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Unconfined Compressive Strength

And

CBR Relationships On LFA Treated Loess

By

Praveen Katamaneni

A thesis submitted in partial fulfillment of the requirment for the degree of

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Committee Approval

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Abstract

Soil stabilization has long been one of the primary and effective means of improving the workability and performance of native problematic soils. Most of the soil deposits in south-central and southeastern Idaho are identified as loess, which are highly collapsible. These native soils as subgrades can result in inadequate pavement support and potential reduction in pavement service life. The main objective of this research study was to investigate the application of different soil stabilizing agents consisting of quicklime, hydrated lime, Class F fly ash, and Class C fly ash as individual components or in combinations and determine the most viable proportion and combination of the admixtures. The performance of both treated and untreated soils was evaluated using Unconfined Compression Strength (UCS) tests conducted in the soils laboratory of Civil and Environmental Engineering Department at Idaho State University. Results of the UCS tests demonstrated that lime-fly ash stabilization resulted in a significant increase in compressive strength of the study soil. The necessity of soaking/curing for an improved compressive strength was firmly established by subjecting the treated samples to representative moist-curing conditions. Furthermore, an attempt was made to study the relationship between the Unconfined Compressive Strength (UCS), companion California Bearing Ratio (CBR), and resilient modulus (M_r) calculated from CBR test results of the study soil.

Chapter 1 – Introduction

1.1 Problem Statement

The subgrade layer forms the basis of a typical pavement structure. In-situ subgrades often do not provide the required stability impacting the long-term performance of pavement structures significantly. The poor engineering properties of soil in its natural state propelled engineers to reinforce the concept of improving soil strength through stabilization techniques. In comparison to the traditional cut-and-fill approach, soil subgrade stabilization offers significant strength improvement economically and is widely accepted as an effective means of improved pavement performance. Research and extensive experimentation on soil stability points out that stabilizing the subgrade results in reduced thickness of other pavement layers (Beeghly, 2003).

Soil stabilization refers to the improvement of the engineering characteristics such as shear strength and stability or bearing capacity of the native subgrade soil. This can be achieved through various techniques such as vibration, surcharge load, strengthening structural reinforcement by structural fill, controlled compaction, admixtures, grouting and other methods (Kazemian et al., 2010). Chemical stabilization of the subgrade soils improves soil stability especially in the case of soft fine-grained and expansive soils. Parsons and Milburn (2003) point out that a single stabilizer may improve performance significantly for some soils while for many soils, it works effectively in combination of different stabilization agents (Parsons and Milburn, 2003). Local availability and financial considerations could also be deciding factors on the stabilizers to be used.

Loess deposits of south central and southeastern Idaho are low plasticity soils with significant shrinkage and swell potential. The low bearing strength of silt soil makes it highly unstable and not a viable pavement subgrade material. The depth of loess over much of this region extends between 50 to 100 cm thick with the thickest sections located south and east of the Snake River (Lewis et al., 1975).

Today, lime-fly ash is used primarily for soil stabilization of silty soils in different states such as Pennsylvania, Kentucky, Texas, Indiana, and others. Lime-fly ash is used to treat the problematic soils considered in the current research. The silty nature of the project soil deemed lime-fly ash stabilization as a viable option. The deficiency of the clay is compensated by the fly ash, which is partly composed of lime. In addition, fly ash is a by-product of industrial waste obtained from burning of coal and is relatively cheaper and easily available.

The stabilizers used in the current study are quicklime, hydrated lime, Class C fly ash and Class F fly ash. The study soil was treated with various combinations of lime-fly ash. It was also treated with lime and fly ash alone to determine the individual contribution of each stabilizer. The performance of the treated soil was evaluated on the basis of Unconfined Compressive Strength (UCS) and a comparative analysis between UCS and resilient modulus (M_r) values calculated from the California Bearing Ratio (CBR) test results.

1.2 Research Objectives

The primary objective of this research study was to investigate the application of different admixtures to the soil samples in laboratory conditions that represent the problematic loess deposits in south-central and southeastern Idaho. The goal of soil stabilization was to evaluate the performance of lime-fly ash treatment in improving the compressive strength and durability of the subgrades. A secondary objective of the concerned study was to determine a viable proportion of lime-fly ash combination to achieve the desired strength and stability in the subgrade soil. The study soil was treated with different proportions of lime and fly ash as individual components or in combinations to evaluate the performance and determine the optimum proportion. The third objective of the study was to investigate noticeable patterns in soil behavior when subjected to similar chemical treatment techniques and curing conditions that best represent the comparison of experimental results from both CBR and unconfined compressive strength tests. The fourth and final objective of this project was to draw a conclusion from the test results and provide appropriate recommendations for practical applications of lime-fly ash stabilization on the field.

1.3 Scope of Study

The scope of this study emphasized a series of activities specific to the research objectives. A representative study soil was excavated from the project site located in southwest Pocatello and was transported to the laboratory in air tight buckets. Unconfined compressive strength (UCS) testing was performed on the samples compacted from the native soil. Soil stabilizing agents consisting of quicklime, hydrated lime, Class F fly ash, and Class C fly ash were acquired for stabilization applications. UCS tests were conducted on soil specimens treated with lime alone, fly ash alone, and a mixture of lime and fly ash. The performance of soil samples was evaluated to determine the optimum proportion of lime and fly ash that enhances the compressive strength of the study soil. Experimental results from companion CBR tests were used to compare and evaluate the performance of the in-situ soil after curing and/or lime-fly ash treatment.

1.4 Outline

Chapter 2 summarizes the stabilizers used in this research study. The classification and characteristics of the admixtures used along with the mechanics of stabilization that occurs in a lime-fly ash treated soil are detailed in this chapter.

Chapter 3 presents an extensive literature review on lime and/or fly ash treated soils. This chapter is divided into sections of soils treated with lime and fly ash only as individual components or in combinations to improve the performance of soils. Prior studies conducted that are comparable in terms of unconfined compressive strength and CBR are also discussed.

Chapter 4 briefly discusses the study soil and its location followed by the sample preparation and methodologies of the experimental testing program implemented in this study to evaluate the laboratory performance of lime-fly ash treated subgrades.

Chapter 5 deals with the complete list of test results and analysis. This chapter summarizes the different proportions of lime-fly ash used in the research study and evaluates how well the study soil performed when treated and/or moist cured as compared to the untreated soils.

Finally, Chapter 6 summarizes and concludes this research study with recommendations for further extension of the work and possible research in the future.

Chapter 2 – Background

2.1 Introduction

Soil stabilization refers to the process of changing natural properties of in-situ soil to improve strength, workability, and durability. It is widely accepted as an effective means of improving soil properties and pavement system performance (Parsons, 2003). The use of stabilized soils in place of natural aggregates can have considerable environmental and economic advantages (Sherwood, P.T., 1993). Stabilizing soils remains the most popular and viable technique where removal and replacement of poor subgrades with a superior material is not always economical. Soil stabilization can be done through chemical and mechanical processes. Mechanical stabilization includes mixing or blending soils of different gradations, thereby altering the structural properties of the soil and compaction to improve strength. Chemical stabilization traditionally involves the use of cement or chemical additives, such as lime, which chemically alter the soil itself through pozzolanic reactions, thereby improving the load-bearing capacity of the native soil.

Recent studies show that stabilizing agents like lime and fly ash can be used as individual components or in combinations to effectively treat the native problematic soils. Lime alone has traditionally been used to treat subgrade soils that are soft and highly cohesive in nature. National Lime Association notes that even though lime can modify almost all fine-grained soils, the most significant improvement occurs in clay soils of moderate to high plasticity. Fly ash is a by-product recovered from the combustion of coal. Rather than being disposed of as a waste, effective use of fly ash as a stabilizing agent can be regarded as economically feasible and environmentally beneficial. Depending on its lime content, fly ash can be used as a stand-alone material in soil stabilization or in combination with a cementitious agent in a pozzolanic reaction. Furthermore, fly ash treatment results in a stabilized soil of less shrinkage in comparison with soft soils treated with lime or cement alone (Natt & Joshi 1984).

Lime-fly ash combinations are often used for stabilization of granular soils. In general, high silt, low active clay subgrades with more than 25% fines passing the No. 200 sieve and Plasticity Index falling in the range of 10 to 20 are viable candidates for lime-fly ash stabilization. These subgrades with low clay content lack reactive silica and alumina. This deficit is made up by the silica and alumina in the fly ash (Beeghly, 2003). Lime increases the soil pH and enhances the solubility of alumina and silica in a pozzolanic reaction forming calcium silicates and calcium aluminates with strong cementitious properties. The pozzolanic reaction of the lime-fly ash combination will improve the bearing capacity and strength of soil subgrades.

This research concentrates on the use of lime and/or fly ash as stabilizing agents and in determining a viable combination of the admixtures to improve the compressive strength of stabilized soil relative to an improved CBR. Chemical stabilization of the problematic soils in the current research study is especially significant because southeastern Idaho loess is collapsible and susceptible to breakdown (Mahar, 2005).

2.2 Stabilizers

Recent technology has led to a variety of soil stabilization techniques and an increased number of traditional additives used for stabilization purposes. Cement, lime, fly ash by itself or in combination with lime or cement, asphalt, bitumen emulsions, tar, cement kiln dust (CKD), and ionic stabilizers are all used as stabilizing agents. While bitumen is not environmentally friendly, use of Portland cement as an additive can often be expensive.

Many of these stabilizers have their own advantages and some disadvantages as mentioned above. The current research is limited in scope to the use of lime (Quicklime, Hydrated Lime) and fly ash (Class C, Class F) as individual components or in combinations.

Lime and fly ash are used primarily in this study, because they are commonly used in practice, offer cost savings, and are easily available. Furthermore, high silt, low clay subgrades with PI in the range of 10 to 20 will have a significant improvement in terms of bearing capacity and strength due to the pozzolanic reaction of lime-fly ash combination in the presence of moisture (Beeghly, 2003). The problematic project soil is classified in the USCS as silt (ML), is similar in characteristics to those mentioned in the literature in terms of potential benefits when subjected to lime and fly ash (LFA) stabilization. A discussion of these stabilizers and their classifications appears in the following sections.

2.2.1 Lime

According to ASTM International standards (2006), lime is described as all classes of quicklime and hydrated lime that could be calcitic (high calcium carbonate; $CaCO_3$), magnesian or dolomitic (mixture of calcium and magnesium carbonate; $CaMg(CO_3)$). In general, lime stabilization is an economical approach for clay subgrades with PI falling in the range of 5 to 20. Two types of lime were used in this study (see Table 2-1). Calcium oxide (CaO), commonly known as quicklime and calcium hydroxide ($Ca(OH)_2$), traditionally called slaked lime or hydrated lime, were used. Commercial grade limes were furnished by Chemical Lime Company.

Table 2-1 Chemical Composition of Lime Used in the Current Research

Lime	Chemical Composition		Source & Location
Quicklime	Calcium Oxide	CaO	Chemical Lime Company, Texas
Hydrated Lime	Calcium Hydroxide	$Ca(OH)_2$	

Quicklime is derived from the calcination (thermal decomposition of ores in presence of oxygen) of limestone and has two primary forms, namely high calcium quicklime containing 0 to 5 percent magnesium carbonate and dolomitic quicklime containing 35 to 46 percent magnesium carbonate. Hydrated lime is a dry powder obtained from the reaction of quicklime with sufficient water converting oxides into hydroxides. Based on the type of quicklime used and amount of water available to react, hydrated limes could be classified as high calcium hydrated lime (72-74% CaO) and dolomitic hydrated lime (46-48% CaO & 32-34% MgO). Quicklime is highly reactive with water, forming calcium hydroxide, and generates considerable heat and an increase in soil pH in the hydration process. Calcium hydroxide splits into ions and the elevated pH increases the solubility of silicates and aluminates in the fly ash to form cementitious materials consisting of calcium silicates and calcium aluminates. The pozzolanic reactions (see section 2.3) result in a gradual increase in shear strength of the soil.

2.2.2 Fly Ash

Fly ash is a by-product recovered from the combustion of pulverized coal. Depending upon the type of coal burned and operational characteristics of the manufacturing plant, the chemical composition of fly ash varies. ASTM C618 - 12a classifies two types of fly ash: Class C and Class F. The primary difference between Class C and Class F fly ash is the relative amount of silicon dioxide (SiO_2), aluminum oxide (Al_2O_3) and iron oxide (Fe_2O_3)

content in the ash. Class F fly ash has relatively higher percentages of silicates and aluminates. According to the ASTM specifications, a minimum of 70 % by dry weight of silicates, aluminates, and ferrites constitute Class F fly ash as compared to a minimum of 50 % dry weight in Class C fly ash. The chemical composition and classification of fly ash are summarized in Table 2-2. The fly ash used for this research study was supplied by the Chemical Lime Company in Texas.

Table 2-2 Chemical Composition of Fly Ash Used in the Current Research

Fly Ash	Chemical Composition		Source & Location
Class C	Silicates/Aluminates/Ferrites	$SiO_2/Al_2O_3/Fe_2O_3$	Chemical Lime Company, Texas
Class F	Silicates/Aluminates/Ferrites	$SiO_2/Al_2O_3/Fe_2O_3$	

Class C fly ash is a self-cementing fly ash produced from burning lignite and sub-bituminous coal. It has higher concentrations of alkali and sulfates and a significant amount of lime (more than 20%). Class C fly ash has self-cementing properties and can be used as an individual component in stabilizing soil subgrades without necessarily combining with lime or cement. In the current research, Class C fly ash was used to evaluate its performance and effectiveness in comparison with Class F fly ash.

Class F fly ash is produced from burning anthracite and bituminous coal predominantly found in the eastern, mid-western, and southern regions of the United States (Schaefer, 1997). With less than 10 to 20 % lime content, Class F fly ash is pozzolanic in nature but not self-cementing and usually requires cementing agents like Portland cement, quicklime, or hydrated lime in tandem to be effective and produce cementitious materials. The majority of the tests in this research study were conducted using Class F fly ash because of the silty nature of problematic loess deposits in southeastern Idaho. The addition of Class F fly ash makes up for the lower clay content in the native soil and will work effectively with lime to form cementitious materials.

2.3 Mechanics of Stabilization

A pozzolanic reaction involves the formation of cementitious materials through the reaction of pozzolans like aluminates (Al_2O_3) and silicates (SiO_2) in fly ash with lime (CaO, CaOH) in the presence of water. Lime-fly ash stabilization occurs in a chain of reactions starting with hydration, followed by coagulation, which leads to the solubility of silica and alumina and eventual formation of the cementitious compounds. The pozzolanic reaction in the stabilization of soil using lime and fly ash can be chemically represented as follows (Transportation Research Board, 1987).

$CaO + H_2O \rightarrow Ca(OH)_2$ + Heat

$Ca(OH)_2 \rightarrow Ca^{++} + 2(OH)$

$Ca^{++} + 2(OH) + SiO_2 \rightarrow$ CSH (Calcium Silicate Hydrate – Gel)

$Ca^{++} + 2(OH) + Al_2O_3 \rightarrow$ CAH (Calcium Aluminate Hydrate – Fibrous)

Chapter 3 - Literature Review

3.1 Introduction

A brief review of the findings of earlier investigations on the mechanisms of lime-fly ash stabilization process and the influence of lime and/or fly ash addition on the properties of in-situ soil-like plasticity index and compressive strength have been presented in this chapter. A comprehensive literature review was conducted in order to develop a valid framework for analyzing and evaluating the effect of lime and/or fly ash on stabilization of the study soils. Most of the research investigations were carried out between 1996 and 2003 and the soil samples were taken from sites located in the South, the Midwest, and northeastern regions of the United States. The typical lime to fly ash ratio was 1:2 by dry weight in these research studies. However, lime to fly ash ratio of 1:3 was used in the current research to make up for the low clay mineral content in the in-situ loess deposits. The admixtures used in the treatment process consisted of lime, fly ash and combination of lime and fly ash.

The concerned research project was done in two parts where in the first part dealt with the classification of the project soil in accordance with AASHTO, USCS, and CBR tests to evaluate the bearing capacity of the soil with and without addition of lime and/or fly ash. Unconfined compressive strength (UCS) test was adopted for this part of the research study. Test results from companion California Bearing Ratio (CBR) tests performed in the first part of the research study were used for comparison and relationship interpretation. Most of the published research findings used unconfined compressive strength, resilient modulus (M_r) and/or freeze-thaw tests to analyze and evaluate the performance and effectiveness of admixtures in soil stabilizations.

3.2 Summary of Literature Review

3.2.1 Introduction

Various laboratory investigations have been conducted independently on stabilization of soil using lime alone, fly ash alone, or a mixture of lime and fly ash. In this study, the literature review focused on UCS test results to evaluate the effectiveness of lime-fly ash additives on soil stabilization. A significant increase in compressive strength ranging from 140% to 450% was recorded with the addition of admixtures on different soil types, which will be discussed in the following sections. Within the treated soils from literature, experimental results were summarized in terms of lime stabilization followed by fly ash stabilization and lime-fly ash stabilization. The effect of plasticity index (PI) of soil before and after treatment is also discussed. The review concentrated on fine grained soils.

3.2.2 Untreated Soils

The unconfined compressive strength test results on untreated soil used in the literature are summarized in Table 3-1. The study was performed on lean clay (CL) samples at different moisture content levels. As the water content increased, UCS decreased. The compressive strength decreased from 26 psi to 3 psi as the saturation increased from 12 to 22%. There was no sufficient data on CBR test results on untreated in-situ soils from literature for comparison.

Table 3-1 Summary of UCS Test Results on Untreated Soil from Literature

Investigation	Year	Soil/Rock Type		Untreated		
		USCS	AASHTO	w%	γdry	qu (psi)
Puppala	1996	CL	A-7-6	12	16.2	26
				14	16.4	22
				17	16.8	13
				20	16.4	6
				22	16.2	3

3.2.3 Treated Soils

The study soils from literature were treated with lime (Quicklime (CaO), Hydrated lime ($Ca(OH)_2$) and non-reported lime), fly ash (Class C and Class F), and lime-fly ash mixture. The typical lime to fly ash ratio was 1:2 by dry weight.

3.2.3.1 Lime Treatment

A summary of UCS test results on lime treated soils from the literature appears in Table 3-2. Two studies were performed on lean clay (CL). Tests were also carried out on clay of high plasticity or fat clay (CH), silty sand (SM), and silt (ML). The tests performed on silt (ML) were comparable with the study soils, but not quite similar due to varying curing conditions. Lime addition had a significant increase (140 to 450%) in compressive strength as reported by Parsons (2003). Untreated compressive strength varying between 39-60 psi increased to 145-232 psi. Clay soils had a higher increase in compressive strength on lime addition. In a study by Yusuf et al. (1998), except in one test, compressive strength increased by an average of 50%, which is effective according to ASTM D 4609. The reason for the exception is unknown as there are not enough data. Lime alone can be used to stabilize clay soils according to a study by Beeghly (2003), but there are no sufficient data on untreated soils to compare.

Table 3-2 Summary of UCS Test Results on Lime Treated Soil from Literature

Investigation	Year	Soil Type		Curing Condition	Untreated		Treated	
		USCS	AASHTO		w%	qu (psi)	Admixture	qu (psi)
Parsons	2003	CH	A-7-6	28 days	18.9	60	L-5.5%	145
					18.6	52	L-3.5%	160
					23.5	44	L-6%	188
		CL	A-6		17	52	L-4%	232
					17.6	42	L-2.5%	230
		ML	A-4		13.7	45	L-1.5%	190
		SM	A-2-4		9.9	39	L-1%	145
Yusuf	1998	-	-	7 days@40°C	-	250	L-4%	400
						315	L-5%	480
						380	L-6%	310
						290	L-6%	500
Beeghly	2003	-	-	3 days@50°C	15	-	L-3%	250
				7 days@40°C				308
				28 days@22°C				220
		-	-	3 days@50°C	-	-	L-6%	100
				28 days@22°C				130
				56 days@22°C				160
		CL	-	3 days@50°C	20.6	-	L-7%	190
				7 days@40°C		40		160
				28 days@22°C		35		120
		-	-	56 days@22°C		-		150
		-	-	3 days@49°C	-	-	L-8%	100

3.2.3.2 Fly Ash Treatment

Fly ash in the presence of moisture reacts chemically and forms cemetitious compounds and attributes to the improvement of strength and compressibility characteristics of soils. It has a long history of use as a stabilizing agent for improvement of expansive soils. Fly ash alone was used as the stabilizer in a study by Parsons & Milburn

(2003). Class C fly ash was used and the tests were carried out on seven soils of types CH, CL, ML and SM. The percent of fly ash used was fixed at 16% by dry weight of soils for all the soil types. The unconfined compressive strength test results on only fly ash treated soils from the literature are summarized in Table 3-3. Addition of fly ash alone had a significant increase in compressive strength on different soils, Parsons & Milburn (2003). Compressive strength increased by 95 to 230% for fat clay (CH) varying from 44-60 psi for untreated samples to 117-146 psi for the treated samples. Clay (CL) soils had a higher increase in compressive strength than silt (ML) or silty sand (SM) from an average untreated compressive strength of 47 psi to a treated average of 200 psi. For silt and silty sand soils, compressive strength increased by 160% (45 to 117 psi) and 280% (39 to 148 psi) respectively.

Table 3-3 Summary of UCS Test Results on Fly Ash Treated Soil from Literature

Investigation	Year	Soil Type		Untreated		Treated		
		USCS	AASHTO	w%	qu (psi)	Curing Condition	Admixture	qu (psi)
Parsons	2003	CH	A-7-6	18.9	60	28 days	FA-16%	117
				18.6	52			145
				23.5	44			146
		CL	A-6	17	52		FA-16%	174
				17.6	42			232
		ML	A-4	13.7	45		FA-16%	117
		SM	A-2-4	9.9	39		FA-16%	148

3.2.3.3 Lime-Fly Ash Treatment

Varying percentages of lime and fly ash were used to treat the soils as reviewed in the literature at different curing conditions. A summary of the test results is given in Table 3-4. Treated compressive strength values were in the range of 80 to 522 psi. There was not enough data from the literature under similar conditions to draw conclusions.

Table 3-4 Summary of UCS Test Results on Lime and Fly Ash Treated Soil from Literature

Investigation	Year	Soil Type		Untreated		Treated		
		USCS	AASHTO	w%	qu (psi)	Curing Condition	Admixture	qu (psi)
Hadi Shirazi	1996	-	A-2-4	-	-	7 days	LFA-6% L-2%, FA-4%	80
						28 days		120
						56 days		200
						7 days	LFA-9% L-3%, FA-6%	100
						28 days		150
						56 days		280
Beeghly	2003	-	-	-	-	4 days@49°C	LFA-9% L-3%, FA-6%	330
		-	-	17	-	3 days@50°C	LFA-9% L-3%, FA-6%	190
						7 days@40°C		140
						28 days@22°C		160
						56 days@22°C		190
		-	-	15	-	3 days@50°C		310
						7 days@40°C		290
						28 days@22°C		250
						56 days@22°C		310
		-	-	20	-	3 days@50°C		350
						7 days@40°C		522
						28 days@22°C		310
		-	-	-	-	3 days@50°C	LFA-9% L-3%, FA-6%	120
						28 days@22°C		150
						56 days@22°C		210
						3 days@50°C	LFA-12% L-4%, FA-8%	170
						28 days@22°C		190
						56 days@22°C		250
		-	-	-	-	3 days@50°C	LFA-12% L-4%, FA-8%	220
					40	7 days@40°C		180
					35	28 days@22°C		170
					-	56 days@22°C		200

3.2.3.4 Curing Effect

Most of the published studies used variable curing conditions in terms of number of days the treated soils were subjected to curing. A summary of curing effect on compressive strength of soils from the literature appears in Table 3-5. Based on the test results from Shirazi (1996), compressive strength increased by approximately 50% (80 to 120 psi and

100 to 150 psi) from 7 to 28 days curing time. 28 to 56 days curing time incrementally increased compressive strength by 67 and 87% (120 to 200 psi and 150 to 280 psi).

Table 3-5 Summary of Curing Effect on Compressive Strength from Literature

Investigation	Year	Soil Type		Untreated		Treated		
		USCS	AASHTO	w%	qu (psi)	Admixture	Curing Condition	qu (psi)
Shirazi	1996	-	A-2-4	-	-	LFA-6% L-2%, FA-4%	7 days	80
							28 days	120
							56 days	200
						LFA-9% L-3%, FA-6%	7 days	100
							28 days	150
							56 days	280
Beeghly	2003	CH	A-7-6	15	-	LFA-9% L-3% FA-6%	28 days	250
							56 days	310
				17	-	LFA-9% L-3% FA-6%	28 days	160
							56 days	190
				-	-	LFA-12% L-4% FA-8%	28 days	190
							56 days	250
				-	-	L-6%	28 days	130
							56 days	160
		CL	-	20.6	35	L-7%	28 days	120
					-		56 days	150
					35	LFA-12% L-4% FA-8%	28 days	170
					-		56 days	200
Parsons	2003	CL	A-6	19	33	L-4%	28 days	232
				20	30	L-2.5%		230

Compressive strength of untreated clay (CL) soils increased by 240 to 380% (35 to 120 and 170 psi) on 28 days curing time with lime and/or fly ash treatment and on 28 to 56 days curing time incremental, compressive strength increased by 18% (170 to 200 psi) to 32% (120 to 150 psi) (Beeghly, 2003). In the study by Parsons & Milburn (2003), 28 days curing time increased the untreated compressive strength by 7 times from 30 to 230 psi. Also, pavement performance improved with increasing curing periods.

3.2.4 Effect on Plasticity Index

The effect of lime and/or fly ash treatment on the plasticity index (PI) of the test soils was reviewed in two published studies (Yusuf et al., 1998; Beeghly 2003). The summary of PI test results from literature is given in Table 3-6. Plasticity Index decreased with the treatment of lime. In a study by Yusuf et al. (1998), lime treatment of 4-6% by weight decreased PI by approximately 50%. Lime added to the soils absorbs water causing a reduction in liquid limit. Calcium ions from lime in the presence of water balance the negative charge around the clay particles in the diffused double layer, which reduces plasticity and leads to improved workability of the soils. Addition of fly ash alone resulted in a small measurable effect on the plasticity of the soils. PI remained essentially the same with the addition of up to 20% of fly ash by dry weight. Lime-fly ash treatment decreased the plasticity index of the test soil by approximately 50% (26 to 14 psi).

Table 3-6 Summary of PI Test Results from Literature

Investigation	Year	Soil/Rock Type		Curing Condition	Untreated	Treated		Ratio (Treated/ Untreated)
		USCS	AASHTO		PI	Admixture	PI	
Yusuf	1998	-	-	7 days@40°C	17	L-4%	9	0.5
					29	L-5%	15	0.5
					28	L-6%	9	0.3
					32		16	0.5
Beeghly	2003	CH	A-7-6	-	26	LFA-13% L-3%, FA-10%	14	0.5
						FA-10%	24	0.9
						FA-20%	24	0.9

3.2.5 UCS vs CBR Test Results

The relation between unconfined compressive strength and CBR values were studied based on test results from the literature. Table 3-7 provides a summary of UCS and CBR test results from the literature. There was not enough data to correlate the desired parameters for untreated samples.

Table 3-7 Summary of UCS and CBR Test Results from Literature

Investigation	Year	Soil Type		Curing Condition	Untreated					Treated				
		USCS	AASHTO		w%	qu (psi)		CBR		Admixture	qu (psi)		CBR	
						Uncured	Cured	Unsoaked	Soaked		Uncured	Cured	Unsoaked	Soaked
Puppala	1996	CL	A-7-6	3 days	12	26	-	-	-	L-4%	29	34	17	-
					14	22					26	30	19	
					17	13					18	25	20	
					20	6					9	15	16	
					22	3					6	13	14	
Yusuf^	1998	-	-	7days@40°C	-	250	2	118	6	L-4%	400	280	196	115
						315	5	81	-	L-5%	480	290	196	140
						290	1	66	-	L-6%	310	245	131	67
						380	6	57	-		500	275	108	45
Beeghly*	2003	CH	A-7-6	7days@40°C	15	-	-	-	-	L-3%	-	308	-	124
					17	-	-	-	-	LFA-9% L-3%, FA-6%	-	140	-	21
					20	-	-	-	-	LFA-9% L-3%, FA-6%	-	522	-	278
		CL	-	7days@40°C	21	-	40	-	-	L-7%	-	160	-	62
						-	40	-	-	LFA-12% L-4%, FA-8%	-	180	-	43

^Cured and Soaked Samples Subjected to 24 hr Capillary Soak and CBR Calculated from Corresponding Resilient Modulus Values Using Formula

*Soaked Samples Under Water for 4 Days

Graphs were plotted individually for published studies with sufficient data. Treated and uncured/unsoaked samples did show a marginal relationship between unconfined compressive strength (q_u) and CBR values. Treated samples that were cured and soaked showed a marginal relationship between compressive strength and CBR values in a study by Yusuf et al. (1998) but had a significant correlation in a study by Beeghly (2003). Lower values of q_u and CBR were recorded in a study by Puppala, Mohammad, and Allen (1996).

Linear regression on the data from treated soil samples in the study showed a marginal relation between uncured compressive strength and unsoaked CBR (see Figure 3-1).

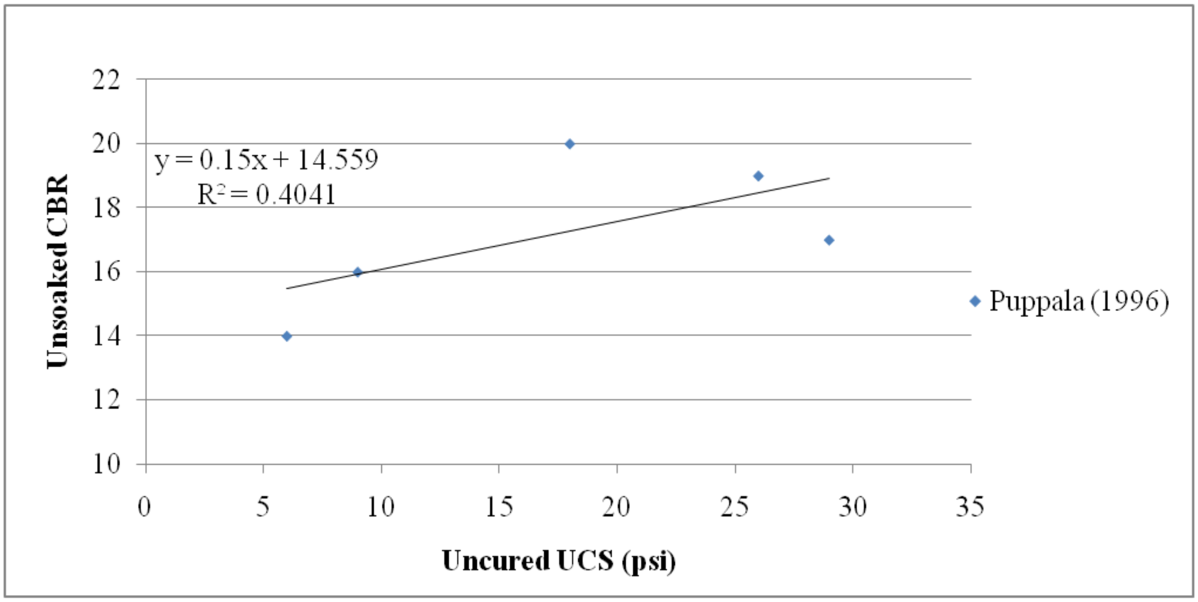


Figure 3-1 Linear Regression on Treated Unsoaked CBR with Respect to Uncured Compressive Strength from Literature (Puppala et al., 1996).

A similar graph plotted using data from study by Yusuf et al. (1998) showed a marginal relation between compressive strength on treated and compacted soils that were cured and soaked CBR (R^2 value about 40%, Figure 3-2). Cured compressive strength was 2 to 6 times more than soaked CBR values at variable percentages of lime treatment. The results are summarized in Table 3-8 and the corresponding linear regression plotted in Figure 3-2.

Table 3-8 Literature Results for Comparison of Treated and Cured Unconfined Compression Strength and Soaked CBR Values

Investigation	Year	Soil Type		Curing Condition	Treated			Ratio*
		USCS	AASHTO		Admixture	Cured UCS (psi)	Soaked CBR	
Yusuf	1998	-	-	7days@40°C	L-4%	280	115	2
					L-5%	290	140	2
					L-6%	245	67	4
					L-6%	275	45	6

*Cured UCS Over Soaked CBR

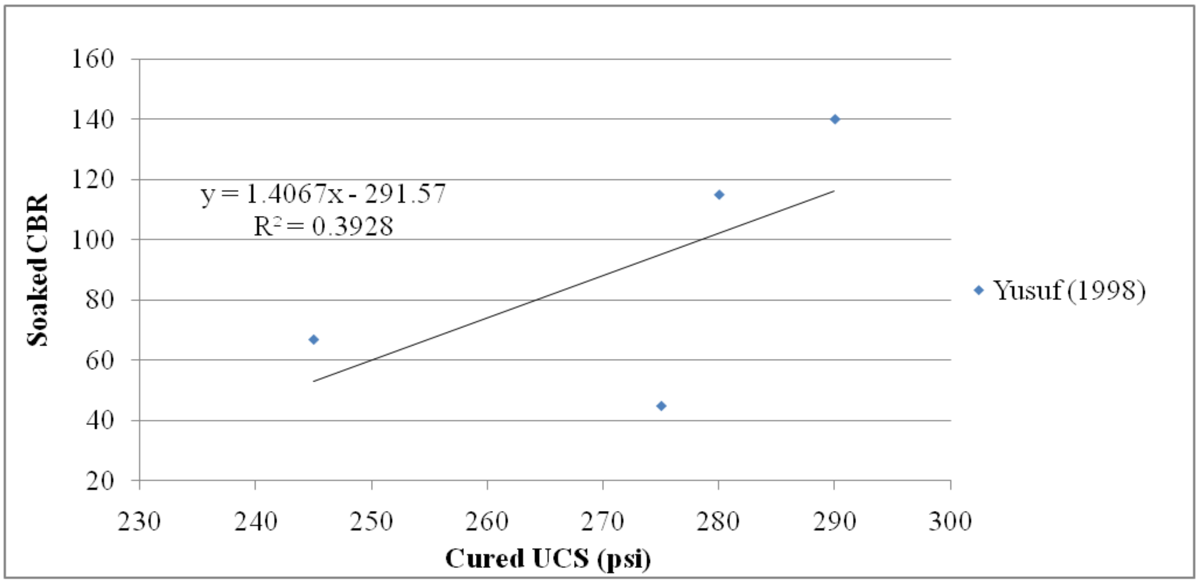


Figure 3-2 Linear Regression on Treated and Soaked CBR with Respect to Cured Compressive Strength from Literature (Yusuf et al., 1998).

Table 3-9 gives a summary of cured unconfined compressive strength and soaked CBR values of treated soils from the literature (Beeghly, 2003) and corresponding ratio of cured UCS over soaked CBR. Compressive strength on curing has a significant correlation with corresponding soaked CBR values. A linear regression conducted on the data gave an R^2 value of 0.98 (Figure 3-3), which represents best fit of the data. For different soil types, cured compressive strength was 2 to 7 times more than soaked CBR values.

Table 3-9 Literature Results for Comparison of Treated and Cured Unconfined Compression Strength and Soaked CBR Values

Investigation	Year	Soil Type		Curing Condition	Treated			Ratio*
		USCS	AASHTO		Admixture	Cured UCS (psi)	Soaked CBR	
Beeghly	2003	CH	A-7-6	7days@40°C	L-3%	308	124	2
					LFA-9% L-3%, FA-6%	140	21	7
					LFA-9% L-3%, FA-6%	522	278	2
		CL	-	7days@40°C	L-7%	160	62	3
					LFA-12% L-4%, FA-8%	180	43	4

*Cured UCS Over Soaked CBR

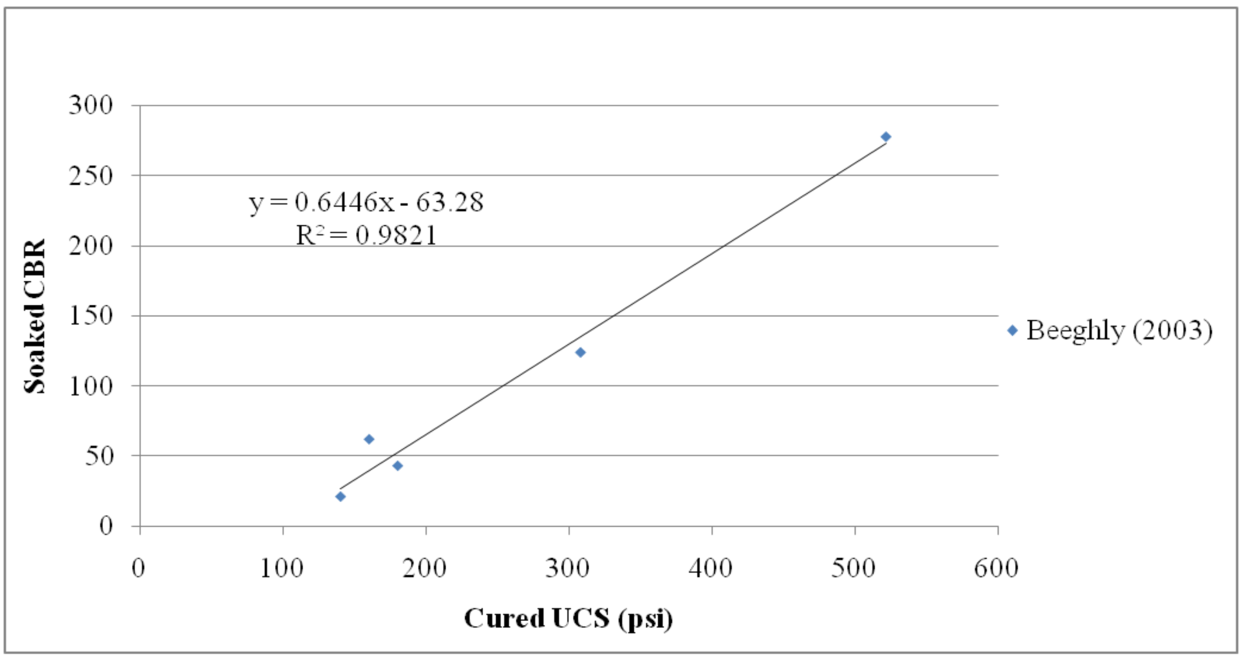


Figure 3-3 Linear Regression on Treated and Soaked CBR with Respect to Cured Compressive Strength from Literature (Beeghly, 2003).

3.2.6 Relationship between UCS and Resilient Modulus

A study by Dallas N. Little suggested that a relationship between unconfined compressive strength (UCS) and resilient modulus can be obtained and graphically represented. Figure 3-4 shows a significant increase in resilient modulus as UCS increases (Little, 1999). This relationship was used in the current research study to estimate the design resilient modulus from measured compressive strength values. The resilient modulus (M_r) values were calculated from the CBR test results in the research study. The calculated M_r values were then used to evaluate them in reference to this relationship in Figure 3-4.

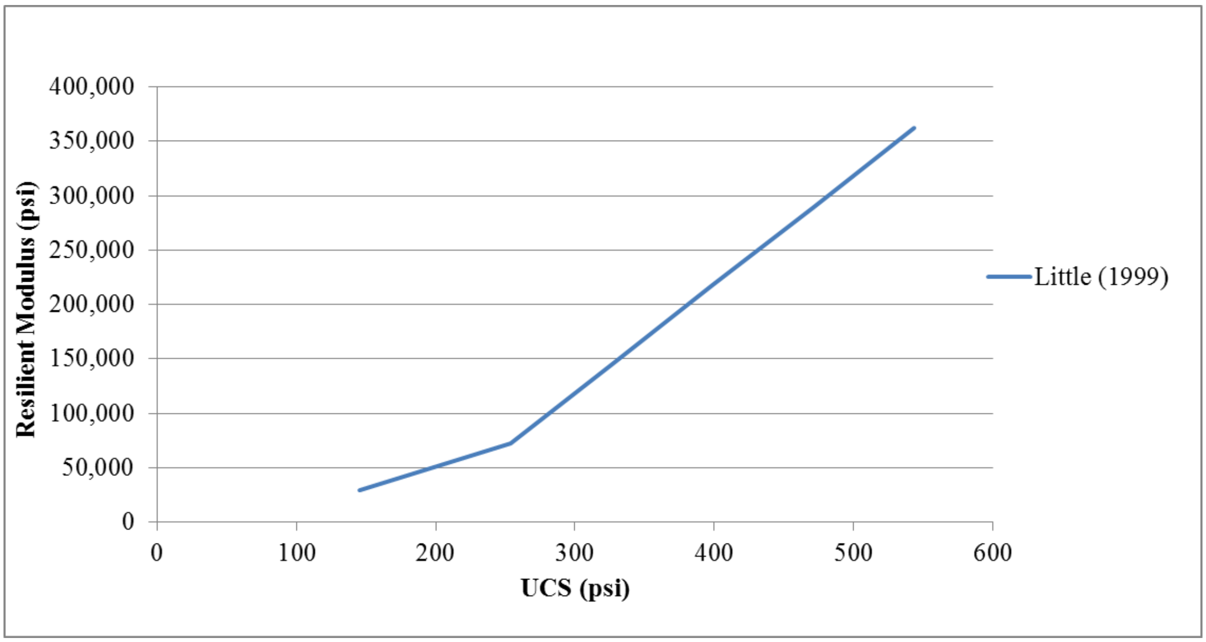


Figure 3-4 Suggested Design Relationship between Unconfined Compressive Strength (UCS) and Resilient Modulus for Lime Stabilized Subgrade Materials (Little, 1999).

Chapter 4 - Test Methodology

4.1 Introduction

In order to determine the effect of lime and/or fly ash treatment on compacted loess and to establish the relationship between California Bearing Ratio and Unconfined Compressive Strength, a series of controlled laboratory tests was performed on the project soils. The intent was to perform the tests in which companion samples were prepared using similar procedures. A summary of the companion CBR and UCS test parameters appears in Table 4-1.

Table 4-1 Summary of Parameters in UCS and CBR Tests

CBR Tests	UCS Tests
Untreated/Unsoaked	Untreated/Uncured
Untreated/Soaked	Untreated/Moist Cured
Treated/Unsoaked	Treated/Uncured
Treated/Soaked	Treated/Moist Cured

4.2 Sample Location

The project site is located in southwest Pocatello, Idaho (see Figure 4-1). The soil samples were taken from an open field south of the Riverside Golf Course and adjacent to the Portneuf River. The soils at the project site are windblown and reworked silt and fine sand deposits made by wind storms and bank overflow during flood stages of the Portneuf River.

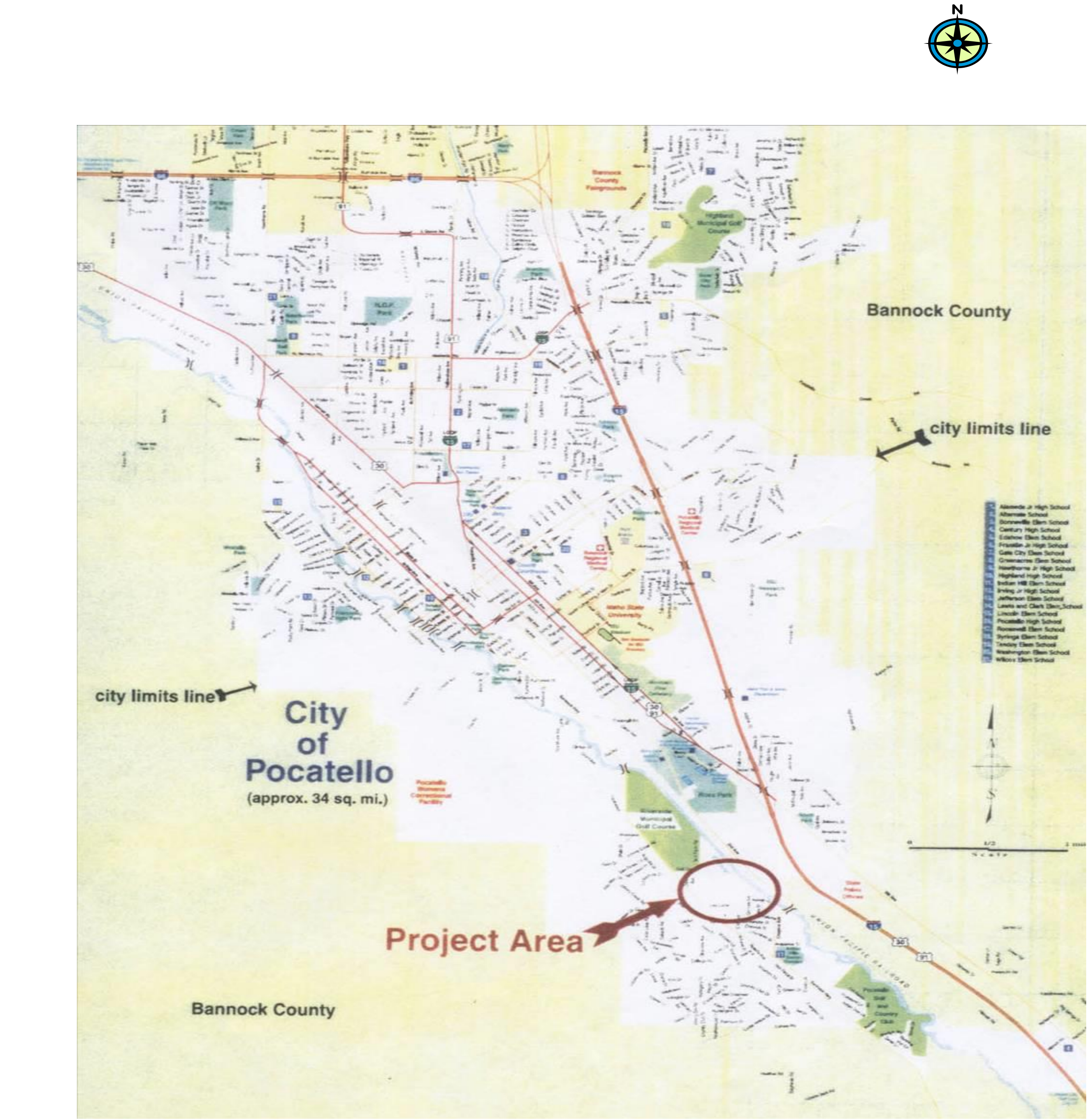


Figure 4-1 Project Site Location Map (Not to Scale)

Courtesy of City of Pocatello

4.3 Sample Preparation

Samples collected from the project site were sealed in air tight buckets and brought to the Materials Laboratory at ISU. There were two or three steps involved prior to the sample preparation to maintain uniformity among the soil samples before performing the engineering property tests. The sample preparation steps involved mechanical disaggregation of the soil, addition of water to reach approximate optimum moisture content, and in some tests, addition of lime and/or fly ash.

4.3.1 Mechanical Disaggregation

The samples collected from the project site had a maximum particle size of approximately two to three inches. The soil was placed on a plastic sheet and mechanically disaggregated to a maximum particle size of about ¼ to ½ inch. The mechanical disaggregation was performed using a shovel (see Figure 4-2). This disaggregation process was not intended to break down individual soil grains, but rather to ensure proper size for uniform blending and absorption of lime and/or fly ash. After disaggregation, the soil was stored in air tight containers. All field samples were prepared in the same manner.

Figure 4-2 Disaggregation Process of Project Soil

4.3.2 Addition of Water to Reach Optimum Moisture Content

The initial moisture content of native soil was approximately 11%. Water was added to obtain an optimum moisture content of 19.5%. The optimum moisture content was determined using Standard Proctor compaction effort and the work was performed as part of a previous ISU-ITD contract. The amount of additional water required to reach optimum was calculated, then measured out and added to the batch. Figure 4-3 shows the blender used to obtain uniform moisture throughout the samples. The soil and water were mixed for 2 to 4 minutes. The blended sample was then allowed to equilibrate for 1 to 2 hours in air tight containers to enhance uniform moisture distribution. The project soils were then compacted using the Standard Proctor method.

Figure 4-3 Mixer Used to Blend the Soil and Water at Optimum Water Content

4.3.3 Addition of Lime and/or Fly Ash

Lime and/or fly ash were added in the test proportions and combinations discussed in Chapter 2. The required amount of lime and /or fly ash was proportioned by dry unit weight of soil and was added to the samples at optimum water content. The lime and/or fly ash were thoroughly mixed with the soil by hand in a plastic container for about 5 minutes until the color was uniform.

4.4 Testing Methods and Procedures

All tests were performed in general accordance with ASTM standards. A summary of the test methods/procedures used in this research study as well as the corresponding ASTM designations and modifications made in some procedures are summarized in Table 4-2.

Table 4-2 Summary of Test Methods/Procedures Used in Research Study

ASTM	Test Method	Standard	Modifications
C 136	Sieve Analysis of Fine and Coarse Aggregates	-	-
D 422	Particle-Size Analysis of Soils	-	-
D 558	Moisture-Density Relations of Soil-Cement Mixtures	-	-
D 698	Laboratory Compaction (Standard Proctor)	-	-
D 1632	Practice for Making and Curing Soil-Cement Compression	• Height to Diameter Ratio=2 • Curing Time:12 hours	• Height to Diameter Ratio < 2 • Curing Time : 96 hours
D 1633	Compressive Strength of Molded Soil-Cement Cylinders	• Immerse Specimens in Water for 4 hours After Curing Period • Applicable to Other Sizes	• Not Immersed in Water After Curing Period • Height to Diameter Ratio < 2
D 1883	California Bearing Ratio (CBR)	-	-
D 2216	Water Content of Soil by Mass	-	-
D 2487	Unified Soil Classification System (USCS)	-	-
D 3551	Preparation of Soil-Lime Mixtures Using a Mechanical Mixer	• Mechanical Mixer • Typical Mellowing Period of 1 hr	• Hand Mixed • Mellowing Time ≈ 10 minutes
D 4318	Liquid Limit, Plastic Limit, and Plasticity Index	-	-
D 4609	Evaluating Effectiveness of Admixtures for Soil Stabilization	• Other Curing Conditions Applicable	• Bucket with Air Tight Lid Used for Moist Curing
D 4972	Soil pH	-	-
D 5102	Unconfined Compressive Strength of Compacted Soil-Lime Mixtures	-	-
D 5239	Characterizing Fly Ash for Use in Soil Stabilization	-	-

Both treated and untreated soils were cured for a period of 96 hours in the compressive strength tests to match the soaking period in the CBR tests. The height of the compressive strength test samples had to be reduced because of the tendency of the specimens to develop fissures during the curing process. The following ASTM correction factor was applied to the test values to account for the length to diameter (L/D) ratio less than 2.

$$\text{Corrected } q_u = \frac{\text{uncorrected qu}}{(0.778+0.222*{}^{D}/_{H})}$$

4.5 Testing Program

The companion California Bearing Ratio (CBR) Test and Unconfined Compression Strength (UCS) tests were performed in the Soil Mechanics Laboratory at Idaho State University. The test apparatus are shown in Figure 4-4.

Figure 4-4 Testing Machines and Computer Used for CBR and UCS Tests

4.5.1 CBR Test

The California Bearing Ratio (CBR) test was performed following the standards specified in ASTM D 1883. Both soaked and unsoaked CBR tests were carried out on untreated and treated soil samples in this investigation. In the soaked CBR tests, the samples were immersed in potable water for 96 hours.

4.5.1.1 Untreated/Unsoaked CBR

The amount of soil required for each CBR test was about 4000 grams. The soil was compacted in a 6-inch diameter by 4-inch high Proctor mold (see Figure 4-5). Procedures specified in ASTM D 1883 were followed. The soil was compacted in 3 layers with 56 blows per layer using a 5.5 lb. rammer falling 12 inches. After compaction, the top of the compacted soil was trimmed flush with top of the mold. The compaction time was 20 minutes and the surface was trimmed in about 15 minutes.

Figure 4-5 Standard Proctor Compaction of Treated Project Soil for CBR Test

The unsoaked CBR tests were initiated roughly 15 minutes after compaction. The unsoaked tests were performed on the compacted soil on the underside (spacer disc side) of the specimen in the mold. The Durham Geo Slope Indicator Machine was used to perform the CBR tests. A 10 lb. surcharge load was placed on top of the sample to provide some confinement as recommended in the ASTM standard (see Figure 4-6). The load application to bearing failure was approximately 5 minutes and real-time load-deflection data were recorded on the computer. Once the test was complete, both uncorrected and corrected CBR values were automatically displayed on the screen. Modulus of Subgrade

Reaction (K_v) values were calculated using the CBR data. A total of 3 hours were needed to complete one CBR test (Table 4-3)

Figure 4-6 CBR Test Set-Up

Table 4-3 Typical Times Taken to Complete an Unsoaked CBR Test

Moisture Conditioning	Compaction Time	Trim Soil Surface to Test	Test Time
1 to 2 hrs	20 min	15 min	5 min

4.5.1.2 Untreated/Soaked CBR

The soaked CBR tests were performed as per ASTM D1883. The soil in the mold from the previous unsoaked test was inverted and the spacer disc replaced inside the mold above the base plate. A 10-lb surcharge load was placed on the sample surface. The mold with the soil was immersed in water for 96 hours. The change in height, either swell or shrinkage, was measured using a 0.001 in. dial gauge fixed outside the mold and bearing on a perforated disc placed on the surface of the soil specimen (see Figure 4-7).

Figure 4-7 Soaked CBR Test with Dial Gauge Measurements

After completion of the 96-hour soaking period, the sample was removed from the water bath and the excess water drained from the soil surface. The 10-lb surcharge was replaced on the soil and the test was performed per ASTM D 1883. The typical times to complete a soaked CBR test appear in Table 4-4.

Table 4-4 Typical Times Taken to Complete Soaked CBR Tests

Dial Gauge Setup	Soaking Period	Prepare Sample Ready for Testing	Test Time
10 min	96 hrs	5 min	5 min

4.5.1.3 Treated/Unsoaked CBR

Lime and/or fly ash were added to the soil samples at optimum moisture content in the required test proportion of 1:3 as discussed in Chapter 2. The admixtures were thoroughly mixed with the soil by hand for about 5 minutes until the color was uniform. Soil compaction started within 5 minutes after lime and/or fly ash addition. Soil compaction and sample testing procedures were similar to those discussed in section 4.5.1.1. A summary of the typical times taken to complete a treated/unsoaked CBR appears in Table 4-5.

Table 4-5 Typical Times Taken to Complete Treated/Unsoaked CBR Tests

Moisture Conditioning	Lime and/or Fly Ash Addition	Compaction Time	Trim Soil Surface to Test	Test Time
1 to 2 hrs	5 min	20 min	15 min	5 min

4.5.1.4 Treated/Soaked CBR

The treated soil samples were soaked in water for 96 hours. Soaking set-up and testing procedures were similar to those of untreated samples discussed in section 4.5.1.2.

4.5.2 General Unconfined Compression Strength (UCS) Procedures

The soil preparation for the unconfined compression tests was similar to that for the CBR test. The native soil was brought to optimum moisture content by adding the required amount of water. The soil and additional water were blended and allowed to equilibrate for 1 to 2 hours in an air-tight container. In the treated soil tests, lime and/or fly ash were then added in required proportions and thoroughly mixed by hand for 5 minutes. Hand and eye protection were worn by the research personnel while mixing the treated soil.

4.5.2.1 UCS Tests on Untreated and Treated Soils

Four different types of unconfined compressive strength (UCS) tests were performed on the project soils to correspond with companion CBR test parameters (see Table 4-1).

- Untreated/Uncured
- Untreated/Moist Cured
- Treated/Uncured
- Treated/Moist Cured

4.5.2.2 Untreated/Uncured

The sample size required for an unconfined compression test was around 400 grams. The moisture content of the soil was increased to optimum, which was done in all of the tests. Moisture conditioning took 1 to 2 hours to allow uniform distribution of the additional water to reach the optimum moisture content. Compaction of the soil specimen was done using Harvard Miniature Compaction Equipment as shown in Figure 4-8.

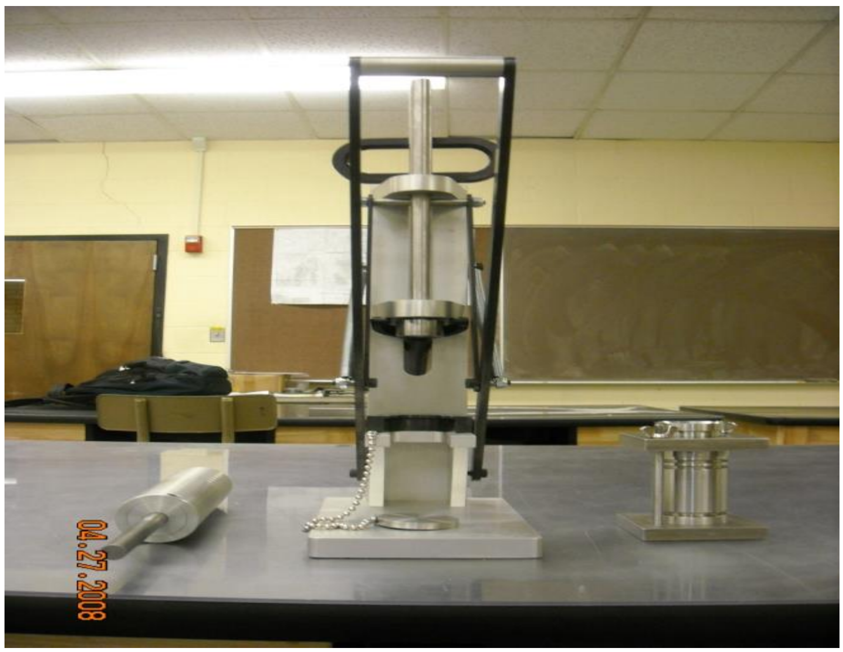


Figure 4-8 Harvard Miniature Compaction Equipment

Sample compaction was carried out following the procedure in the manufacturer's instruction manual, H-4165. The soil was compacted in 5 layers and each layer was compacted 10 times using a compaction tamper (see Figure 4-9).

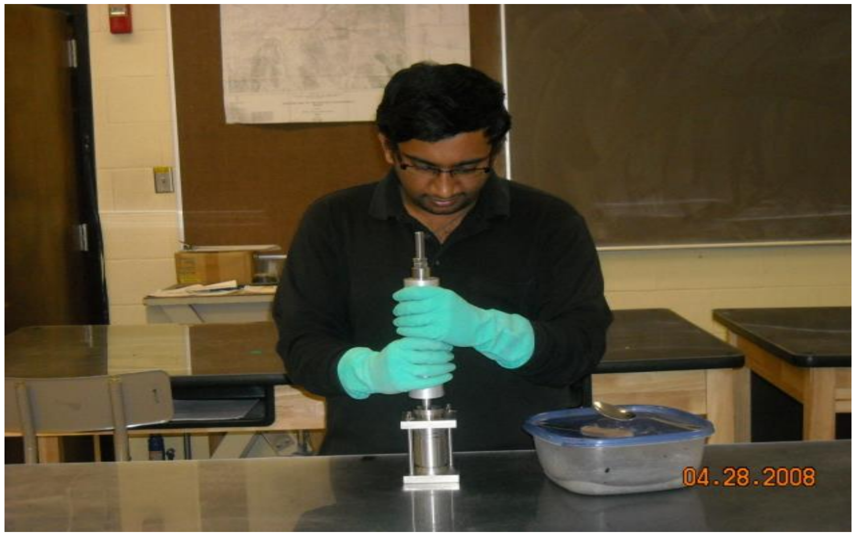


Figure 4-9 UCS Sample Specimen Compaction Using Harvard Miniature Compactor

After compaction, the soil specimen was removed using the ejector equipment and the end surfaces were smoothed and made parallel. Compaction time was 25 minutes and trimming the surface took 5 minutes to complete. The unconfined compressive strength tests were performed using a Durham Geo Slope Indicator Machine (Figure 4-10). Test time was 5 minutes. Typical times to complete an unconfined compressive strength test are summarized in Table 4-6.

Figure 4-10 Geo Slope Indicator Machine

A real-time plot of load versus displacement was displayed on the computer (Figure 4-11). Values of unconfined compressive strength (q_u) and the undrained shear strength (s_u) were calculated from the test data.

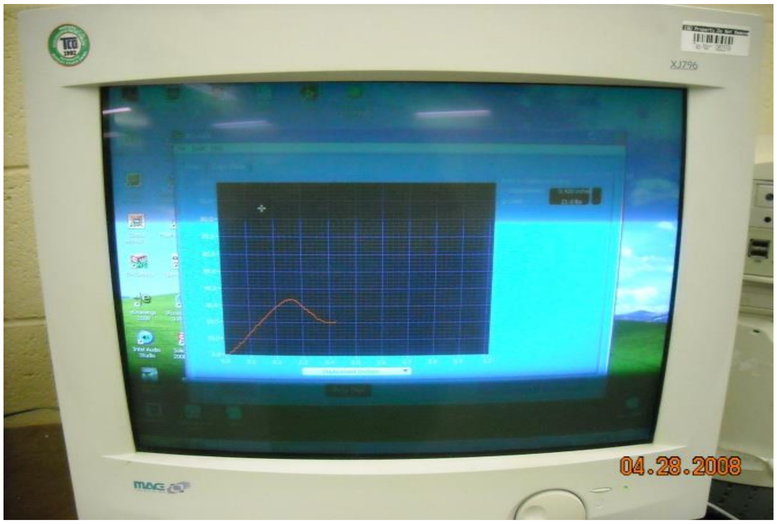


Figure 4-11 Live Data Curve for UCS Test

Table 4-6 Typical Times Taken to Complete Untreated/Uncured UCS Tests

Moisture Conditioning	Compaction Time	Trim Soil Surface to Test	Test Time
1 to 2 hrs	25 min	5 min	5 min

4.5.2.3 Untreated/Moist Cured

The sample preparation for the untreated and moist cured soils was similar to that of uncured specimens. For moist curing, the samples were placed in a tub on an elevated plywood sheet. Water was filled to the base of the plywood and the tub was covered with a lid. Figure 4-12 shows the set-up for moist curing the soil samples. All of the test samples were cured for 96 hours to correspond with the soaking period in the CBR test.

Figure 4-12 Procedure Set-Up Used for Moisture-Cured Samples

4.5.2.4 Treated/Uncured

Prior to the addition of lime and/or fly ash, water was added to the soil to reach optimum moisture content. Lime and/or fly ash were then added and thoroughly mixed by hand for about 5 minutes. Soil compaction and sample testing procedures were similar to those of the untreated/uncured UCS parameter discussed in section 3.5.2.2. A summary of the typical times taken to complete treated/uncured UCS tests appears in Table 4-7.

Table 4-7 Typical Treated UCS Sample Preparation and Test Times

Moisture Conditioning	Lime and/or Fly Ash Addition	Compaction Time	Trim Soil Surface to Test	Test Time
1 to 2 hrs	5 min	25 min	5 min	5 min

4.5.2.5 Treated/Moist Cured

The treated soil samples were soaked in water for 96 hours. Soaking set-up and testing procedures were similar to those of untreated samples discussed in section 4.5.2.3.

Chapter 5 - Test Results and Analysis

5.1 Introduction

Experimental test results and analysis are presented in this chapter with emphasis on the unconfined compressive strength of the project soil and the changes observed by the addition of lime and/or fly ash. The research project was completed in two stages, during which the first stage dealt with the classification of the project soil in accordance with AASHTO, USCS, and CBR tests to evaluate the bearing capacity of the soil with and without addition of lime and/or fly ash. This stage of the research dealt with the effect of added lime and/or fly ash on the unconfined compressive strength of the project soil.

5.2 UCS Test Results on Untreated Soil

The untreated test results serve as a base to measure the effect of added lime and/or fly ash to the project soil. Three samples each were tested for uncured and moist cured condition on untreated soil. The average of the three test values was taken to best represent the results. Table 5-1 provides the summary of unconfined compressive strength test results on untreated soil from research study.

Table 5-1 Summary of Unconfined Compressive Strength Test Results on Untreated Soil

Investigation	Soil Type		Untreated Unconfined Compressive Strength			
	USCS	AASHTO	Uncured (psi)		Moist Cured (psi)	
			Individual	Average	Individual	Average
Research Study	ML	A-6	52	54	85	87
			54		87	
			55		90	

The average untreated and uncured UCS was 54 psi. In moist cured condition, untreated unconfined compression strength increased to 87 psi. The increase in the unconfined compressive strength could be related to the re-cementing of soil with the existing calcium carbonates.

5.3 UCS Test Results on Treated Soil

For the treatment of project soil, the additives were mixed in various proportions and different combinations of Quicklime, Hydrated Lime, Fly ash Class C and Fly ash Class F. Table 5-2 summarizes the combinations and percentages of admixtures used in research study.

Table 5-2 Summary of Combinations and Percentages of Admixtures Used in Research Study

No.	Combinations	% by Dry Weight of Solids
1	Quicklime Only	1%
2	Hydrated Lime Only	1%
3	Fly Ash Class F Only	3%
4	Fly Ash Class C Only	3%
5	Quicklime + Fly Ash Class F	2%, 4%, 6%, 8%, 10%, 12%
6	Quicklime + Fly Ash Class C	4%
7	Hydrated Lime + Fly Ash Class F	4%
8	Hydrated Lime + Fly Ash Class C	4%

As noted in the table, quicklime and fly ash class F with percentages varying from 2 to 12% by dry weight of solids were used for most of the research study. Quicklime hydrates with the moisture in the soil to become hydrated lime which is a better drying agent. Class F fly ash is economical and gives a long term performance (Beeghly, 2003).

Combinations of quicklime, hydrated lime, and fly ash class F and class C were also tested to evaluate their performance in combinations and a 4% by dry weight of solids. Lime and fly ash were mixed in the proportion of 1:3, which means 1 part of lime and 3 parts of fly ash were used for all the combinations. Also, the admixtures were tested individually to evaluate their impact on the stabilization of the soil as follows -

- Quicklime or Hydrated Lime – 1% by dry weight of solids
- Fly Ash classes F and C – 3% by dry weight of solids

This was done to be compatible with the lime-fly ash proportion of 1:3 used for all the combinations of lime and fly ash.

5.3.1 Only Lime Treatment

Compressive strength on lime treatment alone had a measurable increase for uncured samples and a significant increase for moist cures samples. Table 5-3 summarizes the unconfined compressive strength test results on only lime treated soil from the research study.

Table 5-3 Summary of Unconfined Compressive Strength Test Results on Only Lime Treated Soil

Investigation	Soil Type		Untreated		Treated				
	USCS	AASHTO	Uncured q_u (psi)	Moist Cured q_u (psi)	Admixture*	Uncured q_u (psi)		Moist Cured q_u (psi)	
						Individual	Average	Individual	Average
Research Study	ML	A-6	54	87	QL-1%	59	60	116	136
						60		144	
						62		148	
					HL-1%	76	80	135	149
						77		148	
						87		164	

* % by Dry Weight of Solids

From the table, when treated with lime alone, there was a measurable increase in compressive strength. Quicklime alone had a minimal increase in compressive strength from 54 to 60 psi. Hydrated lime 1% by dry weight of solids increased the uncured compressive strength from 54 to 80 psi. For moist cured samples, there was a significant increase in compressive strength on lime addition. Quicklime addition increased the cured compressive strength by 56% from 87 psi to 136 psi. Hydrated lime addition increased the cured compressive strength by 71% from 87 psi to 149 psi. Hydrated lime performed better for both uncured and moist cured samples. Curing had a significant effect on the compressive strength which will be discussed in later section (please see Table 5-8).

5.3.2 Only Fly Ash Treatment

The performance of fly ash alone to stabilize the soil was not significant compared to the addition of lime alone. Table 5-4 summarizes the unconfined compressive strength test results on only fly ash treated soil from the research study.

Table 5-4 Summary of Unconfined Compressive Strength Test Results on Only Fly Ash Treated Soil

Investigation	Soil Type		Untreated		Treated				
	USCS	AASHTO	Uncured q_u (psi)	Moist Cured q_u (psi)	Admixture*	Uncured q_u (psi)		Moist Cured q_u (psi)	
						Individual	Average	Individual	Average
Research Study	ML	A-6	54	87	Fly Ash Class F-3%	55	56	90	117
						57		99	
						57		163	
					Fly Ash Class C-3%	63	68	104	123
						69		113	
						71		152	

* % by Dry Weight of Solids

Addition of fly ash alone had a minimal increase in compressive strength for uncured samples. Compressive strength was 56 psi for added fly ash class F, which does not show any impact. Added fly ash class C had a minimal increase in compressive strength from 54 psi to 68 psi. Curing had a measurable increase in compressive strength on fly ash addition. Fly ash classes F and C increased the compressive strength measurably by 34 and 41 % respectively, which was from 87 psi to 117 and 123 psi. In short, the performance of fly ash alone to stabilize the soil was not significant compared to the addition of lime alone.

5.3.3 Quicklime and Fly Ash Class F Treatment

Table 5-5 Summary of Unconfined Compressive Strength Test Results on Quicklime and Fly Ash Class F Treated Soil

Investigation	Soil Type		Untreated		Treated				
	USCS	AASHTO	Uncured q_u (psi)	Moist Cured q_u (psi)	QL+FA Class F*	Uncured q_u (psi)		Moist Cured q_u (psi)	
						Individual	Average	Individual	Average
Research Study	ML	A-6	54	87	2%	80	85	133	145
						80		146	
						95		157	
	ML	A-4			4%	73	80	120	153
						75		155	
						93		184	
					6%	75	92	98	147
						94		150	
						107		192	
					8%	78	90	174	176
						84		177	
						108		-	
	-	-			10%	84	87	208	262
						85		316	
						92		-	
	-	-			12%	55	77	225	259
						65		294	
						112		-	

* % by Dry Weight of Solids

As noted in the table, lime-fly ash added in varying percentages, increased the untreated compressive strength by 57% from 54 psi to an average of 85 psi. With increased lime-fly ash, there was no significant increase in uncured and treated compressive strength. Curing had a significant impact on moist cured compressive strength values, which is discussed in section 5.4 (Table 5-7).

The pattern of uncured and moist cured compressive strength values appear graphically in Figure 5-1.

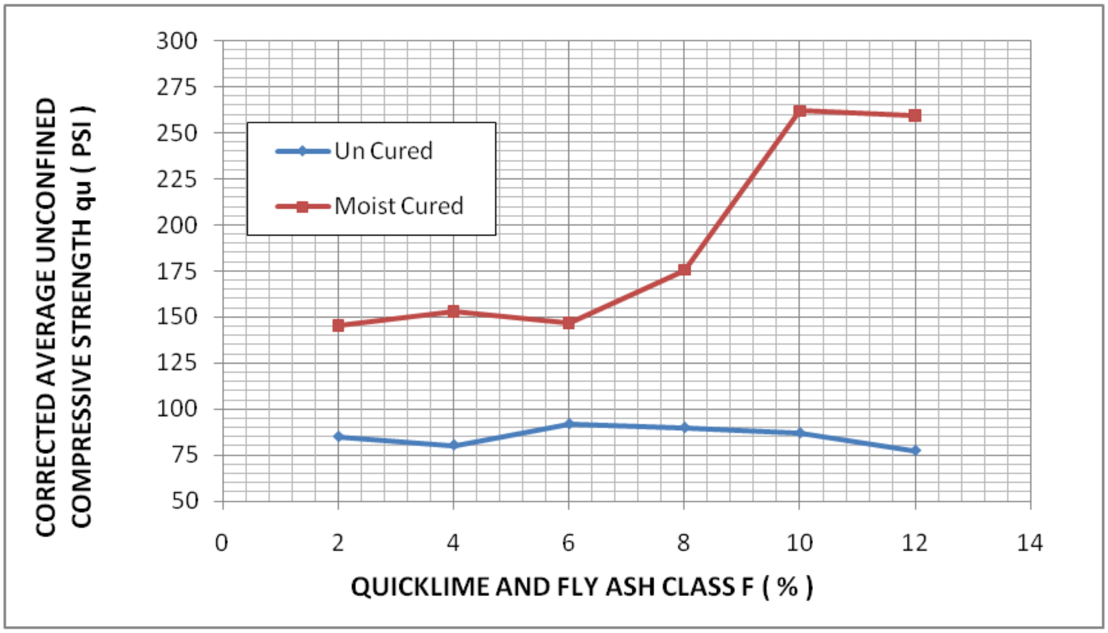


Figure 5-1 Pattern of Uncured and Moist Cured Average Unconfined Compressive Strength vs Percentages of QuickLime and Fly Ash Class F

As noted in the graph, uncured compressive strength values varied slightly with an increase in admixture percentage. The moist cured compressive strength values increased significantly. Based on ASTM D 4609, increase in unconfined compressive strength of 50 psi or more is considered an effective treatment result.

5.3.4 Combinations of Lime and Fly Ash Treatment

The project soil was treated with different combinations of lime and fly ash and all the combinations performed better than the untreated soil specimens. Table 5-6 summarizes the test results of all combinations of lime and fly ash used in the research study.

Table 5-6 Summary of Unconfined Compressive Strength Test Results on Combinations of Lime and Fly Ash

Investigation	Soil Type		Untreated		Treated				
	USCS	AASHTO	Uncured q_u (psi)	Moist Cured q_u (psi)	Admixture*	Uncured q_u (psi)		Moist Cured q_u (psi)	
						Individual	Average	Individual	Average
Research Study	ML	A-6	54	87	QL+FA Class F	73	80	120	153
						75		155	
						93		184	
					QL+FA Class C	86	91	186	213
						87		188	
						99		266	
					HL+FA Class F	89	94	214	239
						96		243	
						96		259	
					HL+FA Class C	92	104	232	264
						102		240	
						119		319	

* 4% by Dry Weight of Solids

As noted in the table, all the combinations performed better than the untreated soils. Curing had a significant increase in compressive srength values for all the combinations as discussed in Table 5-8. Results for all the combinations appear graphically in Figures 5-2 to 5-4.

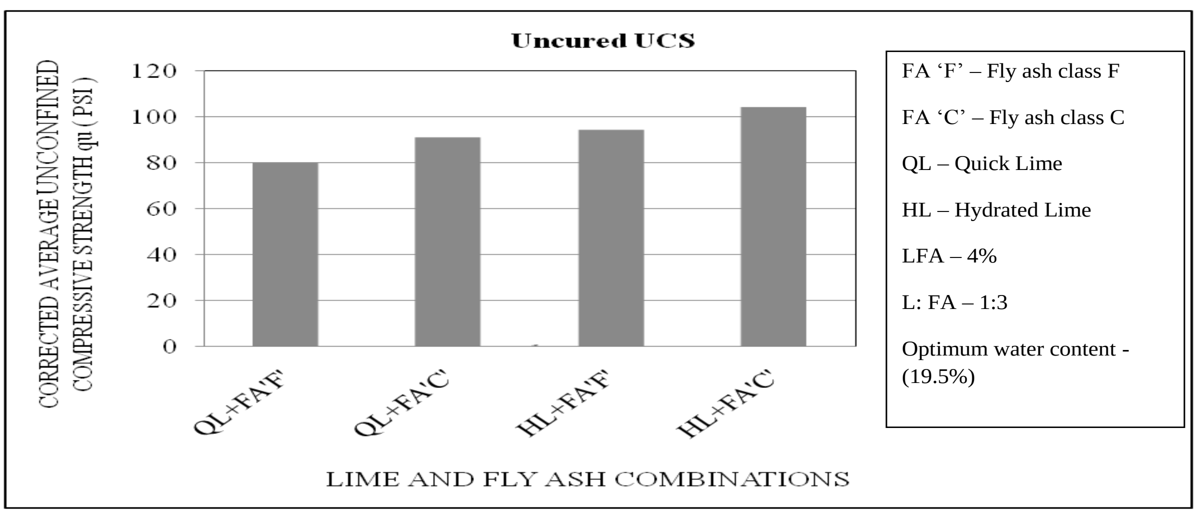


Figure 5-2 Uncured Unconfined Compressive Strength Test Results

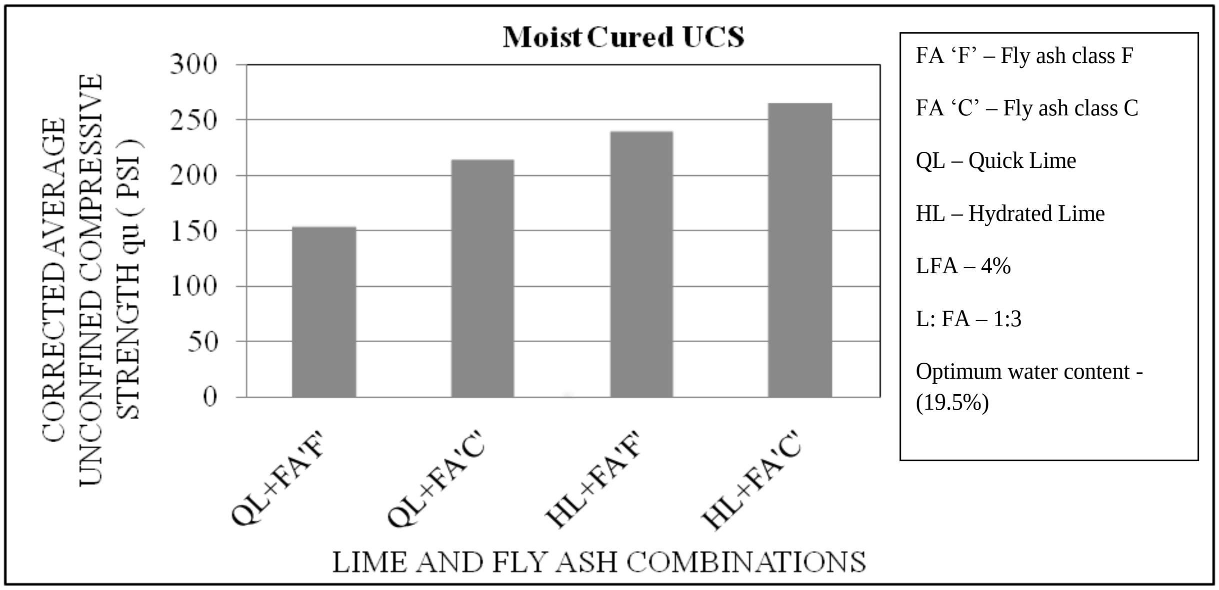


Figure 5-3 Moist Cured Compressive Strength Test Results

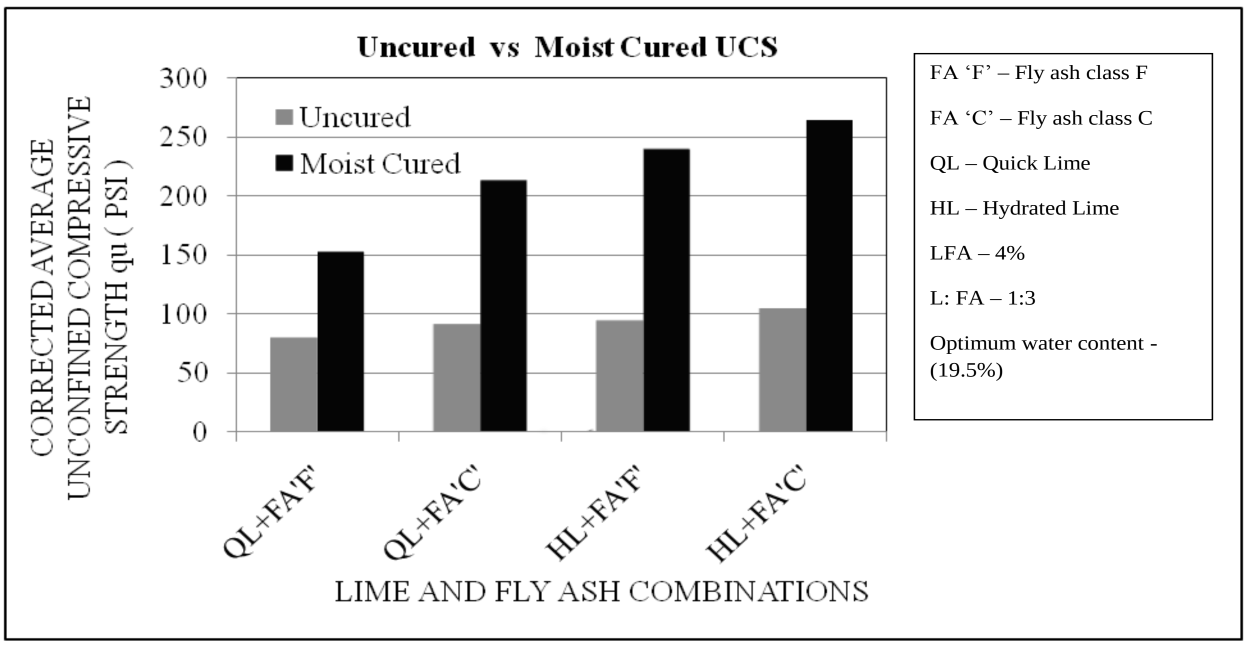


Figure 5-4 Uncured vs Moist Cured Unconfined Compressive Strength Test Results

5.4 Curing Effect on Untreated and Treated Soil

Curing had a measurable effect on untreated soils and a significant increase in the unconfined compressive strength for treated soils. Table 5-7 summarizes the curing effect on compressive strength from the research study.

Table 5-7 Summary of Curing Effect on Compressive Strength

<table>
<tr><th rowspan="3">Investigation</th><th colspan="2">Soil Type</th><th rowspan="3">Curing Condition</th><th colspan="3">Untreated</th><th colspan="2">Soil Type</th><th colspan="3">Treated</th></tr>
<tr><th rowspan="2">USCS</th><th rowspan="2">AASHTO</th><th rowspan="2">w%</th><th colspan="2">q_u (psi)</th><th rowspan="2">USCS</th><th rowspan="2">AASHTO</th><th rowspan="2">Admixture* (%)</th><th colspan="2">q_u (psi)</th></tr>
<tr><th>Uncured</th><th>Cured</th><th>Uncured</th><th>Cured</th></tr>
<tr><td rowspan="6">Research Study</td><td rowspan="6">ML</td><td rowspan="6">A-6</td><td rowspan="6">Moist Cured for 4 Days</td><td rowspan="6">19.5</td><td rowspan="6">54</td><td rowspan="6">87</td><td>ML</td><td>A-6</td><td>2</td><td>85</td><td>145</td></tr>
<tr><td rowspan="3">ML</td><td rowspan="3">A-4</td><td>4</td><td>80</td><td>153</td></tr>
<tr><td>6</td><td>92</td><td>147</td></tr>
<tr><td>8</td><td>90</td><td>176</td></tr>
<tr><td rowspan="2">-</td><td rowspan="2">-</td><td>10</td><td>87</td><td>262</td></tr>
<tr><td>12</td><td>77</td><td>259</td></tr>
<tr><td colspan="12">*Quicklime and Class F Fly Ash</td></tr>
</table>

For untreated samples, compressive strength increased by 61% on moist curing over 4 days. Calcium carbonate in the native materials accounts for the hydration and re-cementing of soil particles. Compressive strength increased by 60-95% for samples treated with 2-8% lime-fly ash and moist cured for 4 days. For 10 and 12% treatment with lime-fly ash, compressive strength increased by 200% and 236% respectively. Higher percentages of lime-fly ash had higher corresponding values of compressive strength on curing.

5.5 Effect of Lime and/or Fly Ash on PI

The untreated project soil was classified as ML or A-7-6 soil with plasticity index (PI) of 13 based on tests performed earlier. Table 5-8 summarizes the PI test results from the research study with and without the addition of lime and fly ash.

Table 5-8 Summary of PI Test Results

Investigation	Soil Type		Untreated	Soil Type		Treated		Ratio (Treated/ Untreated)
	USCS	AASHTO	PI	USCS	AASHTO	Admixture	PI	
Research Study	ML	A-6	13	ML	A-6	LFA-2%	12	0.9
					A-4	LFA-4%	10	0.8
						LFA-6%	7	0.5
						LFA-8%	7	0.5

As noted in the table above, as the lime-fly ash percentage increases, PI decreases. Addition of lime-fly ash 4% by dry weight of soils decreases PI slightly by 23% (13 to 10 psi). LFA treatment of 6 and 8% by dry weight of soils decreases PI by about 50% (13 to 7 psi). In summary, a lime-fly ash treatment greater than or equal to 6% produces a significant reduction in PI.

5.6 Unconfined Compressive Strength (UCS) vs. CBR Test Results

Table 5-9 summarizes the unconfined compressive strength and CBR test results on both untreated and treated soil from the research study.

Table 5-9 Summary of Unconfined Compressive Strength and CBR Test Results on Untreated and Treated Soil

Investigation	Soil Type		Untreated				Soil Type		Treated				
	USCS	AASHTO	Uncured UCS (psi)	Unsoaked CBR	Moist Cured UCS (psi)	Soaked CBR	USCS	AASHTO	Admixture*	Uncured UCS (psi)	Unsoaked CBR	Moist Cured UCS (psi)	Soaked CBR
Research Study	ML	A-6	54	3	87	2	ML	A-6	LFA-2%	85	16	145	28
							ML	A-4	LFA-4%	80	26	153	56
									LFA-6%	92	28	147	86
									LFA-8%	90	30	176	91
							-	-	LFA-10%	87	31	262	96
									LFA-12%	77	46	259	173

*Quicklime and Class F Fly Ash

Table 5-10 summarizes the treated UCS and CBR test results from the research study with the corresponding ratio of moist cured UCS over soaked CBR values.

Table 5-10 Summary of Treated UCS and CBR Test Results

Investigation	Soil Type		Treated			Ratio*
	USCS	AASHTO	Admixture	Moist Cured UCS (psi)	Soaked CBR	
Research Study	ML	A-6	LFA-2%	145	28	5
	ML	A-4	LFA-4%	153	56	3
			LFA-6%	147	86	2
			LFA-8%	176	91	2
	-	-	LFA-10%	262	96	3
			LFA-12%	259	173	1.5

*Moist Cured UCS Over Soaked CBR

A linear regression on soaked CBR and moist cured compressive strength test results from Table 5-10 can be graphically shown in Figure 5-5.

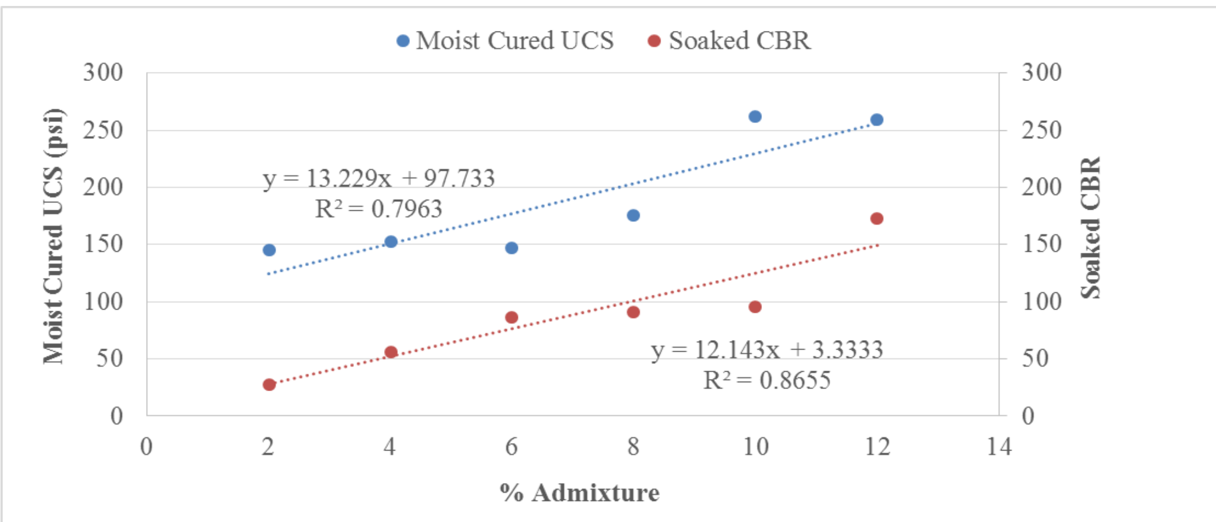


Figure 5-5 Linear Regression on Soaked CBR and Moist Cured Compressive Strength

From the graph, an adjusted R^2 value of about 70% represents a reasonable relation between moist cured compressive strength and soaked CBR values. The curing condition is not the same in compressive strength and CBR tests. Samples for both tests were allowed access to water. Compressive test samples were moist cured in an enclosed bucket for 96 hours while CBR test samples were soaked under water for 96 hours. Moist cured compressive strength was 1.5 to 5 times that of soaked CBR values as measured in the research study (Table 5-10); this fell within the proportion found in the literature about 2-7 times. It can also be noted that smaller samples were tested for unconfined compressive strength with height to diameter ratio less than 2 and corrections were made for samples with height over diameter ratio less than 2.

5.7 Unconfined Compressive Strength (UCS) vs. Resilient Modulus (M_r) Values Calculated from CBR Test Results

There is a suggested relationship between the unconfined compressive strength and resilient modulus as discussed in the literature (Little 1999) which is graphically shown in Figure 5-6.

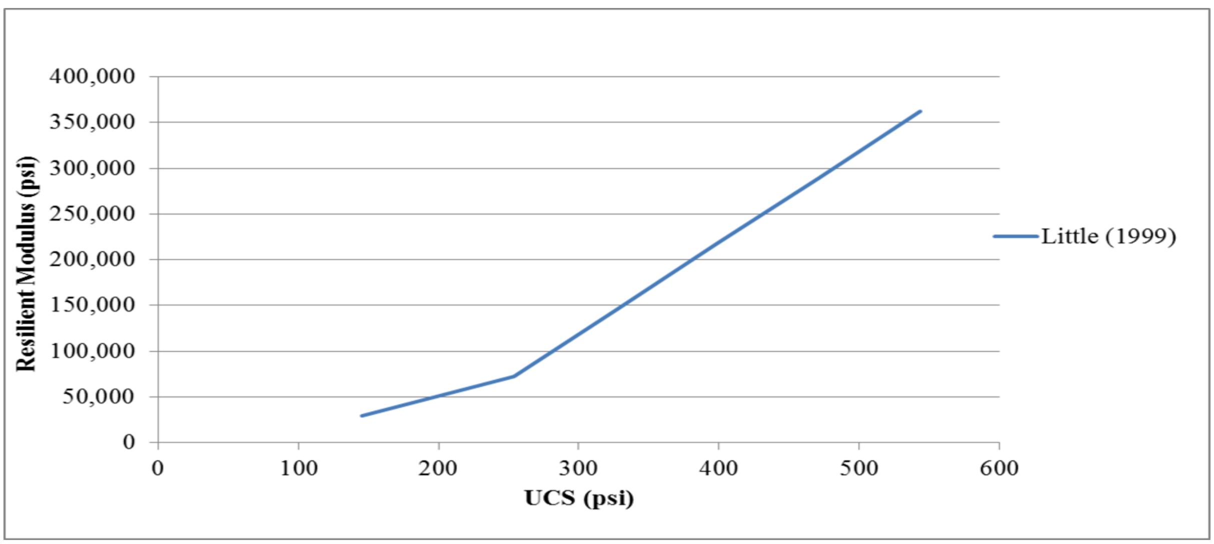


Figure 5-6 Suggested Design Relationship between Unconfined Compressive Strength (UCS) and Resilient Modulus for Lime Stabilized Subgrade Materials (Little, 1999)

The suggested relationship from Little (1999) was used to estimate the design resilient modulus from measured compressive strength values. For compressive strengths between 140 and 300 psi, the resilient modulus varied between 29000 to 73000 psi. The resilient modulus (M_r) values were calculated from the CBR test results in the research study to evaluate them in reference to this relationship. Table 5-11 summarizes the treated compressive strength, CBR test results, and calculated resilient modulus values from the research study. Resilient modulus corresponding to both unsoaked and soaked CBR values were calculated using the AASHTO formula –

$$Mr = 2555\ CBR^{0.64}$$

Table 5-11 Summary of Treated Compressive Strength, CBR Test Results and Calculated Resilient Modulus Values

Investigation	Soil Type		Treated						
	USCS	AASHTO	Admixture*	Uncured UCS (psi)	Unsoaked CBR	Resilient Modulus (psi)	Moist Cured UCS (psi)	Soaked CBR	Resilien Modulu (psi)
Research Study	ML	A-6	LFA-2%	85	16	43946	145	28	61770
	ML	A-4	LFA-4%	80	26	42206	153	56	63930
			LFA-6%	92	28	46122	147	86	62314
			LFA-8%	90	30	45542	176	91	69925
	-	-	LFA-10%	87	31	44527	262	96	90202
			LFA-12%	77	46	41191	259	173	89540

*Quicklime and Class F Fly Ash

As noted in Table 5-11, the calculated resilient modulus values were plotted against the UCS values per the relation discussed above. Figure 5-7 shows the linear regression on resilient modulus calculated from unsoaked and soaked CBR values and unconfined compressive strength of treated samples.

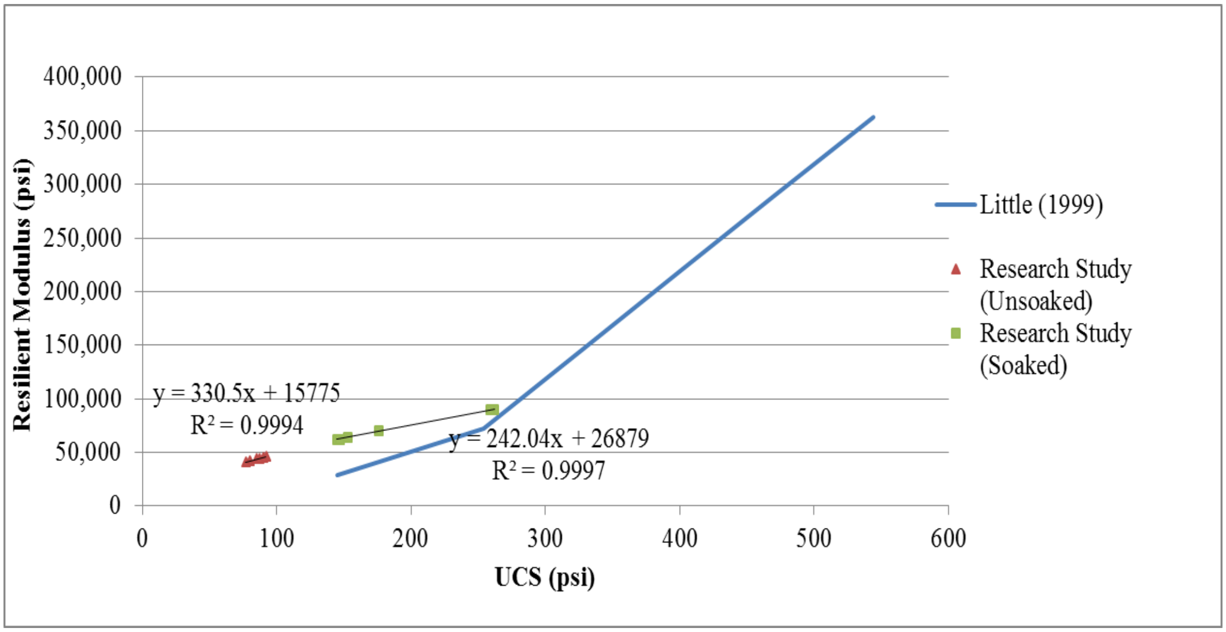


Figure 5-7 Linear Regression on Resilient Modulus Calculated from Unsoaked and Soaked CBR Values and Unconfined Compressive Strength of Treated Samples

As seen in the graph, values were plotted using linear regression. R square value of 0.99 represents best fit of the data. The aforementioned relationship from the research study followed a similar path or had a similar slope as presented by Little (1999). Resilient modulus corresponding to unsoaked CBR values fall below the range of suggested relationship by Little (1999). This can be due to reduced hydration and lower strength of the soil specimen. Lower resilient modulus corresponding to soaked CBR values are more representative with respect to Figure 5-6. The necessity of soaking/curing for a better compressive strength is reflected in Figure 5-7.

Chapter 6 – Conclusions and Recommendations

6.1 Conclusions

The objective of this research study was to perform a series of unconfined compression strength tests to establish correlations in relation to the strength improvement of treated soils and their corresponding CBR values. Representative soil samples were obtained from the problematic loess soils in south central and southeastern Idaho. The soil was mixed with different proportions of lime, fly ash, and combinations of lime and/or fly ash to determine the optimum lime-fly ash mix proportion. Moist cured and uncured UCS tests were conducted on both untreated and treated soils to provide a basis for strength improvement and corresponding correlations. The beneficial effect of lime-fly ash treatment of the project soil was analyzed on the basis of these UCS test results and CBR correlations.

Different admixtures involving quicklime, hydrated lime, Class C fly ash, and Class F fly ash were considered for the use of soil stabilization. The admixtures were tested individually and in combinations to evaluate their impact on soil stabilization and strength improvement. Thorough investigation of the applications of aforementioned admixtures revealed that quicklime and Class F fly ash were more viable choices for the current research study. The core of the experiments conducted focused on combination effects of these additives varying between 2 to 12 percentages by dry weight of solids.

In practice, the recommended minimum quantity of additive is 4 percent. For the current study, lime to fly ash ratio of 1:3 was determined to be an effective treatment proportion making up the recommended total additive quantity of 4% by dry weight of solids. Besides offering significant strength improvement, mixing soil with 3 parts of fly ash and only 1 part of lime is also cost effective. Unconfined compressive strengths improved as a result of mixing quicklime and Class F fly ash with the study soil. The increase in UCS was significant in the case of moist cured soil samples. The unconfined compressive strength for uncured samples reached an average strength of 80 psi while the moist cured samples displayed an average strength of 153 psi. These results indicate a significant improvement in the bearing strength of the stabilized soil. A reasonable relationship existed between the moist cured compressive strength and soaked CBR values under variable curing conditions.

6.2 Recommendations

The experiments in this study were conducted under controlled laboratory conditions. The soil was disaggregated to a maximum particle size of about ¼ to ½ inch to ensure proper size for uniform blending and absorption of lime and/or fly ash. This mechanical disaggregation of soil to uniform maximum particle size may not be feasible under vast actual field condition. It is recommended that test pavement structures should be constructed to evaluate and monitor the performance of the admixtures and stabilization procedures used in this study under field conditions. This will help analyze if the current laboratory findings could be extrapolated to actual field conditions.

Current research focused on loess deposits in south central and southeastern Idaho and an optimum lime-fly ash percentage was determined based on the silty nature of the problematic soil. Future research could investigate other representative soils and corresponding lime-fly ash proportions that best fit the soil type. Admixtures including lime and fly ash could be varied based on their availability and economic feasibility of the intended project. A lime to fly ash ratio of 1:3 was suggested an appropriate proportion in the current research based on recommendations from the literature and trial mixing proportions. Also, fly ash is cheap and readily available, which offsets the higher cost of lime and constitutes ¾ of the recommended additive quantity of 4% by dry weight of solids.

Curing time was limited to 4 days and curing conditions varied in unconfined compression tests and CBR tests in the current study. Future research could establish longer curing times and more identical curing conditions to better correlate unconfined compressive strength results with CBR values.

REFERENCES

1. Beeghly, J.H. (2003). *Recent Experiences with Lime-Fly Ash Stabilization of Pavement Subgrade Soils, Base and Recycled Asphalt*. Pittsburg, PA: International Ash Utilization Symposium. Retrieved January 17, 2008, from http://www.flyash.info/2003/46beeg.pdf

2. Kazemian, S., Huat, B.K., Prasad, A., and Barghchi, M. (2010). *A Review of Stabilization of Soft Soils by Injection of Chemical Grouting.* Australian Journal of Basic and Applied Sciences, 4(12): 5862-5868, 2010

3. Parsons, R. L. and Milburn, J. P. (2003). *Engineering Behavior of Stabilized Soils. Transportation Research Record*, Journal of the Transportation Research Board, 1837, 20-29. http://dx.doi.org/10.3141/1837-03

4. Lewis, G.C., Fosberg, M.A., McDole, R.E., and Chugg, J.C. (1975). *Distribution and Some Properties of Loess in Southcentral and Southeastern Idaho,* Soil Science Society of America J., 39: 1165-1168

5. Sherwood, P.T. (1993). *Soil Stabilization with Cement and Lime.* TRL State of the Art Review, London: HMSO, 1993. p. 153.

6. Natt, G.S. and Joshi, R.C. (1984). *Properties of Cement and Lime – Fly Ash Stabilized Aggregate.* Transportation Research Record 998, Transportation Research Board, 1984, p. 32-40.

7. Mahar, J. W. (2005). *Particle Breakdown in Collapsible Loess Deposits in SE Idaho*. Proceedings of the Geological Society of America Annual Meeting, Salt Lake City, Utah, Vol. 37, No. 7, Paper No. 144-3: p. 327.

8. ASTM International Standards Worldwide. (2006). *ASTM C593 - 06 Standard Specification for Fly Ash and Other Pozzolans for Use with Lime for Soil Stabilization*. ASTM International.
9. Schaeffer, R.V. (1997). *Ground Improvement, Ground Reinforcement and Ground Treatment, Developments.* Proceedings of the First National Conference of the ASCE Geo-Institutes, Logan, Utah, Geotechnical Special Publication No. 69: 45-62
10. Puppala, J. A., Mohammad, N. L. and Allen, A. (1996). *Engineering Behavior of Lime-Treated Louisiana Subgrade Soil*. Journal of Transportation Research Record, 1546, 24-31.
11. Yusuf, S., Little N. D., & Sarkar, L., S. (1998). *Evaluation of Structural Contribution of Lime Stabilization of Subgrade Soils in Mississippi*. Transportation Research Record, 1757, 22-23.
12. Shirazi, H. (1999). *Field and Laboratory Evaluation of the Use of Lime Fly Ash to Replace Soil Cement as a Base Course*. Journal of Transporation Research Record, 1652, 270-275.
13. Little, D.N. (1999). *Evaluation of Structural Properties of Lime Stabilized Soils and Aggregates.* Volume 1: Summary of Findings, National Lime Association, Arlington, Va., p. 89.
14. ASTM (2008). *ASTM Book of Standards Cement and Concrete*. Cross Ref ASTM International.
15. ASTM International Standards Worldwide. (2007). Annual Book of ASTM Standards. Library of Congress.

16. Barbu, B. (2004). *Identification and Stabilization of Problematic Silts*. Retrieved April 7, 2008, from ProQuest Digital Dissertations database. (AT 765940401).
17. Baoshan, H., Qubain, S. B., Heirendt, M. K. & Li, J. (2006). *Quality Assurance and Quality Control Requirements for Lime and Cement Subgrade Stabilization*. Pavement Mechanics and Performance: Proceedings of Sessions of GeoShanghai, GSP 154.
18. Department of the Army, The Navy and The Air Force. (1994). *Soil Stabilization for Pavements* (TM 5-822-14/AFJMAN 32-1019). Washington, DC: Retrieved from http://www.army.mil/USAPA/eng/DR_pubs/dr_a/pdf/tm5_822_14.pdf
19. Hussein, H.A. (2006). *Treatment and Improvement of the Geotechnical Properties of Different Soft Fine-Grained Soils Using Chemical Stabilization*. Retrieved from http://sundoc.bibliothek.uni-halle.de/diss-online/06/06H107/t1.pdf
20. Lovewell, C.E. (1975). *History of Development of ASTM Lime Specifications*. In. Masonry: Past and Present. ASTM STP 589, American Society for Testing and Materials, pp. 3-9.
21. Lukens, P. R., & Cornillot, L.J. (1980). Annual Book of ASTM Standard, Part 13.
22. McGrath, C. L. (1987). *Soil Survey of Bannock County Area, Idaho*. National Cooperative Soil Survey.
23. National Cooperative Highway Research Program (NCHRP) & ARA, Incorporated, ERES Consultants Division. (2004). *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*. Transportation Research Board. Retrieved April 17, 2009, from http://www.trb.org/mepdg/guide.htm

24. Qubain, S. B., Seksinsky, J. E. & Li, J. (2000). *Incorporating Subgrade Lime Stabilization into Pavement Design*. Journal of Transportation Research Record, 1721, 3-8.

25. U.S. Department of Transportation, Federal Highway Administration. (1988). *Use of Coal Ash in Embankments and Bases* (publication No. T 5080.9). Retrieved from http://www.fhwa.dot.gov/legsregs/directives/techadvs/t508009.htm

26. USDA Natural Resources Conservation Service Soil Survey Data. (1987). Engineering Properties of Bannock County.

27. Yoder, J. E. & Witczak, W. M. (1975). *Principles of Pavement Design*. New York: A Wiley-Interscience Publication.

Appendix 1

Unconfined Compression Strength Test Results for Untreated Soil from STP-5

PROJECT: LIME-FLY ASH TREATED SOIL NAME: MY/PK
LOCATION: IRON GATE POND SITE DATE: 1/16/2008
TEST METHOD: ASTM D 2166 - 06

UNCONFINED COMPRESSION TEST UNTREATED SOIL

UCT TEST NUMBER #1
SAMPLE LOCATION OR NUMBER: STP 5, B-5/4, AVG. OPTIMUM MOISTURE
DEPTH OF SAMPLE: DEPTH = 0.75FT TO 1.4FT CONTENT 19.5%
DESCRIPTION OF SOIL: Untreated soil

SPECIMEN DATA
TYPE OF SPECIMEN ___ Undisturbed X Remolded
SHAPE OF SPECIMEN X Cylinderical ___ Prismatic
DIAMETER OF SPECIMEN, D 1.309 in
INITIAL AREA OF SPECIMEN, A 1.346 sq in
INTIAL HEIGHT OF SPECIMEN, H 3.336 in
HEIGHT-TO-DIAMETER RATIO, H/D 2.549
VOLUME OF SPECIMEN, V = A*H 4.489 cu. in.
MASS OF SPECIMEN, M 145.96 gm

WET UNIT WEIGHT OF SPECIMEN, γ_{wet} (M/453.6)*(1728/V 123.85 pcf

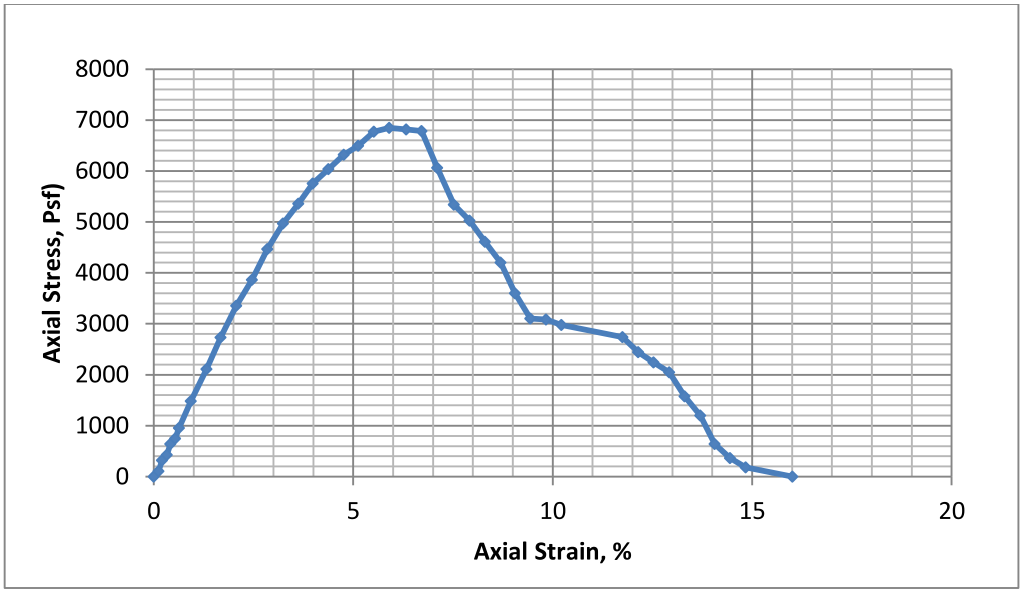
MOISTURE CONTENT

TEST #	1
CUP #	2
CUP TARE	90.58
WET+TARE	149.38
DRY+TARE	139.74
ω, %	19.61

DRY UNIT WEIGHT OF SPECIMEN, γ_{dry}: $(\gamma_{wet}/(100+\omega))*100$ **103.55 pcf**

MODE OF FAILURE: crack along the height of the sample

UNCONFINED COMPRESSIVE TEST								
UCC TEST #1								
LOAD - DISPLACEMENT CURVE				ACTUAL DATE/TIME START : 01/16/08				
Dial reading	Load	ΔL	Strain	Strain	Area (A')	Stress		
(in)	(lbs)	(in)	(ε)	(%)	(sq ft)	(psf)		
0	0	0	0	0	0.00935	0		
0.004	1	0.004	0.0012	0.1199	0.00936	106.862		
0.007	3	0.007	0.0021	0.20983	0.00937	320.298		
0.011	4	0.011	0.0033	0.32974	0.00938	426.551		
0.014	6	0.014	0.0042	0.41966	0.00939	639.249		
0.018	7	0.018	0.0054	0.53957	0.0094	744.893		
0.021	9	0.021	0.00629	0.6295	0.00941	956.853		
0.031	14	0.031	0.00929	0.92926	0.00943	1483.95		
0.044	20	0.044	0.01319	1.31894	0.00947	2111.59		
0.056	26	0.056	0.01679	1.67866	0.00951	2735.06		
0.069	32	0.069	0.02068	2.06835	0.00954	3352.88		
0.082	37	0.082	0.02458	2.45803	0.00958	3861.34		
0.095	43	0.095	0.02848	2.84772	0.00962	4469.58		
0.108	48	0.108	0.03237	3.23741	0.00966	4969.29		
0.121	52	0.121	0.03627	3.6271	0.0097	5361.71		
0.133	56	0.133	0.03987	3.98681	0.00973	5752.6		
0.146	59	0.146	0.04376	4.3765	0.00977	6036.18		
0.159	62	0.159	0.04766	4.76619	0.00981	6317.25		
0.171	64	0.171	0.05126	5.1259	0.00985	6496.4		
0.184	67	0.184	0.05516	5.51559	0.00989	6772.99		
0.197	68	0.197	0.05905	5.90528	0.00993	**6845.72**	Maximum stress	
0.211	68	0.211	0.06325	6.32494	0.00998	6815.19		
0.224	68	0.224	0.06715	6.71463	0.01002	6786.84		
0.237	61	0.237	0.07104	7.10432	0.01006	6062.76		
0.251	54	0.251	0.07524	7.52398	0.01011	5342.79		
0.264	51	0.264	0.07914	7.91367	0.01015	5024.71		
0.277	47	0.277	0.08303	8.30336	0.01019	4611.01		
0.29	43	0.29	0.08693	8.69305	0.01024	4200.66		
0.302	37	0.302	0.09053	9.05276	0.01028	3600.28		
0.315	32	0.315	0.09442	9.44245	0.01032	3100.42		
0.328	32	0.328	0.09832	9.83213	0.01037	3087.07		
0.341	31	0.341	0.10222	10.2218	0.01041	2977.68		
0.392	29	0.392	0.11751	11.7506	0.01059	2738.14		
0.405	26	0.405	0.1214	12.1403	0.01064	2444.04		
0.418	24	0.418	0.1253	12.53	0.01069	2246.03		
0.431	22	0.431	0.1292	12.9197	0.01073	2049.69		
0.444	17	0.444	0.13309	13.3094	0.01078	1576.76		



From the stress-strain curve above,					
Unconfined Compressive Strength (q_u) =			6845.72	**lb/sq. ft.**	
			=	48	**psi**
Corrected UCC			=	55	**psi**