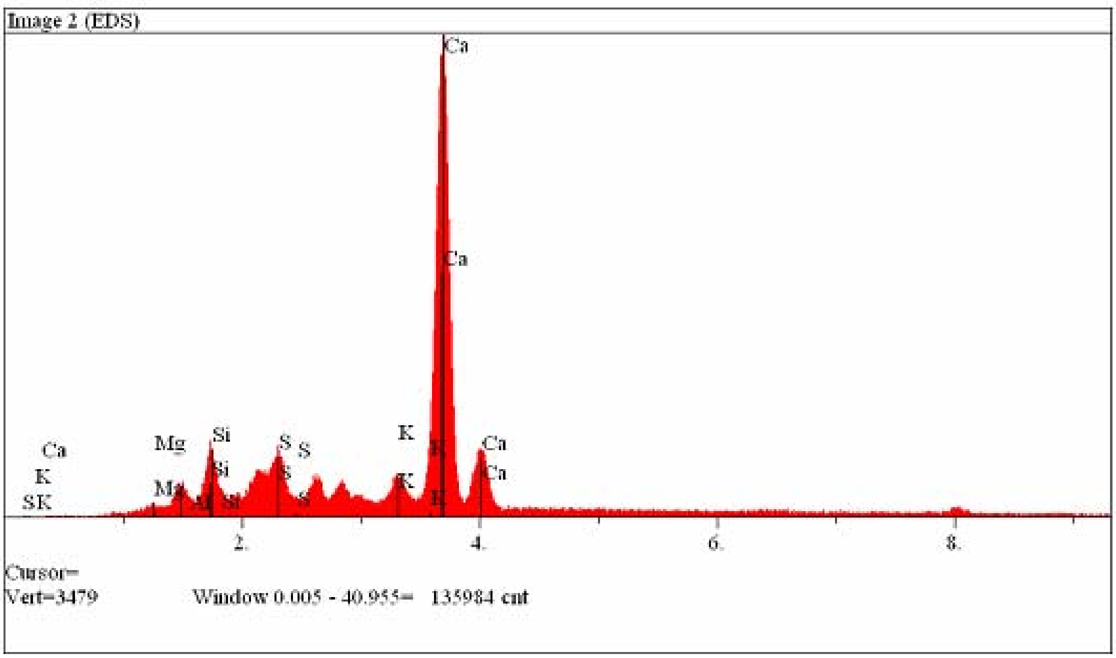
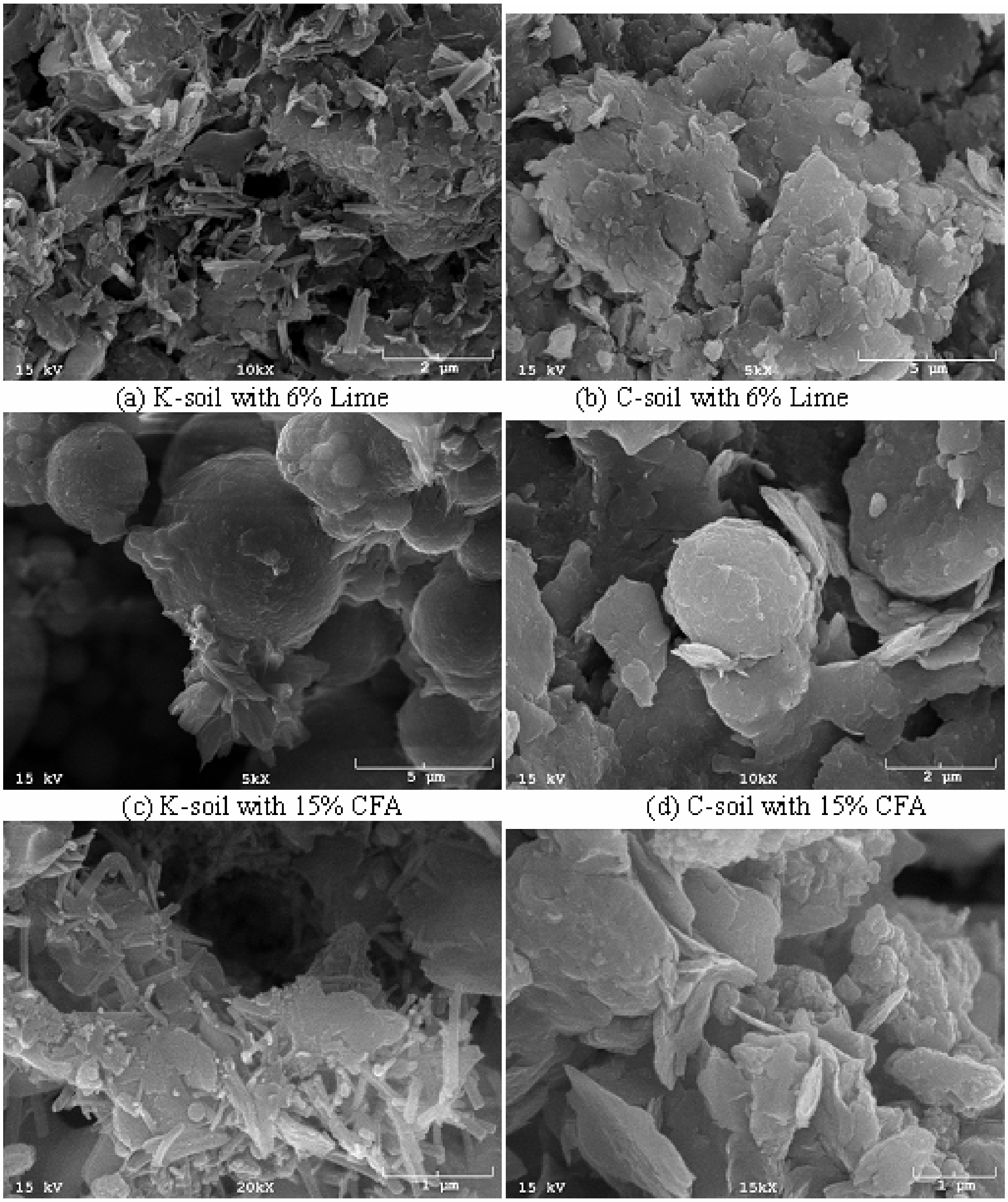


Figure 5.10 SEM/EDS of Raw CKD Powder



(a) K-soil with 6% Lime
(b) C-soil with 6% Lime
(c) K-soil with 15% CFA
(d) C-soil with 15% CFA
(e) K-soil with 15% CKD
(f) C-soil with 15% CKD

Figure 5.11 SEM Micrographs of the Indicated 28-Day Stabilized Soil Specimens

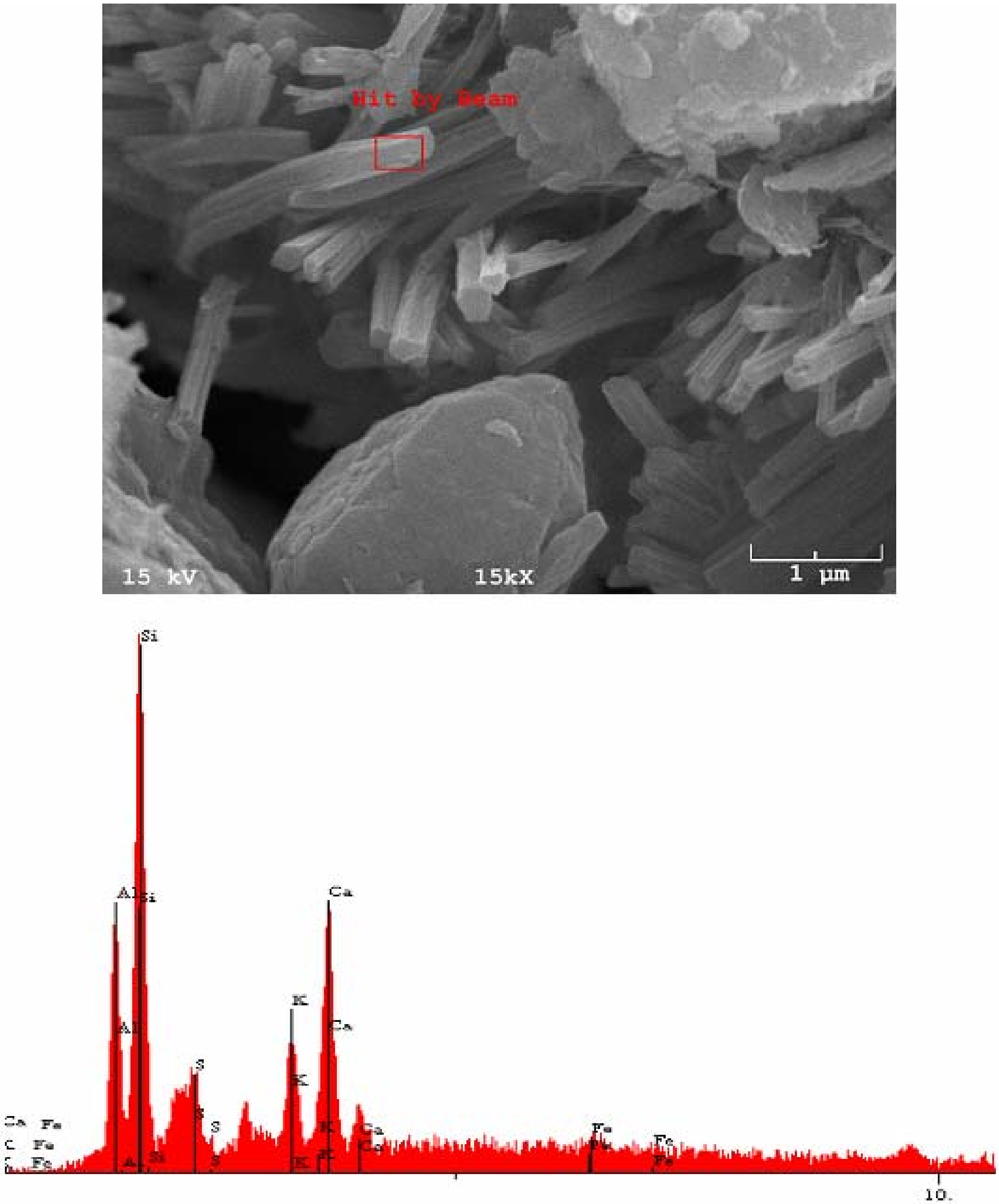


Figure 5.12 SEM/EDS of Ettringite Deposited in the 15% CKD-Stabilized K-soil Specimens (After 60-Day Swell)

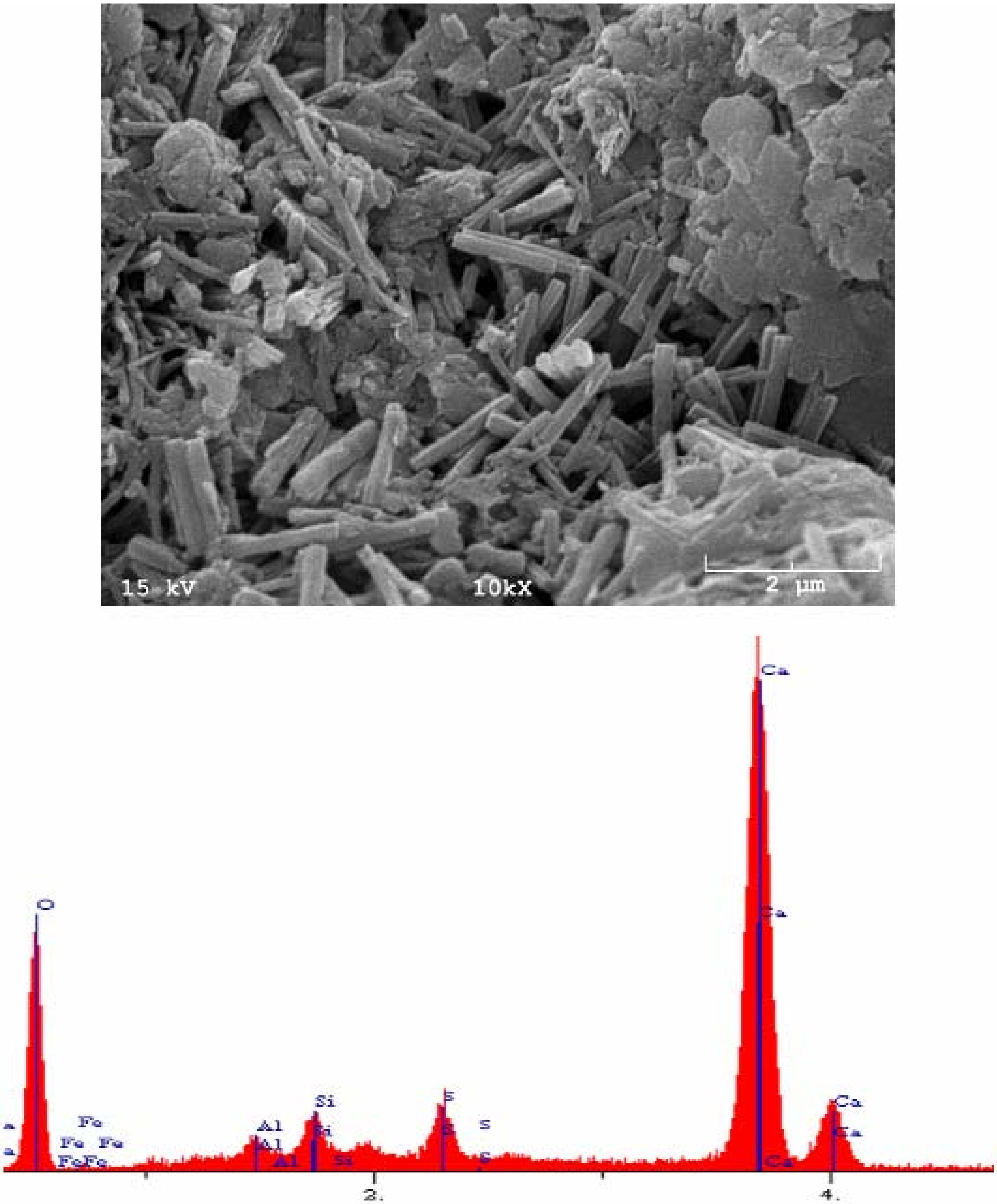


Figure 5.13 SEM/EDS of Ettringite Deposited in the 9% Lime-Stabilized V-soil Specimens (After 60-Day Swell)

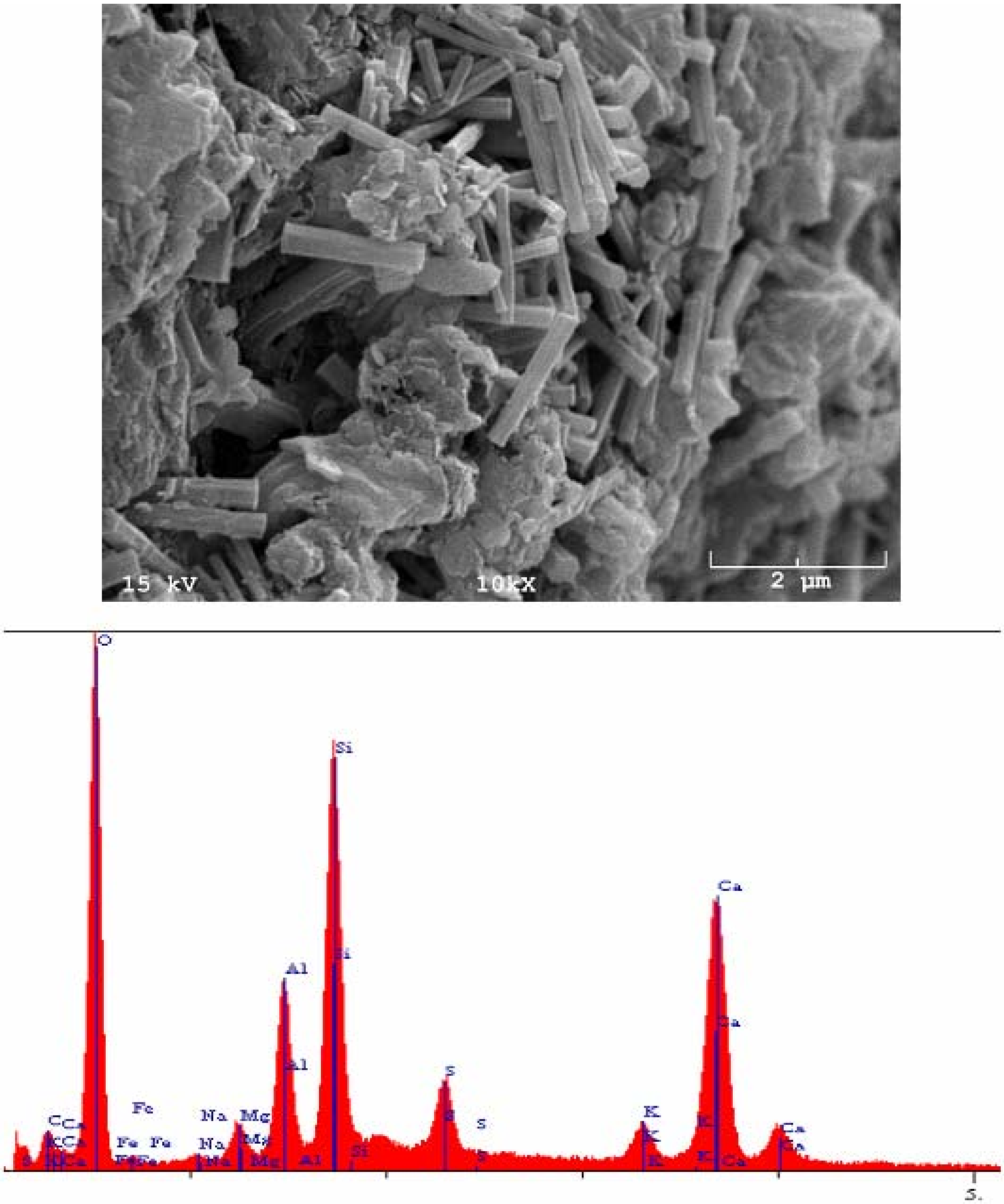


Figure 5.14 SEM/EDS of Ettringite Deposited in the 15% CKD-Stabilized V-soil Specimens (After 60-Day Swell)

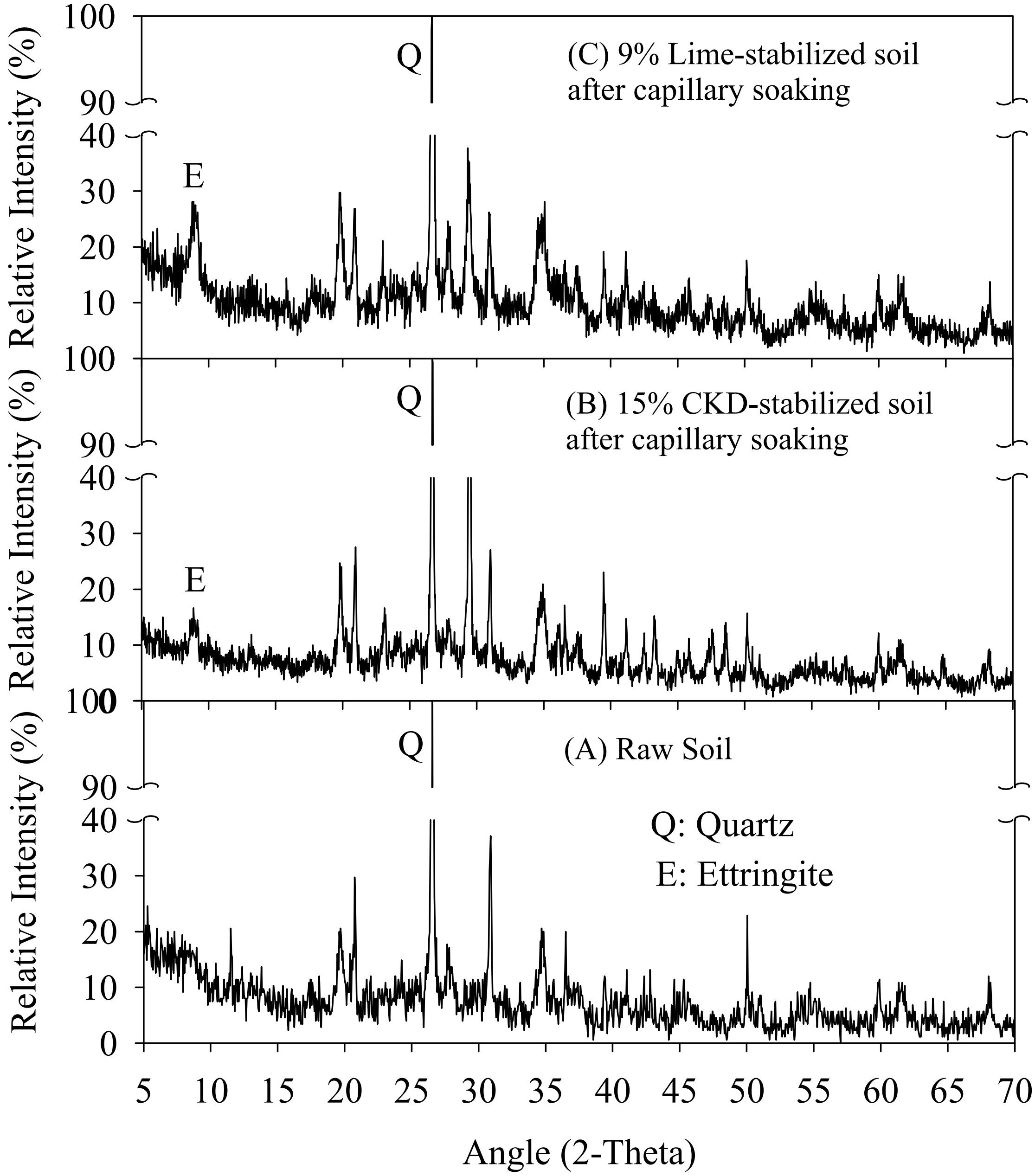


Figure 5.15 X-Ray Diffraction Results of Stabilized V-soil Specimens (After 60-Day Swell)

6.1 Conclusions

From the laboratory tests and analysis of data presented in the preceding chapters, the following conclusions can be drawn:

1. The Proctor results on all of the soils showed an increase in OMC and a decrease in MDD with increasing amount of lime and CKD. However, no such specific trend was observed with CFA.
2. The resilient modulus, modulus of elasticity and unconfined compressive strength of stabilized soil specimens are higher than the corresponding resilient modulus of raw specimens. The percentage of increase depends upon many factors such as type of stabilizing agent, percentage of stabilized agent, and soil type.
3. All three stabilizers improved the resilient modulus of P-, K-, V- and C-soil specimens. At lower application rates (3% to 6%), the lime-stabilized soil specimens showed the highest improvement in the M_r values. At higher application rates (10% to 15%), however, CKD treatment provided maximum enhancements. Overall, K-soil and C-soil specimens showed the highest and the lowest improvements in the M_r values. One of the explanations could be differences in the pH values of K- and C-soil. For example, raw K- and C-soil had the highest and the lowest pH value of 9.07 and 4.17, respectively, among the four soils used in this study.
4. The addition of additive, namely, lime, CFA or CKD, increased the unconfined compressive strength and reduced the failure strain.

5. The TST results revealed that lime- and CFA-treatment helps reduce the moisture susceptibility. CKD-stabilization, however, makes stabilized specimens more susceptible to moisture, as compared to raw soil specimens.
6. The three-dimensional swelling tests on non-sulfate bearing soil (P-, K- and C-soil) showed that lime is more effective in reducing the swell of raw specimens, as compared to CFA and CKD. In contrast to lime and CFA, an increase in the percentage of CKD makes specimens more susceptible to moisture and three-dimensional swell. It is believed that such an increase in volume is due to the presence of high sulfate content (28,133 ppm) in CKD causing sulfate-induced heaving (ettringite formation).
7. The three-dimensional swelling test on sulfate bearing soil (V-soil) showed an increase in volume for lime- and CKD-stabilized specimens while a reduction in volume for CFA-stabilized specimens was observed, as compared to raw soil specimens. This increase in volume is attributed to sulfate-induced heaving which results in the formation of expansive mineral ettringite. Further, presence of ettringite was verified using SEM/EDS tests in conjunction with XRD analysis.
8. All the three additive used in this study, namely, lime, CFA and CKD, are effective in reducing the plasticity of soils. However, lime-stabilization is more effective as compared to CFA and CKD-stabilzation in reducing the PI of soils. In addition, the percentage of reduction in PI was observed maximum with K-soil among all the three soils (K-, V- and C-soil). This could also be one of the reasons for the highest improvement in M_r values of stabilized K-soil specimen, as reported in conclusion # 3.
9. Ranking of all additives on the basis of different properties evaluated in this study suggested that 9% lime is the best additive for non-sulfate bearing soil (K- and C-soil). On

10. Regression equations were developed for the lime-, CFA- and CKD-stabilized soil to estimate M_r values. Predicted values were well correlated with measured values.
11. The SEM analysis shows formation of hydration products with soil matrix as a result of stabilization. It is reasoned that the hydration products within the matrix provide better interlocking between the particles and possible higher resistance to shear deformation and also reduce void within the matrix resulting in overall strength gain. The results of the analysis conform to the results of the M_r, M_E and UCS tests.
12. To rationalize swelling behavior of CKD-stabilized specimens, presence of ettringite was verified using SEM/EDS tests. This was also conformed using XRD analysis.

6.2 Recommendations

The following recommendations are made for further studies:

1. As indicated in this study, strength (UCS) and stiffness (M_r, M_E) evaluation alone can be misleading. In the present study, for example, CKD showed better UCS, M_r and M_E values but increase in volume during 3-D swell testing. It is also worth noticing that properties of CKD can vary significantly from plant to plant depending on the raw materials and type of collection process used (Miller and Zaman 2000). Similarly, fly ash properties may be unique to same source while it may differ from ashes obtained from other sources (Ferguson and Levorson 1999). These differences in physical and chemical properties can lead to different performance of stabilized soil specimens. In the present study, for example, CKD showed swelling of specimens due to high sulfate content (28,133 ppm) while CFA with lower sulfate content (3,280 ppm) helped by reducing swelling. Hence, it is suggested that a proper mix design be done with locally available

2. This study showed that CFA is the best additive for stabilizing sulfate bearing soil. However, this study was limited to only one sulfate bearing soil. Further, performance of CFA should be evaluated with other sulfate bearing soils.
3. This study evaluated only three (strength, stiffness and durability) out of the required four categories that have been identified as key to performance (AASHTO, 2004). Further study is needed to evaluate and compare the fatigue fracture of subgrade soils stabilized with lime, CFA and CKD, for an overall pavement performance evaluation.
4. From the literature review conducted, there is no standard test available to evaluate the durability of soil specimens stabilized with lime, CFA and CKD. The "conventional" ASTM test (ASTM D 559/560) for soil-cement, however, are considered overly severe and abrasive and do not simulate the field conditions adequately (Kalankamary and Donald 1963; Miller and Zaman 2000). Moreover, Little et al. (2005) have emphasized the need for developing a rapid and reliable test method for assessing the impact of moisture on stabilized materials. Hence, it is important to conduct additional studies to develop standardized durability test procedures addressing the effects of F-T/W-D actions on stabilized subgrade soil. Also, it is important to explore the combined effect of both F-T and W-D cycles on M_r values and other properties. A current research study entitled "*Tube Suction Test for Evaluating Durability of Cementitiously Stabilized Soils*" at the University of Oklahoma is an attempt to verify that the tube suction test for evaluating durability of stabilized soil specimens.

5. Flexural strength and fatigue life influence the structural response and fatigue performance of a stabilized subgrade soil layer. Therefore, it is recommended that studies be conducted focusing on the evaluation of fatigue parameters for soil layer stabilized with lime, CFA or CKD, commonly used additives by Oklahoma Department of Transportation.

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