EVALUATING SUBGRADE SOIL STABILIZATION USING CEMENT AND LIME, THEIR COST AND DAMAGE PROGRESSION ANALYSIS

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DEDICATED

TO

MY PARENTS AND TEACHERS

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(Abdul Hanan Tunio)

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ABSTRACT

This research study evaluates subgrade soil stabilization using cement and lime and their cost and damage progression analysis. Since treatments to soil are generally classified as processes of soil stabilization or soil modification. Specifically purpose of subgrade stabilization is to enhance the subgrade soil strength. This increased strength is then taken into account in the pavement design process.

In this research cement and lime being easily available material has been selected as stabilizing agents with their proportions as 2%, 4% and 6% of the dry weight of soil in the mixture. Parameters considered for analysis of results were Dry Density, California Bearing Ratio (CBR), Unconfined Compressive Strength (UCS), and moisture susceptibility. A low plastic clay (CL), A-6 soil from Karachi Lahore Motorway (KLM) Hyderabad-Sukkur section near Nawabshah has been used in the research. Methodology adopted entails characterization of physical and chemical properties of soil material, determination of optimum stabilizer contents (for cement and lime), specimen preparation and testing for California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) test in dry and moist condition. This research also determine cost effectiveness of easily available soil stabilizers like cement and lime and cost comparison with conventional methods used to increase the strength and CBR of unsuitable subgrade material.

It is observed that cement works well to stabilize any type of soil, except soils with organic content greater than 2% or having pH lower than 5.3. Other significant effects of cement-soil stabilization process are reduction in shrinkage and swell potential with increase in strength. Moreover soil treatment with lime produced significant increase in strength of soil. The immediate increase in strength results from flocculation-agglomeration reaction and results increased workability, whereas long-term strength gain is due to pozzolanic reactions. Results indicate that under the defined test conditioned and site specific soil, cement and lime treatment can be 25% to 30% cost effective compared to the current practice of replacement of poor subgrade soil, besides enhancing the long term durability of the pavement.

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Treatments to soil are generally classified as process of soil stabilization or soil modification. Soil stabilization is a method of adding admixtures to soil to enhance its strength, durability, volume stability and permeability (Bell, 1993).The objective for subgrade soil modification is to achieve a working platform for movement of a construction equipment. No credit is accounted for in this modification in the design of pavement considerations. Whereas purpose of subgrade stabilization is to enhance the subgrade soil strength. This increased strength is then taken into account in the pavement design process. Stabilization requires thorough design methodology during construction as compared to modification. Subgrade modification or stabilization include physical processes such as soil densification, blends with granular material, use of reinforcements (Geogrids), undercutting and replacement, and chemical processes such as mixing with cement, fly ash, lime, lime byproducts, and blends of any of these materials. Soil properties such as strength, compressibility, hydraulic conductivity, workability, swell potential and volume change potential may be altered by different soil stabilization methods.

Portland Cement as an additive can modify and improve the quality of soil with increased strength and durability. That can also be used to control the erosion of inorganic soils (Oswell, and Joshi, 1986). Oswell, and Joshi (1986) found a good correlation between unconfined compressive strength and erosion resistance characteristic. As the compressive strength increases the rate of erosion gets on decreasing. Cement works well to stabilize any type of soil, except soils with organic content greater than 2% or having pH lower than 5.3 (ACI 230.1R-90, 1990). Kezdi (1979) reported that cement treatment slightly increases the maximum dry density of sand and highly plastic clays but it decreases the maximum dry density of silt. In studies by Tabatabi (1997) concludes that cement increases the optimum water content but decreases the maximum dry density of sandy soils. Cement increases plastic limit and reduces liquid limit which mainly reduces plasticity index (Kezdi, 1979). Other significant effects of cement-soil stabilization process are reduction in shrinkage and swell potential, increase in strength and elastic modulus and resistance against the effect of moisture including freeze and thaw. Cement treated soils show brittle behavior as compared to non-treated soils. Cement increases both cohesion and angle of internal friction of the soil (Uddin et al., 1997; Bragdo eral, 1996)

Soil treatment with lime can produce significant increase in strength of soil. The immediate increase in strength results from flocculation-agglomeration reaction and results workability, whereas long-term strength gain is due to pozzolanic reactions (Thompson, 1966). With increase in lime content it results in increased unconfined compressive strength (Giffen et al., 1978). Lime has been used in stabilizing clayey soils and has been found to impart long-term strength gain as reported in the literature (Bell and Coulthard, 1990; Little, 1995; Mallela et al., 2004; Amu et al., 2011; Herrier et al., 2012). Lime treatment to natural soil has already been used for several years for improving soil strength and stiffness properties. This treatment allows to reuse fine soils obtained from earthworks by reducing their plasticity index and compactive effort and improvement their bearing capacity. For the long term durability, lime treated soils prove to have improved compressive strength and CBR [ASTM D (1883-94)], as well as increased resistance to frost.

Soil stabilization techniques have not been practiced much in Pakistan, while there is a big need to attain better strength of weak subgrade in many regions. Also a little research work has been carried out for determining the effect and for proportions of stabilizing agents like cement and lime. Now when in Pakistan national trade corridor development is in progress and huge work is expected to be undertaken such research is the need of hour. many areas having lower bearing capacity can be made part of highway network with soil stabilization techniques. Therefore in this research cement and lime has been selected as stabilizing agents with their proportions as 2%, 4% and 6% of the dry weight of soil in the mixtures.

Scope of this research has been to obtain intelligent decision in the achievement of most economical pavement section with due diligence for value engineering considerations. For a soil with weak subgrade was to enhance its strength with the use of most suitable stabilizing material with easily executable procedure on large scale on site of construction. Parameters considered for analysis of results were Dry Density, California Bearing Ratio (CBR), Unconfined Compressive Strength (UCS), and moisture susceptibility. The ratio of conditioned soil strength and unconditioned soil strength gives the value of moisture susceptibility. This research was carried out by determining the optimum moisture content (OMC) with respect to each proportion of stabilizing agent by modified proctor test. The OMC of control specimen has also been made found out. Three types of specimens for carrying out CBR and UCS tests were prepared according to their respective optimum moisture contents. . With the attainment of these tests, evaluation and analysis of results, this research work will produce a workable solution for stabilization of problematic soils in Pakistan.

1.2 PURPOSE, RATIONALE AND JUSTIFICATION FOR THIS RESEARCH

Since there are areas in Pakistan with weak subgrade soil for the construction of highways there. Due to that problematic soil with CBR much less than required various activities are carried out to counter this issue including removal of such soil and replacement with most suitable soil borrowed from other areas. This all incur huge cost with permanent weak layer under lying all construction over it insinuating failure and recurrent costly maintenance works. Through this research it has been tried to find out an economical solution with long life for the constructed structures on such problematic soils. When stabilizing materials are easily available and with suggested design considerations and algorithm through the outcome of this research, a rationale solution of this issue can be have for future construction activities.

1.3 PROBLEM STATEMENT

Selection of stabilizing agent is crucial for the attainment of desired results for an specific soil type. Soil constituents including organic matter, percentage of fines, sand, silt and clay and existence of moisture may play variant role in bonding with stabilizing agent. It is not always common that the soil type may be accepting increasing percentage of stabilizing agent for a strength gain. Some stabilizing agents can be enhancing strength of soil up to a particular quantity and by increasing the volume of stabilizing agent for that particular soil beyond that optimized limit may inhibit the strength gain. This is to be discovered based on accepted standards for the control soil specimen.

1.4 RESEARCH OBJECTIVES

Research objectives entail determination of how to achieve most efficient, economical, practicable solution for sub-grade stabilization based on value engineering concepts. Which may contain systematic method to improve the value of constructed structures by using an examination of functions which will reduce these cost to deliver the required functions at lowest cost while meeting quality, performance, and reliability specifications. Following are the objectives of this research:

- To evaluate efficacy of soil stabilizing agents to enhance the bearing capacity, strength and other engineering properties of subgrade.
- To study the effect of varying proportions of cement and lime on California Bearing Ratio (CBR), Unconfined Compressive Strength (UCS) on the pavement subgrade material and selecting the optimum percentage of modifier for a given soil type.
- To determine cost effectiveness of soil stabilizers mainly Portland cement and lime and their comparison with conventional methods used to increase the strength and bearing capacity of unsuitable subgrade material.
- To quantify the reduction in moisture induced damage of lime and cement stabilized/ treated subgrade soils

1.5 METHODOLOGY / EXPERIMENTAL DESIGN

Methodology adopted for this research consisted of determination of engineering properties of control soil specimen and evaluation of soil material type according to AASHTO classification standards. Selection of most suitable stabilizing material and assessment of optimum proportion for conditioning of specimen carried out following their related standards. Experimental design takes into consideration the attainment of required strength with economical percentage of stabilizing agent added in the soil specimen which may result desired design life expectancy, safety and fulfillment of value engineering considerations. Figure 1.1 presents the research methodology.

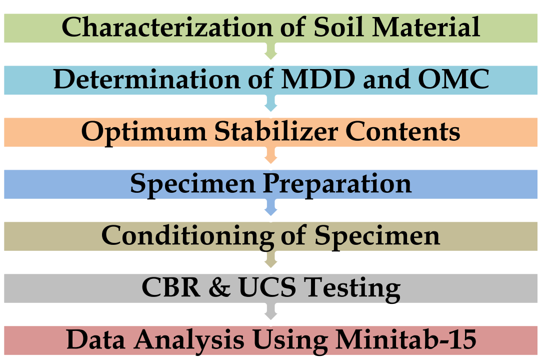


Figure 1.1: Research Methodology

1.6 SCOPE OF THE THESIS

To achieve the above-mentioned research objectives, a research plan was prepared and the following research tasks were outlined:

- ❖ Literature review pertaining to the soil stabilization methods including previous research on application of different additives used in modification and stabilization of subgrade material and effect of curing on the strength related properties of stabilized soils and test procedures required to evaluate the properties of stabilized soil mixtures.
- ❖ Development of particle size distribution curves by sieve analysis (ASTM C 136).
- ❖ Evaluation of Atterberg limits (liquid limit, plastic limit and shrinkage limit) (ASTM D 4318).
- ❖ Soil Classification according to AASHTO method (AASHTO M 145-82).
- ❖ Determination of Organic Contents and Sulfate contents of soil specimens using their related standards ASTM D 2974-00 and AASHTO T-290 respectively.
- ❖ To check the suitability of soil for stabilizing with Portland Cement and Hydrated Lime $Ca(OH)_2$.
- ❖ Determination of Optimum Lime Contents by Eades and Grim pH test using ASTM D 6276-99.
- ❖ Selection of Optimum Cement Contents on the bases of prevailing cost comparison following NHA Composite Schedule of Rates (CSR-2014).
- ❖ Determination of Optimum Moisture Content (OMC) and Maximum Dry Unit Weight for the raw and stabilized soil mixtures following ASTM D 1557-12.
- ❖ Laboratory preparation of specimens for Unconfined Compressive Strength (UCS) test and CBR test by varying the proportion of stabilizing agents Portland cement and lime as 2%, 4% and 6%.
- ❖ For California Bearing Ratio (CBR), the Control and Modified specimens were tested following test procedure ASTM D 1883-99.
- ❖ Cement modified specimens for UCS test were prepared and cured according to ASTM D 1632-96 and were tested using ASTM D 1633 on their respective equipment.

- ❖ The Lime modified specimens were tested for Unconfined Compressive Strength (UCS) on its respective equipment according to ASTM D 5102.
- ❖ Statistical analysis and discussion.

1.7 ORGANIZATION OF THE THESIS

This research is organized into five chapters:

Chapter 1 includes a brief introduction to soil stabilization, role of different stabilizers, and their optimum amount required to improve the soil strength parameters, objectives and scope of the research.

The second chapter includes a brief literature review on sub grade stabilization, functions of hydrated lime and impact of Portland cement on the performance of subgade material. A short introduction to Modified proctor test and Unconfined Compressive Strength test is included. Finally, the result analysis technique has been reviewed.

The third chapter describes the methodology used to achieve the objectives of the study. This explains the characterization of soil material, determination of optimum stabilizer contents (for cement and lime), preparation of specimen for California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) test. Conditioning and testing of CBR and UCS test.

Chapter 4 presents the experimental results and their analysis using the software Minitab-15. This chapter describes stabilization results and the full factorial design and regression analysis for cement stabilization of UCS testing data.

Chapter 5 summarizes the findings and conclusions of laboratory testing and statistical analysis. The future work and suggestions are also discussed.

CHAPTER 2

REVIEW OF THE RELEVANT LITERATURE

2.1 INTRODUCTION

This chapter is a review of the literature about stabilization with different types of stabilizers mostly used for soil stabilization and their methods of application to soil is also discussed. The basic properties of soil relevant to the phenomenon of stabilization are discussed in this chapter. Effectiveness of Portland Cement and Hydrated Lime $Ca(OH)_2$for stabilizing subgrade soil including their mechanism is described. The mix design for cement and Lime stabilization with its procedural steps and factors affecting strength of stabilized soil are explained. Benefits of soil stabilization are also elaborated. At the end the tests used in this particular research work are discussed mainly Modified Proctor Test, California Bearing Ratio (CBR) Test and Unconfined Compressive Strength test.

2.2 SOIL STABILIZATION

It is altering the soil properties entailing mechanical or chemical techniques resulting better engineering characteristics. The modification and stabilization definition can be confusing. Modification is soil improvement in short period (within hours) during mixing or shortly after mixing. Modification improves consistency of soil (reduces plasticity) to the required level and also improves strength for short-term only. While no significant cementitious or pozzolanic reaction occurs but measurable strength improvement occurs due to textural changes and consistency improvements. Stabilization takes place when long-term significant reactions occur. The hydration of calcium-aluminates and calcium silicates causes this long-term reaction in Portland cement or between soil pozzolans and free lime due to pozzolanic reactivity. Around 50 psi increase in strength or greater can be reasonable criteria for stabilization compared with strength of untreated soil under similar conditions of preparation and curing. (TRB 144).

The geotechnical properties of unsuitable soils (such as stiffness and strength) can be improved by different methods and techniques. These techniques include pore water pressure reduction, densifying by compaction or preloading, use of reinforcing elements and bonding of soil particles. For many works including geotechnical applications such as roadways, pavement structures, channel and reservoir linings, building foundations, sewer lines and water lines, chemical stabilization of problematic soil is important to avoid the breakage due to the swell action of expansive soil or due to the settlement of the soft soil .Generally, the stabilization concept is older up to 5000 years ago. History reveals that stabilized earthen roads were used in Egypt and ancient Mesopotamia and Romans and the Greeks used soil-lime mixture for that purpose. The initial experiments on subgrade stabilization were attained in the USA with clay-sand mixtures around 1906. Soil stabilization pertinent to road construction was applied in Europe in the 20th century. Pokalwar et al., (2014)

2.3 WORLDWIDE STABILIZATION TECHNIQUES AND METHODS

Depending on the purpose and type of soil to be stabilized there are various stabilization methods and materials. Following table 2.1 depicts stabilizing materials to be used which are governed by different situations;

Table 2.1: Soil Stabilization Techniques used Worldwide

S No	Description	S No	Description
1	Lime Stabilization	12	Tree Resin
2	Fly Ash Stabilization	13	Bamboo Leaf Ash Stabilization
3	Asphalt Stabilization	14	Electro kinetic Stabilization
4	Cement Stabilization	15	Soil Verification
5	Rice Husk Stabilization	16	Stabilization with Pulp & Paper
6	Soil Reinforcement Method	17	Liquefied Soil Stabilization Method
7	Stabilization with Scrap Tires	18	Expanded Polystyrene Blocks (EPS)
8	Acids	19	Renolith Stabilization
9	Lignosulfonates	20	Permazymes
10	Polymers	21	Fujibeton
11	Petroleum Emulsion	22	Terrazymes

2.4 STABILIZATION COMPONENTS

Soil stabilization involves the use of stabilizing agents (binder materials) for weaker soils to improve their geotechnical properties. Improved results can be such as compressibility, strength, permeability and durability. Stabilization components include soil and or soil minerals and stabilizing agents or binders (cementitious materials).

2.4.1 Soils

Most of stabilization has to be undertaken in soft soils (silty soil, clayey peat or organic soils) to achieve desirable engineering properties. According to Sherwood (1993) fine-grained granular materials are the easiest to stabilize due to their large surface area in relation to their particle diameter. A clayey soil compared to others has a large surface area due to flat and elongated particle shapes. Silty materials can be sensitive to small changes in moisture which may cause difficulty during stabilization (Sherwood, 1993). Peat soils and organic soils are rich in water content of up to about 2000% contain high porosity and high organic content. The consistency of peat soil can vary from muddy to fibrous and in most cases, the deposit is shallow, but in worst cases, it can extend to several meters below the surface (Pousette, et al 1999; Cortellazzo and Cola, 1999; Ahnberg and Holm, 1999). Organic soils have high exchange capacity; it can hinder the hydration process by retaining the calcium ions liberated during the hydration of calcium silicate and calcium aluminate in the cement to satisfy the exchange capacity. In such soils, successful stabilization has to depend on the proper selection of binder and amount of binder as added (Hebib and Farrell, 1999; Lahtinen and Jyrava, 1999, Ahnberg et al, 2003).

2.4.2 Stabilization Agents

Stabilizing agents can be reactive and inert admixtures or reinforcement agents (Geogrids & Geotextiles etc). Geotextiles are permeable fabrics which, when used in association with soil, have the ability to separate, filter, reinforce, protect, or drain. Geogrids represent a rapidly growing segment within geosynthetics. Stabilizing agent when come in contact with water or react with water in the presence of pozzolanic minerals they form composite cementitious materials. Most commonly used stabilizing agents are; Cement, Lime, Fly ash and Blast furnace slag

2.5 SOIL PROPERTIES

2.5.1 Atterberg Limits

2.5.1.1 Liquid Limit:

When water content of the soil keeps soil between plastic state and liquid state is called liquid limit. It is minimum water content at which the soil is in liquid state and shows little shearing strength against flowing. Casagrande's apparatus is used to measure the liquid limit and is denoted by *wL*.

2.5.1.2 Plastic Limit:

Plastic limit of soil lies between its semi-solid state and plastic state. It is established by rolling out a thread of the soil on a non-porous flat surface. Least water content present in soil at which it just begins to crumble while rolling into approximately 3mm diameter thread. Plastic limit is denoted by *wP*.

2.5.1.3 Shrinkage Limit:

This limit is attained when further loss of water does not affect reduction in the volume of the soil . It is lowest water content at which the soil can still be said as completely saturated. It is denoted by *wS*.

2.5.2 Particle Size Distribution

Soil is composed of particles of different sizes and shapes. Particle sizes in the soil sample can be of ranging from microns to centimeters (Krishna Reddy, 2008). Density, strength and permeability in soil are determined by particle size distribution in a soil sample.

There are two methods used for describing particle size distribution. One for coarse grained soils is sieve analysis and second for fine grained soil sample sedimentation analysis is used. The results from both the methods are plotted. Graphs give us the information about gradation and type of the soil. If the curve is more towards the right, it means that the concentration of coarse grained particles is more; if it is higher towards left, we can deduce that the percentage of finer particles is more.

2.5.3 Specific Gravity

It can be defined "as the ratio between the mass of any substance of a definite volume divided by mass of equal volume of water". Specific gravity is obtained by the number of times the soil substance is heavier than the same volume of water. Specific gravities is different for different soil types.

2.5.4 Shear Strength

The stresses induced in a loaded soil are shearing stresses at the limiting value of these stresses deformation of the soil mass takes place which leads ultimately to failure of the soil specimen. "The shear strength of a soil is its resistance to the deformation caused by the shear stresses acting on the loaded soil". It is one of the most important property of a soil. UCS test can be used to determine shear strength of soil.

2.5.5 Bearing Capacity

Bearing capacity is the ability of soil to safely carry the pressure placed on the soil from any engineered structure without undergoing a shear failure with accompanying large settlements. Applying a bearing pressure which is safe with respect to failure does not ensure that settlement of the foundation will be within acceptable limits.

2.6 MECHANISMS OF STABILIZATION

The mechanism of stabilization may include coating the particles surfaces by stabilizing agents to control the moisture sensitivity and new compounds are formed of finer particles as bonded. That's why a proper understanding of each additive for stabilization mechanisms is required before selection of an additive. In chemical stabilization the soil is mixed with chemically active compounds such as lime, Portland cement, fly ash, sodium or calcium chloride or with bitumen (visco-elastic materials). There are broadly three groups of chemical stabilizers: Traditional stabilizers; Non-traditional stabilizers; and By-product stabilizers.

2.6.1 Traditional Stabilizers

The important traditional stabilizers are Portland cement, Hydrated lime and fly ash. The most commonly used chemical additives are Portland cement, lime, and fly ash (Petry, T. M., and Little, D. N (2002))

Traditional stabilizers broadly rely on pozzolanic reactions and exchange of cat-ions to stabilize soil. Commonly lime is used for that objective. Decomposing limestone at elevated temperatures produces lime. Soil-Lime reactions are two step complex processes. The primary reaction engross flocculation/agglomeration and exchange of cat-ion and cause the textural and plasticity changes (Little, D. N. 1995).Larger particle agglomerates are formed due to short term pozzolanic reaction and cat-ion exchange which make the soil more friable and more workable. In the second step a cementing process among agglomerates and flocculates occur that leads to pozzolanic reaction. Considerable strength increase depends on quantity of pozzolanic matter. Which depends on reactivity of soil minerals with the additives (TRB 144).

2.6.2 Non - Traditional Stabilizers

Stabilization mechanism for non-traditional stabilizers may include enzymes, ionic stabilizers sulfonated oils, etc. which are narrated by Little and Petry (2002). Stabilizers in presence of higher sulfate contents in the soil play a great role in stabilization and limits the applicability of traditional stabilizers.

2.6.3 By-Product Stabilizers

For most of the by-product stabilizers, cat-ion exchange and pozzolanic reactions are the primary processes, like traditional stabilizers. The by-product from Portland cement and Lime are Cement Kiln Dust (CKD) and Lime Kiln Dust (LKD) respectively. Lime Kiln Dust (LKD) generally is composed of 30 to 40 percent lime. The lime is either combined with pozzolans or free lime in the kiln. The fuel used to provide energy is the source of these pozzolans. Cement Kiln Dust (CKD)is obtained as by product from the production of Portland cement. The level of pozzolanic reactivity is supported by the reactive pozzolans generated by Portland cement production. CKD generally contains 20 to 25 percent pozzolanic material and about30 and 40 percent calcium oxide CaO(TRB 144).

2.7 CEMENT STABILIZATION

2.7.1 General

As per TRB reports, since 1915 cement-stabilized soil bases of equivalent 24 ft (7.5m) are provided for more than 100,000 mile roads. It has been found that cement stabilization is effective for clays, silts and sandy materials.

There are generally two classes of cement-stabilized materials one is cement modified soil and other is soil-cement. Soil-cement is produced by mixture of soil, aggregates, Portland cement and water compacted at more density. The quantity of cement added is higher enough to produce a strong and durable subbase for rigid pavements or structural base layer for flexible pavements. Recycled flexible pavements and cement-treated aggregate base are considered soil-cement products. Cement-modified soil is a mixture that has been treated with a comparatively small proportion of Portland cement. Therefore the chemical properties of mixture cannot be altered by the modification (TRB).

2.7.2 Mechanism of Cement Stabilization

This contains mainly hydration process which starts when cement, soil and water are mixed to achieve hardened product. The cement hardening (setting) will make soil as glue, but the structure of soil remains same (EuroSoilStab, 2002). Cement hydration with a series of unknown chemical reactions is a complex process (MacLaren and White, 2003). Governing factors can from following;

- Type of additive
- Curing temperature
- Specific surface of the mixture
- Water-cement ratio
- Presence of foreign matters or impurities

Gain in strength of cement stabilized soil and the ultimate effect on setting may vary depending on factors involved. Therefore, during mix design this should be taken into account to achieve the desired strength. Two main cementitious properties responsible for strength development are calcium silicates, *C3S* and *C2S* respectively (Al-Tabbaa and Perera, 2005; EuroSoilStab, 2002). Portland cement hydration reaction produces calcium hydroxide that reacts with available pozzolanic materials which results cementing material (Sherwood, 1993). Presence of cement is enough to alter the soil properties which improves the intensity of cat-ion exchange. The properties of cement stabilized soil are as follow:

• Cohesiveness reduction

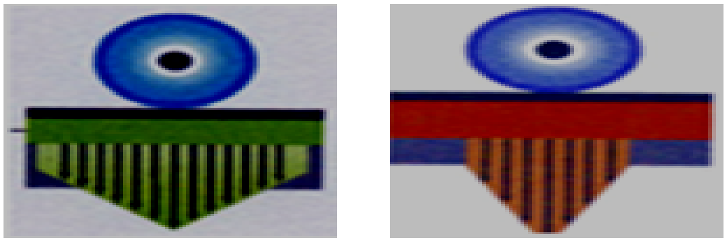
• Increased compressibility

• Strength gain

2.7.3 Engineering Benefits of Cement Stabilization

2.7.3.1 Stiffness

Soil-cement offers higher stiffness as compared with unstabilized subgrade. Due to higher stiffness soil-cement transfer loads on a broad area, therefore reduced stresses on subgrade and the original grade is maintained for longer time with no resurfacing or costly repair. PCA (2005)



Cement Stabilized Base **Unstabilized Base**

Figure 2.1: Stabilized Vs Unstabilized Base

The better stiffness make soil-cement as rut resistant. When exposed to water it does not get softened. With a stabilized subgrade, rutting and settlements are confined to the asphaltic layers of pavement and are relatively easy and less costly to correct.

2.7.3.2 Improved Strength

Soil cement base gains strength gradually with time. Because the hydration process continues for several years and that's why soil cement becomes stronger and "reserve" strength. The soil cement provides load distribution on a wider area due to its slab like characteristics. Strong soil-cement having tendency to bear cyclic load and freeze-thaw damage. PCA (2005).

2.7.3.3 Superior Performance

Different types of soil-cement mixtures designed for specific pavement applications have achieved superior performance since 70 years of diverse experience. The application of soil-cement pavement lengthening thousands of miles in all Canadian provinces and in the every state of United States of America, providing excellent service at low preservation costs.

The higher strength of soil-cement mixtures produces smaller deflections in pavement structures, eventually reduce asphalt strains and therefore better fatigue life is achieved for asphalt concrete. Fatigue cracking a well known flexible pavement distress is reduced by the application of soil-cement.

2.8 LIME STABILIZATION

2.8.1 General

Treatment of natural soil with lime is an advanced technology which has been applicable in highway construction for several years, successfully used for improving engineering properties of weak soil. Lime treatment proves to have long term improvement in compressive strength and CBR of subgrade soil (ASTM D 1883-94). Construction of roads, airports and railways involves volumes of material and lime utilization is the option that have a profound impact on surroundings, particularly preservation of natural resources, such as mineral aggregates also limiting the transport of material to construction sites and dumping of waste material such as unsuitable soil is used for construction purposes. In this framework, the main objective of the project management is the proper planning of earth work that allows one to govern utilization of supply material for capping layer and for embankment also awareness about environmental issues (Celauro B. & Celauro C. (2011). In fact, due to the long-term increase in engineering properties of the subgrade, the pavement designer must consider the strength of lime-stabilized soil for subsequent layer thicknesses (Little, D.L. &Shafee F.A.M.Y.(2001).

Lime is the most common and oldest stabilizer being used in construction works. Lime treatment is more beneficial for fine grained soil and high-plasticity clays. The lime application is a very common method for soil stabilization or modification for airfields, roadways and parking lots. Fig. 2.2 elaborates the effectiveness of lime treatment for pavement structure design. The increase in subgrade CBR form 8% to 15% of an interstate highway in Pennsylvania, yielded a 20% overall project cost saving (CARMEUSE 2002). The layer thicknesses reduced while increase in CBR value, therefore the construction cost reduced.

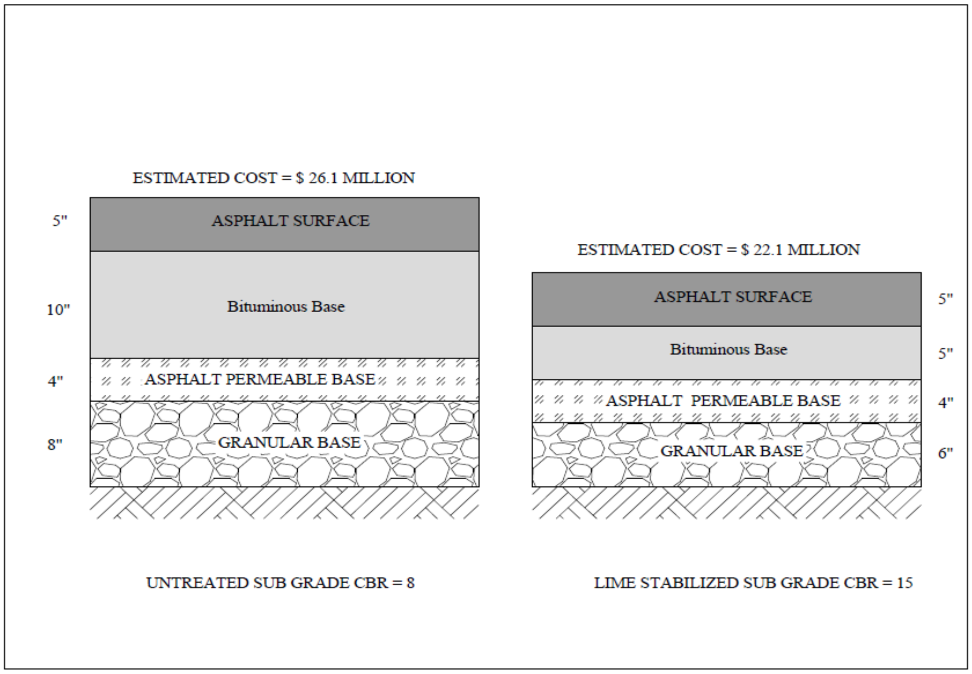


Figure 2.2: Reduction in thickness by lime treatment (Adopted from CARMEUSE 2002)

2.8.2 Soil-Lime Reactions

Most of the soils contain a sufficient amount of alumina and silica, addition of hydrated lime ($Ca(OH)_2$ or quick lime (CaO) and required quantity of water, produces a chemical reaction which ultimately formed cementitious products. The reactions between lime and soil have been explained by various theories. One theory suggests that when lime is adsorbed on the clay surface and cementitious products are formed when reacting with other surfaces. According to another theory, under a high pH of 12.4, the clay lattices released the alumina and silica, both of them react with calcium of lime. Still another theory put forwards that a cementitious network formed when lime directly react with clay edges. Winterkorn, H.F. and Pamukcu, S. (1991). The soil-lime chemical reactions are explained below:

<u>2.8.2.1 Cat-Ion Exchange</u>

A surplus of Ca^{++} cat-ions are created during this reaction, which have a tendency to substitute monovalent cat-ions (Na^{+} or H^{+}). When this cat-ion exchange process occurs in clayey soil it becomes less susceptible to moisture (i.e., in term of volume change it becomes more stable). This is a speedy reaction which occurs immediately when the soil being stabilized is mixed with lime.

2.8.2.2 Flocculation-Agglomeration

In this reaction, lumping together of smaller particles occurs to create the apparent larger particles. This results in a variation in gradation and texture, making the soil very friable and change its behavior like sand. Flocculation-agglomeration is also a quick and immediate reaction it occurs very quickly.

2.8.2.3 Pozzolanic Reactions

The pozzolanic reaction depends on the amount of Ca^{++} available and the clay mineralogy, to replace Alumina and Silica of clay, following the initial reactions do take place. The strength gain due to pozzolanic reaction is long term which continue for years. Pozzolanic reaction also promote autogenously healing that results in strength regain after loss of strength like thaw weakening. The pozzolanic reaction and formation of cementitious products is illustrated in figure 2.3 below.

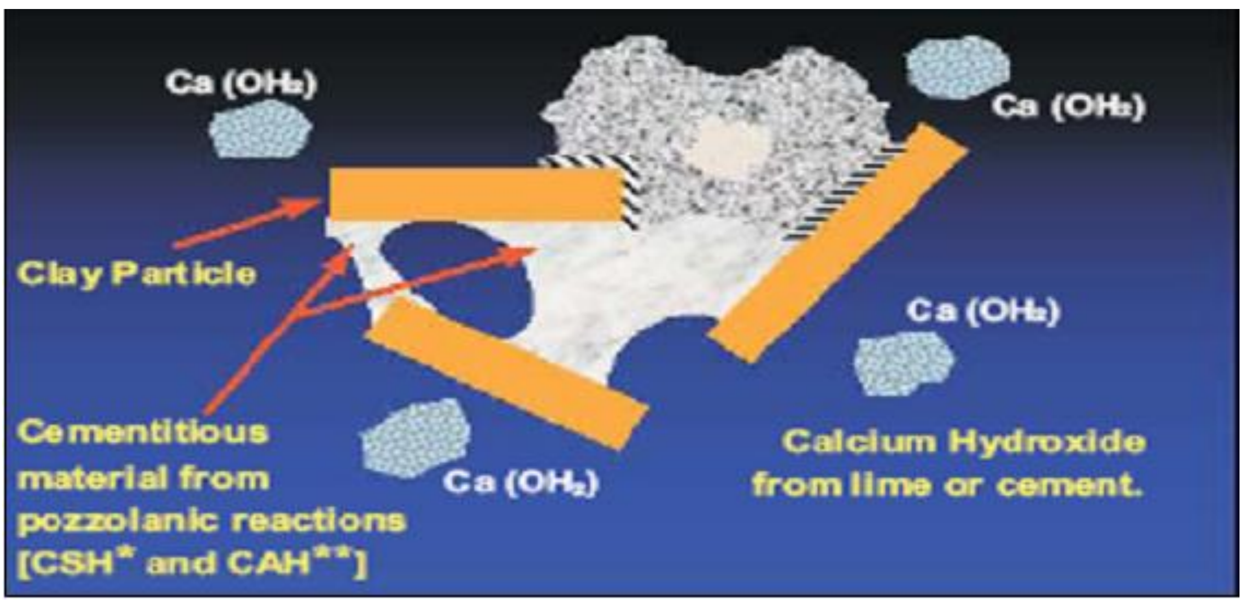


Figure 2.3 : Formation of cementitious products by pozzolanic reaction

2.8.2.4 Carbonation

This reaction when there is a shortage of reactive pozzolanic clay or if the quantity of lime added is more than what is required for stabilization (Winterkorn, H.F. and Pamukcu, S. (1991).The carbonation reaction produced a plastic material Calcium Carbonate ($CaCO_3$) that enhances the plasticity of the soil. Therefore it is very important to ascertain the clay mineralogy and the required amount of lime before the actual work being start.

2.8.3 Feasibility of Soil for Stabilization with Lime

2.8.3.1 Initial Soil Evaluation

According to NLA (2006), if a soil have at least 25% passing No. 200 sieve and its plasticity index is 10 or more, then it is considered as suitable for lime stabilization.

2.8.3.2 Optimum Lime Demand For Stabilization

A pH of 12.4 is best suitable for soil lime reactions. The optimum amount of lime is that percentage which produces a laboratory pH of 12.4 (NLA 2006).

2.8.4 Effects Of Lime On Soil Properties

2.8.4.1 Atterberg's Limits

Plastic limit of lime treated soil increases which reduces its plasticity index, however, the liquid limit may vary (decrease or increase) depending on the type of soil. (Hausmann 1990). Plasticity index can be reduced to an extent that the treated soil becomes non-plastic (Little et al. 1987).

2.8.4.2 Moisture–Density Relationship

Lime decreases the maximum dry density (MDD) while at the same time increases the optimum moisture content of soil specimen at a given compaction efforts. The reduction in MDD is because of the cementation and Flocculation which makes the compaction difficult. Generally OMC increases by 2 to 4% while MDD reduces by 3 to 5 pcf (Hausmann 1990). Lime effects on OMC and MDD are shown in figure 2.4 .

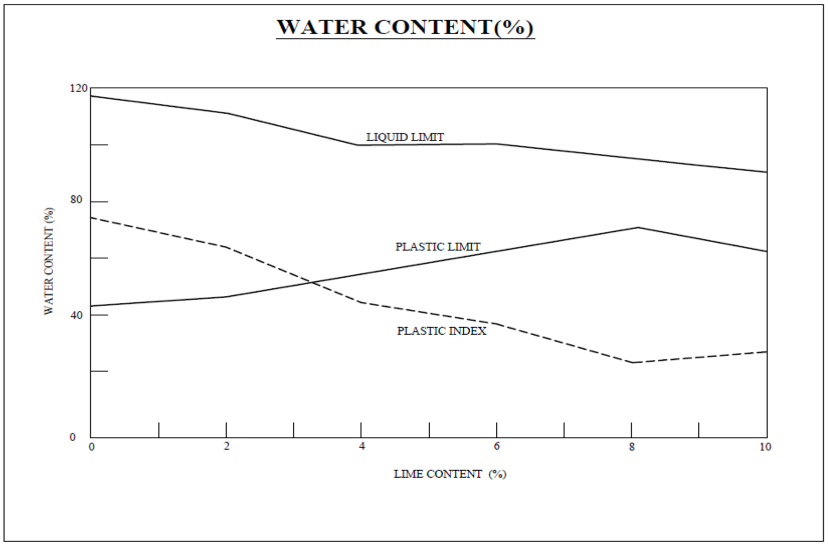


Figure 2.4: Effect of lime on Atterberg's limits

2.8.4.3 Grain Size Distribution

The fraction passing No. 40sieve decreases due to agglomeration thus the lime treated soil becomes coarser in gradation than the original untreated soil. (Winterkorn et al. 1991). The workability of soil improved by these textural changes.

2.8.4.4 Swell Potential

The lime treatment significantly reduces the swell potential of the soil. Little et al. 1987 have concluded that for most of the soil swell values from CBR swell test after 96 hours soaking are less than 0.1%.

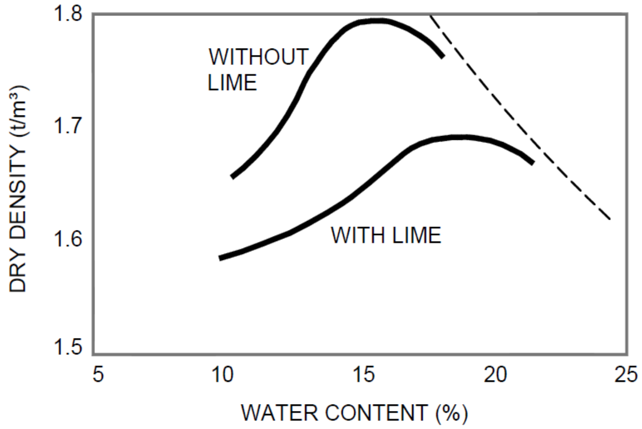


Figure 2.5: Effects of lime on moisture-density relationship (Adopted from Hausmann, 1990)

2.8.4.5 Unconfined Compressive Strength (UCS)

The lime treatment significantly improves the UCS of soil. The gain in strength depends upon a number of variables, curing period is one of that which promotes the strength rise. Little et al. 1987 analyzed that many lime treated soils achieved a strength increase of more than 100 psi for a curing period of 28 days.

2.8.4.6 California Bearing Ratio (CBR)

Like other basic properties CBR is also greatly enhances by lime treatment. For many instances CBR test is used to compare the stabilized and unstabilized soil with and without curing. The CBR test is performed for specific moisture and density usually after a soaking period of four days. CBR values of a treated soil are used to determine the needs for subgrade stabilization and overall pavement layer thicknesses above the subgrade. The other strength tests like resilient modulus stiffness and dynamic cone penetrometer testing can be compared with Lab CBR results. (Beeghly 2003).

2.8.4.7 Deformation Properties

To analyze the behavior of pavement structure stress–strain properties are very essential. An increase in failure stress and decrease in ultimate strain developed in lime treated soil as compared with natural soil. Regardless of the soil type the ultimate strain of a lime stabilized soil is about 1% (Little et al. 1987).

2.8.4.8 Curing Conditions

Moisture, temperature and time are the three important factors for strength gain of lime stabilized soils and these must be controlled strictly. Various agencies have recommended accelerated curing at for different curing periods at different temperatures. NLA (2006) recommends curing of soil lime samples at 40^{o}C. According to ASTM D 5102, curing temperature must below 48.9^{o}C and for a desired period accelerated curing is to be done at 40.6^{o}C.For preventing carbonation of lime and loss of moisture, specimens must be sealed. For this purpose steel metal cans or plastic bags can be used (Little et al. 1987).

2.8.5 Optimum Lime Content

For lime stabilization assessment of optimum lime contents to ensure the optimal strength development is the primary step. Eades and Grim pH test is most commonly used for this purpose. Lime used in the pH test be stored in a way to prevent carbonation and for consistency in test results. The lime whether it is CaO or $Ca(OH)_2$, must meet the purity requirements following AASHTO M 216 (ASTM C 977). The design pH of 12.45 at 25^{o} C is determined according to ASTM D 6276. To establish the proportion of lime needed for required lime-soil reactions provision of substantial volume of calcium will be making pH high resulting good pozzolanic reactions. Based on strength testing the optimum lime contents must be validated.

2.9 FACTORS AFFECTING THE STRENGTH OF STABILIZED SOIL

Undesirable strength of stabilized material is obtained when soil contains enough amount of sulphates, sulphides, organic matters, and carbon dioxide. (Sherwood, 1993, Netterberg and Paige-Green, 1984).

2.9.1 Organic Matter

The upper most layers of soil constitute enough amount of organic matters, in many cases. But concentration of organic matter may be 1.5m deep for well drained soil. (Sherwood, 1993). The reaction between soil organic matter and calcium hydroxide results into lower pH value. Hydration process may be slowed due to low pH which may decrease hardening of soil under stabilization with lesser compact.

2.9.2 Sulphates

Calcium based stabilizers in sulphate rich soils in the presence of moisture, sulphate rich soil will react to produce thamausite and or, calcium sulphoaluminate. This resulting product covers greater volume than the whole volume of reactants (Sherwood, 1993, Little and Nair, 2009;).

2.9.3 Sulphides

In many of industrial by-product and waste materials, sulphides or iron pyrites (FeS_2) may be present. Sulphuric acid is produced by oxidation of FeS2, which react with calcium carbonate and may form hydrated calcium sulphate (gypsum), as shown in equation 1 and 2.

$$2FeS_2 + 2H_2O + 7O_2 = 2FeSO_4 + 2H_2SO_4 \qquad (1)$$

$$CaCO_3 + H_2SO_4 + H_2O = CaSO_4.2\,H_2O + CO_2 \qquad (2)$$

Calcium sulphate if hydrated may react with the stabilized material containing sulphates will lower down pH value (Sherwood, 1993). Even so, the natural soil can also contain gypsum (Little and Nair, 2009).

2.9.4 Compaction

Addition of additive for densification of soil is of quite significant. For a given degree of compaction dry density of stabilized mixture is higher than that of unstabilized soil. The increasing additive quantity also increase the optimum moisture contents (Sherwood, 1993). Hardening of soil mix occur during this process which means that it is needed to compact mixture as early as possible. Hardening of stabilized mixture may results if there is any delay in compaction and more effort will have to exert for desired results. Which may lead to loss of strength and hence serious bond breakage may occur (Sherwood, 1993). Whereas delay in compaction is advantageous to lime-treated soils. The diffusion of lime through the soil require mellowing period thus maximize the effect on plasticity. Lime treated soil may be remixed after mellowing period, and provided its final compaction which results into a significant strength than otherwise (Sherwood, 1993).

2.9.5 Moisture Content

For efficient compaction and hydration process in stabilized soils, sufficient amount of moisture content is necessary (Sherwood, 1993; Roger et al, 1993). The binders compete with soils in order to attain this moisture content if the moisture content is insufficient. For organic soil, clay and peat having higher soil-water attraction, slowed hydration process may be result of lack of moisture content, affecting strength.

2.9.6 Temperature

Temperature changes greatly affect the pozzolanic reaction. Throughout the day, temperature varies continuously in the field. At low temperature pozzolanic reactions between soil particles and additives will slow down and result into lesser strength of the treated mixture.

Stabilization process in low temperature areas may be suggested during hot season (Maher et al, 1994; Sherwood, 1993).

2.10 BENEFITS OF SOIL STABILIZATION

2.10.1 Substantial Savings

When stabilization of existing subgrade is considered, the costs for excavation of existing soil, its removal from site and its replacement with suitable materials are eliminated. This can result in significant cost saving.

2.10.2 Preparation Of Working Platform In Poor Soils

Soil stabilization may be used to treat unsuitable soils in order to carry on site work for the areas where the weather and climate conditions prevent the site jobs. It is beneficial and cost saving for the owner who have to continue his work without waiting for good weather.

2.10.3 Conservation Of Materials

The areas which are in short supply of suitable material to replace existing material or the remote areas where aggregate import is cost prohibitive, In such circumstances soil stabilization proves to be a cost effective solution. Thus leading to conservation of natural materials.

2.10.4 Additional Material Reduction

In context to pavement design, if the stabilized subgrade provides sufficient CBR value the subsequent layer thicknesses above subgrade, asphalt concrete, base course and subbase course may be reduced. This reduction in layer thicknesses results cost saving to the owner Mukesh A. Patel and Dr. H. S. Patel (2012)

2.11 MODIFIED PROCTOR COMPACTION TEST

The modified proctor test slightly differs in procedure from the standard proctor test; water is added to each soil specimen to bring it to the required moisture content. In a standard four inch mold five layers of the soil are compacted using modified Proctor hammer in accordance to ASTM D 1557 (AASHTO T180). The T180 procedure specifies a 10 pounds hammer and a drop height of 18 inches, which produces 56,000 ft-lbf/ft³ of energy. The compactive effort is increase significantly by heavier hammer and longer drop distance.

This test provides a lucid relationship between the moisture content and dry density of the soil. The particulars of experimental setup are (i) cylindrical metal mould, (ii) rammer (4.5 kg), (iii) detachable base plate, (iv) 5 cm high collar. The compaction process eliminates air voids thus helps in increasing the density. The theory beyond this experiment is that for compaction effort, moisture content in the soil is the controlling factor for dry density. The dry density achieved when almost all the air is driven out and the soil compacted at comparatively high moisture content, is the maximum dry density and this water content is named as optimum moisture content (OMC). In broad, the increase in soil density will improves the most engineering properties of the soil, such as the stiffness, strength, imperviousness and resistance to shrinkage.

Figure 2.6: Equipments for modified proctor test

We can obtain MDD and OMC by plotting the experimental data such as dry density on the ordinate and water content on the abscissa. Relevant formulae are as follows:

$$\text{Wet density(gm/cc)} = \frac{\text{weight of wet soil in mould}}{\text{volume of mould}} \quad (3)$$

$$\text{Moisture content (\%)} = \frac{\text{weight of water}}{\text{weight of dry soil}} * 100 \quad (4)$$

$$\text{Dry density (gm/cc)} = \text{wet density}*(1 + \frac{\text{moisture content}}{100}) \quad (5)$$

2.12 UNCONFINED COMPRESSIVE STRENGTHTEST

"The unconfined compressive strength (q_u) is the load per unit area at which the cylindrical specimen of a cohesive soil falls in compression"**.** For unconfined compression test a cylindrical sample of soil is prepared such that its ends are reasonably smooth and the diameter to length ratio is on the order of two. The soil sample is placed on a metal plate consists of a loading frame; by turning a crank, the level of the bottom plate is raised. A top plate is used to restrained the top of the soil sample, this plate is also attached with a calibrated proving ring. An axial load is applied to the sample as the bottom plate is raised. The readings of force applied to the specimen and the resulting deformation are taken periodically as the load increased gradually to shear the sample. The loading is continued until the deformation crack becomes visible. The measured data are used to determine the strength of the soil specimen and the stress-strain characteristics. Finally, water contents are determined by oven drying the sample. The unconfined compressive strength is the maximum load per unit area. "The unconfined compressive strength (qu) is the compressive stress at which the unconfined cylindrical soil sample fails under simple compressive test". The setup for experiment may consist of dial gauges and compression device for load and deformation measurement. The load is taken corresponding to strain dial gauge readings starting from $\varepsilon = 0.005$ and incrementing by 0.005 at each step.

Figure 2.7: UCS testing equipment

2.13 CBR TEST

The California bearing ratio (CBR) test is generally a penetration test used for evaluating the strength of soil sub grade and base course materials for flexible pavements. The CBR test results are arbitrary that cannot be exactly related to any of the fundamental properties of soil like angel of in eternal friction and cohesion governing the soil strength.

The California Bearing Ratio (CBR) test is a simple strength test that compares the bearing capacity of a material with that of a well-graded crushed stone (thus, a good quality crushed stone material should have a 100 % CBR value). AASHTO 2000 recommends that for evaluating the strength of cohesive material the maximum particle size should be less than 19 mm (0.75 mm). The CBR test is extensively used for the subgrade control and selection of material.

The basic CBR test involves applying load to a small penetration piston at a rate of 1.3 mm (0.05″) per minute and recording the total load at penetrations ranging from 0.64 mm (0.025 in.) up to 7.62 mm (0.300 in.). Figure 1 is a sketch of a typical CBR sample.

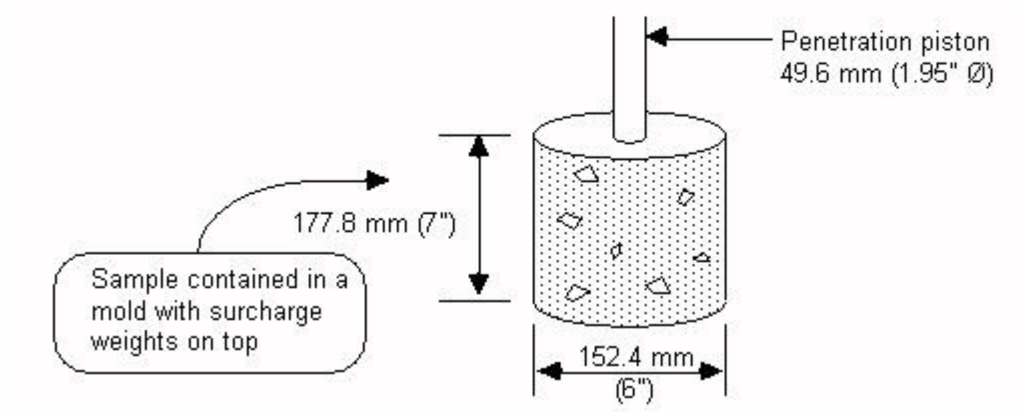


Figure 2.8: Typical sketch of CBR sample

Values obtained are inserted into the following equation to obtain a CBR value:

$$\text{CBR (\%)} = 100^* \left(\frac{x}{y} \right) \qquad (2.6)$$

where: x = material resistance or the unit load on the piston (pressure) for 2.5 mm (0.1 in) or 5.08 mm (0.2 in) of penetration

y =standard unit load (pressure) for well graded crushed stone (1000 psi for 2.5 mm and 1500 psi for 5.08 mm).

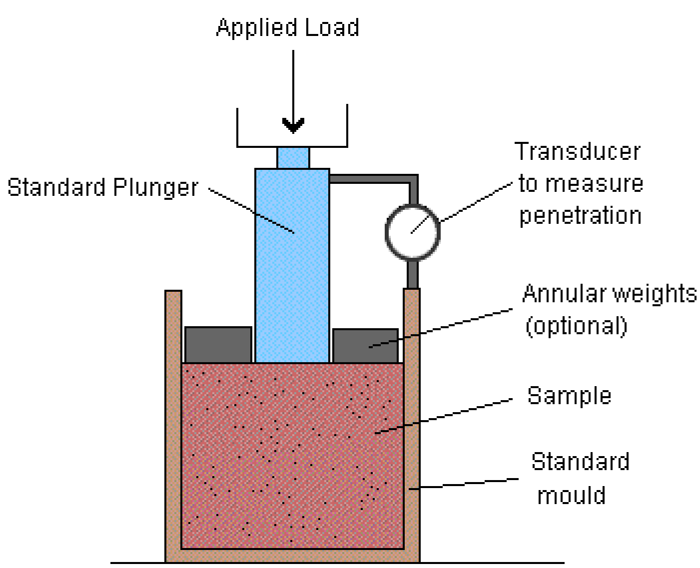


Figure 2.9: Schematic diagram for CBR testing

Usually the CBR value at 2.5 mm (0.1 in) penetration is selected. If the C.B. R. at 5 mm (0.2 in) penetration is greater than at 2. 5 mm then this greater value is adopted to define CBR.

2.13.1 Swell Potential

Swell Potential of a soil specimen is the ratio of the increase in thickness to the original thickness of a soil specimen compacted at OMC in a consolidation ring, soaked under a surcharge load of 7 KPa and is expressed as a percentage.

2.14 SUMMARY

This chapter firstly defines the stabilization, soil properties and agents of stabilization. A description of stabilization mechanism, and different methods of stabilization are included. After this Cement and Lime stabilization is explained in detail. From the literature review it is found that the cement and hydrated lime are most effective stabilizing agents. The engineering properties of subgrade soil are enhanced by the application of these distinctive stabilizers. The strength gain due to pozzolanic reaction in chemical stabilization is explained. The mix design for Cement and Lime stabilization is elaborated precisely. The benefits of stabilization are discussed. At the end, in view to the scope of this particular research work the test involved are summarized. Which includes the procedural steps and usefulness of Modified Proctor Test, California bearing Ratio Test and Unconfined compressive strength test.

CHAPTER 3

RESEARCH METHODOLOGY

3.1 INTRODUCTION

The research methodology adopted for the study including preparation of specimen, testing and analyzing the implication of different evaluated factors. The detailed research methodology is presented in figure 3.1. The engineering properties of soil are evaluated with their respective standards including sieve analysis, Atterberg's limit, soil classification, MDD and OMC. The organic content test and sulfate content test are conducted to check the suitability of soil material for stabilization. Since the amount of organic contents is less than 1 percent, therefore the given soil can be used for lime and cement stabilization. After this the optimum stabilizer contents for cement and lime stabilization are determined. For lime, Eades and Grim pH test is conducted to determine the optimum lime contents. The optimum or efective cement contents are selected at prevailing cost comparison. The proportion of stabilizing agents (lime and cement) is varied as 2%, 4% and 6% for specimen preparation. The optimum moisture contents for cement and lime stabilization are determined at each dosage percentage. Further CBR and UCS test specimen are prepared for control and treated soil mixtures, following their respective test standards. Table 3.1 describes the test matrix developed to fulfill the objective of this research work.

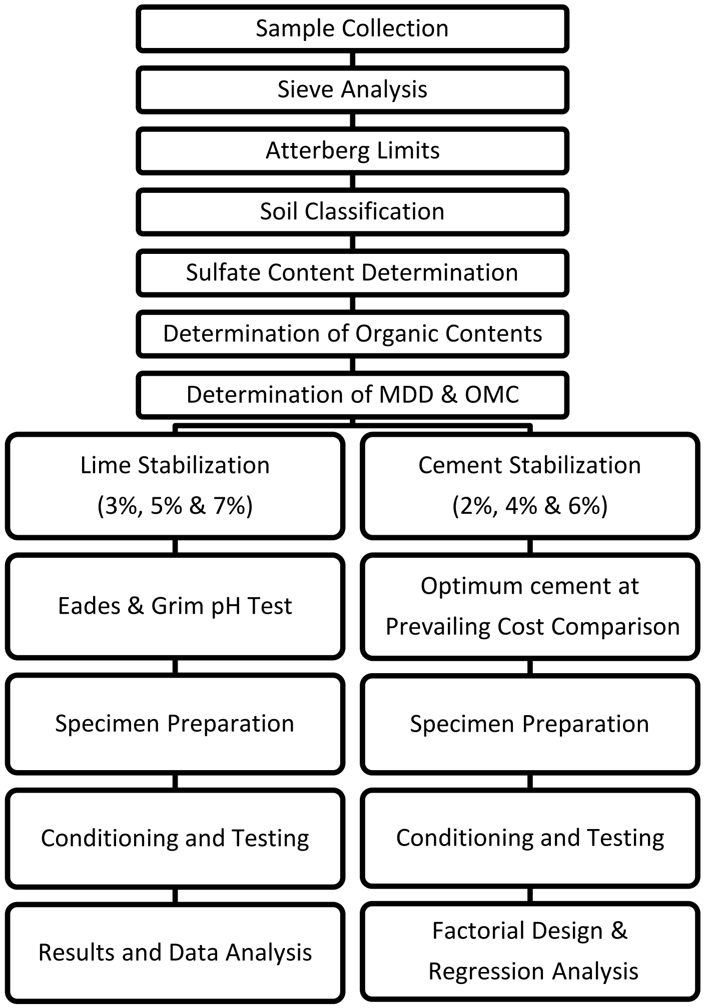


Figure3.1: Research methodology

Table 3.1: Test Matrix

Specimen Type	**Optimum Additive Amount Criteria**	**% of Additive**	**pH Test**	**MDD & OMC**	**CBR Test**		**UCS Test**	
					Unsoaked	**Soaked**	**Unsoaked**	**Soaked**
Control	×	0%	×	√	√	√	√	√
Cement Modified	×	2%	×	√	√	√	√	√
	At Prevailing Cost	4%	×	√	√	√	√	√
	×	6%	×	√	√	√	√	√
Lime Modified	×	3%	×	√	√	√	√	√
	Optimum	5%	√	√	√	√	√	√
	×	7%	×	√	√	√	√	√

3.2 SAMPLE COLLECTION

In this research the subgrade soil sample from Karachi Lahore Motorway (KLM) at Nawabshah site was selected and procured to NIT laboratory. The soil in this particular area is very weak and un-suitable; it cannot be used directly for the pavement construction. Due to poor strength and CBR, cement and lime stabilization are adopted for this soil material.

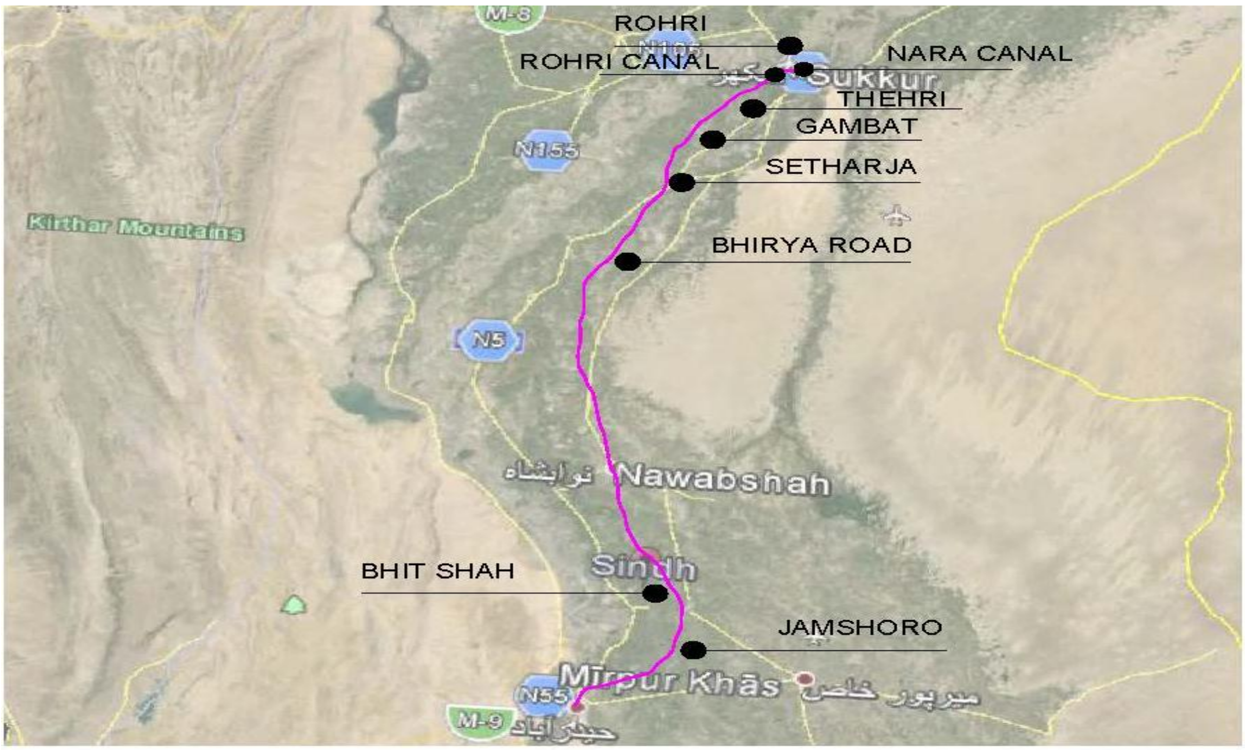


Figure 3.2: Location of soil samples along the proposed road

3.3 EVALUATION OF ENGINEERING PROPERTIES OF SOIL

In phase one of this research work the engineering properties of subgrade soil material were evaluated. The basic tests required for soil characterization were performed according to their respective standards. The soil type and behavior has been determined which leads to further stabilization phase.

3.3.1 Sieve Analysis

The composition of subgrade soil particles at any place is different in sizes and shapes. Permeability, strength and density properties of soil are generally based upon particle distribution analysis. The particle size distribution of soil sample was developed by carrying the sieve analysis and sedimentation analysis for coarse grained and fine grained soil using the standard test method ASTM C 136-96 and ASTM D 422-63 respectively. The results were plotted as a line chart/graph. In which particle size was taken on abscissa and percent passing as ordinate on a logarithmic scale as shown in figure 3.3.

Table 3.2: Gradation of Control Soil Specimen

Sieve No	Percent (%) Passing									
	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6	Sample 7	Sample 8	Sample 9	Sample 10
3/4 in	100	100.0	100	100	100	100	100	100	100	100
1/2 in	97.4	98.1	99.4	99	100	100	99.5	97.5	100	98.2
3/8 in	94.9	95.2	96.8	96.2	98.8	99.5	98.3	95.6	98.6	96.3
No 4	90.2	91.1	91.6	93	96.3	96.3	97.3	93	95.4	92.3
No 10	87.6	88.9	88.4	89	92.3	90.8	93.3	88.5	88.6	88.2
No 40	83.6	84.8	81.2	82.7	87.1	85.8	86.1	77.3	81.4	80.9
No 100	76.6	77.6	70.7	72.2	77.9	83.2	78.6	66.8	72.9	72.6
No 200	63	65.0	55.2	58.1	68.1	71.3	72.4	59.3	65.7	62.4

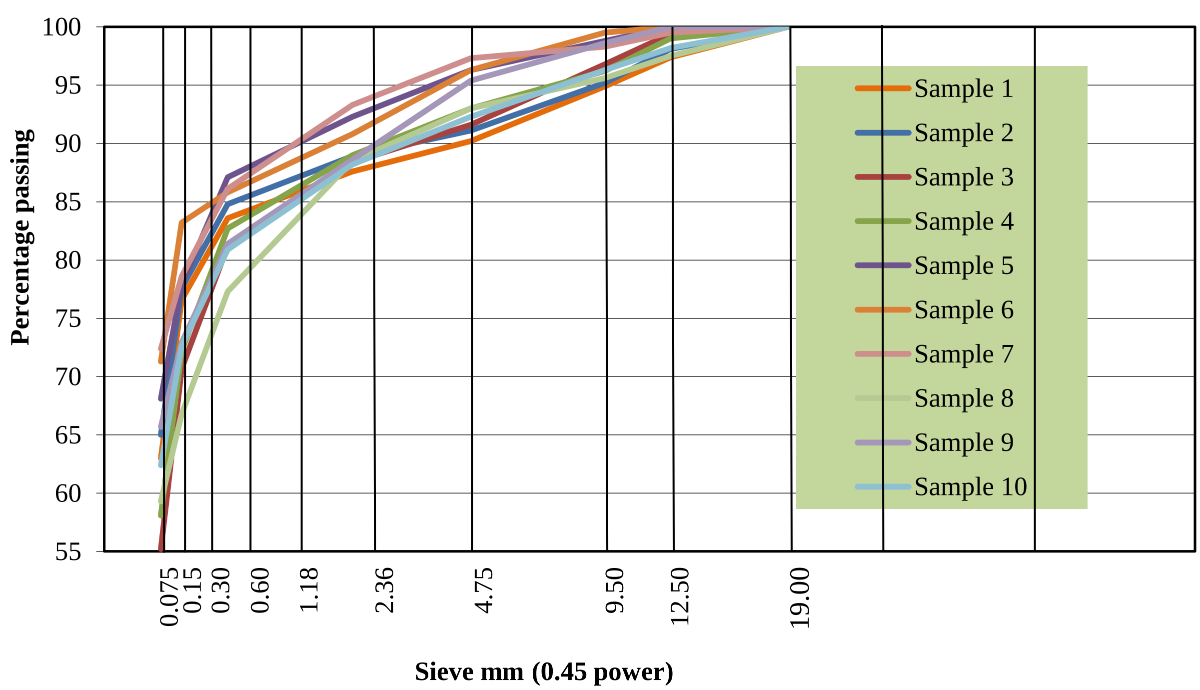


Figure 3.3: Control Soil particle size distribution curves

3.3.2 Atterberg's Limits

The plasticity of any soil is determined by evaluating its index properties, which are mainly liquid limit, plastic limit and shrinkage limit. Liquid limit, plastic limit and plasticity index were found out following standard test method ASTM D 4318.

3.3.2.1 Liquid Limit

Casagrande's apparatus was used for liquid limit determination as shown in figure 3.4. The air dried soil sample was pass through sieve no. 40 and pulverized. It was thoroughly mixed with distilled water until a uniform paste has formed and then placed to the cup. A clean straight groove has been made by grooving tool. The crank of the apparatus was turned at a rate of approximately two drops per second and the number of drops were counted, N, that takes to make the two halves of the soil pat come into contact at the bottom of the groove along a distance of 13 mm (1/2 in.).

Figure 3.4: Casagrande's Apparatus for liquid limit test

The small amount of soil was placed into can, after weighing it was placed into oven for 16 hours and the moisture contents were determined. A graph between number of drops, N, versus the water content (w) was plotted. The best-fit straight line was drawn through the plotted points and the liquid limit (LL) as the water content at 25 drops was determined. The procedure revised by thrice.

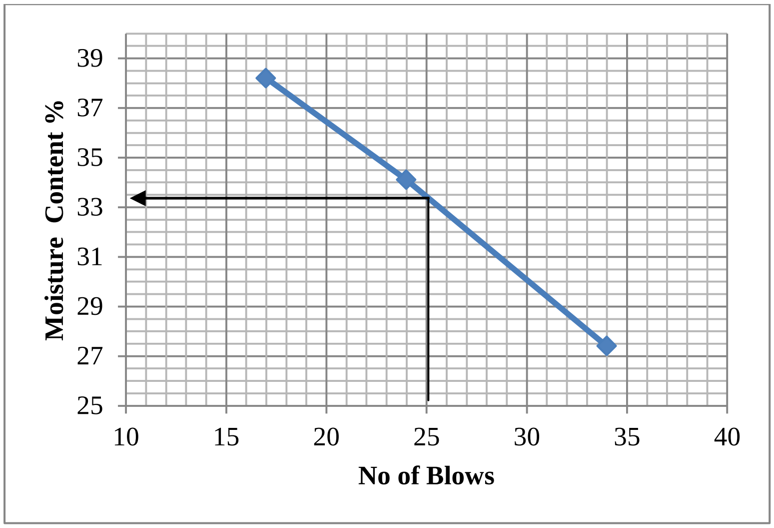


Figure 3.5: Liquid limit at 25 No. of blows

3.3.2.2 Plastic Limit

The remaining 1/4 of the original soil sample was taken and distilled water was added to it until the soil is at a consistency where it can be rolled without sticking to the hands. The soil mass was rolled between the palm or the fingers and the glass plate as shown in figure 3.6. Sufficient pressure was applied during rolling to made the mass into a thread of uniform diameter by using about 90 strokes per minute. The thread was deformed so that its diameter reached 3.2 mm (1/8 in.), then the thread was broken into several pieces. Kneading and reforming the pieces into ellipsoidal masses was carried out and was re-rolled, until the thread crumbled under the pressure required for rolling and can no longer be rolled into a 3.2 mm diameter thread. The portions of the crumbled thread were gathered and soil was placed into a moisture can, immediately weighed and it was placed into oven for 16 hours. Finally the water contents were determined.

Figure 3.6: Sample preparation for Plastic limit

3.3.2.4 Plasticity Index

The plasticity index is a range of moisture in which a soil remains in a plastic state while passing from a semisolid state to liquid state. It is the numerical difference between Liquid Limit and Plastic Limit of a soil. Plasticity index was calculated by following equation 1.

$$PI = \text{Liquid Limit} - \text{Plastic Limit} \quad (1)$$

3.3.3 Soil Classification

In this research AASHTO soil classification system was used for soil classification. As this classification system is based on particle size analysis and consistency limits, which were firstly evaluated. The classification results show that it was A-6 type soil.

3.3.4 Determination Of Sulfate Contents

The water-soluble sulfate content was determined for soil by mixing oven-dried samples with a known amount of water following the standard AASHTO T 290-95. The mixture was then placed in a centrifuge, and the sulfate ion concentration of the supernatant was measured. The sulfate content was then reported as mg of sulfate per kg of dry soil. The value of water soluble sulfate contents was less than 0.3% (3000 ppm). This lies in the acceptable range provided by Transportation Research Board (TRB 2009).

3.3.5 Determination Of Organic Contents

The organic content of soil sample was determined according to ASTM D 2974-00 by firstly a representative sample of soil was oven dried at 105°C for 24 hours, then moisture contents were recorded. The sample was then placed in a muffle furnace, heated to 440°C, then reweighed after a nearly constant mass was achieved. The ash content of the sample was then recorded as the weight loss due to ignition divided by the initial dry weight. The organic content was then calculated as 1 minus the ash content. The value of organic content was 0.75 percent, therefore it is the indication to conduct the further tests.

3.4 LABORATORY PREPARATION OF HYDRATED LIME

Calcium carbonate (CaCO3) in the form of solid stones was procured from local market. It was soaked in water and let hydrate and later cool down to room temperature as shown in Figure 3.7. This lime slurry solidifies and cools down after complete hydration, which is later pulverized and passed through No.200 sieve. This is hydrated lime in powder form, which stored in air tight containers and used as an anti-stripping agent in the preparation of mixes for resilient modulus and dynamic modulus tests.

Figure 3.7: Hydrated lime Prepared in laboratory

3.5 SELECTION OF OPTIMUM STABILIZERS CONTENTS

As the scope of this study indicates the use of cement and hydrated lime for stabilization of weak subgrade soil and the evaluation of basic engineering properties of soil demonstrate that for this particular type soil, cement and hydrated lime are most suitable stabilizing agents according to the criteria of Transportation Research Board (TRB). Therefore the next step of research methodology was the determination of optimum stabilizers contents. Those were evaluated as follow:

3.5.1 Optimum Lime Contents For Lime Stabilization

The standard test method ASTM D 6276 - 99 was used, which basically estimates the optimum lime contents required for soil stabilization by pH value. Five specimen, each of 25 g were obtained from oven-dried soil. Each specimen was placed into dry plastic bottle which was caped tightly. Five representative specimen were obtained; which were representative of 2, 3, 4, 5, and 6% of the equivalent 25-g oven dried soil. The sixth specimen represents a saturated lime solution that contains 2 .0 g of lime. The first five specimen were added to soil specimen in plastic bottles and a 100ml of water was added to each soil lime mixture also to the bottle containing 2.0 g of lime. A 45 minutes shaking period was given and temperature was maintained to 25ºC. After shaking pH of each soil-lime-water and lime-water mixture was determined and recorded. Graph between percent lime contents and pH value is plotted as shown in figure 3.8. The optimum lime contents from this pH test are 5%.

Figure 3.8: Laboratory pH test

3.6 DETERMINATION OF MDD AND OMC

Modified Proctor tests of the control soil specimen as well as treated soil specimen were performed in accordance with ASTM D 1557-02 standard procedure to evaluate the maximum dry density and the optimum moisture content associated with that density. The tests were repeated for control, cement modified and lime modified specimen with three replicates of each. In case of lime stabilization the OMC and MDD were determined at previously obtained optimum lime contents (OLC) by pH test and at 2% above and below of that OLC. While for cement stabilization the OMC and MDD were determined for each cement percentage (2%, 4%, 6%) to be used for stabilization, because hydration losses are prominent at different cement percentages.

The compaction energy of 56000 ft lb/ft^3 was applied by dropping 10 lb hammer from a height of 18 inch in three different layers with 25 number of blows/layer. Soil compacted at relatively high moisture content with almost all the air driven out, maximum dry density (MDD) was achieved, which was its optimum moisture content (OMC). The OMC, MDD and water were plotting as abscissa and dry density as ordinate as shown in figure 3.9.

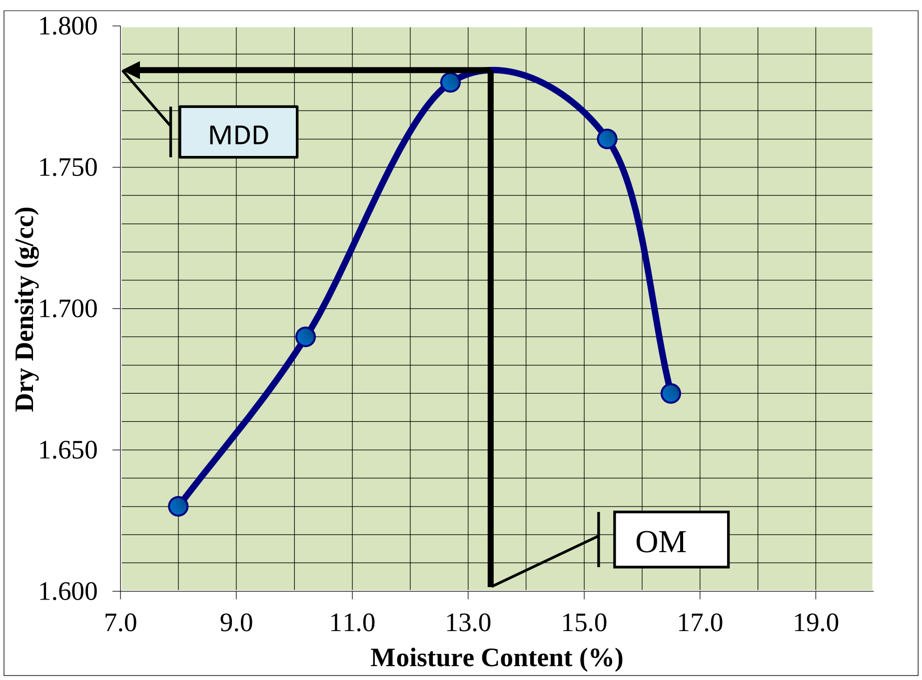


Figure 3.9: MDD Vs. OMC

The equations used in this experiment are as follows:

$$\text{Wet Density} = \frac{\text{weight of wet soil in mould gms}}{\text{volume of mould cc}} \quad (2)$$

$$\text{Moisture content \%} = \left(\frac{\text{weight of water gms}}{\text{weight of dry soil gms}}\right) * 100 \quad (3)$$

$$\text{Dry density } \gamma d \text{ (gm/cc)} = \frac{\text{wet density}}{(1+\text{moisture content})} \quad (4)$$

Table 3.3: MDD and OMC of modified specimen

Specimen Type	Additive (%)	OMC (%)	MDD (g/cm^3)
Cement Treated	2%	10.20	2.26
	4%	9.25	2.41
	6%	8.50	2.63
Lime Treated	3%	11.0	1.92
	5%	15.5	1.75
	7%	12.3	1.85

3.7 DETERMINATION OF CBR

The CBR test was carried out on compacted controlled and treated soil specimen using CBR mould 150 mm in diameter and 175 mm in height, provided with detachable collar of 50 mm and a detachable perforated base plate. The two sets of specimen (Unsoaked and soaked) for CBR testing were prepared according to ASTM D 1883 - 99. A total of 42 specimen were prepared for CBR testing. Generally the specimen were controlled, lime modified and cement modified. All the specimen were compacted at their respective optimum moisture contents determined by standard proctor test for control and varying proportion of stabilizing agents lime and cement (2, 4 and 6%). The Unsoaked set of specimen was tested immediately after compaction, and mould before being placed in the testing machine was inverted. A 4 days (96 hours) soaking was carried out for the soaked set of specimen. A surcharge weight of 4.54 kg was applied during soaking. During soaking a constant water level was maintained. At the end of 96 hours, final swell measurements were taken and the swell as a percentage of the initial height of the specimen was calculated. For 15 minutes the specimen was allowed to drain out downward. As per AASHTO, rate of deformation was .05 inch/per minute. Measurements of test were recorded by two digital displays for displacement and load. Plot of the deformation versus load and CBR at 0.1 inch deformation was established. (In case it was lesser than the CBR at 0.2 inch, CBR at 0.2 inch deformation was also reported).

Figure 3.10: Specimen in CBR testing machine

3.8 UNCONFINED COMPRESSIVE STRENGTH DETERMINATION

3.8.1 Specimen Preparation

A total of 42 specimen (6 control, 18 lime stabilized and 18 cement stabilized, at different additive percentages) were prepared for UCS test at OMC. Control soil sample was prepared in a way that it was air dried, pulverized, mixed and oven dried. The standard test method ASTM D 5102-96 procedure A was followed for soil lime mixture preparation. The specimen height to diameter ratio was 2. For lime stabilization the oven dried soil and lime (at varying proportion 3, 5 and 7%) were mixed in dry state and OMC added as determined by modified proctor test. Before compaction soil, lime, and water were mixed, sealed in a container and mellowed for one hour.

For cement stabilization the oven dried natural soil and Portland cement at different proportions (2, 4 and 6%) were mixed in dry state then specimen prepared at optimum water contents with respect to each proportion of cement content following the standard ASTM D 1632 - 96. ASTM D 1633 procedure B was adopted for soil cement mixture testing. The specimen height to diameter ratio was 2. In this case no mellowing period was required.

3.8.2 Soil-Lime Specimen Curing

After compaction, specimen sealed in plastic bag and placed in a 23°C oven according to ASTM D 5102-96 for 48 hours as illustrated in figure 3.11.

Figure 3.11: Curing modified specimen of lime

3.8.3 Compaction of Specimen

Each specimen was compacted in a 3 in. diameter by 6 in. high mold for both lime and cement stabilization. The cylindrical mold was filled in five equal layers, with each layer receiving 20 blows of a 10-lb hammer falling from a height of 18 inches. For this specimen size, the compaction effort applied was equivalent to that of the Modified Proctor compaction (ASTM D 1557-02). Once the compaction was completed, the specimen were extracted from the molds, trimmed and weighed. A moisture sample was taken before and after the compaction was complete. The specimen was also weighed at this point.

3.8.4 Soil-Lime Specimen Soaking

The samples, after the desired curing period, were wrapped in a porous cloth and placed over a porous stone in a container. The container was filled with water till the top of porous stones. As required by NLA (2006) design procedure, direct contact of sample with water was avoided. The samples were allowed to soak for 24 hours and then tested. In this case also, three samples were tested and their average strength was reported. Soaking arrangements are shown in Fig 3.12.

Figure 3.12: Soil lime mixture soaking

3.8.5 Curing of Soil-Cement Specimen

The 12 hour curing of cement modified specimen in a humid room in the mold was carried out according to ASTM D 1632, immediately after compaction has been completed.

Figure 3.13: Curing of soil cement mixtures

3.8.6 Soaking of Soil-Cement Specimen

Cement treated/stabilized samples were submerged in the water bath for approximately 3 to 4 hour (ASTM D 1633-00) prior to testing as shown in figure 3.14.

Figure 3.14: Soaking of soil cement mixtures

3.8.7 Testing of Specimen

The controlled soil samples were tested immediately after compaction. ASTM D 5102 procedure A was followed for lime modified specimen (Unsoaked and soaked) while ASTM D 1633 for cement treated specimen (Unsoaked and soaked). **S**pecimen were tested at a constant rate of .05 inches per minute until failure (Figure 3.15). For each test a load versus deformation plot was obtained. Ultimate load was recorded, for the average of the specimen as reported to find the unconfined compressive strength. After the test, sample for moisture from each specimen was also taken.

Figure 3.15: Unconfined Compressive Strength Tester

3.9 SUMMARY

This chapter is divided into three parts; in the first soil material characterization in explained. In which the engineering properties of control soil specimen has been evaluated and the soil material type according to AASHTO classification has been established. The basic tests for suitability of stabilizer to be used with soil were elaborated. That were sulfate content test and organic content test and the selection of cement and lime for that particular soil was identified. In the next portion the optimum amount of stabilizer agent were determined. The optimum amount of lime was selected based on Eades and Grim pH test and the percentage of which is obtained as 5%. The amount of cement was selected at prevailing cost comparison made between the convention method stabilization of soil to the other conventional methods applicable in the field. The optimum amount of cement came as 4% of the dry weight of soil. The percentage of lime was taken as 3%, 5% and 7%. While the proportion of cement was selected as 2%, 4% and 6% for further testing. In third part the OMC and MDD were determined and specimen for California bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) were prepared, conditioned and were tested by their respective standards.

CHAPTER 4

RESULTS AND ANALYSIS OF EXPERIMENTAL DATA

4.1 INTRODUCTION

The basic properties of ten control soil specimen are presented In the beginning. The National Highway Authority (NHA) and Transportation Research Board (TRB) criterion for stabilization are discussed and the suitability of soil for lime and cement stabilization is checked. The results of lime stabilization of soil are discussed in detail. Which includes the selection of optimum lime contents by Eades and Grim test, the strength tests UCS and CBR on varying proportion of lime contents. The gain in UCS and CBR due to lime stabilization is elaborated by tables and charts. In the further part cement stabilization of subject soil is discussed. The effects of cement stabilization for that particular soil are plotted in charts and tabular form to indicate the benefits of cement stabilization. After this the analysis of UCS data is carried out using Minitab 15 software for cement stabilization. At the end cost analysis is made to amass the benefits of stabilization.

4.2 EVALUATION OF BASIC PROPERTIES OF CONTROL SOIL SPECIMEN

Ten subgrade soil samples from Nawabshah (Sindh) site were taken at a spacing of 500 m along the road site and were brought to laboratory for testing. The basic properties of subgrade soil material; particle size distribution, Atterberg's limits, optimum moisture contents, maximum dry density, soaked CBR and swelling potential were evaluated to characterize the subgrade soil. Nature, type, pattern and behavior of the soil was determined from these basic properties. Table 4.1 summarizes these basic properties of control soil material.

Table 4.1: Summary of basic properties for ten control soil samples

Sr No.	%Passing No 200 Sieve (Clay fraction)	Natural Water Content w (%)	Max Dry Density γ (g/cm^3)	OMC (%)	Liquid limit $w_{L(\%)}$	Plastic Limit $w_{p\,(\%)}$	Plastic Index I_p	CBR (%)	Swelling Ratio δ_{ef} (%)
1	55.0	12.2	1.78	13.50	35.5	18.3	17.2	2.5	2.6
2	58.0	11.6	1.72	13.00	34.4	15.9	18.5	2.6	2.5
3	68.0	12.5	1.75	14.80	36.8	15.7	21.1	3.4	2.1
4	71.0	10.6	1.73	14.40	38.0	16.5	21.5	2.4	2.8
5	63.0	11.4	1.81	14.60	37.5	17.9	19.6	2.0	3.1
6	72.0	10.8	1.74	12.20	36.5	17.3	19.2	3.3	2.1
7	59.0	12.5	1.81	15.10	33.5	13.8	19.7	3.0	2.7
8	52.0	10.7	1.85	13.60	39	25.3	13.7	3.9	2.7
9	65.0	11.9	1.84	14.20	33.0	16.4	16.6	2.5	2.9
10	62.0	12.8	1.78	14.60	36.0	19.1	17.0	2.8	3.2

Table 4.2: Physical / Chemical properties of Control Soil specimen

Plasticity Index	17.5
Percent Passing Sieve No. 200	63
Soil pH	11
Organic Content, (%)	0.75
Sulfate Contents, (%)	0.22

Preparation of subgrade is very important in pavement design. Subgrade material quality and strength is checked out and maintained in the field by the agency standards. In this particular research National Highway Authority (NHA) general specifications and Transportation Research Board (TRB) Criterion were followed to select the soil material.

4.3 CRITERION FOR STABILIZERS

As the scope of this research work includes the stabilization of unsuitable soil with cement and hydrated lime. The selection of cement and hydrated lime for stabilization was verified by National Highway Authority (NHA) and Transportation Research Board (TRB) standard criteria's of soil stabilization. According to item 111.2.2 and 112.2.1 of NHA general specifications, the properties of soil for cement and lime stabilization are summarized in figure 4.1. Figure 4.2 presents the TRB criteria for the selection of stabilizers.

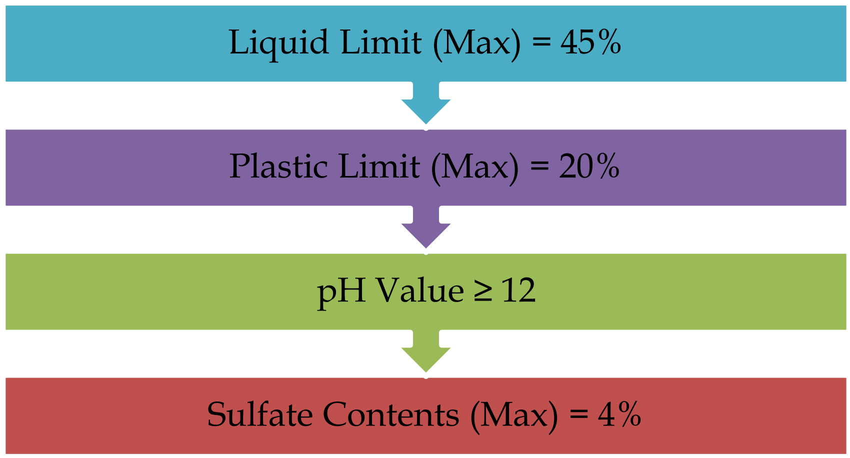


Figure 4.1: Properties of Soil for Cement and Lime Stabilization (NHA Specifications)

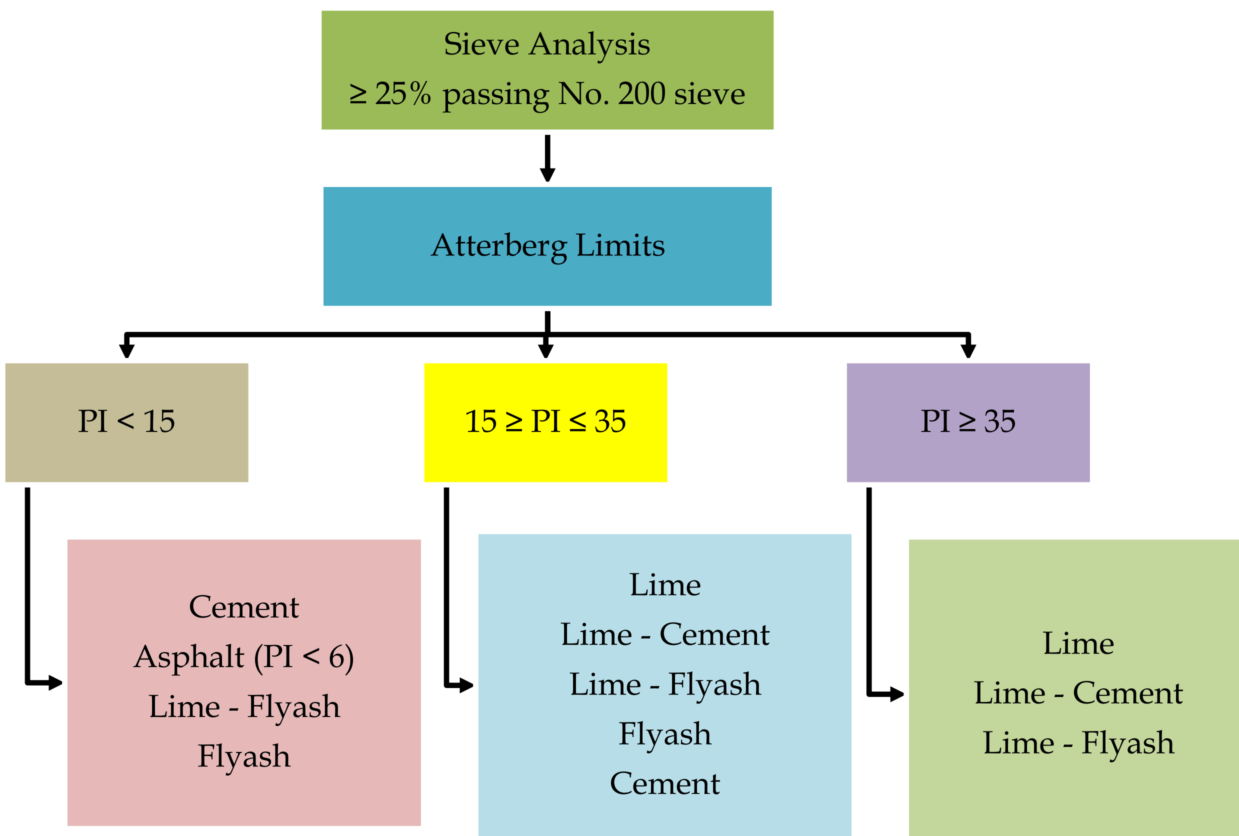


Figure 4.2: TRB criteria for stabilizer selection

4.4 LIME STABILIZATION OF SUBJECT SOIL

Determination of optimum lime contents was the requirement for lime stabilization. Therefore Eades and Grim pH test was conducted to find out the optimum lime contents. After which the CBR and UCS test were performed. The results are presented below:

4.4.1 Optimum Lime Content For Subject Soil

The results of Eades and Grim pH test indicates that the 5 % lime content is the optimum value for lime stabilization. Figure 4.3 elaborates the optimum lime contents against the pH value of 12.4. Not any significant change in pH value was observed beyond 5% lime contents. The validation of optimum lime contents was carried out by the conduction of UCS test at variable proportions of lime. After pH test specimens prepared for proctor test to determine the OMC and MDD at variable proportions of lime (3%, 4%, 5%, 6% and 7%). The behavior of lime with the soil is such that it decreases the maximum dry density while at the same time it increases the OMC of the soil as evident from figure 4.4 and 4.5. It is concluded that the amount of lime that giving the maximum reduction in dry density while at the same time maximum increase in optimum moisture content is the effective dose for lime stabilization.

Maximum OMC of 15.5 percent was obtained at 5% lime that was about 42 % rise from untreated soil OMC of 10.2 %.

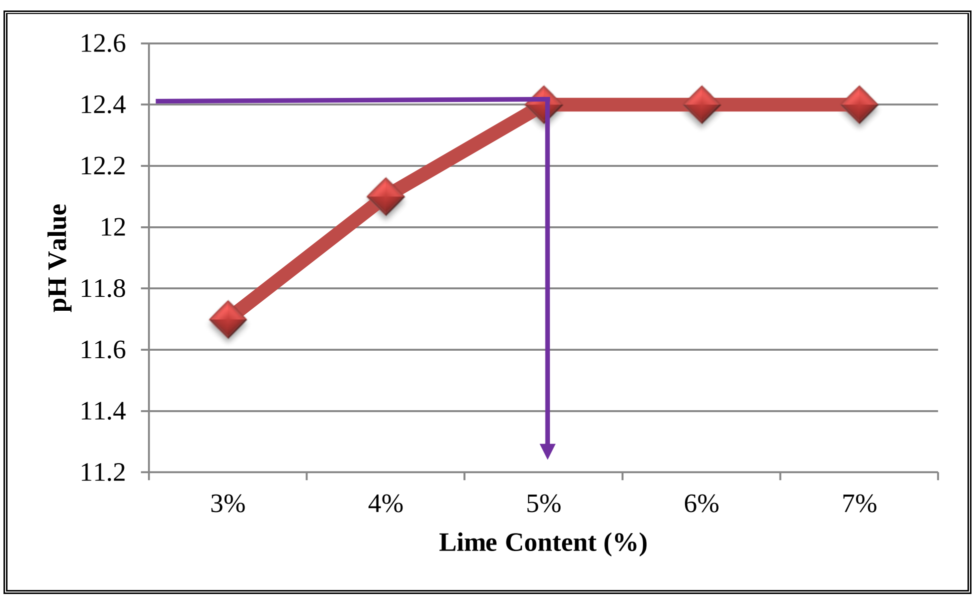


Figure 4.3: Optimum Lime contents by pH test

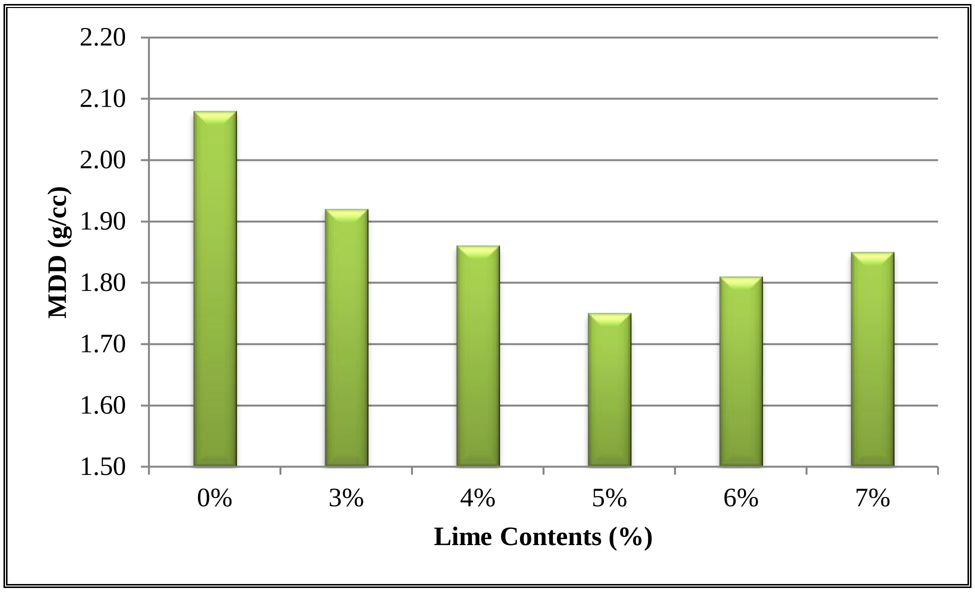


Figure 4.4: MDD at varying proportions of Lime

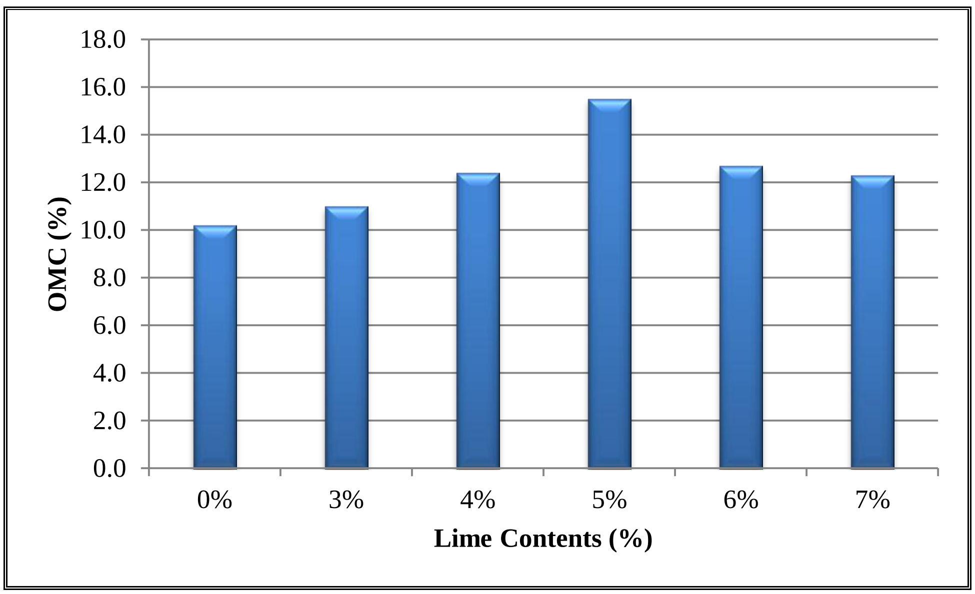


Figure 4.5: Effect of Lime on OMC

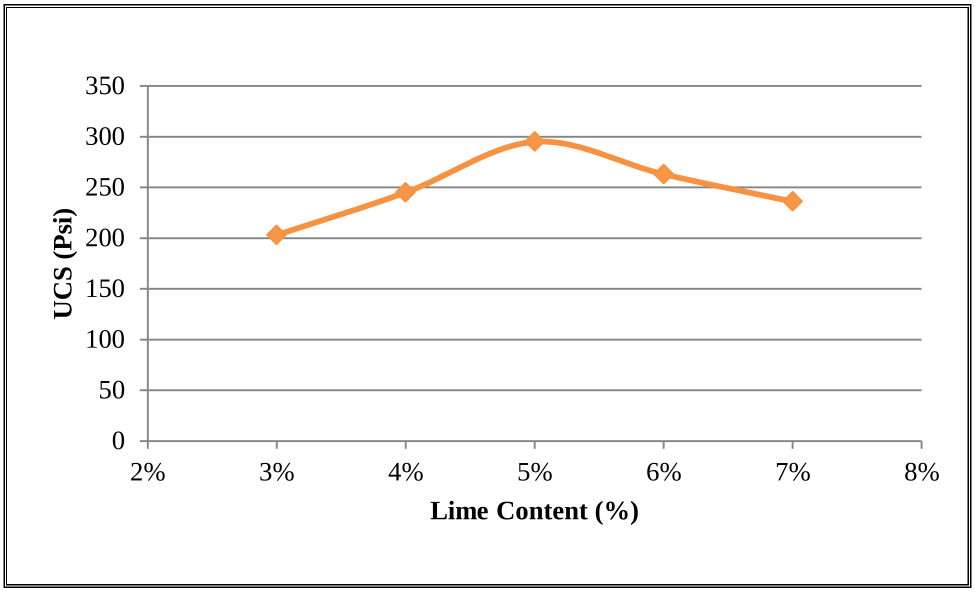


Figure 4.6: UCS at different lime contents

Figure 4.6 elaborates the UCS at varying proportion of lime contents. UCS of treated soil was maximum at 5% lime content that was 295 psi. It was confirmed from all these tests that 5% lime content is the optimum value of lime for soil lime stabilization.

4.4.2 Strength Gain Due to Lime Stabilization of the Subject Soil

The average values of UCS test for lime stabilization are presented in table 4.3. Fig. 4.7 shows the variation in UCS with respect to percent Lime Contents. It can be seen that the increase in strength is maximum at 5 % lime contents which was the optimum amount.

Table 4.3 : UCS of lime treated mixtures

% lime	UCS (Psi)	
	Unsoaked	Soaked
0%	41	22
3%	203	125
5%	295	192
7%	236	145

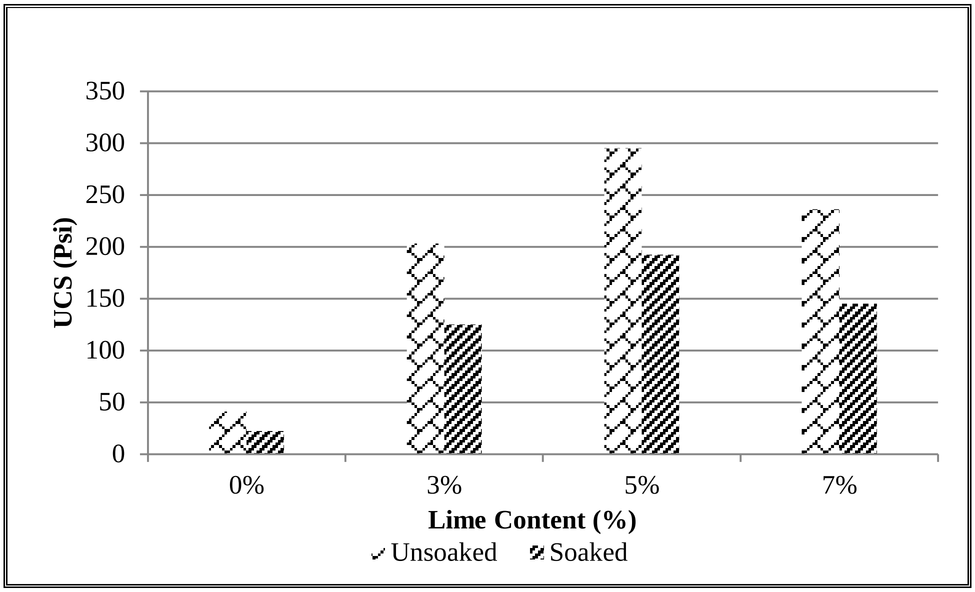


Figure 4.7: Soaked and Unsoaked UCS at different lime contents

4.4.3 CBR of Lime Treated Soil

The results of CBR test at varying proportion of soil-lime mixtures are presented in table 4.5. CBR test indicates that by the addition of lime the CBR value of specimens significantly increased as compared with control specimen. The maximum CBR was observed for the mixture compacted at optimum lime content of 5%. CBR value increased gradually by the addition of lime while beyond optimum lime it decreased. The rise in soaked CBR due to lime stabilization is displayed in table 4.4. The maximum increase in soaked CBR is about 1150% at optimum lime content in the mixtures. Figure 4.8 elaborates the CBR of control soil specimen and soil-lime mixtures against percent lime contents for Unsoaked and soaked conditions. The trend is almost similar for both Unsoaked and soaked specimens.

Table 4.4 : Increase in CBR of lime treated mixtures

Specimen Type	Additive (%)	CBR (%)		% increase in Unsoaked CBR	% increase in Soaked CBR
		Unsoaked	Soaked		
Control	0%	4	2	NA	NA
Lime Treated	3%	24	14	500	600
	5%	40	25	900	1150
	7%	30	19	650	850

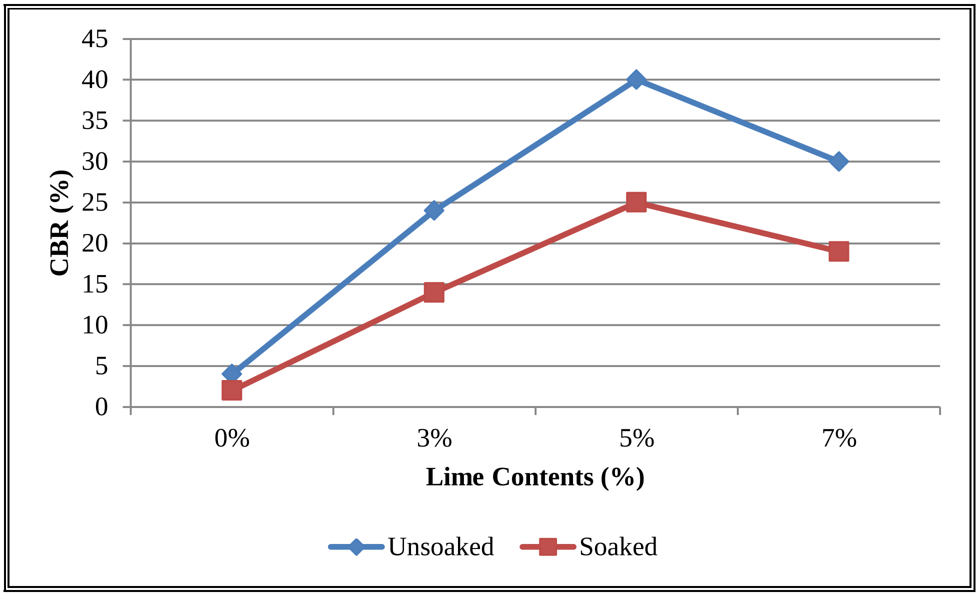


Figure 4.8: Soaked and Unsoaked CBR at varying proportion of lime-soil mixtures

4.4.3.1 Correlation Between Unsoaked And Soaked CBR of Lime Treated Mixtures

Figure 4.9 presenting correlation between Unsoaked and soaked CBR of Lime treated mixtures prepared at varying proportion of lime. The plot shows that there is a strong positive correlation between Unsoaked and soaked CBR. The R^2 for this particular relation is 89 %.

$$\text{Soaked CBR} = 0.61\ \text{Unsoaked CBR} - 0.35 \qquad (4.1)$$

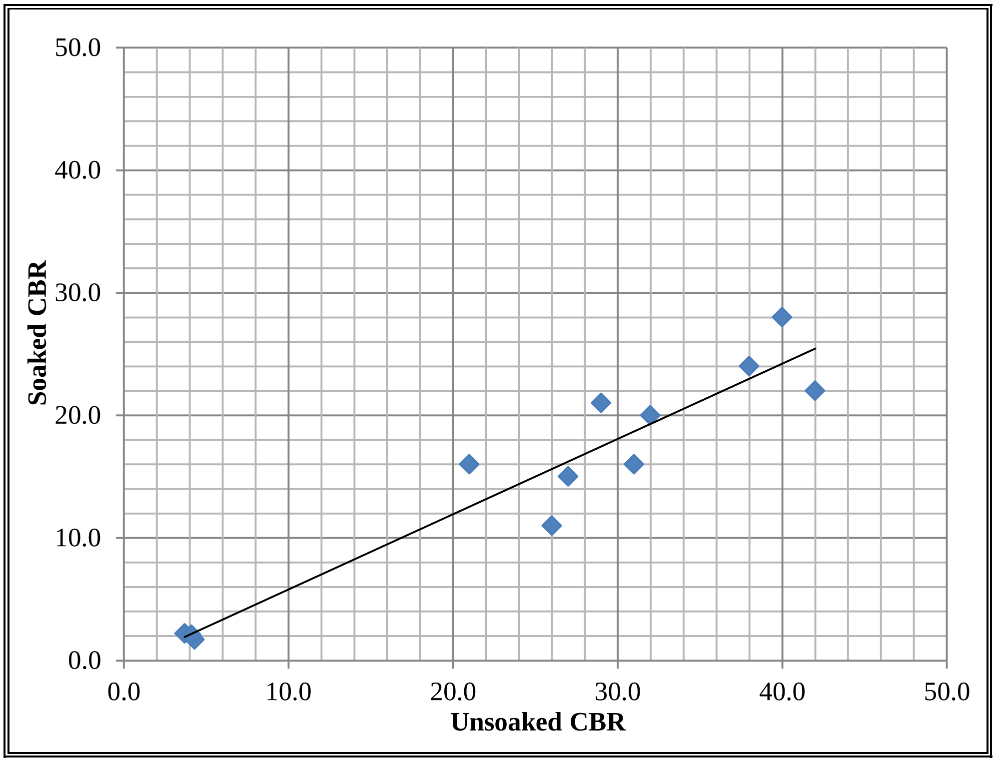


Figure 4.9: Correlation between Unsoaked and Soaked CBR

4.4.4 Strength Gain, Effect of Soaking and Strength Development Index for Lime Stabilization

The strength gain, effect of soaking and strength development index of UCS data for lime stabilization were calculated and presented in table 4.5. Effect of soaking and strength development index were calculated by equation 4.2 and 4.3 respectively.

$$\text{Effect of Soaking} = \left(\frac{\text{Unsoaked Strength} - \text{Soaked Strength}}{\text{Unsoaked Strength}}\right) * 100 \qquad (4.2)$$

$$\text{SDI} = \frac{(\text{Strength of Stabilized sample} - \text{Strength of untreated sample})}{\text{Strength of untreated sample}} \qquad (4.3)$$

$$\text{Lime Reactivity} = \text{Qu with Lime (Psi)} - \text{Qu without Lime (Psi)} \qquad (4.4)$$

Table 4.5: Effect of soaking and SDI of UCS for lime treated mixtures

% lime	Qu UCS (Psi)		Ratio US / S	Effect of Soaking (%)	SDI	Lime Reactivity
	Unsoaked	Soaked				
0%	41	22	0.54	46.34		
3%	203	125	0.62	38.42	3.95	162
5%	295	192	0.65	34.92	6.20	254
7%	236	145	0.61	38.56	4.76	195

Figure 4.10 shows the effect of soaking versus lime contents. Effect of soaking due to soaking is maximum for untreated soil while it is minimum of 35% for lime stabilized specimen having 5% lime. The strength development index for varying lime proportions is presented in figure 4.11. It is depicted from the figure that SDI is maximum of 6.20 for specimen compacted at optimum amount of lime. SDI is decreased on both sides of optimum. The ratio of Unsoaked to soaked soil specimen at varying proportion of lime is presented in figure 4.12. Figure indicates that the strength gain while soaking is maximum for optimum proportion of lime contents. Lime reactivity was determined by equation 4.4, by subtracting Qu without lime to Qu with lime and shown in table 4.5.

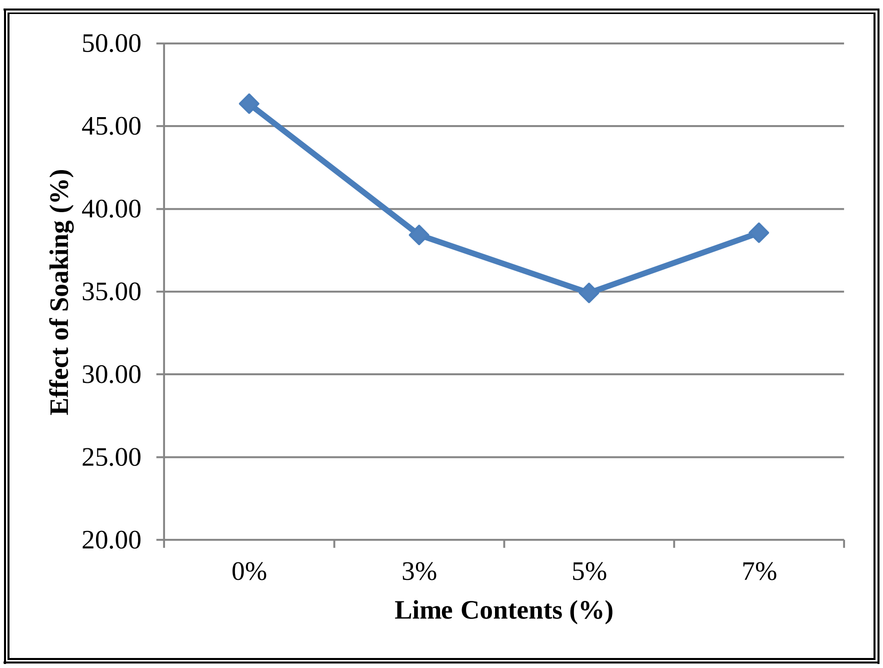


Figure 4.10: Effect of soaking on lime treated mixtures

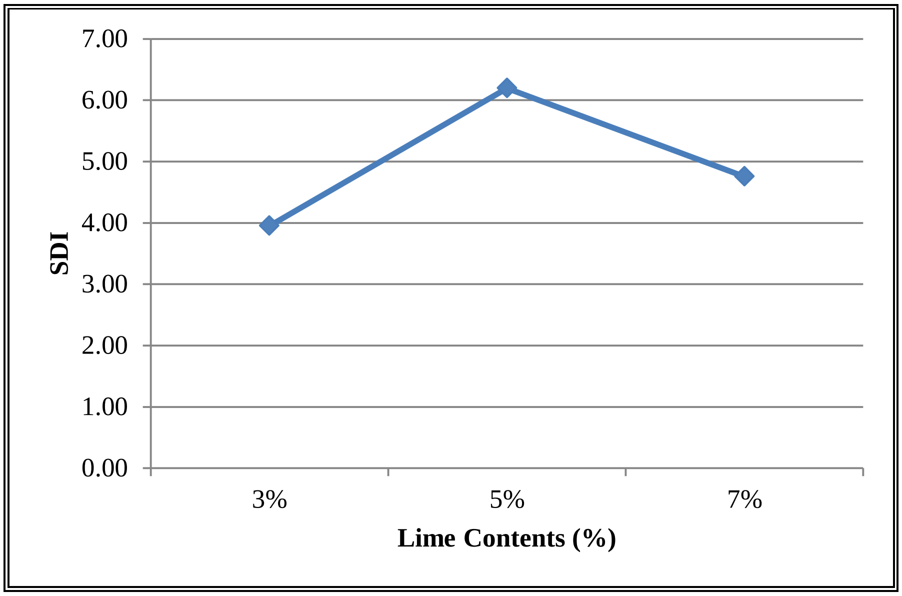


Figure 4.11: Strength development index for lime treated mixtures

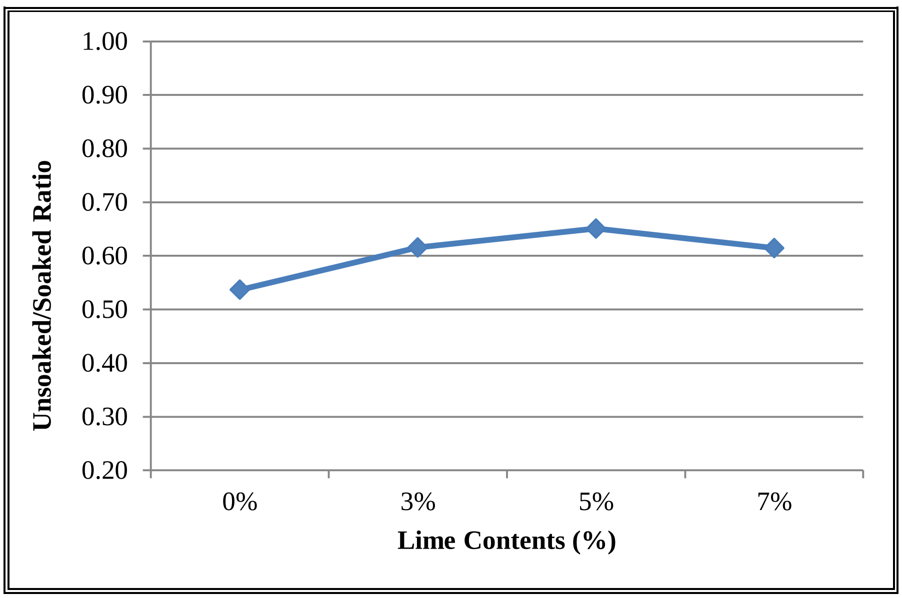


Figure 4.12: Unsoaked to Soaked ratio for Lime treated mixtures

4.4.5 Stress–Strain Behavior of Lime Treated Mixtures

Lime increases the ultimate stress and decreases the ultimate strain. Also ultimate strain of lime treated soil is approximately 1% regardless of the soil type. The test results show a variation in ultimate strain which occurs at an average strain of about 2% as shown in Fig. 4.13. However, the ultimate stress is increased and ultimate strain is decreased substantially as compared to unstabilized soil. It is inferred that the ability of specimen to bear stress was maximum for the 5% lime-soil sample. The control soil specimen failed under a little amount of stress while producing very higher strain as compared with treated specimen.

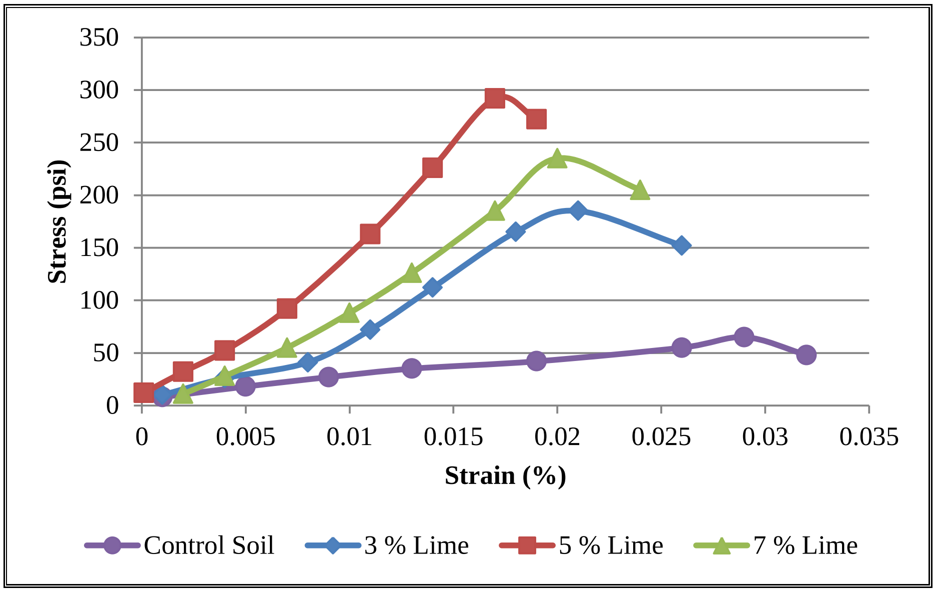


Figure 4.13: Stress strain relationship of control soil and soil-lime mixtures

4.5 CEMENT STABILIZATION

The average values of UCS test for cement stabilization are presented in table 4.6. Fig. 4.14 shows the variation in UCS with respect to percent cement Contents and curing periods. It can be seen that the strength increased gradually by the addition of cement and is maximum at 6 % cement contents. The UCS data shows that the increase in strength occur due to curing of specimens and it is obvious that it raised when the curing period changes from zero to seven days.

Table 4.6: UCS of cement treated mixtures at various curing periods

% Cement	UCS (Psi),0-day Curing		UCS (Psi),2-day Curing		UCS (Psi), 7-day Curing	
	Unsoaked	Soaked	Unsoaked	Soaked	Unsoaked	Soaked
0%	41	22	48	26	52	29
2%	210	152	250	180	272	195
4%	236	172	280	205	302	219
6%	275	210	330	235	350	252

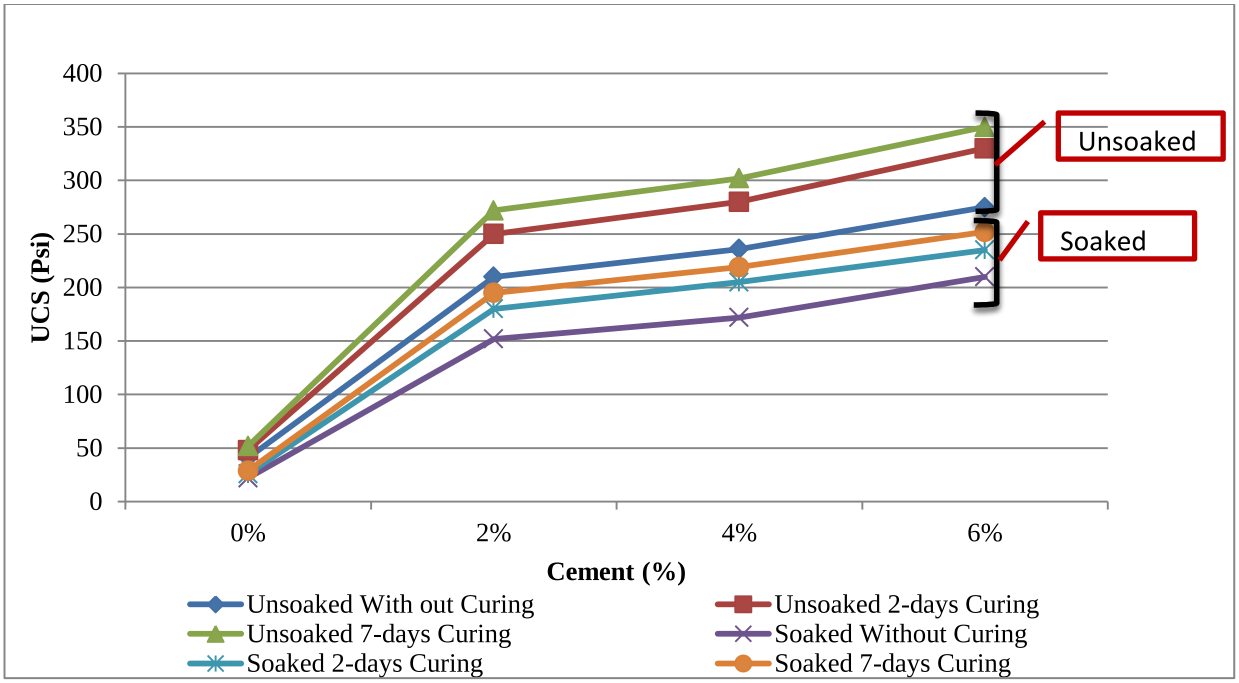


Figure 4.14: UCS of cement treated mixtures

4.5.1 CBR of Cement Treated Soil

The results of CBR test at varying proportion of soil-cement mixtures are presented in table 4.7. CBR test indicates that by the addition of cement the CBR value of specimens significantly increased as compared with control specimen. The CBR value increased gradually with the addition of cement contents. The rise in Unsoaked and soaked CBR due to cement stabilization is displayed in table 4.7. The maximum increase in soaked CBR is about 2600% at 6 % cement contents as compared with control mixture. Figure 4.15 elaborates the CBR of control soil specimen and soil-cement mixtures against percent cement contents for Unsoaked and soaked conditions. The trend is almost similar for both Unsoaked and soaked specimens.

Table 4.7: Increase in CBR of cement treated mixtures

Specimen Type	Additive (%)	CBR (%)		% increase in Unsoaked CBR	% increase in soaked CBR
		Unsoaked	Soaked		
Control	0%	4	2	NA	NA
Cement Treated	2%	32	18	700	800
	4%	45	26	1025	1200
	6%	52	30	1200	1400

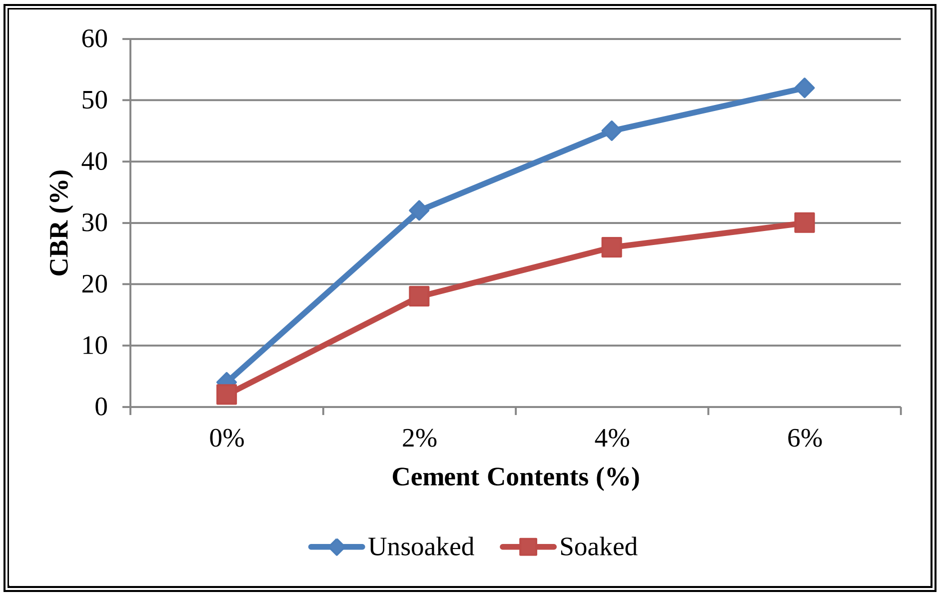


Figure 4.15: CBR at varying proportion of soil-cement mixtures

4.5.1.1 Correlation Between Unsoaked And Soaked Cbr Of Cement Treated Mixtures

Figure 4.16 presenting correlation between Unsoaked and soaked CBR of cement treated mixtures prepared at varying proportion of cement (0%, 2%, 4%, and 6%). The plot shows that there is a strong positive correlation between Unsoaked and soaked CBR. The R^2 for this particular relation is 93 %.

Soaked CBR = 0.58 Unsoaked CBR - 0.01 (4.5)

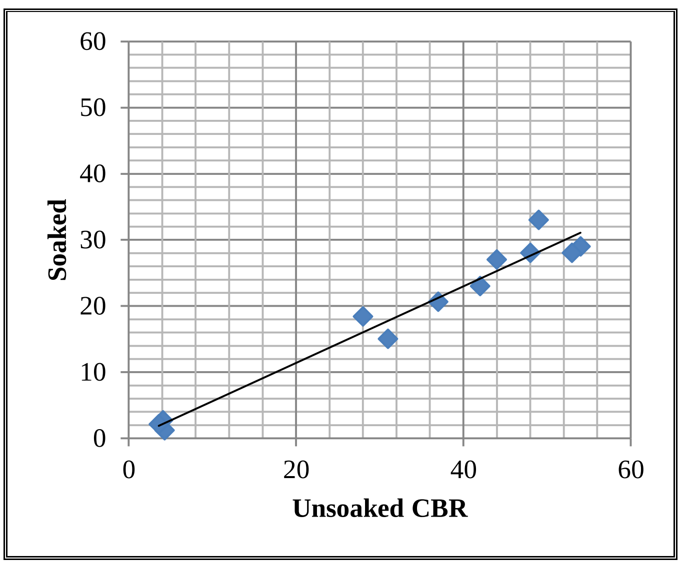


Figure 4.16: Correlation between Unsoaked and soaked CBR

4.5.2 Strength Gain With Cement Stabilization

The strength gain, loss of strength and strength development index of UCS data for cement stabilization were calculated and presented in table 4.8. Figure 4.17 shows the effect of soaking versus cement contents. Effect of soaking is maximum for untreated soil while it is minimum of 7.64% for cement stabilized specimen having 6% cement. The strength development index for varying cement proportions is presented in figure 4.18. It is depicted from the figure that SDI is maximum of 2.57 for specimen compacted at 6% cement contents. The ratio of Unsoaked to soaked soil specimen at varying proportion of cement is presented in figure 4.19. Cement reactivity was determined by equation 4.6, by subtracting Qu without cement to Qu with cement and shown in table 4.8.

$$\text{Cement Reactivity} = \text{Qu with Cement (Psi)} - \text{Qu without Cement (Psi)} \qquad (4.6)$$

Table 4.8: Increase in UCS of cement treated mixtures

% Cement	UCS (Psi)		Ratio US / S	Effect of Soaking (%)	SDI	Cement Reactivity
	Unsoaked	Soaked				
0%	41	22	0.54	46.34		
2%	210	152	0.72	27.62	4.12	169
4%	236	172	0.73	27.12	4.76	195
6%	275	210	0.76	23.64	5.71	234

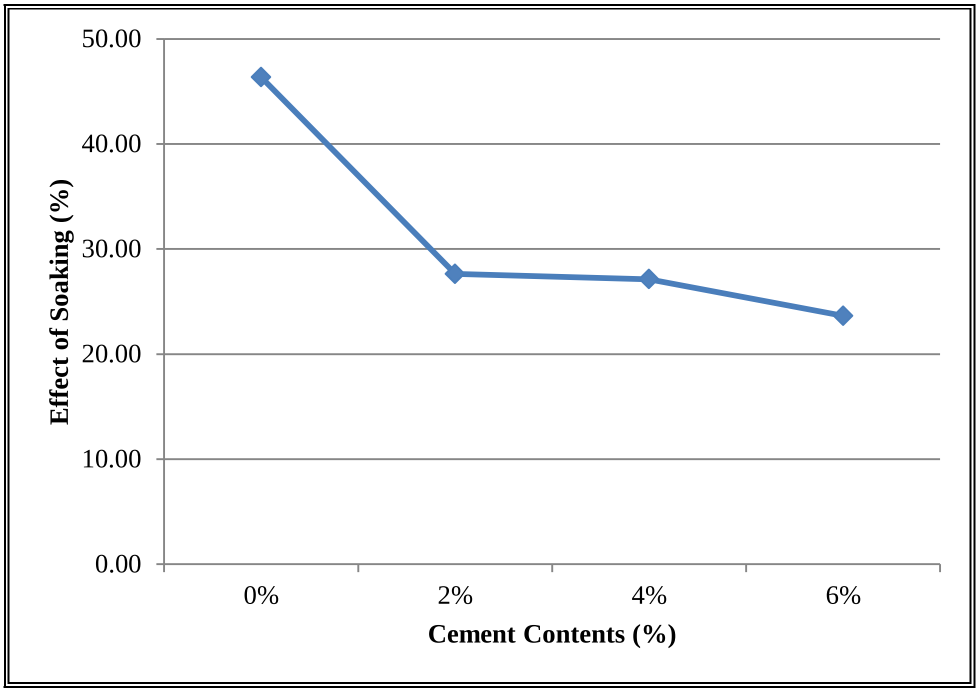


Figure 4.17: Effect of soaking on cement treated mixtures

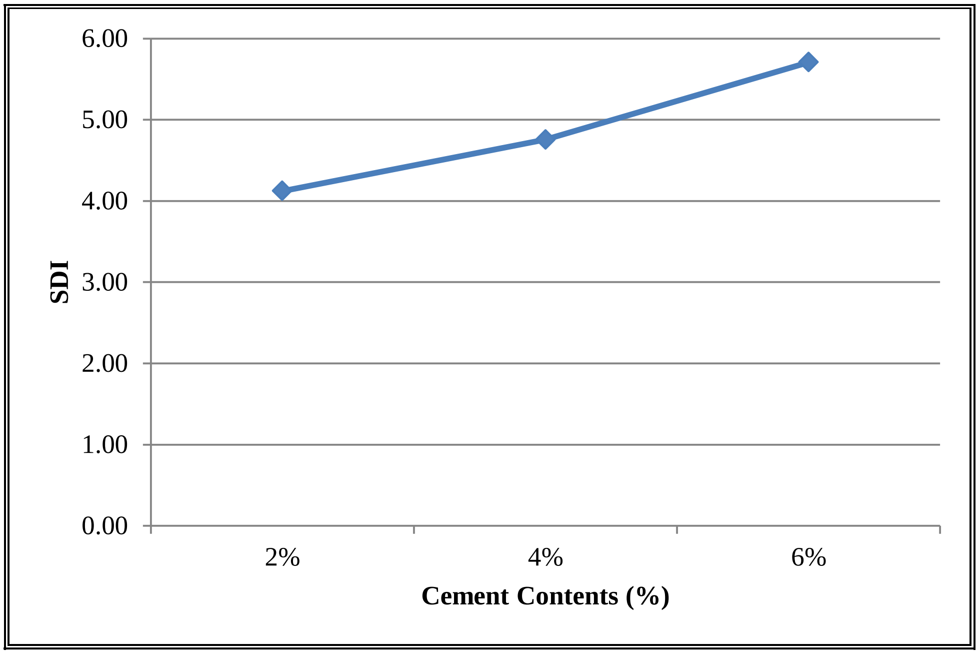


Figure 4.18: Strength development index for cement treated mixtures

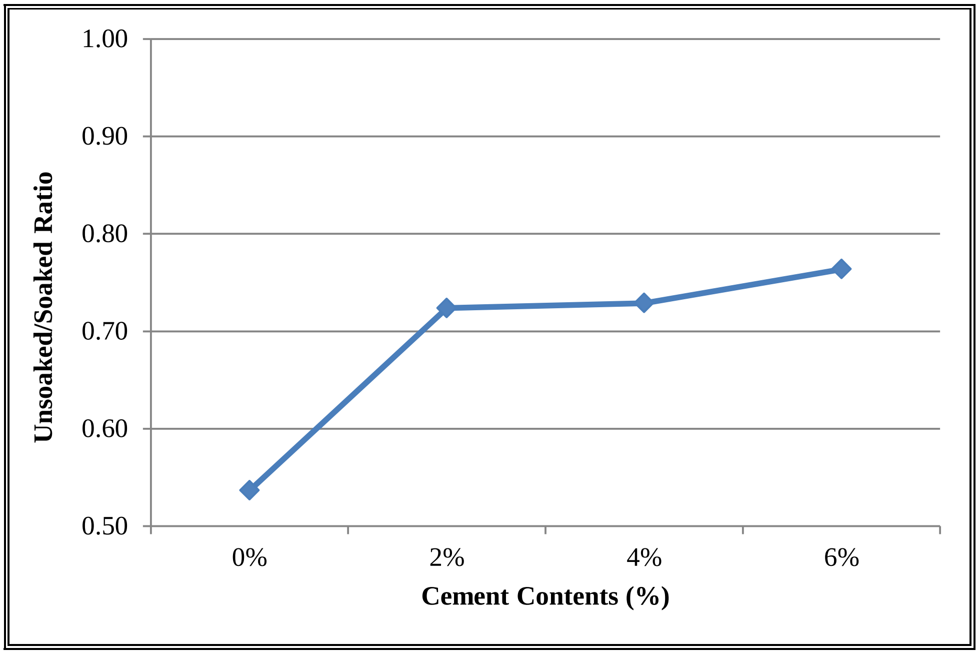


Figure 4.19: Unsoaked to Soaked ratio for cement treated mixtures

4.5.3 Stress–Strain Behavior of Cement Treated Mixtures

Cement increases the ultimate stress and decreases the ultimate strain. Also ultimate strain of cement treated soil is approximately 1% regardless of the soil type. The test results show a variation in ultimate strain which occurs at an average strain of about 2% as shown in Fig. 4.20. However, the ultimate stress is increased and ultimate strain is decreased substantially as compared to unstabilized soil. Slope of stress-strain curve is modulus (stiffness parameter). Cement stabilization caused increase in the failure stress and decrease in the ultimate strain of treated soil as compared to natural soil. The ability of specimen to bear stress was maximum for 6% cement producing minimum strain.

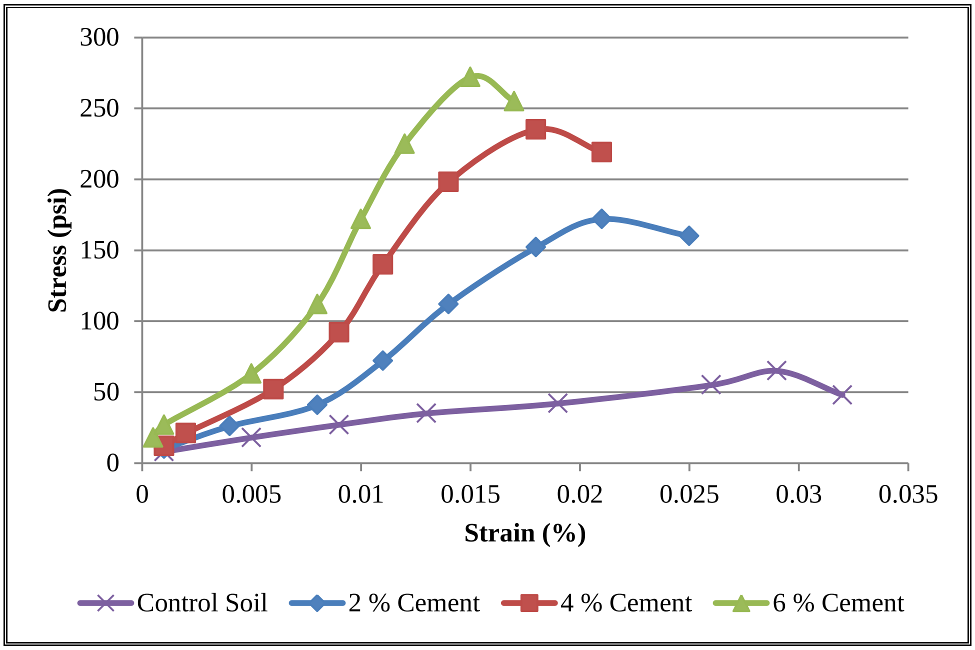


Figure 4.20: Stress strain relationship of control soil and soil-cement mixtures

4.6 REGRESSION ANALYSIS FOR UCS OF CEMENT TREATED MIXTURES

Regression analysis of Unconfined Compressive Strength (UCS) test data has been performed for different proportions of cement, conditioning and curing time by considering UCS as response variable and percent cement, conditioning, and curing time as predictors. The number of data values used in the regression analysis were 72. The general form of regression model is as follow:

UCS = f (% Cement, Curing & Conditioning) (4.7)

Where,

- UCS = Dependent Variable
- % Cement = Independent Variable
- Curing = Independent Variable
- Conditioning = Independent Variable

The regression equation for UCS data is presented as equation 4.8 below:

UCS = 92.2 - 62.4 Conditioning + 37.1 Cement (%) + 5.66 Curing (4.8)

Where

- UCS = PSI
- Conditioning = Soaked or Unsoaked
- Cement = Percentage (2%, 4% & 6%)
- Curing = Days (0,2 &7 days)

Table 4.9: T-Statistics

Predictor	Coefficient	SE Coefficient	T-test	p Value
Constant	92.22	11.36	8.12	0.000
Conditioning (A)	-62.44	10.33	-6.05	0.000
Cement (%) (C)	37.117	2.31	16.07	0.000
Curing (B)	5.662	1.754	3.23	0.002

The value of R^2 for this model is 81% which is good. Figure 4.21 shows the normal probability plot of residuals, the normality assumption is satisfied because the plot resembles a straight line which shows that the errors are normally distributed. Figure 4.22 is the plot of residual vs. fitted values. The plot indicates that the residuals are structure less and the plotted points made approximately horizontal band that satisfy the assumption.

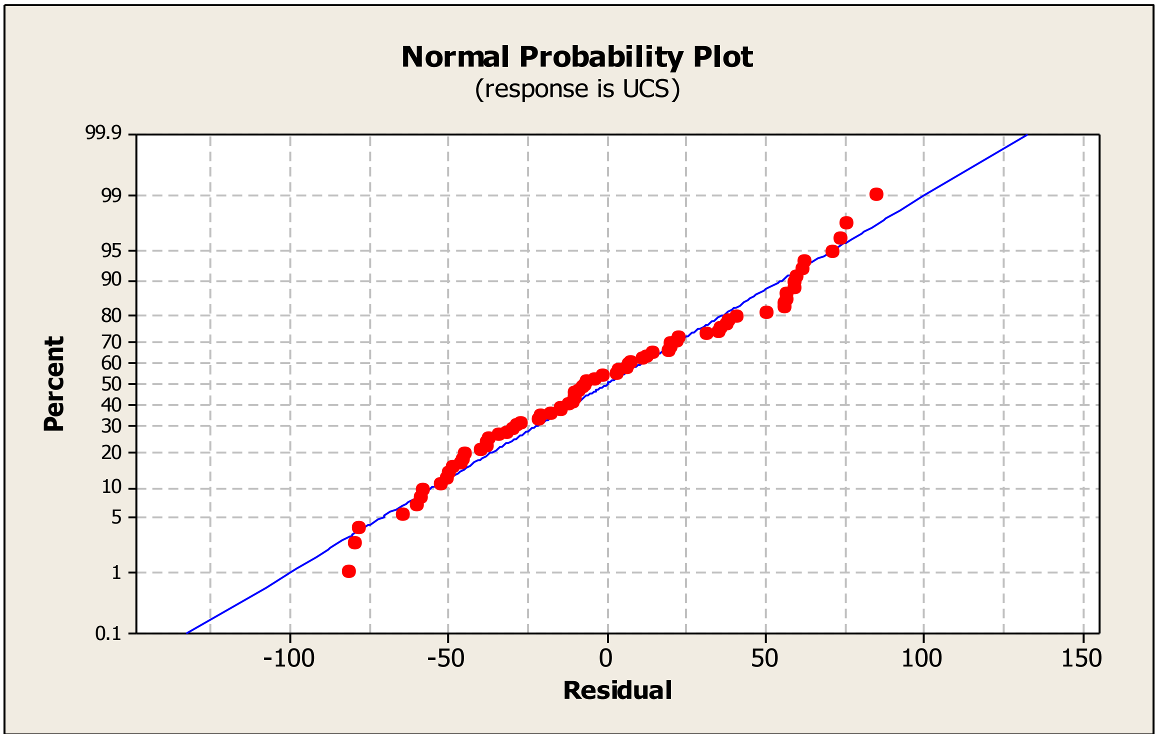


Figure 4.21: Normal Probability Plot

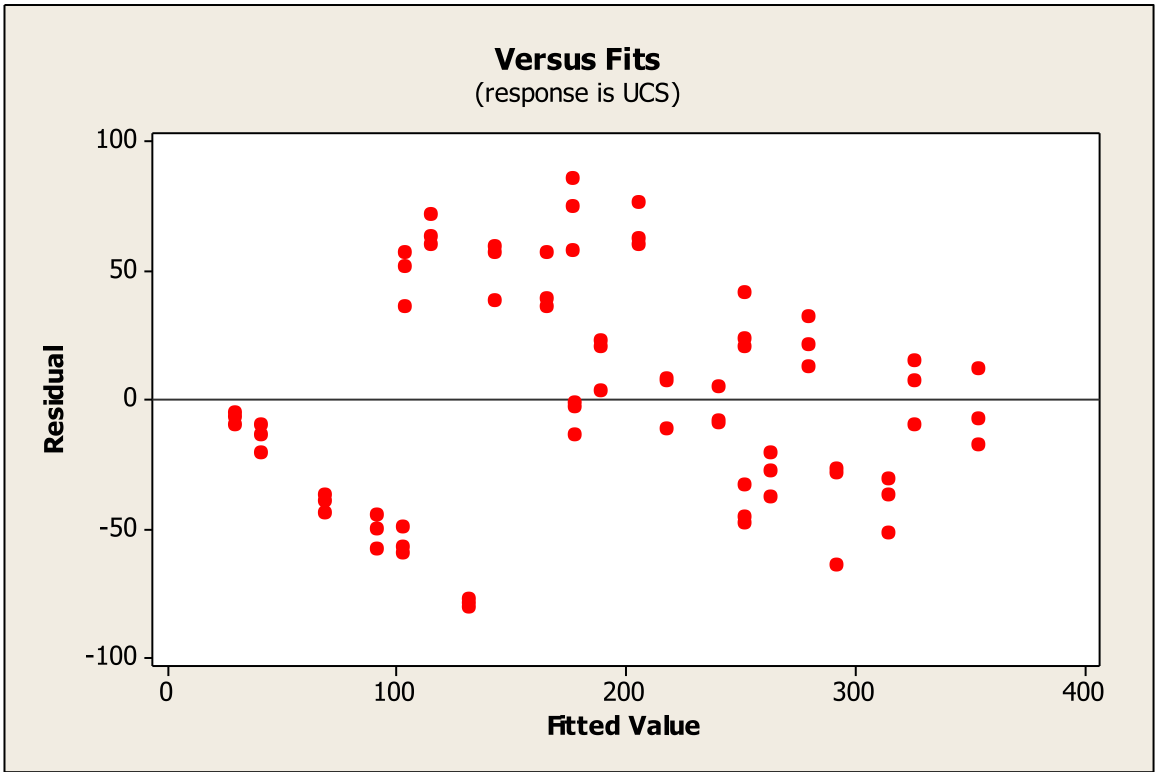


Figure 4.22: Residual vs. Fitted Values

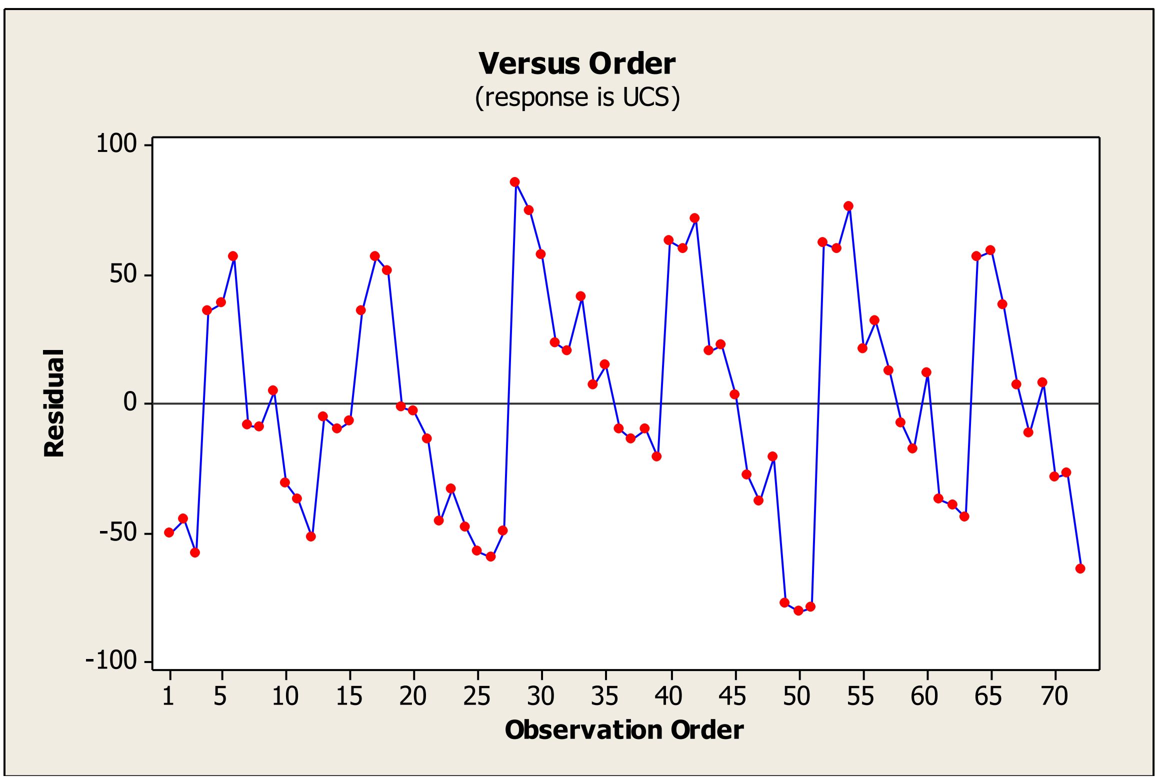


Figure 4.23: Residual vs. Observation order plot

The correlation between the residuals is detected by plotting the residuals in time order of data collection. Figure 4.23 is the plot of residuals vs. observation order. It indicates that the residuals are independent and they are not correlated. The indefinite pattern of the plot shows that it is structure less.

4.7 REGRESSION MODEL VALIDATION

Validation of model was carried by Mean Absolute percentage Error (MAPE) which is defined as deviation of predicted value from observed value in percentage for a fitted time series. It is generally expressed in percentage and can be formulated as:

$$S.D = \frac{1}{N}\sum_{i}^{n}\left(\frac{Observed - Predicted}{Observed}\right) \tag{4.9}$$

Where S. D is Standard deviation and N is no of values.

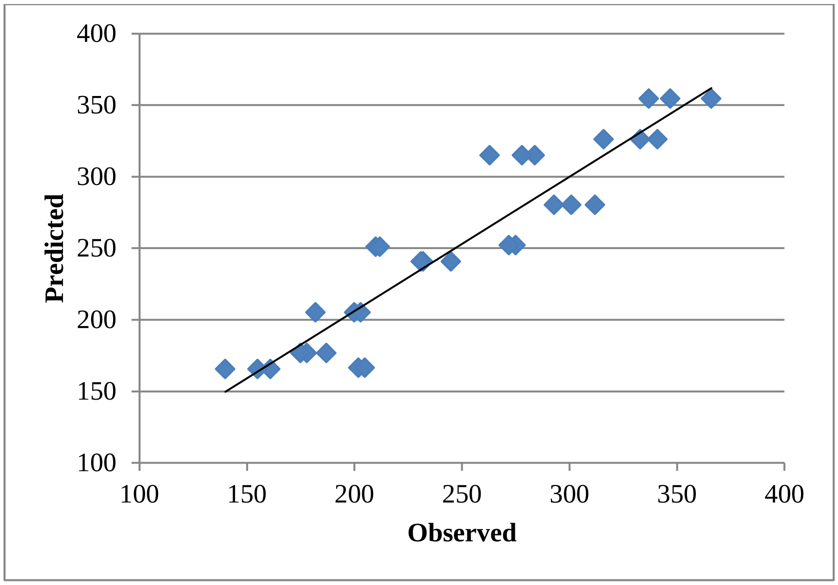


Figure 4.24: Validation Plot

Before the development of model, 50% of data was randomly selected for validation of developed model. The predictive capability of model was tested by calculating MAPE using model parameters and validation data set. The MAPE value was 7 for the validation data set which is satisfactory and hence model is significant. The observed versus predicted values were plotted as shown in figure 4.24 which demonstrate the predictive capability of model. The model for which predicted data points were located close to the 100% validation line were considered the best predictive.

4.8 FULL FACTORIAL DESIGN FOR UCS OF CEMENT TREATED MIXTURE

The analysis of UCS data for the varying proportions of cement, conditioning and curing time was carried out by considering three factors i.e. percent cement, conditioning and curing time, each with two levels. Therefore 2^3 full factorial design of experiment was adopted using MINITAB-15 software. Table 4.10 shows the factors that have been considered in the 2^3 full factorial design of experiment with their particular notations and low and high severity levels. Inputting these three factors in software resulted in eight combinations. To obtain a reasonable estimate of error each experimental condition was replicated three times. Therefore total 24 tests were conducted. Table 4.11 shows the combination of factors generated by software for the full factorial design of experiment.

Table 4.10: Factor's notations and their levels for analysis

Factors	**Notation**	**Levels**	
Conditioning	A	Soaked	Unsoaked
Curing Days	B	0	7
Cement (%)	C	2	6

Table 4.11: Measured values of UCS along with factors

Conditioning	**Curing Days**	**Cement (%)**	**UCS**
soaked	7	2	263
soaked	0	6	207
soaked	0	6	219
Unsoaked	7	2	268
Unsoaked	0	2	202
Unsoaked	7	2	266
soaked	7	6	263
soaked	7	2	265
soaked	0	2	140
Unsoaked	7	6	347

Unsoaked	0	6	284
soaked	0	6	204
soaked	7	6	265
Unsoaked	0	6	278
soaked	0	2	161
Unsoaked	0	6	263
Unsoaked	7	6	337
soaked	7	6	228
soaked	0	2	155
Unsoaked	0	2	223
soaked	7	2	228
Unsoaked	7	2	282
Unsoaked	0	2	205

4.8.1 Effects and Coefficient Table

Table 4.12 shows the effects and coefficients values generated by MINITAB-15 for the significant effects. The factors and interaction of factors with high values of effect and coefficient indicate that they have large impact on the UCS of the mix. The effect of each term is equal to the twice of the coefficient.

Table 4.12: Effects and Coefficients for UCS

Term	Effect	Coefficient	SE Coef	t-test	p-value
Constant		246.625	2.873	85.84	0.000
Conditioning	60.250	30.125	2.873	10.48	0.000
Curing Days	69.750	34.875	2.873	12.14	0.000
Cement (%)	50.250	25.125	2.873	8.74	0.000
Conditioning * Curing Days	-1.250	-0.625	2.873	-0.22	0.831
Conditioning * Cement (%)	21.250	10.625	2.873	3.70	0.002
Curing Days * Cement (%)	-11.250	-5.625	2.873	-1.96	0.068
Conditioning*Curing Days*Cement (%)	17.750	8.875	2.873	3.09	0.007

The calculated value of t-statistics for most of the terms is greater than the critical value of t-statistics ($t_{critical}$ = 2.001 for the degree of freedom 23 and significance level of 5%) and p-value of all individual factors is less than the significance level which shows that all the factors are significant at 5% significance level. Dividing the coefficient by its standard error calculates a t-value.

4.8.2 Significant Effects and Interaction Plots

The factors and interaction of factors that are most significant and affect the UCS of the mix are also shown with Pareto plot, Half Normal probability plot and normal probability plot obtained from the analysis using MINITAB-15 software. Figure 4.25 shows the Pareto plot with the absolute values of effects and a reference line drawn which indicates the critical value of student-t. It is quite interesting to note that all main factors Conditioning, Curing days, percent cement and 2-way interactions of Cement percent and conditioning also the 3-way interaction of conditioning, curing time and cement percent are beyond the reference line which shows that these factors and their interactions are significant at 5% significance level. The Pareto plot shows that the Curing time is most significant factor when its level changes from 0 to 7. The conditioning and cement percent having the similar affect at 5% significance level.

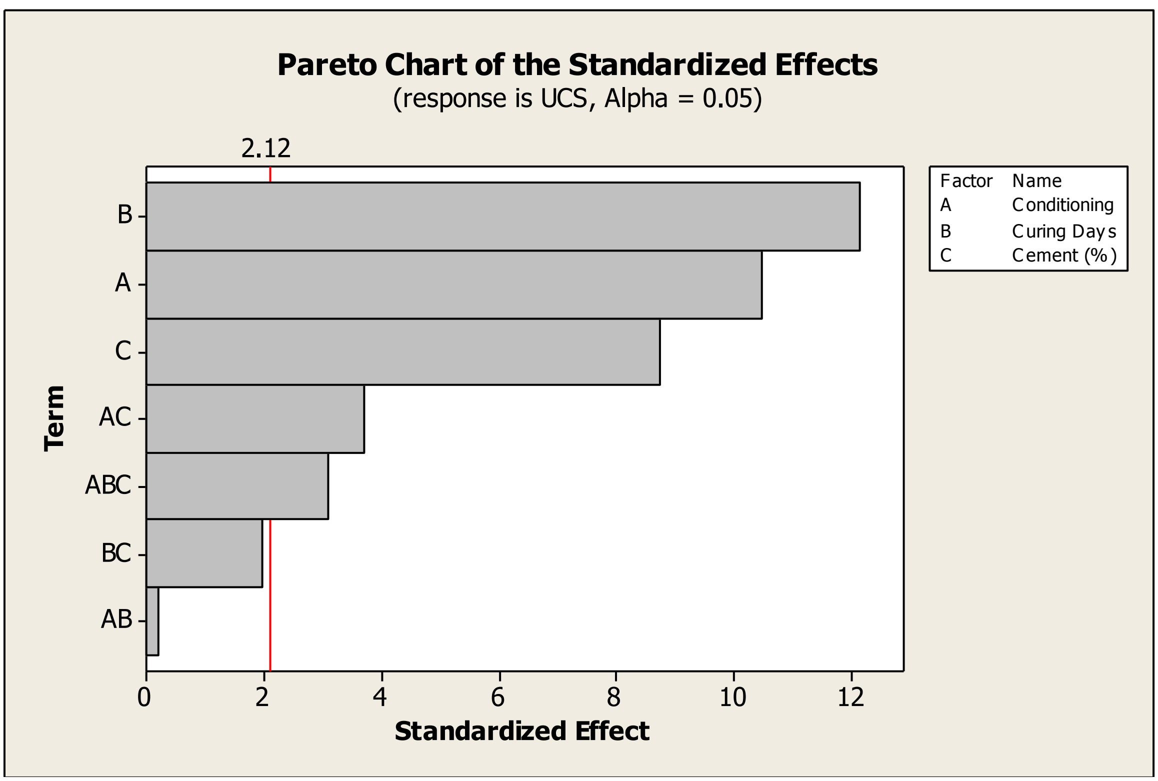


Figure 4.25: Pareto plot of the standardized effects

The other plots for obtaining the significant main effects and interaction are half normal probability plot and normal probability plot of standardized effects as shown in figure 4.26 and 4.27. The significant effect from the half normal probability plot or normal probability plot is judge in the sense that how far a factor or interaction of factor from the reference line. Both of the plots show the significance of factors and there interaction at 5% significance level but curing days is the most significant factor, which greatly affect the UCS test.

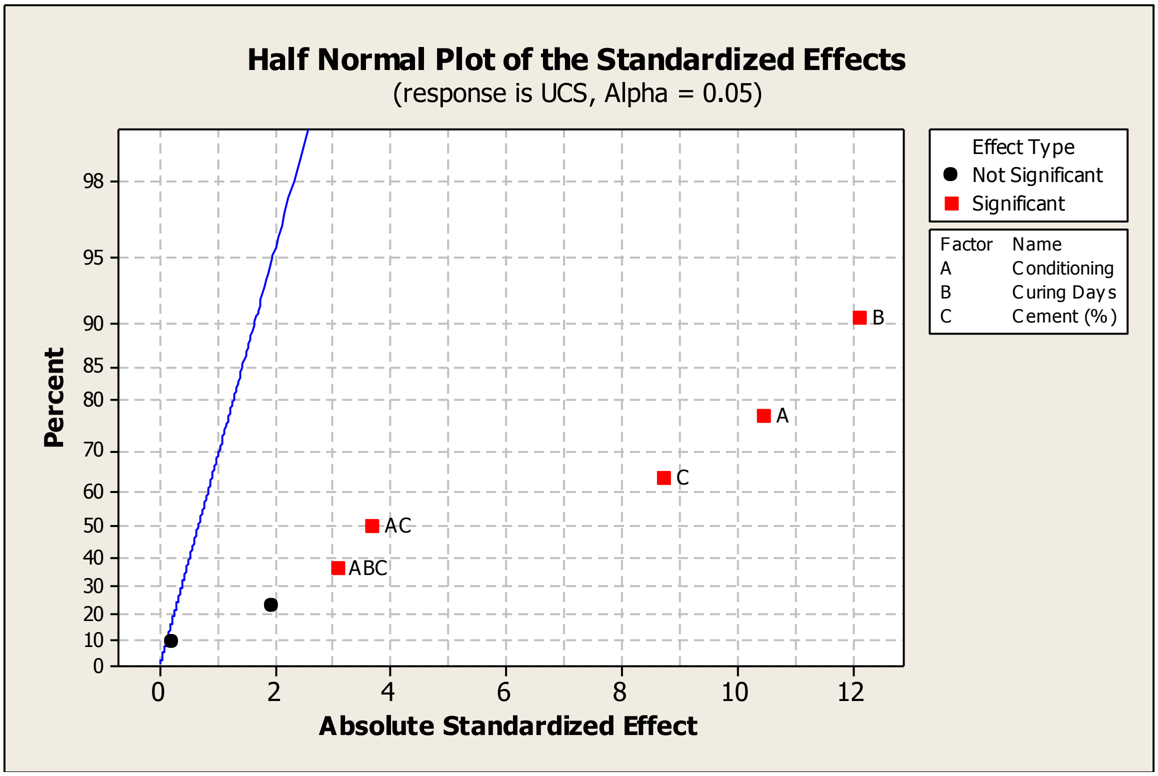


Figure 4.26: Half Normal plot of the Absolute Standardized Effects

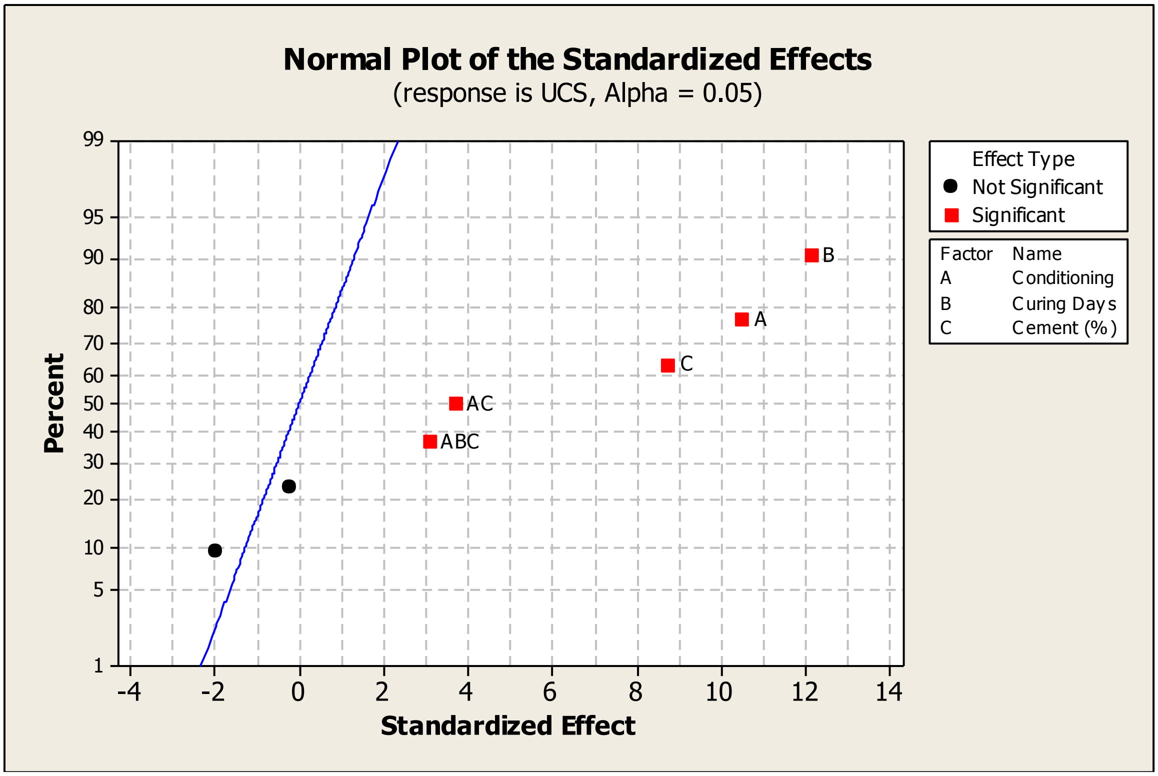


Figure 4.27: Normal plot of the Standardized Effects

4.8.3 Factorial Plots

The significant effects and interactions obtained from the Pareto plot, half normal probability plot and normal probability plot are explained in detail in factorial plots. The effects of individual factors are shown by main effect plot, 2-way interaction with interaction matrix and 3-way interaction with cubic plot.

4.8.3.1 Main Effect Plots

The effects of conditioning, curing days and percent cement are shown in figure 4.28. It is quite clear from the curing days plot that UCS is much higher at 7 days as compared to 0 days curing. This is due to the fact that mix gains strength due to hydration of cement with time.

The fines plot shows the effect of soaking on the UCS of mix. The larger UCS occurs for the Unsoaked specimens while the UCS is lower for the soaked specimens. It is due to the fact that soaking in water makes specimen soft.

The percent cement plot shows that the UCS is lower for the specimens having 2 % cement and higher for the specimens containing 6 % cement.

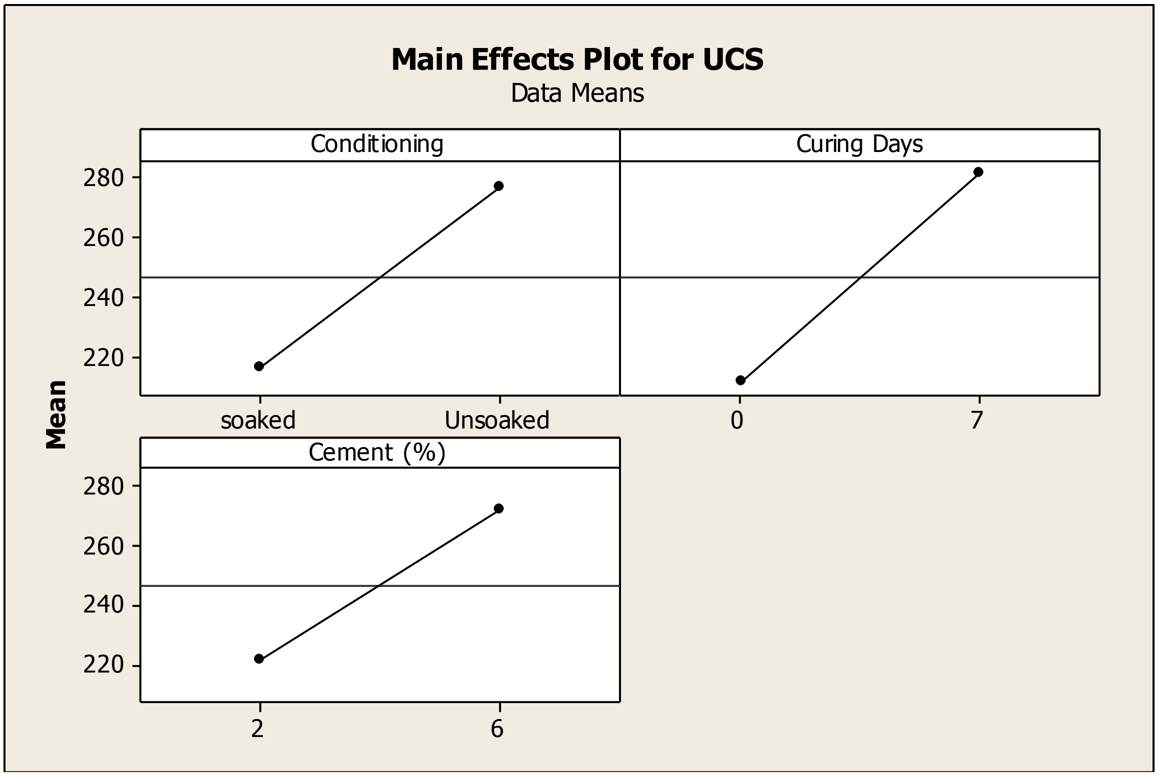


Figure 4.28: Main Effects plots for UCS

4.8.3.2 Interaction Plots

Figure 4.29 shows the interaction plot of factors. It is clear from the plot that all the interactions Conditioning and curing days, percent cement and conditioning, curing days and percent cement all are significant and represented by non-parallel lines.

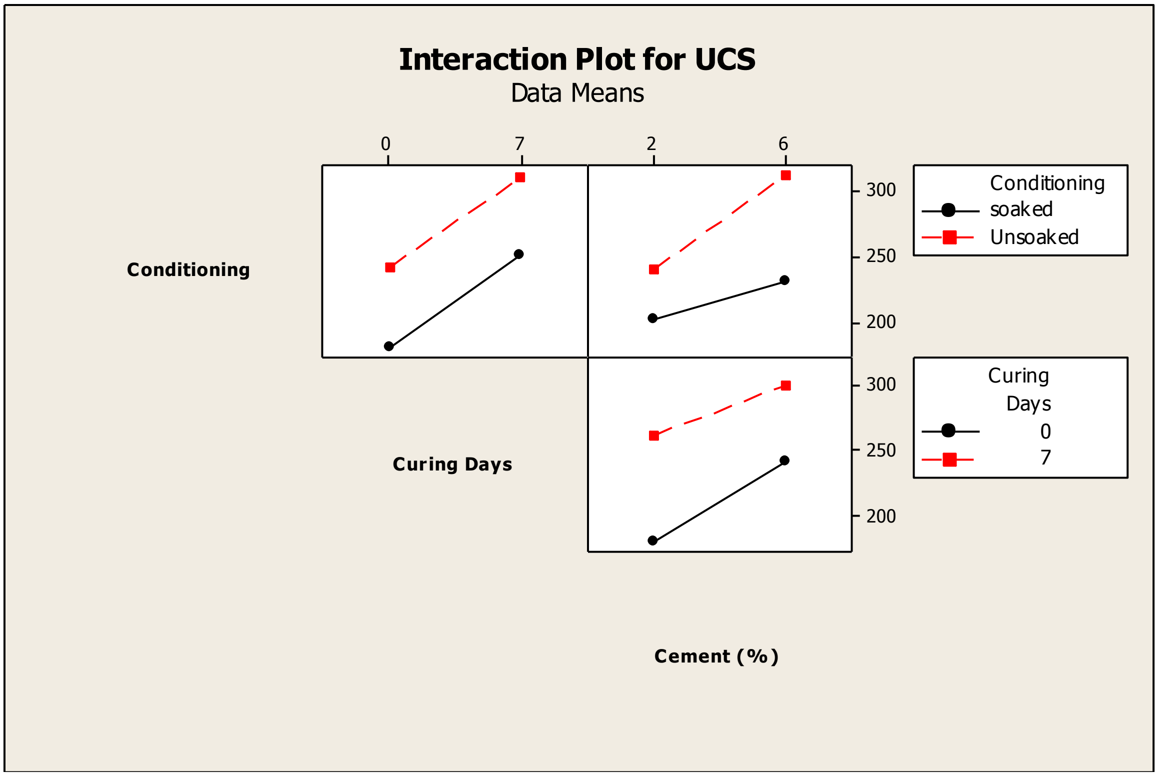


Figure 4.29: Interaction plots for UCS

4.8.3.3 Cubic Plot

From the Pareto plot, Normal and Half Normal plot it is clear that interaction of conditioning, curing days and percent cement is significant. Figure 4.30 shows the cubic plot for the UCS. It shows that maximum UCS value is at 6 % cement, for 7 days curing when the specimen was Unsoaked.

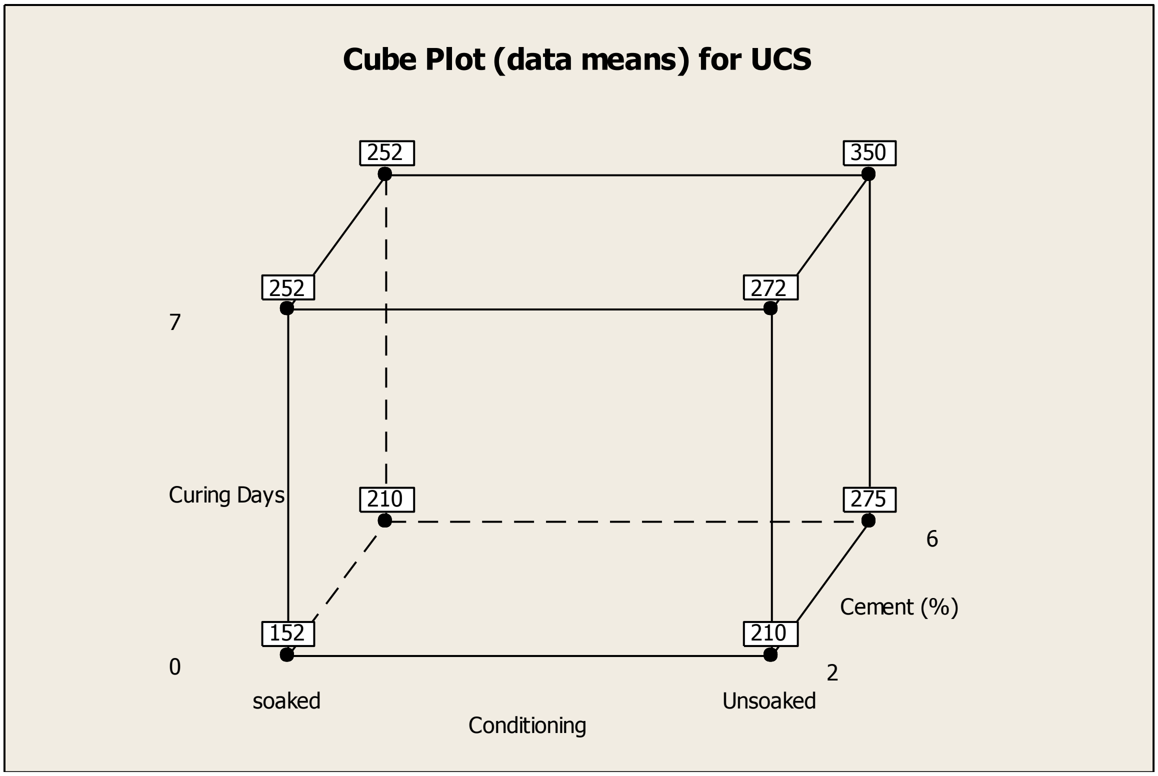


Figure 4.30: Cubic Plot for UCS

4.8.4 Analysis Of Variance (ANOVA)

In analysis of variance, three F-tests are made. The probability values for the evaluation of these tests are given as shown in ANOVA table 4.13:

Table 4.13: Analysis of Variance for UCS

Source	DF	Sum of Square (SS)	Mean Sum of Square (MSS)	F-test	P-value
Main Effects	3	66121	22040	111.24	0.000
2-Way Interactions	3	3478	1159	5.85	0.007
3-Way Interactions	1	1890	1890	9.54	0.007
Residual Error	16	3170	198		
Total	23	74659			

The first three tests judge the significance of individual factors, 2-way interactions and 3-way interactions. The p-value < 0.05 shows that these tests are satisfied.

4.9 COST ANALYSIS

The detailed cost analysis for currently in practice techniques with soil stabilization was made. For this purpose National Highway Authority (NHA) General Specifications (1998) and Composite Schedule of Rates (CSR 2014) were used.

4.9.1 Cost Comparison of Stabilization With In-Practice Field Techniques

Removing and replacing unsuitable common material is the technique most commonly used at national level for the construction of roads in the areas having lower subgrade CBR. Table 4.14 describes the cost of removing and replacing of unsuitable common material with suitable material. Rates were taken from NHA Composite Schedule of Rates (NHA CSR 2014) Table 4.15 presents the exertion cost for stabilizing a one kilometer two lane road. Total cost for stabilization including stabilization agent (Cement) at different proportions and exertion cost is calculated and presented in table 4.16. The cost of stabilization for a two lane road at four percent cement contents is lesser than the cost of conventional removing and replacement method.

Table 4.14: Cost for conventional removing and replacing technique of unsuitable material

In Practice		
Activity No.	**Description**	**Rate / m^3 (PKR)**
1	Excavation of unsuitable common material	361.89
2	Loading, hauling and disposal of excavated surplus material	
3	Borrow excavation, loading and dumping of suitable material	447.49
4	Compaction of suitable material	
Total rate for removal and replacement		809.38
Volume of a two lane road 1 km length (m^3)		2190
Cost/km in PKRs		**1772542.2**

Table 4.15: Execution cost for stabilizing a 1 km road

Proposed Practice (Stabilization)		
Activity No.	**Description**	**Rate / m3 PKRs.**
1	Simple excavation, ploughing and stripping	50
2	Spreading of stabilizer (Rate from compaction)	3.5
3	Mixing, ploughing, stripping	50
4	Watering and compaction	5.54
Total Rate		109.04
Volume of a two lane road 1 km length (m^3)		2190
Execution Cost/km		238797.6

Table 4.16: Total cost of stabilizing 1(one) km road for different proportions of cement

Cement (%)	Quantity of cement for 1 Km (ton/ km)	Rate per Ton (204c) (PKRs.)	Cost (PKRs.)	Execution Cost/km	Total Cost/km PKRs.
2%	65.95	10456	689587.9639	238797.6	928386
4%	131.90	10456	1379175.928		1617974
6%	197.85	10456	2068763.892		2307561

4.9.2 Optimum Cement Contents at Prevailing Cost

The optimum cement contents for cement stabilization were selected at prevailing cost of cement stabilization compared with other applicable methods adopted in field in contrast to stabilization following Composite Schedule of Rates (CSR) of National Highway Authority (NHA). The amount in rupees for excavation and replacement with borrow soil material of 1 kilometer road having standard lane width of 3.65 and the selected depth was 1 ft was evaluated for the district Nawabshah. While the cost for cement stabilization was calculated at 2, 4, and 6 % of cement contents including cement cost, mixing and curing etc. The detailed working is presented in Tables as shown below.

Table 4.17: Cost of subgrade layer for1 Km road with conventional method

1.(A) Excavation of unsuitable common material (item no. 106a)						
Location	**Length (m)**	**Width (m)**	**Depth (m)**	**Volume (m^3)**	**Rate/ (m^3)**	**Cost**
Nawabshah	1000	7.3	0.3	2190	361.89	792539.1
1.(B) Formation of embankment from borrow excavation in common material						
Nawabshah	1000	7.3	0.3	2190	447.49	980003.1
Total Cost (A+B) Rs.						1772542

Table 4.18: Cost of subgrade layer for1 Km road with improved subgrade

2.Improved Subgrade CBR (>20%) (Item no. 110)						
Location	**Length (m)**	**Width (m)**	**Depth (m)**	**Volume (m^3)**	**Rate/ (m^3)**	**Cost**
Nawabshah	1000	7.3	0.3	2190	866.12	1896803

Table 4.19: Summary of cost for cement stabilization

Cement (%)	Quantity of cement for 1 Km (ton/ km)	Rate per Ton (204 c) (Rs.)	Cost (Rs.)	Mixing and Curing (15%) (Rs.)	OH-Profit (20%) (Rs.)	Total Cost (Rs.)
2%	65.95	10456	689588	137918	27584	855089
4%	131.90	10456	1379176	275835	55167	1710178
6%	197.85	10456	2068764	413753	82751	2565267

Table 4.20: Summary of cost for Lime stabilization

Lime (%)	Quantity of Lime for 1 Km (ton/ km)	Rate per Ton (MRS 2015) (Rs.)	Cost (Rs.)	Mixing and Curing (20%) (Rs.)	OH-Profit (20%) (Rs.)	Total Cost (Rs.)
3%	31.60	13000	410822.1	82164	16433	509,419
5%	52.67	13000	684703.5	136941	27388	849,032
7%	73.74	13000	958584.9	191717	38343	1,188,645

The cost for stabilization with 4% cement contents was the prevailing cost at which the expenses for cement stabilization were lesser than the other conventional methods used in the field. Therefore the cement contents of 4% were selected as optimum cement content. Further strength testing was carried out at 2%, 4% and 6% cement contents.

4.9.3 Pavement Structure Design By AASHTO

Pavement layer thicknesses of the subject road section were calculated with and without subgrade soil stabilization by AASHTO method. It was prepared for making cost comparison and for determining the cost saving due to stabilization. AASHTO design guide 1993 was followed for determining the pavement layer thicknesses. The traffic on the subject road for entire 10 year design life was 30 million ESAL's, data provided by NHA design wing. Table 4.21 shows the CBR and resilient modulus values of subgrade soil before and after Cement & Lime stabilization. For this purpose a stretch of 2 lane highway having of 1 kilometer length was considered. The pavement layer thicknesses with and without stabilization are shown in table 4.22. It is evident that the reduction in layer thicknesses has been occurred due to subgrade soil stabilization. Table 4.23 and 4.24 describes the cost of a standard two lane road for a length of one kilometer when control soil (CBR = 2%) and subgrade (CBR = 8%) were considered. The cost saving due to stabilization is worked out using NHA Composite Schedule of Rates (CSR) 2014 and presented in table 4.25 and 4.26 for control soil and subgrade. The cost saving was manipulated by considering asphaltic base course and granular subbase course. The net cost saving of pavement structure due to cement stabilization (4 % cement or 5% hydrated lime) was obtained 21.6 million and 9.30 million per kilometer or 49% and 27 % of total pavement structure cost per kilometer.

Table 4.21: Subgrade for pavement design

	CBR (%)	M_R (psi)
Control Soil	2%	4000
Minimum subgrade (NHA)	8%	10500
After Cement Stabilization	26%	21000
After Lime Stabilization	24%	20500

Table 4.22: Layer thicknesses by AASHTO standard procedure

Course Type	Thickness (cm)			
	Control soil CBR=2%	Subgrade CBR=8%	Cement (4%) Stabilized	Lime (5%) Stabilized
Asphaltic Wearing Course	5.00	5.00	5.00	5.00
Asphaltic Base Course	17.00	14.00	8.00	8.00
Aggregate Base Course	40.00	25.00	25.00	25.00
Granular Subbase Course	45.00	25.00	20.00	20.00
Total Thickness (cm)	107.00	69.00	58.00	58.00

Table 4.23: Cost of a Standard 2-Lane Pavement Structure (per kilometer) for Control Soil

NHA CSR Item No.	Description	Length (m)	Width (m)	Depth (m)	Quantity	Rate	Cost PKRs.
203a	Asphaltic Wearing Course	1000	7.3	0.05	365	18501	6752865
305a	Asphaltic Base Course	1000	7.3	0.17	1241	19669	24409229
201	Granular Subbase Course	1000	7.3	0.40	2920	1864	5442880
202	Aggregate Base Course	1000	7.3	0.45	3285	2286	7509510
Total Cost per Km (PKRs.)							44,114,484

Table 4.24: Cost of a Standard 2-Lane Pavement Structure for Subgrade CBR = 8%

Cost (per kilometer) without Stabilization							
NHA CSR Item No.	Description	Length (m)	Width (m)	Depth (m)	Quantity	Rate	Cost PKRs.
203a	Asphaltic Wearing Course	1000	7.3	0.05	365	18501	6752865
305a	Asphaltic Base Course	1000	7.3	0.14	1022	19669	20101718
202	Aggregate Base Course	1000	7.3	0.25	1825	2286	3401800
201	Granular Subbase Course	1000	7.3	0.25	2555	1864	4762520
Total Cost per Km (PKRs.)							34,428,333

Table 4.25: Cost Saving due to Cement / Lime Stabilization for Control Soil CBR = 2%

NHA CSR Item No.	Description	Length (m)	Width (m)	Depth (m)	Volume (m3)	Rate/ (m3)	Cost PKRs.
305a	Asphaltic Base Course	1000	7.3	0.11	803	19669	8615022
		1000	7.3	0.15	1095	2286	2503170
201	Granular Subbase Course	1000	7.3	0.25	1825	1864	3401800
Total Cost Saving per Km (PKRs.)							21699177
Cost Saving (%)							49%

Table 4.26: Cost Saving due to Cement / Lime Stabilization for Subgrade CBR = 8%

NHA CSR Item No.	Description	Length (m)	Width (m)	Depth (m)	Volume (m3)	Rate/ (m3)	Cost PKRs.
305a	Asphaltic Base Course	1000	7.3	0.06	438	19669	8615022
201	Granular Subbase Course	1000	7.3	0.05	365	1864	680360
Total Cost Saving per Km (PKRs.)							9295382
Cost Saving (%)							27%

For same traffic ESAL's the cost saving in required thickness of pavement structure with stabilization of subgrade or without stabilization of this specific control soil specimen comes to be 27% and 49% respectively.

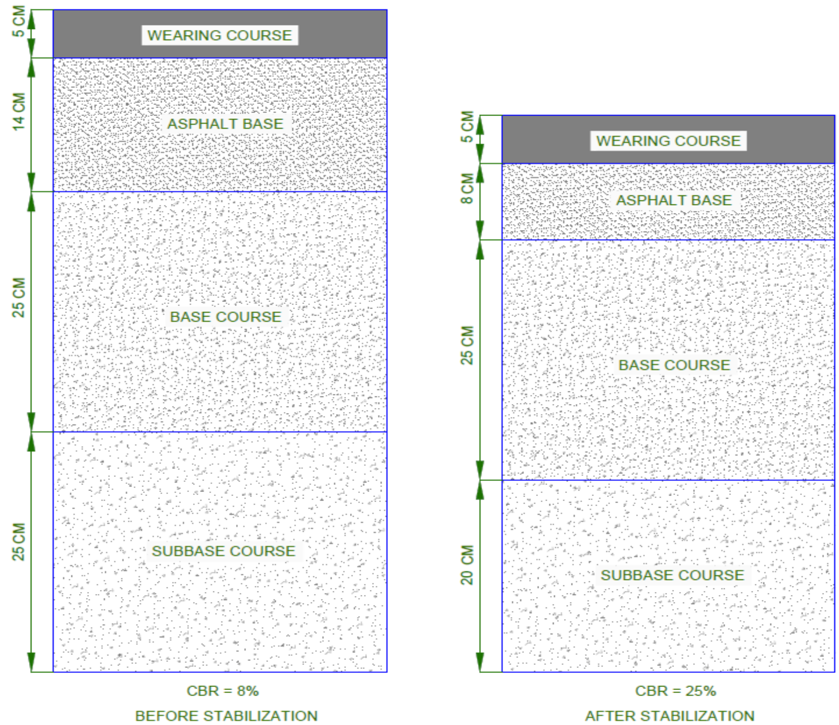


Figure 4.31: Pavement cross section before and after stabilization

4.10 DAMAGE PROGRESSION ANALYSIS

The damage progression is assessed by comparing the swelling ratio while soaking and difference in unsoaked and soaked CBR for control and stabilized soil specimens respectively. Figure 4.32 and 4.33 elaborates that the swell ratio decreased by the addition of stabilizing agents either hydrated lime or cement. Swell ratio was observed minimum when 5% hydrated lime content was used for stabilization. Which shows that the damage progression is lesser when optimum amount of lime (5%) was used during stabilization. While in case of cement the trend is linear and the swelling ratio/damage progression always decreased by increasing the amount of cement for stabilization. The swell ratio was found zero with addition of cement at 2 percent and beyond for cement stabilization. The difference in unsoaked and soaked CBR is presented separately for lime and cement stabilization in figure 4.34 and 4.35. When considering lime treated specimens that difference in unsoaked and soaked CBR was initially increased by adding lime but it decreased up to a certain limit upto optimum lime contents but beyond optimum lime it again increased. Therefore damage progression is minimum for specimen having optimum lime contents. In case of cement stabilization difference in unsoaked and soaked CBR decreased gradually by increasing cement proportion. Which means that increase in cement percentage results in decrease for the damage progression caused by soaking. Following graphs depict the tendencies as described above for both lime and cement mixed soil specimens.

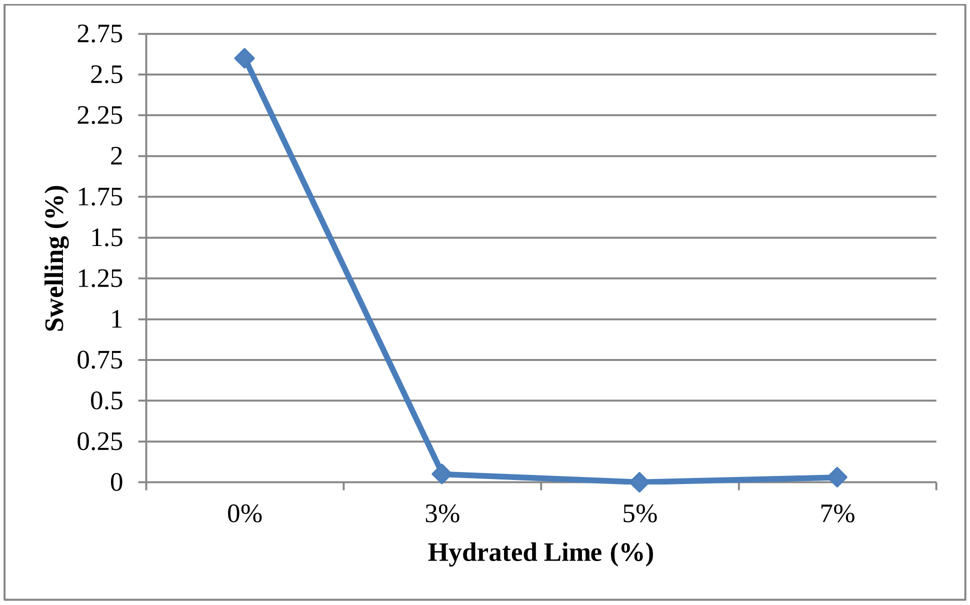


Figure 4.32: Swelling ratio for control and lime stabilized soil specimen

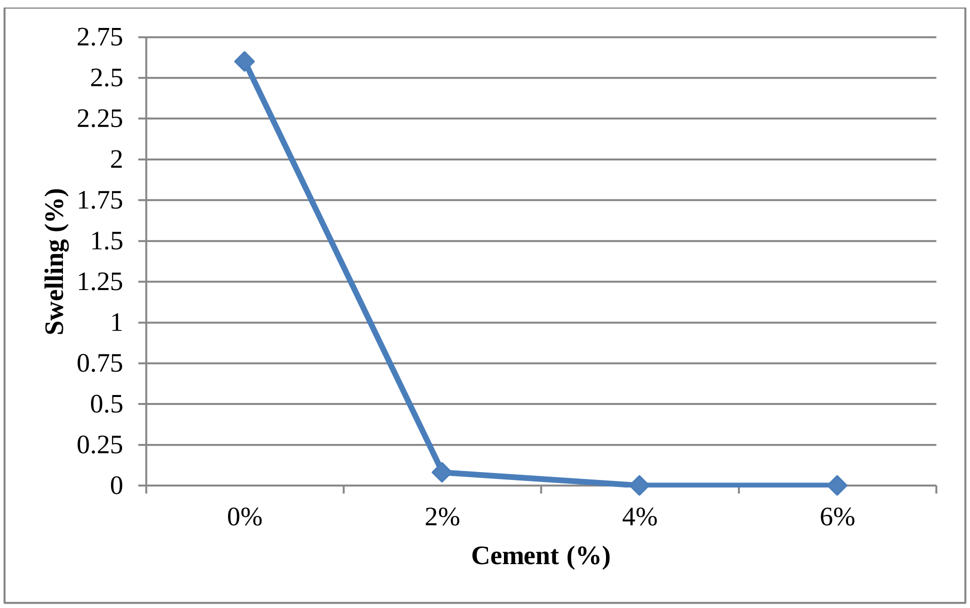


Figure 4.33: Swelling ratio for control and cement stabilized soil specimen

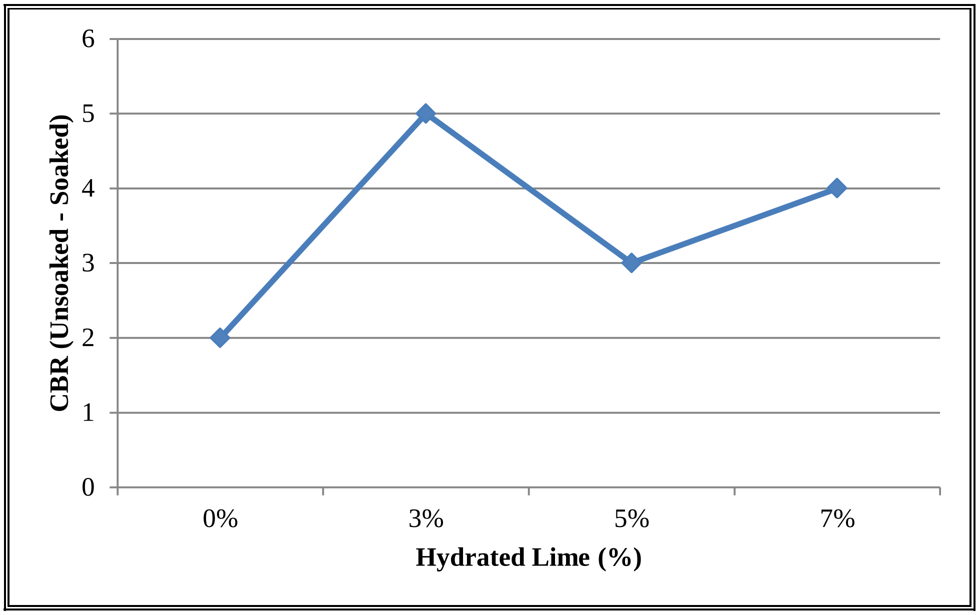


Figure 4.34: Difference in Unsoaked / soaked CBR Vs. hydrated lime

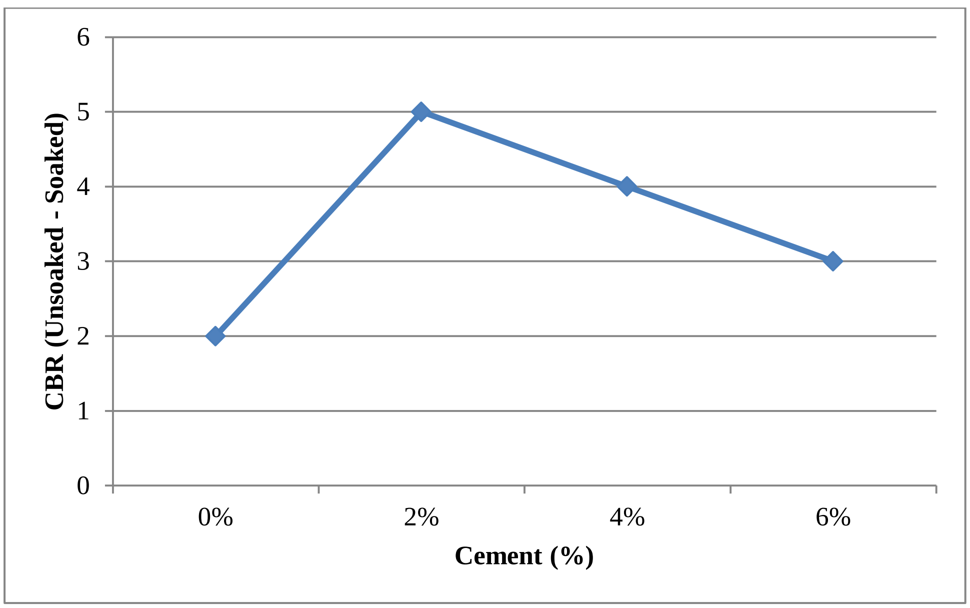


Figure 4.35: Difference in Unsoaked and soaked CBR Vs. cement percentage

4.11 SUMMARY

At start of this chapter the properties of control subgrade soil were presented and suitability of subject soil was checked by NHA and TRB criterion. The results of Eades and Grim pH test were elaborated and the optimum amount of lime for that particular subgrade were found as 5% of the dry weight of soil. Further strength testing results of CBR and UCS test were discussed for varying proportions of lime (3%, 5% and 7% respectively). It was concluded from strength testing results of lime stabilization that the specimen contained optimum amount of lime provides the highest UCS strength and CBR in case of Lime stabilization. The results of cement stabilization were also summarized in the form of tables and figures. The cement stabilization strength testing showed that as the amount of cement contents for stabilization increased that will cause to increase the CBR and UCS gradually. The analysis of UCS data for lime and cement stabilization has been presented and explained with graphs generated by Minitab 15. At the end cost analysis of lime and cement stabilization has been made to amass the benefits of stabilization.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

The purpose of this research was to prove the effectiveness of hydrated lime and Portland cement for the stabilization of unsuitable subgrade soil that have naturally low CBR and UCS. The basic engineering properties of subgrade soil were evaluated and soil material was characterized. Keeping in view the criterion of stabilization further soil stabilization with hydrated lime and cement were proceeded from their respective optimum stabilizers contents. The optimum lime contents for lime stabilization were determined as 5 % by Eades and Grim pH test. For lime stabilization the proctor test was conducted for 3%, 5% and 7% of lime contents and their respective OMC's were determined. Specimens for CBR and UCS tests were prepared for varying proportions (3%, 5% and 7%) of lime at their respective OMC's and were tested according to their respective standards following the unsoaked and soaked conditioning. On the other hand 4 % cement contents were selected as effective cement contents for stabilization by working with prevailing cost comparison (NHA rates). While three proportions of cement 2%, 4% and 6% were selected to check the effectiveness of cement stabilization for the subject soil. Firstly effective or optimum cement contents were determined for each percentage of cement and further specimen were prepared and tested for CBR and UCS tests. The major findings from compaction tests, CBR test, UCS test, analysis of experimental results and cost comparison are concluded as follows:

5.2 CONCLUSIONS

Based on engineering properties, strength testing and analysis of results following have been concluded for this specific soil that:

- The subject soil from Karachi Lahore Motorway (KLM) Hyderabad-Sukkur section near Setharja-Nawabshah region is low plastic clay (CL), classified as A-6 soil. Under wet conditions the soil is highly susceptible to loss of strength.
- Due to low CBR and extremely low compressive strength under moist conditions, the soil in present state cannot be considered as a subgrade layer for pavement construction.

- The soil is highly reactive with lime due to abundant clay content present in the soil. The optimum lime percentage required for stabilization has been found 5% of the dry weight of soil.
- The MDD and OMC for the control soil specimen were found as 1.8 g/cc and 14.5% respectively.
- The OMC for 3%, 5% and 7% lime specimen was found as 11%, 15.5% and 12.3% respectively. While it was 10.2%, 9.25% and 8.5% for 2%, 4% and 6% soil cement specimen respectively.
- The MDD for 3%, 5% and 7% lime specimen was 1.92, 1.75 and 1.85 g/cc respectively. While it was 2.26, 2.41 and 2.63 g/cc for 2%, 4% and 6% soil cement specimen.
- The specimen prepared at optimum lime contents produced the maximum CBR and unconfined compressive strength (UCS).
- The enhanced soaked CBR of lime treated soil specimen prepared at optimum amount of lime (5%) was observed as 1150% high as compared with control soil with maximum soaked UCS value as obtained as 295 psi at that optimum lime contents.
- The effect of soaking stood maximum at 46.34 % for control soil specimen while it was obtained 38.42, 34.92 and 38.56 for 3%, 5% and 7% lime-soil mixtures.
- The strength development index (SDI) and lime reactivity were found as 6.2 and 254 respectively for soil lime mixture having optimum 5 percent lime contents.
- The effective or optimum amount of cement by prevailing cost comparison following NHA composite schedule of rates (CSR - 2014) was found as 4% of the dry weight of soil.
- It was concluded by the CBR and UCS test results that the amount of cement content (% by weight) was directly proportional to the CBR and UCS strengths gain.
- The soaked CBR for 2%, 4% and 6% soil cement specimen was 18%, 26% and 30% respectively.
- The soaked UCS for 2%, 4% and 6% soil cement specimen was obtained 152 psi, 172 psi and 210 psi respectively.
- The strength development index (SDI) was 4.12, 4.76 and 5.71 for 2%, 4% and 6% soil cement respectively.

- Unsoaked to soaked strength ratio showed always increase by increasing cement contents. Its values were 0.54, 0.72, 0.73 and 0.76 for control and for 2%, 4% and 6% soil cement specimens respectively.
- Cement reactivity was observed maximum for the specimen having highest cement contents of 6%. Its values as obtained were 169, 195 and 234 for 2%, 4% and 6% soil cement respectively.
- The effect of soaking was about 23.64% for 6% cement contents and it was minimum. While it was maximum at 46.34% for the control soil specimen.
- The analysis of UCS data by 2^3 full factorial design of experiment indicated that all three factors including percentage of stabilizer, conditioning and curing period were significant.
- It was observed that percentage stabilizer is the most significant factor in case of cement stabilization. When the stabilizer value changed from lower to higher proportion, substantial rise in UCS was observed.
- As per AASHTTO design method, significant decrease in structural number was observed due to addition of lime and cement with control soil as stabilization agents. Hence reduction in the highway construction cost is deduced.
- In this research it was concluded that soil subgrade with CBR at 8% stabilized with 5% lime and 4 % cement reduced 27% of highway pavement construction cost.
- Whereas it was also observed that control soil subgrade with CBR at 2% stabilized with 5% lime and 4 % cement reduced 49% of highway pavement construction cost.
- The damage progression was assessed by comparing the swelling ratio and difference in unsoaked to soaked CBR of control and treated soil specimens.
- Damage progression for lime treated specimen was minimum at the optimum lime contents comparing with control and at other lime percentage treated specimen.
- In case of cement stabilization damage progression reduced by increasing cement proportions. Which showed that swelling and difference in conditioning was minimum for 6% soil-cement specimen.

- Cement and lime stabilization are most economical and feasible techniques than all of other conventional method applicable in the field.

5.3 RECOMMENDATIONS

- These recommendations are peculiar to a case study as discussed and findings are specifically applicable in the particular soil. Any other soil having same characteristics can be have applicability for the applicability of the observed values for the purpose of stabilization.
- As discussed soil stabilization techniques in highway construction are not very common in Pakistan. Lime and cement stabilization offers technically and financially feasible solution to many highway engineering problems. These techniques need to be employed more frequently to improve the strength properties of unsuitable soil.
- In this research only soaking conditions were considered while further research work may be carried out for taking into account freeze-thaw conditioning.
- Hydrated lime was used for lime stabilization in this research while other different lime types like slaked lime and quick lime may be used for the adaption of best lime type for the subgrade soil stabilization.
- Variable temperature and moisture curing may lead to the damage progression analysis, further testing can be done by considering variable temperature curing.
- Field investigations should be carried out to implement the findings of research. Trial sections can be planned in coordination with Highway Research and Training Center (HRTC) NHA.
- As the scope of this thesis was to evaluate the strength properties of lime and cement stabilized soil with CBR and UCS tests. It can become more beneficial by including repeated load tests, resilient modulus and dynamic modulus tests.
- On a test section more savings can be quantified in terms of service life, maintenance management, reduced wear & tear, less fuel consumption, time saving and damage retardation due to stabilized subgrade.

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