EFFECTS OF CEMENT AND LIME STABILIZATION ON GEOTECHNICAL CHARACTERISTICS OF A REGIONAL CLAY

by
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ABSTRACT

In the present investigations cement and lime stabilization of a regional medium expansive clay has been carried out to find the effects of cement and lime stabilization on the selected soil. Here the said clay sample was collected from Mouchak in Gazipur district. The collected sample of the selected site was inorganic clay of low plasticity and medium expansive. As additives, ordinary Portland cement and slaked lime were used in amount of 2%, 5% and 8% of dry weight of collected soil sample individually for index tests, shrinkage limit tests, linear shrinkage tests, standard proctor tests and unconfined compressive strength tests. The said additives were 5% of dry weight of collected soil individually for direct shear tests. Unconfined compressive strength tests were performed on soil-additive mixtures compacted at optimum moisture content (OMC) and then consolidated undrained direct shear tests were performed on soil-additive mixtures compacted at wet side of OMC at 95% of maximum dry density. Cement stabilized samples for both unconfined compressive strength tests and consolidated undrained direct shear tests were cured for 7, 14, 28 and 56 days individually. On the other hand, lime stabilized samples for unconfined compressive strength tests were cured for 7, 14, 28 and 56 days individually but for consolidated direct shear tests the samples were cured for 7, 28 and 56 days individually. Comparison among unconfined compressive strengths which were obtained from different researches on cement and lime stabilization have also been studied.

Compared with the untreated samples; plasticity indices and percentages of linear shrinkages of the selected soil-additive mixtures do not change significantly while shrinkage limits of the selected soil-additive mixtures increased significantly. Change in OMC and maximum dry density due to selected cement and lime stabilization is also not significant.

Most of the data found from unconfined compressive strength tests show that cement is better choice than lime to increase unconfined compressive strength of the selected soil, although more study is required to determine the effects of longer curing period and more admixture content. The range of unconfined compressive strength of the selected soil-cement mixtures is 530 kN/m^2 to 2195 kN/m^2. For lime treated soil, the said range is 605 kN/m^2 to 1990 kN/m^2. It is found that compressive strengths of samples treated with 8% cement and cured for 7 and 28 days satisfied the PCA (1956) for the compressive strength

of soil cement mix. It is also found that for all cement contents and all curing ages of the present investigation except 2% cement content with 56 days curing, compressive strength of the stabilized samples fulfilled the requirements of soil-cement mix for use in road sub-base and base subjected to light traffic, as proposed by Ingles and Metcalf (1972).

For selected cement treated samples, axial failure strains and initial tangent moduli do not show a specific trend. The ranges of axial failure strains and initial tangent moduli for selected cement treated samples are 0.57% to 1.2% and 60200 kN/m^2 to 526700 kN/m^2 respectively. For selected lime treated samples too, initial tangent moduli do not show a specific trend. Most of the data of unconfined compressive strength tests show that axial failure strains of selected lime treated samples decrease with increments of lime contents in the samples. The ranges of axial failure strains and initial tangent moduli for selected lime treated samples are 0.76% to 4.6% and 45300 kN/m^2 to 95800 kN/m^2 respectively.

It is observed that consolidated undrained cohesions of lime treated samples decrease with increases of lime in the samples while the said cohesions of cement treated samples increase with increases of cement in the samples. On the other hand, consolidated undrained angles of internal frictions increase with increases of lime in the samples while the said angles of internal frictions of cement treated samples decrease with increases of cement in the samples.

For both cement and lime treated selected clay samples, shear stress vs. shear displacement curves show that the clay samples are over consolidated clay samples but all shear displacement vs. corresponding changes in height curves do not show the said nature.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1
INTRODUCTION

1.1 General

In Bangladesh, land development activities have been increased significantly in recent time. Every year new residential, commercial and recreational areas are being developed by raising low lands. Houses, markets, roads etc. are frequently being constructed on newly reclaimed ground. In the reclamation process, filling soils are generally collected from readily available borrow pits. But the properties of these soils do not always comply with the specified requirements and thus create problems in the construction phase. Longer monsoon, heavy rainfall and flood are other problems in land development works in Bangladesh. To mitigate the problems it is necessary to stabilize the in-situ soil and the filling soil.

Winterkorn (1975) defined soil stabilization as the collective term for any physical, chemical or biological methods, employed to improve certain properties of a natural soil to make it serve adequately an intended engineering purpose. The intended engineering purpose may be increasing strength, reducing erodibility, reducing compressibility, reducing distortion under stress, controlling shrinking and swelling, controlling permeability, reducing water pressures, prevention of detrimental physical or chemical changes due to environmental conditions (freezing/thawing, wetting/drying), reducing susceptibility to liquefaction, reducing natural variability of borrow materials or foundation soils, etc.

In recent years, the stabilization of soil with suitable admixtures such as lime, cement and bitumen have been successfully used on an increasing scale for the construction of road foundation in U.S.A., U.K., South Africa, India and in many other countries.

Since soils exist in a broad range of types, differing markedly in their properties and since different soils react differently to various stabilizing agents, it is expected that a wide range of stabilization process may be in existence. However, in short four groups of soil stabilization techniques are being mentioned as follows:

i) Mechanical stabilization: Soil density is increased by the application of short-term external mechanical forces, including compaction of surface layers by static, vibratory, or impact rollers and plate vibrators, and deep compaction by heavy tamping at the surface or vibration at depth.

ii) Hydraulic stabilization: Free-pore water is forced out of the soil via drains or wells. In coarse-grained soils this is achieved by lowering the groundwater level through pumping from boreholes or trenches; in fine-grained soils the long-term application of external loads (preloading) or electrical forces (electrokinetic stabilization) is required. Traditional techniques have benefited from the development of geosynthetics, as in the case of vertical drains.

iii) Stabilization by inclusions and confinement: Reinforcement by fibers, strips, bars, meshes, and fabrics imparts tensile strength to a constructed soil mass. In situ reinforcement is achieved by nails and anchors. Stable earth-retaining structures can also be formed by confining soil with concrete, steel, or fabric element (sand bags).

iv) Physical and chemical stabilization: Stabilization by physically mixing additives with surface layers or columns of soil at depth, by heating the ground, freezing the ground, etc. are discussed in this group. Additives include natural soils, industrial by-products or waste materials and cementitious and other chemicals which react with each other and/or the ground. Heating evaporates water and causes permanent changes in the mineral structure of soils, freezing solidifies part or all of the water and bonds individual particles together.

The choice of a method of ground improvement for a particular object will depend on many factors. Type and degree of improvement required, type of soil, geological structure, seepage conditions, cost (the size of the project may be decisive), availability of equipment and materials, the quality of work required, construction time available, possible damage to adjacent structures or pollution of groundwater resources, durability of the materials involved (as related to the expected life of the structure for the given environmental and stress conditions), toxicity or corrosivity of any chemical additives, reversibility or irreversibility of the process, reusability of components, reliability of methods of analysis and design, feasibility of construction control and performance measurements, etc. are the major factors which play vital role on the selection of Soil stabilization procedure.

Increased strength and stiffness, better volume stability (less moisture sensitivity), increased durability, etc. are the engineering benefits which encourage the selection of stabilization using cement. On the other hand, increase of workability, increase of optimum water content for compaction, increase of strength of clayey soil, improved volume stability, reduction of wheel path rutting and potholing, etc. are the engineering benefits which encourage the selection of stabilization using lime. Lime is primarily used for the treatment of clayey soils. It is not very effective for cohesion less soils unless other materials are also added, such as fly ash, furnace slag, or other pozzolans. A soil where lime treatment leads to cementation is termed as ―reactive” soil. The most reactive clays are those containing minerals belonging to the montmorillonite group, distinguished by a three-layer primary element in their crystalline structure and a high base exchange capacity. Less reactive are illites, kaolinites and chlorites. So, it is sensible to carry out laboratory soil testing in order to evaluate the reactivity of a particular material.

Anyway, cementation is not the only characteristics of stabilized soil which is to be evaluated. Stress-strain-strength properties, moisture-density relationship, liquid limit, plastic limit, shrinkage limit, percentage of linear shrinkage, etc. are the important characteristics of stabilized soil which are to be known also. The initial tangent modulus and ultimate failure strain are often used for estimation of soil settlement and elastic deformation analysis. Moisture-density relationship is required to know the optimum moisture content which is used to find out optimum binder content. Liquid limit and plastic limit of soil are very important properties of fine grained soil and its value is used to classify fine grained soil and calculate activity of clays and toughness index of soil. Moreover, they also give us information regarding the state of consistency of soil on site. In addition, they also can be used to predict the consolidation properties of soil while calculating settlement of foundation. A shrinkage limit test gives a quantitative indication of how much moisture can change before any significant volume change. The shrinkage limit is useful in areas where soils undergo large volume changes when going through wet and dry cycles (e.g. earth dams). It is necessary to know the percentage of linear shrinkage in order to determine the exact size of moulds needed for producing bricks of given dimensions. To know degree of expansion linear shrinkage is also important.

1.2 Objectives of the Present Research Work

This project work has been undertaken to investigate about the stress-strain-strength properties, moisture-density relationships, liquid limits, plastic limits, shrinkage limits and percentages of linear shrinkages of selected clay which has been stabilized with three different cement contents (2%, 5% and 8% of soil's oven dried weight respectively) and lime contents (2%, 5% and 8% of soil's oven dried weight respectively) respectively.

The above mentioned investigations have fulfilled some objectives as follows:

(i) investigation on the effects of admixture types (cement and lime) and admixture contents on liquid limits, plastic limits, shrinkage limits, percentages of linear shrinkages and moisture-density relations,

(ii) investigation on the effects of admixture types (cement and lime), admixture contents and curing periods on unconfined compressive strengths and

(iii) finding out the effects of admixture types (cement and lime), and curing periods on undrained shear strength parameters of stabilized soil respectively.

1.3 Research Scheme

Sample of soil was collected from Mouchak in Gazipur district (the north latitude and east longitude of the soil collection point are about 24.027868^0 and 90.299284^0 respectively). Before collecting sample, 2~3 feet top soil was removed and then the sample was collected.

The sample was air dried for five days. Afterwards, the following tests procedures were adopted to determine the various parameters.

(i) Liquid limit and plastic limit of the sample were obtained using ASTM D4318 - 10.
(ii) Shrinkage limit of the sample was obtained using ASTM D427-9804.
(iii) Linear shrinkage of the sample was determined using BS 1377.
(iv) Optimum moisture content and maximum dry density of the sample were calculated out using ASTM D698 – 12.
(v) Unconfined compressive strength of the sample which was compacted using compaction method used in ASTM D698-12 at optimum moisture content was found out using unconfined compression strength test (ASTM D2166/D2166M-13).
(vi) The sample was mixed with water to achieve more water content than optimum moisture content at which 95% of corresponding maximum dry density was gained. Then the sample was cured for 50 minute. After curing, the sample was compacted using compaction method used in ASTM D698-12. Then undrained consolidated shear strength parameters were investigated with consolidated undrained direct shear test (ASTM D6528-17).

The sample of the collected soil was mixed with cement at various proportions. Similarly, the sample was also mixed with lime to the same proportions. Then the tests described in 1.3 (i) to 1.3 (iv) were carried out. The prepared soil samples with various lime and cement contents will be cured for 7, 14, 28 and 56 days respectively. Then the tests described in 1.3 (v) to 1.3 (vi) will be carried out.

CHAPTER 2
LITERATURE REVIEW

2.1 General

For improving volume stability, strength and stress-strain properties, permeability, and durability it needs to stabilize soils. The development of high strength and stiffness is achieved by reduction of void space, by bonding particles and aggregates together, by maintenance of flocculent structures, and by prevention of swelling. Good mixing of stabilizers with soil is the most important factor affecting the quality of results. Two most commonly used stabilizers for improving the physical and engineering properties of soils are cement and lime.

The improvement of in-situ and reclaimed soils undergone by cement and lime treatment have been more widely employed in the past recent years, especially in stabilization of soils for various applications (Ingles and Metcalf, 1972; Mitchell, 1981; IRC, 1976; Macham et al., 1977; Kezdi, 1979; Broms and Boman, 1979; Markus et al., 1979; NAASRA, 1986; TRB, 1987; Hausmann, 1990; TRL, 1993; Bell, 1993).

This review looks into the fundamental concepts, mechanisms of cement and lime treatments, factors influencing the properties of cement and lime stabilized soils, characteristics of cement-treated and lime-treated soils, and the applications of cement and lime stabilization.

2.2 Cement Stabilization

2.2.1 General

The most commonly and successfully used stabilizer for soil stabilization is ordinary Portland cement due to its availability, easy quality control and easy handling characteristics. Soil stabilization with cement is also currently one of the most widely used methods. Portland cement and soil mix of the proper moisture content produce soil-cement, a structural material that is hard and durable. Soil-cement has been used mainly as bases under concrete pavement for highway and airfields. It is also used for construction of rural roads, earth dams and foundation of buildings. Any type of cement may be used for soil stabilization but ordinary Portland cement is most widely used. The two principal factors that determine the suitability of a soil for stabilization with ordinary Portland cement are, firstly, whether the soil and cement can be mixed satisfactorily and, secondly, whether, after mixing and compacting, the soil-cement will harden adequately. Cement has the following two important effects on soil behavior (NAASRA, 1986):

(i) It greatly reduces the moisture susceptibility of soils, giving to stabilized materials enhanced volume and strength stability under variable moisture conditions.

(ii) It can cause the development of inter particle bonds in granular materials, endowing the stabilized material with a useful tensile strength and high elastic modulus.

2.2.2 Materials for Cement Stabilization

The materials to be considered in cement stabilization are the cement, soil and water. Water, both in quantity and quality, and a number of undesirable materials principally organic matter and sulphate salts are important.

2.2.2.1 Cement

Portland cement is the most commonly used and effective additive for soil stabilization. It has both adhesive and cohesive properties, enabling it to bind mineral fragments into a solid mass, i.e., those that can set and harden in the presence of water and so it is termed as "Hydraulic Cement". These consist primarily of silicates and aluminates of lime, made from lime stones and clays (or shales), which are grounded, blended and fused in a kiln and crushed to a powder. The usual hydraulic cement used is known as Portland cement. ASTM C150 defined Portland cement as a hydraulic cement produced by pulverizing clinker consisting essentially of hydraulic calcium silicates, usually containing one or more of the forms of calcium sulphate. Type I Portland cement is the most widely used in soil stabilization. Typical composition of ordinary Portland cement is presented in Table 2.1.

2.2.2.2 Soil

Any soil, with the exception of highly organic materials, may be treated with cement. Ingles and Metcalf (1972) reported that for cement stabilization, the upper limit of particle size is about 8 cm (3 in.) or one-third of the thickness of the compacted layer, but a maximum size of 2 cm (3/4 in.) is to be preferred to give a good surface finish. The lower limit is about 50 percent passing the B.S. No. 200 sieve (0.08 mm), with a liquid limit not greater than 50 and a plasticity index not greater than 18.

Table 2.1: Typical composition of ordinary Portland cement (after Mindess and Young, 1981)

Chemical Name	Chemical Formula	Weight (Percent)
Tricalcium silicate	$3CaO.SiO_2$	50
Dicalcium silicate	$2CaO.SiO_2$	25
Tricalcium aluminate	$3CaO.Al_2O_3$	12
Tetracalcium-aluminoferrite	$4CaO.Al_2O_3.Fe_2O_3$	8
Calcium sulfate dihydrate (gypsum)	$CaSO_4.2H_2O$	3.5

NAASRA (1986) provides the following guide to property limits for effective cement stabilization:

Table 2.2: Property of soil for efficient treatment of soil with cement

Property Limit	Value
(a) Particle Size	
Maximum size*	75mm
Passing 4.75 mm	>50%
Passing 425 µm sieve	>15%
Passing 75 µm sieve	<50%
Finer than 2 µm+	<30%
(b) Plasticity	
Liquid limit	<40
Plasticity Index	<20

* Depends on mixing plant

\+ At upper limit may need pretreatment with lime

2.2.2.3 Water

There is no precise measure of the quality of water required, it is being generally regarded that "potable" water is satisfactory. However, highly organic water or water containing high concentration of sulphates (e.g., above 0.05 percent) may cause problems and should be avoided. Water with a high salt content (chloride in sea water) may be used, provided efflorescence is not likely to be a problem. Most importantly, the quantity of water added to cement-treated mix is determined by the requirements of the maximum dry density and not that needed for cement hydration.

2.2.3 Mechanism of Soil-Cement Stabilization

The reaction between cement and clay has been investigated by a number of investigators (Herzog, 1963; Saitoh et al., 1985). Major constituents of cement, which have a distinct effect on the strength aspect of soil-cement mix, are calcium di-silicate, calcium tri-silicate, and free lime. Calcium tri-silicate sets fast and is responsible for immediate strength gain. Free lime may bring about base exchange capacity and change the texture of the soil. Calcium di-silicate is responsible for long term strength due to hydration reaction (Jah and Sinha, 1977).

Shetty (1982) explained that anhydrous cement compounds when mixed with water, react with each other to form hydrated compounds of very low solubility. The hydration of cement can be visualized in two ways. The first is "through solution" mechanism. In this the cement compounds dissolve to produce a supersaturated solution from which different hydrated products get precipitated. The second possibility is that water attacks cement compounds in the solid state converting the compounds into hydrated products starting from the surface and proceeding to the interior of the compounds with time. It is probable that both "through solution" and "solid state" types of mechanism may occur during the course of reactions between cement and water. The former mechanism may predominate in the early stage of hydration in view of large quantities of water being available, and the latter mechanism may operate during the later stages of hydration. Shetty (1982) estimated that on an average 23 percent of water by weight of cement is required for chemical reaction. This 23 percent of water chemically combine with cement and therefore it is called bound water. A certain quantity of water is imbibed within the gel-pores this water is known as gel-water. It can be said that bound water and gel-water are complementary to each other. It has been further estimated that about 15 percent by weight of cement is required to fill the gel-pores. Therefore, a total 38 percent of water by weight of cement is required for the complete chemical reactions and occupy the space within gel-pores.

In addition, the hydration of cement leads to a rise in the pH value of the pore water, which is caused by the dissociation of the hydrated lime. The strong bases dissolved the soil silica and alumina (which are inherently acidic) from both the clay minerals and amorphous materials on the clay particle surfaces, in a manner similar to the reaction between a weak acid and a strong base. The hydrous silica and alumina will then gradually react with the calcium ions liberated from the hydrolysis of cement, to form insoluble compounds (secondary cementitious products) which harden when cured to stabilize the soil. This secondary reaction is known as the "pozzolanic reaction".

The reactions which take place in cement stabilization can be represented in the following qualitative equations; the reactions given there are for the tri-calcium silicates (C_3S) only, because they are the most important constituents of Portland cement:

$$C_3\mathrm{S} + H_2\mathrm{O} \rightarrow C_3S_2H_x(\text{hydrated gels}) + \mathrm{Ca}(OH)_2 \qquad (\ )$$

$$\mathrm{Ca}(OH)_2 \rightarrow Ca^{++} + (\mathrm{OH})^- \qquad (\ )$$

$$Ca^{++} + (\mathrm{OH})^- + SiO_2(soil\ silica) \rightarrow \mathrm{CSH} \qquad (\ )$$

$$Ca^{++} + (OH)^- + Al_2O_3\ (soil\ alumina) \rightarrow CAH \qquad (\ )$$

The cementation strength of the primary cementitious products is much stronger than that of the secondary ones. At low pH values (pH < 12.6), the following reaction will occur:

$$C_3S_2H_x \rightarrow CSH + Ca(OH) \qquad (\ )$$

In Equation (1), $C_3S_2H_x$ and $Ca(OH)_2$ are primary cementitious products while in equations (3) and (4), CSH and CAH are secondary cementitious products. However, during the pozzolanic reaction, the pH drops, and a drop in the pH tends to promote the hydrolysis of $C_3S_2H_x$, to form CSH. The formation of CSH is beneficial only if it is formed by the (pozzolanic) reaction of lime and soil particles, but it is detrimental when it (CSH) is formed at the expense of the formation of the $C_3S_2H_x$, whose strength-generating characteristics are superior to those of CSH. The cement hydration and the pozzolanic reaction can last for months, or even years, after the mixing, and so, the strength of cement-treated clay is expected to increase with time.

2.2.4 Factors Governing the Characteristics of Soil-Cement Mix

The hardening characteristics of cement treated soil mixtures are developed by a number of factors. A sound understanding of the behavior of the mixture is possible only by an extensive study of the nature and extent of these factors. Factors affecting the properties of soil-cement mix are broadly classified as soil factors and production factors. Soil factors deal with the composition of the untreated soil and its response to cement and the production factors include the quality of water and cement, the uniformity of mixing, compaction and curing conditions. A brief review of the important factors is presented in the following several sections.

2.2.4.1 Characteristics of Soil

Any type of soil, with the exception of highly organic soils or some highly plastic clays, may be stabilized with cement. Bell (1993) reported that although particles larger than 20 mm diameter have been incorporated in soil-cement, a maximum size of 20 mm is preferable since this allows a good surface finish. At the other extreme, not more than about 50% of the soil should be finer than 0.08 mm. Typically soils containing between 5 and 35% fines provide the most economical soil-cement stabilization. Indian Road Congress (1973a) does not recommend cement stabilization for clay soils having plasticity index greater than 22. Soils with large clay content are difficult to mix and high additive contents are required for an appreciable change in properties. Under laboratory conditions, with elaborate attention to mixing, such heavy clays may be successfully stabilized but, in practice, it is not usual to attempt directly to stabilize with cement a clay soil with a liquid limit exceeding 45 and plasticity indices above 18% (Croft, 1968).

It is often possible, however, to stabilize with cement such heavy clays after pre-treatment (modification), with either cement or, more commonly hydrated lime. The purpose of the pre-treatment with 2-3 percent of cement or lime content is to reduce the plasticity and render the soil more workable. After curing (compacted or loose) for one to three days, the modified soil is then stabilized with cement in the usual manner. IRC (1973a) does not recommend cement stabilization for road construction for soils having organic matter content greater than 2 percent. Ahmed (1984) showed for a silty soil of ASSHO group A-5 having organic matter content of about 4% by weight, the strength increase of the soil beyond 8% cement content is insignificant.

2.2.4.2 Cement Content

For a given soil that reacts normally with cement, the cement content determines the nature of the cement-treated soil mixture. The proportion of cement alters the plasticity, the volume change, the elastic properties, the resistance to wet-dry alternations and other properties in different degrees for different soils. Catton (1940) and Portland Cement Association, PCA (1956) recommended average cement requirement for moisture-density and wetting-drying tests of various fine-grained soils, which have been reported by Hossain (1986). Cement content significantly influences the physical and engineering properties of fine-grained soils (Mitchell, 1976; Ahmed, 1984; Hossain, 1986; Serajuddin and Azmal, 1991; Serajuddin, 1992; Bell, 1993). In general, increasing cement content has got the following effects on soil-cement mix:

(i) Increase in the values of liquid and plastic limits

(ii) Reduction in the values of plasticity index, shrinkage limit, swell, volume change and linear shrinkage

(iii) Increase or reduction in maximum dry density

(iv) Increase in unconfined compressive strength, flexural strength and stiffness, and CBR value.

It is to be noted that quantity of cement required for stabilization increases as soil-plasticity increases. For highly plastic soil as much as 15 to 20% cement by weight is required to bring about the hardening of the soil (Yoder and Witczak, 1995).

2.2.4.3 Types of Cement

Felt (1955) made experiments on three different types of soils to find out the effects of cement type on cement-treated soil mixtures. Felt (1955) compared the results of compaction tests, compressive strength tests and the wet-dry tests made on soils treated by normal Portland cement (Type-I) and air-entraining Portland cement (Type-IA). It was found that moisture-density relationships, compressive strengths and the soil-cement losses in the wet-dry tests were almost the same. This indicates that these two types of cement can be used interchangeably in soil-cement construction.

It was further observed on experimentation with Type-III cement that the optimum moisture contents and maximum densities obtained are approximately the same for Type-I and Type-III cements. Felt (1955) also found that influence of Type-III cement on strength of different soils varies. For loamy sand, the 7 and 28-days strength for Type-III cement were about 2 and 1.4 times those for Type-I cement respectively. For a silty-clay loam, the strength for Type-III was only slightly higher than that for Type-I cement.

2.2.4.4 Mixing and Compaction

To achieve better results by cement stabilization, efficient mixing and compaction are essential pre-requisites. Equipment used and the time lag between mixing and compaction also influence both the strength and durability characteristics of soil-cement mixtures. The degree of mixing using particular equipment and following a specific procedure depends on the soil type as well as on its degree of pulverization and its moisture content. The efficiency of mixing also depends on the mixing time. An increased wet mixing time usually increases the optimum moisture content, reduces the compressive strength and increases the weight losses during the wet-dry tests.

Studies on cement hardening and certain in-situ experiences give rise to the idea that waiting between wet mixing and compaction could increase the compression strength of the soil-cement mix. Hungarian experience supported this assumption. But Marshall (1954) claimed that this waiting period would lead to strength reduction in case of several soils. Felt (1955) also showed that the compressive strengths of cement treated soil mixtures are reduced with the increasing periods of mixings. In Britain, the current specifications require that compaction be completed within 2 hours of mixing being initiated (Maclean and Lewis, 1963). Ingles and Metcalf (1972) also reported that prolonged delay between mixing and compaction reduce the magnitude of unconfined compressive strength significantly.

2.2.4.5 Curing Time and Curing Temperature

The environmental conditions under which curing takes place have significant influences on the extent to which a soil may be stabilized with cement. The unconfined compressive strength, flexural strength and stiffness of soil-cement mix increase with the increase in the curing age. Soil-cement must be moist cured during the initial stages of its life so that moisture is sufficient to meet the hydration needs of the cement can be maintained in the mixture. Curing in the laboratory moist room meets the requirements of humidity and temperature. But in field a loose material such as straw, foliage, reed, earth etc. must cover the fresh surface. Another way is to cover the surface with a waterproof protective coating, usually bituminous, which then keeps the water in the pavement.

Curing temperature also markedly influences the strengths of cement-treated soil mixtures. Clare and Pollard (1953) showed that when the test-temperature is around 25°C (77°F), the 7-days compressive strength increases with the increase in temperature by 2 to 2.5 percent per degree. They also found that taking the compressive strength as the sole-criterion of quality of cement-treated soil mixture, less cement is needed in warm weather than in cold weather.

2.2.5 Properties of Cement Stabilized Soil

The properties of soil-cement mixtures vary with several factors as mentioned in the previous sections. The major benefits of cement stabilized soils are increased strength and stiffness, better volume stability and increased durability. The properties of soil-cement mix have been summarized by a number of investigators (Ingles and Metcalf, 1972; Kezdi, 1979; Mitchell, 1981; NAASRA, 1986; Bell, 1993). In the following several sections the various physical and engineering properties of cement-treated soil have been reviewed.

2.2.5.1 Unconfined Compressive Strength

The compression strength value can characterize the degree of soil-cement-water reaction and the progress of hardening. It is usually the compression strength value, which serves as a criterion for determining cement requirements for the construction of soil-cement structure. Evaluation of stabilized soil with admixture like cement is widely made with the help of compressive strength of stabilized mix. In Britain, usual practice is to specify the desired stabilities of most soil-cement mixtures in terms of minimum unconfined compressive strengths. The most recent specification for soil-cement requires a minimum 7-days value of 400 psi for moist-cured cylindrical specimens having a height/diameter ratio of 2:1 and 500 psi for cubical specimens (Ministry of Transport, UK, 1969).

Portland Cement Association, PCA (1956) established the range of compressive strength of cement treated soils under three broad textural soils groups, namely, sandy and gravelly soils, silty soils and clayey soils. The range of 7-days and 28-days unconfined compressive strengths for soil-cement is shown in Table 2.3.

In general, unconfined compressive strength increases linearly with cement content, but at different rates for fine-grained and coarse-grained soils as shown in Figure 2.1. Curing time is also important because strength increases gradually with age of curing. The effects of curing ages on unconfined compressive strengths for fine-grained and coarse-grained soils stabilized with 10% cement are shown in Figure 2.2.

Table 2.3: Ranges of compressive strengths of soil-cement mixtures (PCA, 1956)

Soil Type	Compressive Strength (psi)	
	7 days	28 days
Sandy and Gravelly soils: AASHO Group A-1, A-2, A-3. Unified group GW, GC, CP, GM, SW, SC, SP, SM	300-600	400-1000
Silty soil: AASHO group A-4, A-5. Unified group ML and CL	250-500	300-900
Clayey soils: AASHO group A-6, A-7. Unified group MH, CH	200-400	250-600

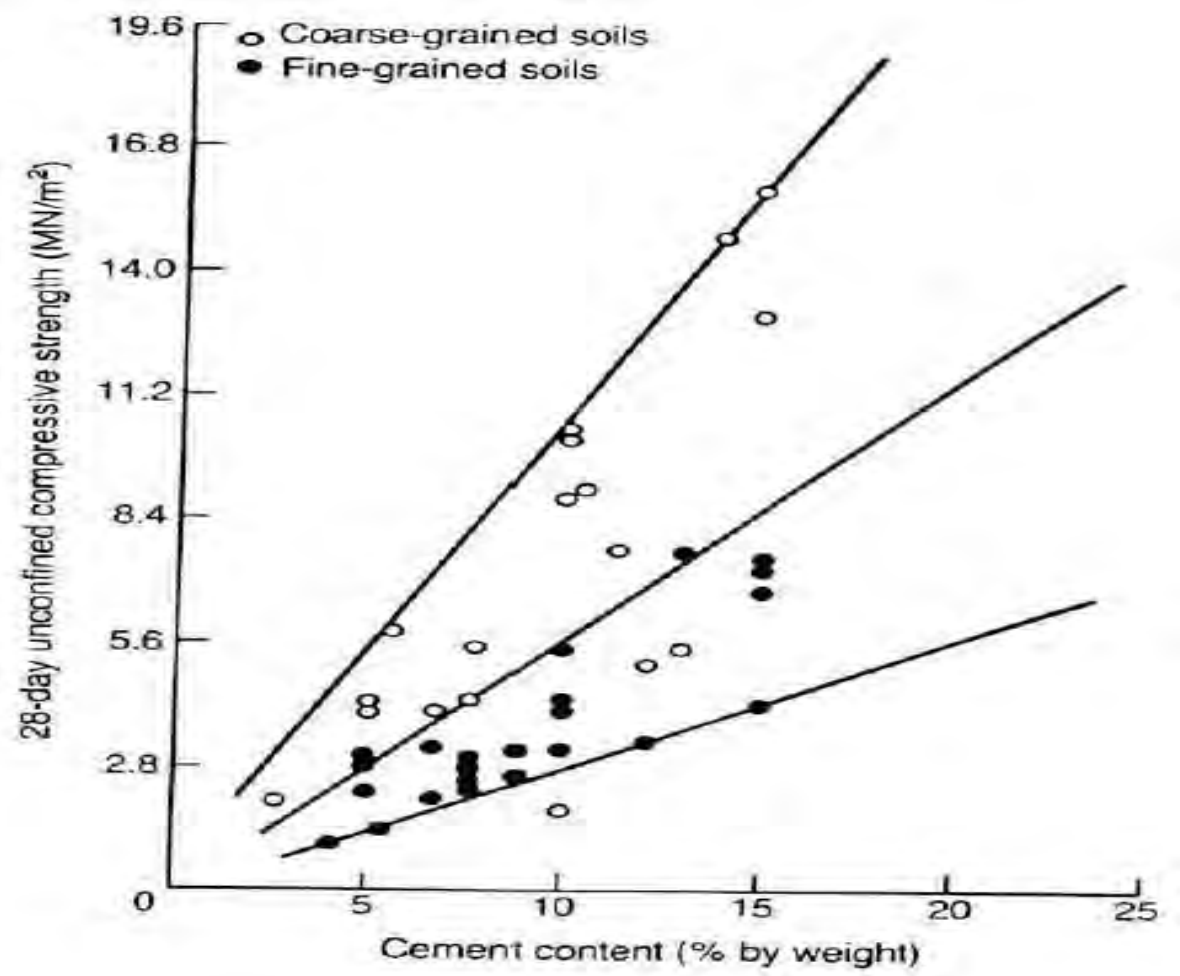

Figure 2.1: Effects of cement contents on unconfined compressive strengths of cement stabilized coarse-grained and fine-grained soils (after Anon, 1990)

A most comprehensive review of the strength properties of cement stabilization was reported by Mitchell (1981). The unconfined compressive strength, q_u is generally described as increasing linearly with the cement content percentage, C. This increase is more pronounced for coarse-grained soil than for silt and clays.

Mitchell (1981) reported the following relationship between curing time and q_u

$$q_u(d) = q_u(d_0) + K log \frac{d}{d} \qquad (\, . \,)$$

where, $q_u(d)$ =Unconfined compressive strength at d days, kPa

$q_u(d\ )$ =Unconfined compressive strength at d_o days, kPa

K = 480C for granular soils and 70C for fine-grained soil

C = Cement content, % by weight

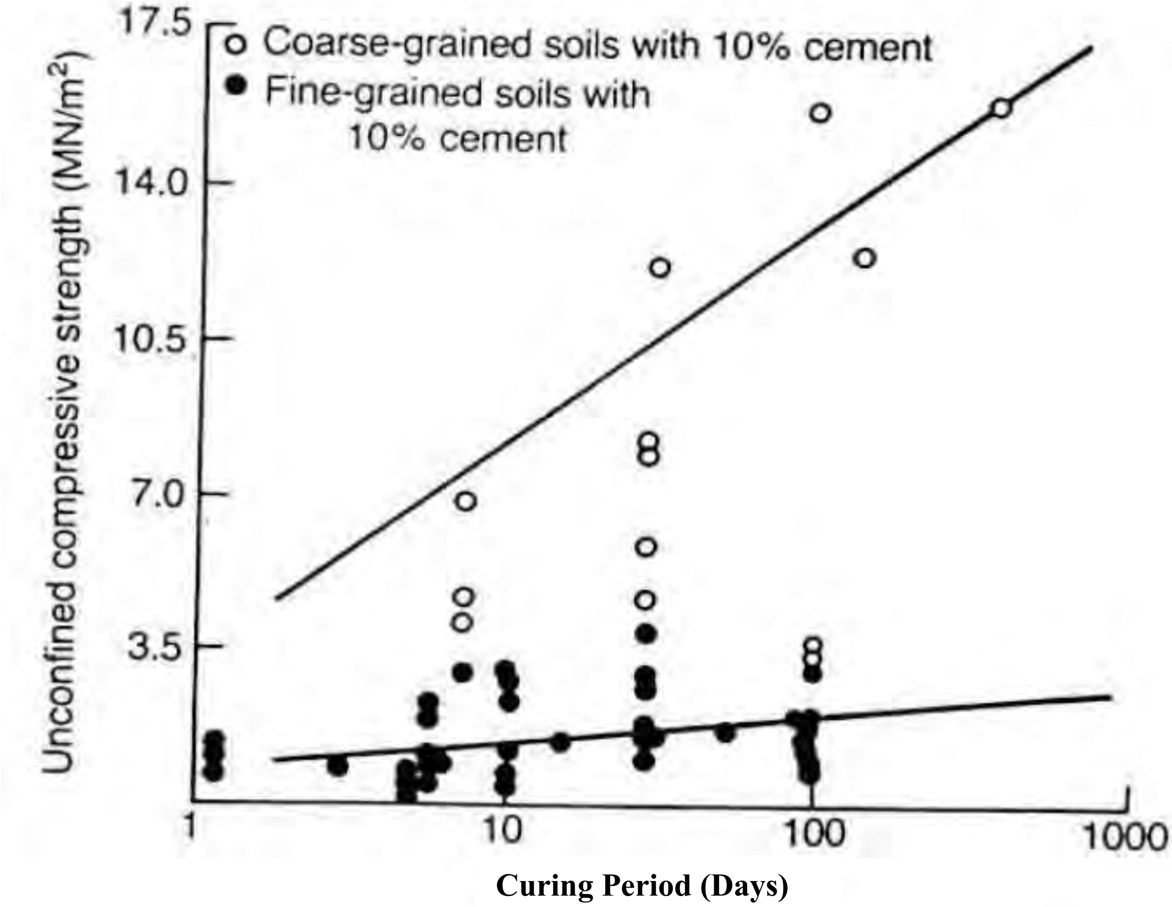

Figure 2.2: Effects of curing times on unconfined compressive strengths of cement stabilized coarse-grained and fine-grained soils (after Anon, 1990)

Ramaswamy et al. (1984) reported that the values of q_u of cement-treated silty clay subgrade soil samples for road construction continued to increase with the increases in cement contents and curing ages.

Ahmed (1984), Hossain (1986) and Rajbongshi (1997) investigated the effects of cement stabilization on unconfined compressive strengths (1.4 in. diameter by 2.8 in. high samples) of a number of regional soils of Bangladesh. Ahmed (1984) and Hossain (1986) found that compared with the untreated soil, unconfined compressive strengths of the cement-treated samples increased markedly, depending on the cement contents and curing ages. The effects of cement contents and ages on compressive strengths, and the rates of gain in strengths with cement contents for coastal soils reported from Rajbongshi (1997) are shown in Figure 2.3 and Figure 2.4 respectively.

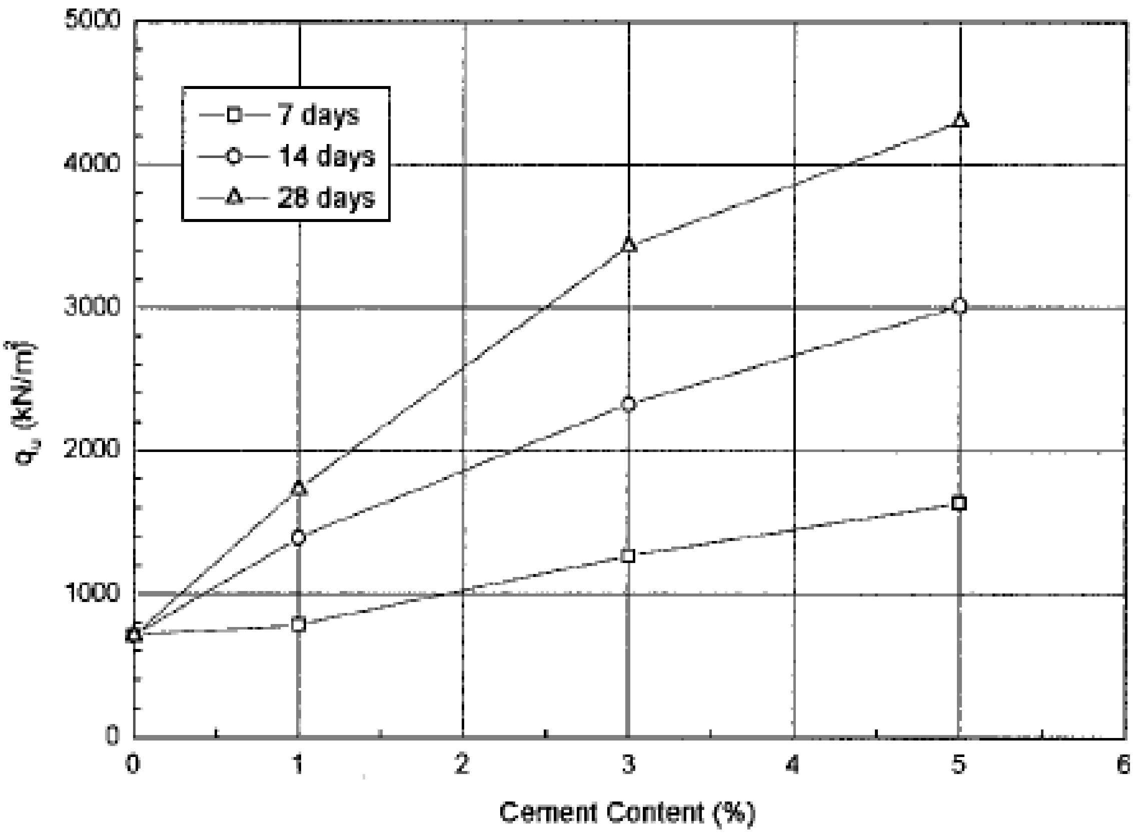

Figure 2.3: Effects of cement contents on unconfined compressive strengths of a sandy silt (reproduced after Rajbongshi, 1997)

Serajuddin and Azmal (1991) and Serajuddin (1992) reported the effects of cement contents and curing ages on unconfined compressive strengths (50 mm diameter and 100 mm high samples) of regional alluvial soils of Bangladesh. Typical results are presented in Figure 2.5 and Figure 2.6. Both Figure 2.5 and Figure 2.6 show that compressive strengths of samples stabilized with cement increases with the increase in cement content and curing age.

Hong (1989) and Uddin (1995) reported the effect of cement content and curing age on unconfined compressive strength of soft Rangsit clay of Bangkok. Hong (1989) reported results of samples (LL = 104, PI = 63) stabilized with 5% to 15% cement and cured for 7 days to 56 days while Uddin (1995) reported results of samples (LL= 70 to 117, PI = 50 to 78) treated with 5% to 40% cement and cured for 1 week to 40 weeks. Hong (1989) and Uddin (1995) found considerable increase in unconfined compressive strength, depending on the cement content and curing age.

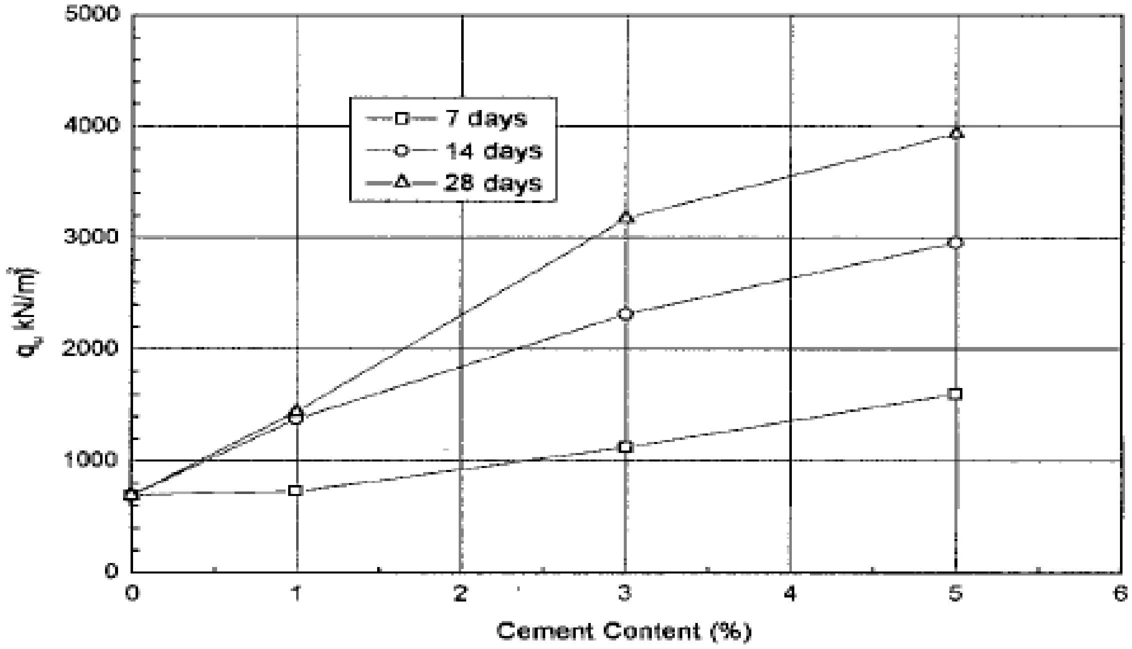

Figure 2.4: Gain in unconfined compressive strengths for a cement-treated silty clay over untreated soil (reproduced after Rajbongshi, 1997)

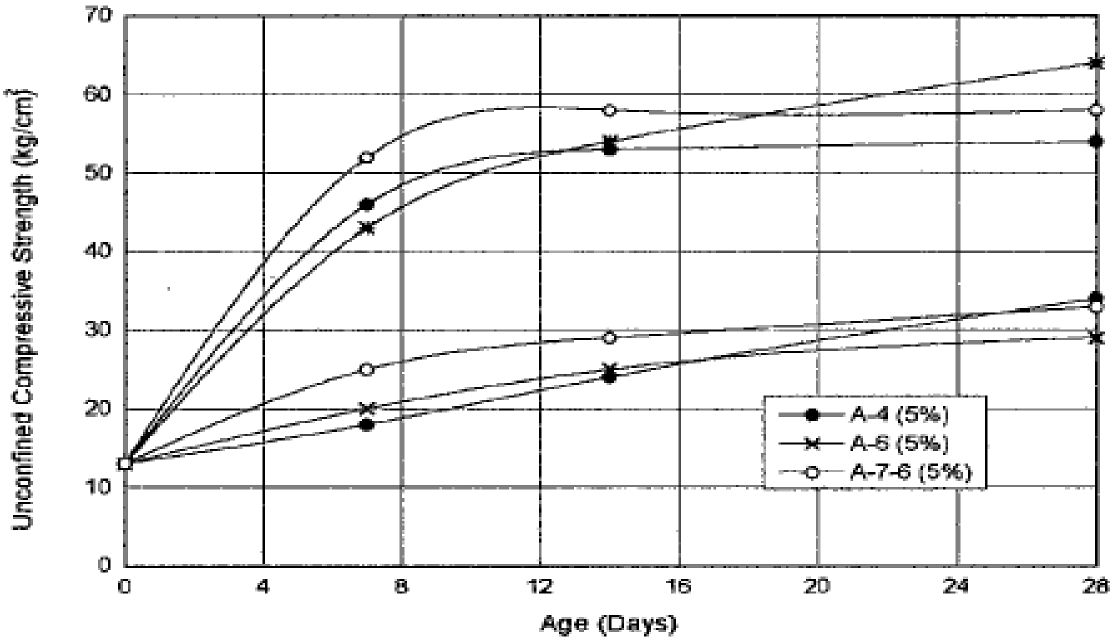

Figure 2.5: Effects of curing ages on unconfined compressive strengths for soil-cement mix specimens of three typical silty soils (reproduced after Serajuddin and Azmal, 1991)

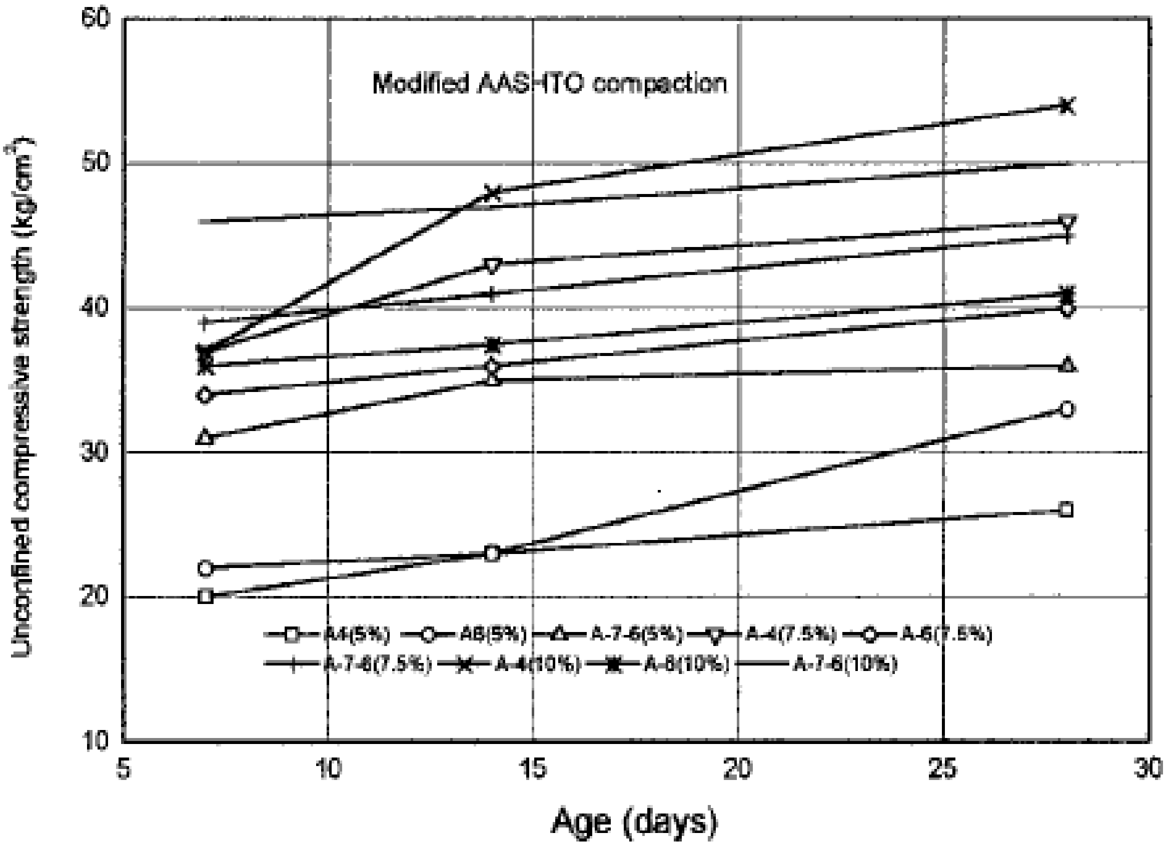

Figure 2.6: Effects of curing ages and cement contents on unconfined compressive strengths for cement stabilized three typical regional soils (reproduced after Serajuddin, 1992)

2.2.5.2 Shear Strength

The Mohr-Coulomb shear strength envelops of the cement based composite soils were shown by Sarkar et al. (2012). The said authors found that the slopes of the curves (shear stress vs. normal stress relationship) increased with the increases of cement. It is observed that the value of cohesion gradually increased with the addition of 5% cement content and it increased rapidly up to 7.5% of cement content for 7 days and 28 days soaking. After that, the value increased gradually for the further amount of cement content. Besides that, there was no significant improvement in cohesion for uncured samples. The value of angle of internal friction increased rapidly for the 5% of cement content and this value increased gradually for the further amount of cement content.

2.2.5.3 Plasticity and Shrinkage Properties

In general, liquid limit and plastic limit of the soil increase with increasing cement, while the plasticity index reduces with the increase in cement content. Felt (1955) showed that the plasticity index for a plastic granular soil reduced considerably when treated with cement.

Willis (1947), however, showed that the cement admixture reduces slightly the liquid limit of mixtures made from soils having liquid limit greater than 40. Willis (1947) also showed that liquid limit increases for soils having liquid limits less than 40 when treated with cement.

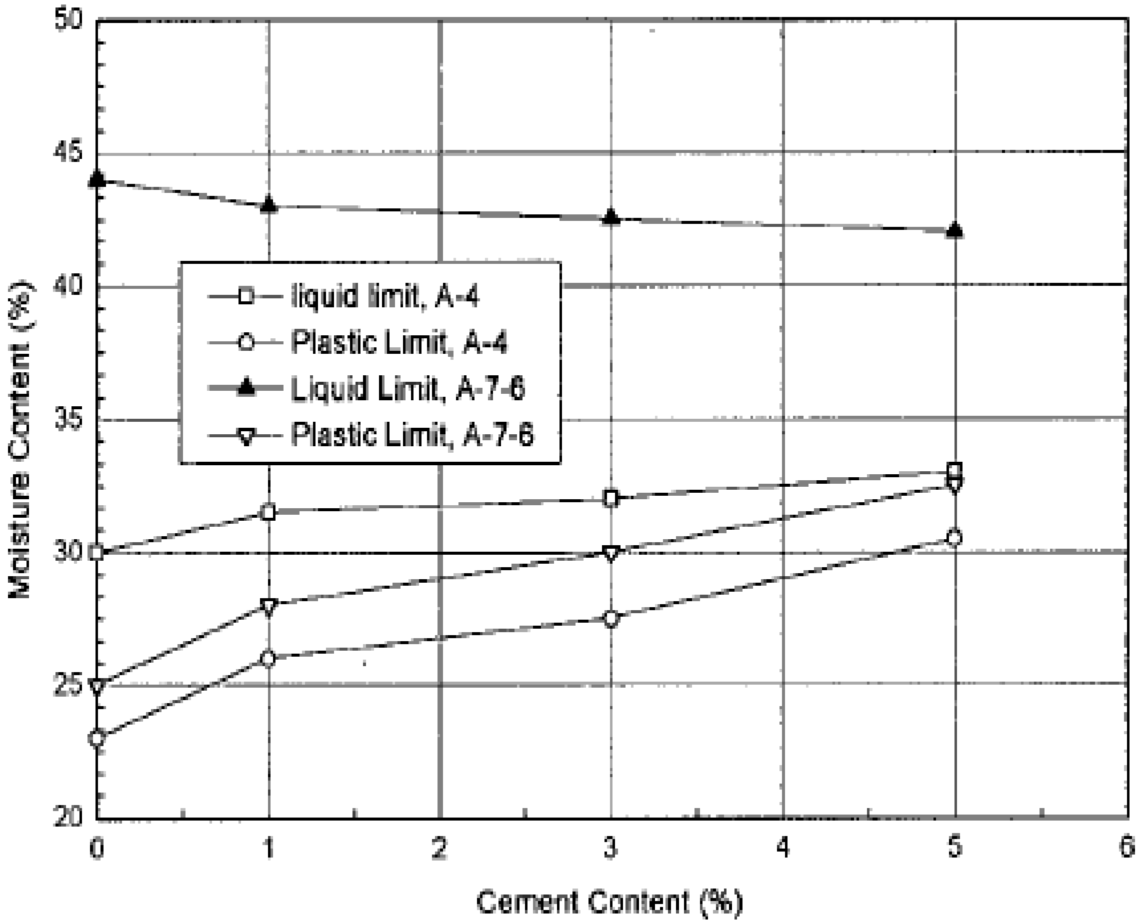

Figure 2.7: Effects of cement contents on Atterberg limits for a coastal soil (reproduced after Rajbongshi, 1997)

Ahmed (1984) showed that for sandy silt (LL = 40, PI = 10) and silty clay (LL = 43, PI = 21), plastic limits increased while plasticity indices reduced as cement contents increased. Rajbongshi (1997) found that with the increases in cement contents, for coastal soil (Type: A-4, LL=41, PI=7) liquid limits and plastic limits increased while plasticity indices reduced. For coastal soil (Type: A-7-6, LL = 44, PI = 19) liquid limits reduced and plastic limits increased with increases of cement contents while plasticity indices increased with increases of cement contents. The said phenomenon is presented in Figure 2.7 and Figure 2.8.

Hasan K. A. (2002) proposed the following Figure 2.9 which shows that for sandy silt (LL = 40%, PI = 10%) and fat clay with sand (LL = 52%, PI = 29%) shrinkage limits decreased as cement contents were increased. He also showed in Figure 2.10 that sandy silt (LL=40%, PI=10%) loses the percentages of linear shrinkages with increments of cement contents.

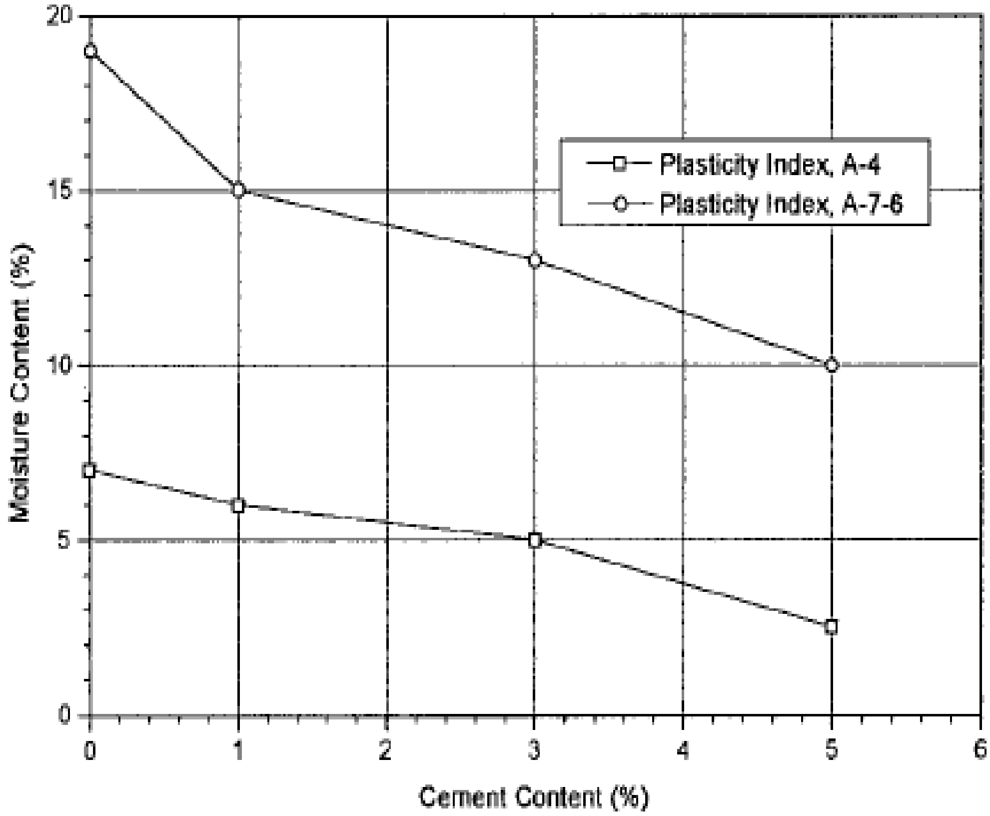

Figure 2.8: Effects of cement contents on plasticity indices for a coastal soil (reproduced after Rajbongshi, 1997)

2.2.5.4 Moisture-Density Relations

Adequate compaction is essential for successful stabilization but prolonged delays between mixing and compaction reduce the maximum density attainable (Ingles and Metcalf, 1972). The addition of cement produces small increases in the compacted densities of both kaolinitic and illitic clay soils, but not those containing montmorillonite (Bell, 1993). With the addition of cement, maximum dry density of sand or sandy soil increases; little or no change is observed for light to medium clays whereas density increases slightly for fat clays and density decreases for silts (Kezdi, 1979).

Small changes can also be observed in the optimum moisture contents. These have been illustrated in Figure 2.11. Felt (1955) also reported that for sand and sandy soils the density increases with the increasing cement content.

2.2.5.5 Failure Strain

Wang et al. (2018) studied on the dredged marine soils whose liquid and plastic limit, determined by the percussion-cup and rolling thread method according to NF P 94–051

(Association Française de Normalisation 1993) and NF P 94–052-1 (Association Française de Normalisation 1995), are, respectively, 76.1 and 35.3%.

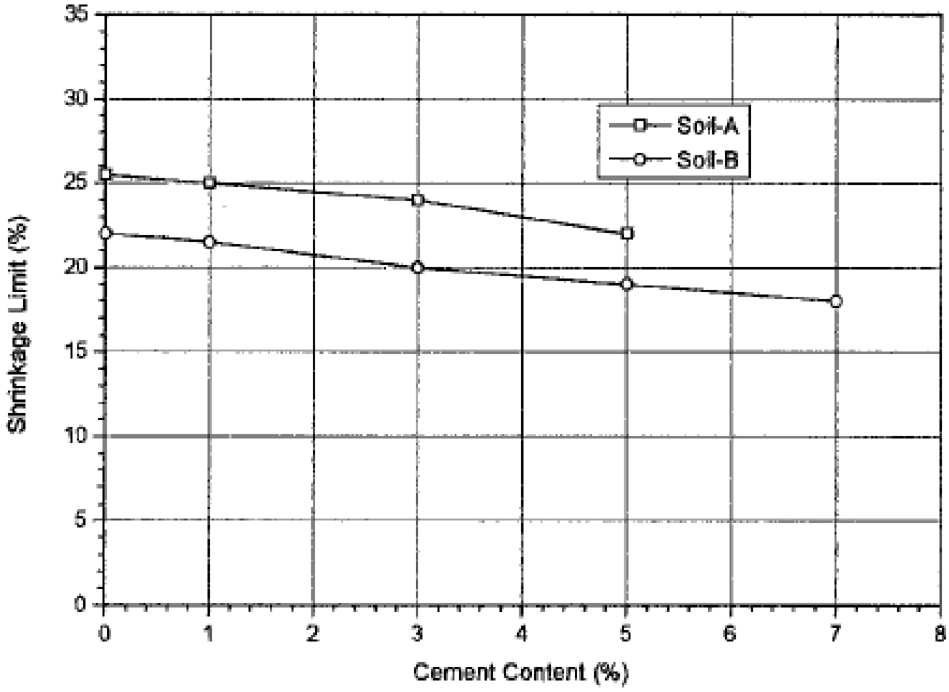

Figure 2.9: Effects of cement contents on shrinkage limits of soil-A (sandy silt) and soil-B (fat clay with sand)

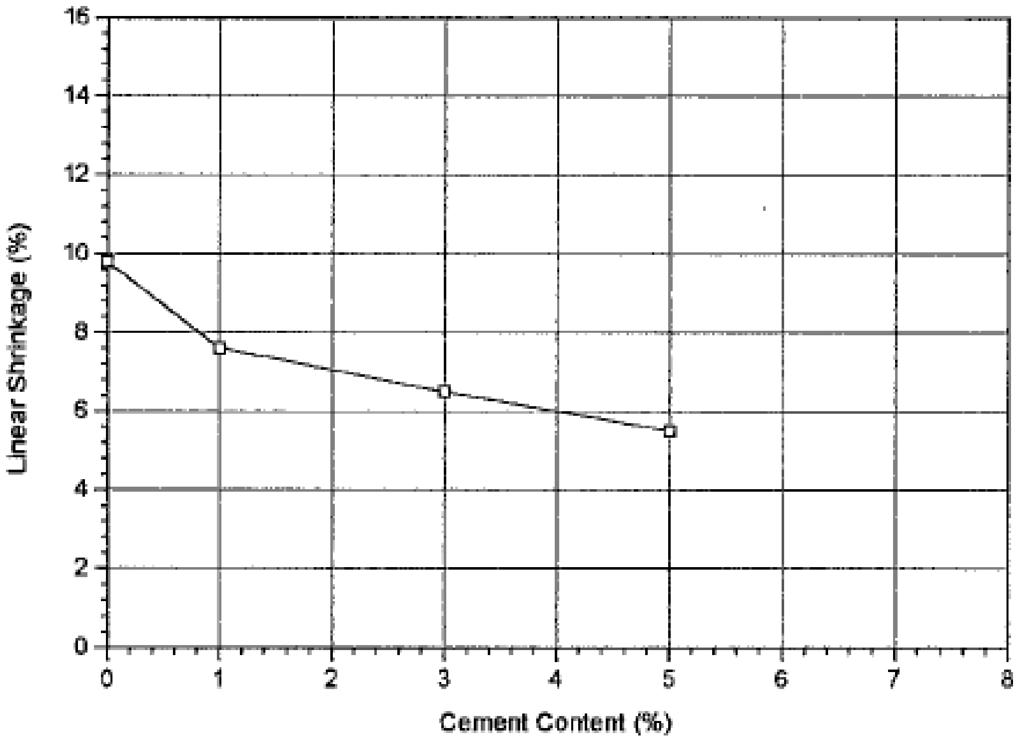

Figure 2.10: Effect of cement contents on linear shrinkage of sandy silt

In his study he found that as cement contents increase, failure strains vary from 1.81% for soil-cement mixture (3% cement and 97% soil), via 1.58% for soil-cement mixture (6% cement and 94% soil), to 1.10% for soil-cement mixture (9% cement and 91% soil) when these mixtures were subjected to thawing-freezing damage. He also found that as cement contents increase, failure strains vary from 1.18% for soil-cement mixture (3% cement and 97% soil), via 0.88% for soil-cement mixture (6% cement and 94% soil), to 1.02% for soil-cement mixture (9% cement and 91% soil) when these mixtures were subjected to water immersion aging.

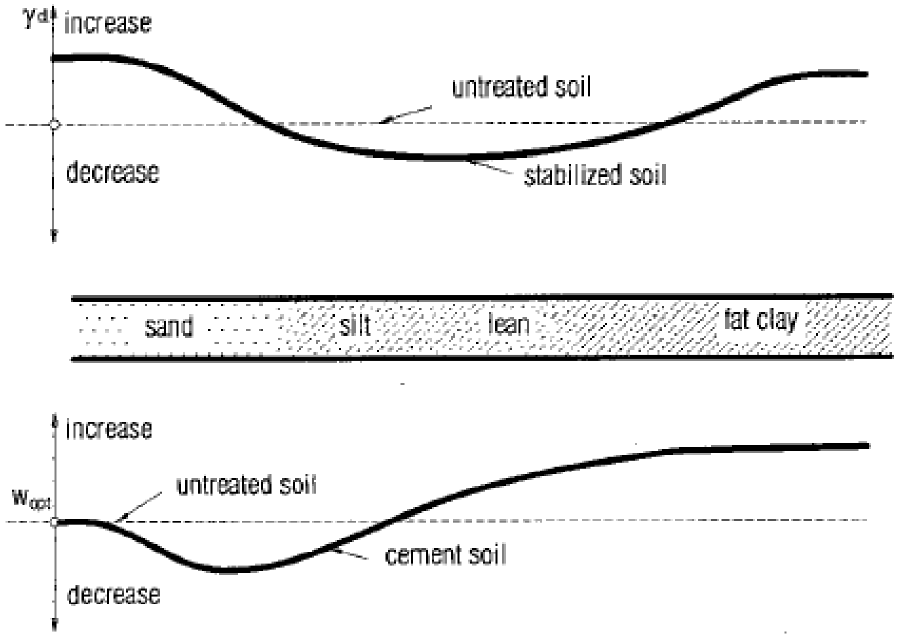

Figure 2.11: Effects of cement contents on maximum dry densities and optimum moisture contents of different soils (reproduced after Kezdi, 1979)

2.2.6 Applications of Soil-Cement

The principal use of soil-cement is as a base material underlying pavements. One of the reasons of using soil-cement as a base is to prevent pumping of fine-grained sub grade soils into the pavement above. The thickness of the soil-cement base depends upon subgrade strength, pavement design period, traffic and loading conditions and thickness of the wearing surface. Frequently, however, soil-cement bases are around 150-200 mm in thickness. Ingles and Metcalf (1972) recommended the values of various engineering properties of cement-treated materials for various purposes that are shown in Table 2.4. In Table 2.4, the recommended values of unconfined compression strength, CBR, swell and loss in wetting and drying tests are presented. Ingles and Metcalf (1972) also suggested the cement contents for various soil types for pavement construction those are shown in Table 2.5.

Bell (1993) reported various other uses of soil-cement. Soil-cement has been used to afford slope protection to embankment dams, soil-cement made from sandy soils giving a durable erosion-resistant facing. Soil-cement has also provided slope protection for canals, river banks, highway and railway embankments and coastal cliffs. In addition to water storage reservoirs, soil-cement has been used to line wastewater treatment lagoons, sludge-drying beds, ash-settling ponds and sanitary landfills. The soil-cement linings are commonly 100 mm to 150 mm thick.

2.3 Lime Stabilization

2.3.1 General

Stabilization of soils with lime is broadly similar to cement stabilization in that similar criteria and testing and construction techniques are employed. There are however, significant differences in the nature and rate of the cementitious reactions and these often permit a clear basis of choice between cement and lime.

Lime is an effective additive for clayey soils for improving workability, strength and volume stability. Lime stabilization is suitable for more plastic clayey soils and is less suitable for granular materials. It is used more widely as a construction expedient, that is to prepare a soil for further treatment or to render a sufficient improvement to support construction traffic.

Table 2.4: Values of engineering properties of soil-cement (after Ingles and Metcalf, 1972)

Purpose	q_u (7- Day Cured)		Four-Day Soaked CBR	Swell	Loss in Wet-Dry Test
	kg/cm^2	psi		%	%
Road sub-base, formation backfill for trenches etc	3.5-10.5	50-150	20-80	2	7
Road sub-base, base for light traffic*	7-14	100-200	50-150	2	10
Base for heavy traffic* Building blocks	14-56	200-800	200-600	2	14
Embankment protection Floodways (too strong for general use under thin surfacing)	>56	800	600	2	14

* Lower strengths may be adequate for well-drained areas in the tropics

2.3.2 Materials for Lime Stabilization

The materials to be considered in lime stabilization are lime, soil and water, and it is important that the type of lime to be used is clearly defined.

2.3.2.1 Lime

Lime, refers to hydrated or slaked lime (calcium hydroxide), quicklime (calcium oxide), or dolomitic limes (calcium/magnesium oxide), that is, the highly alkaline (pH> 12.3) lime products. Agricultural lime (calcium carbonate) is not suitable for stabilization. Dolomitic lime is usually not as effective as calcium lime (i.e., hydrated or slaked lime and quicklime). In order to give a common quantitative base, lime contents are expressed as equivalent 100 percent pure hydrated lime. On a mass basis pure quicklime is equivalent to 1.32 units of hydrated lime.

Hydrated lime comes in the form of a dry, very fine powder or as slurry. Quicklime and dolomitic limes are commonly much more granular than the hydrated products and are available only as a dry product. These limes rapidly react with any available water producing hydrated lime, releasing considerable amounts of heat. The water content of common slurry limes can range from 80 to 200 percent.

Table 2.5: Cement contents for various soil types for pavement construction (after Ingles and Metcalf, 1972)

Soil Type	Cement Requirement (per cent)
Fine crushed rock	½ - 2[1]
Well graded sandy clay gravels	2 - 4
Well graded sand	2 - 4
Poorly graded sand	4 - 6[2]
Sandy clay	4 - 6
Silty clay	6 - 8
Heavy clay	8 - 12
Very heavy clay	12 - 15[3]
Organic soils	10 - 15[4]

[1] Used as a construction expedient to aid "set up" on compaction, to reduce sensitivity to compaction moisture content and prevent reveling under construction traffic.
[2] Compaction may be very difficult, and segregation of the cement may occur.
[3] Mixing may be very difficult - pretreatment with lime may help.

[4] Pretreatment with lime or addition of 2 percent calcium chloride may help.

The efficiency of lime stabilization depends in part on the type of lime material used. Quicklime is generally more effective than hydrated lime (Kezdi, 1979), but generally it needs care in handling for soils with high moisture contents. Unslaked lime or quicklime is more effective since water will be absorbed from the soil and more importantly, the hydration will cause an increase in temperature which is favorable to strength gain (Broms, 1986).

Table 2.6: Properties of lime (after NAASRA, 1986)

Parameters	**Hydrated Lime**	**Quick Lime**	**Slurry Lime**
Composition	$Ca(OH)_2$	CaO	$Ca(OH)_2$
Form	Fine Powder	Granular	Slurry
Equivalent $Ca(OH)_2$ / Unit Mass	1.00	1.32	0.56 to 0.33
Bulk Density (kg/m^3)	450 to 560	1050	1250

2.3.2.2 Soil

The addition of lime has little effect on soils that contain either a small clay content or none at all. Lime has also little effect in highly organic soils. Lime usually reacts with most soils with a plasticity index ranging from 10% to 50%. Those soils with a plasticity index of less than 10% require a pozzolan for the necessary reaction with lime to take place, fly ash being commonly used. Lime is particularly suited to stabilize highly plastic clay soils. In such soils the lime will immediately create a more friable structure, which is easier to work and compact, although a lower maximum density will be achieved, and lime may be used solely for this reason as a pre-treatment to further additions of lime. Lime reacts more quickly with montmorillonitic clays than with kaolinitic clays. In montmorillonitic clays the plasticity is reduced, but this may not happen with kaolinitic clays.

The effect of soil moisture content is important only where it affects the operation of compacting or pulverizing equipment by being either too low or too high. In wet clays the use of lime to effect rapid changes in plasticity is the basis of the application of lime stabilization as a construction expedient.

2.3.2.3 Water

Potable water is preferred for lime stabilization. Acidic (organic) water should be avoided. Seawater can be used but should be avoided where a bituminous seal is to be placed. The amount of water used in lime stabilization is governed by the requirements of compaction. However, if quicklime is used then extra water may be required in soils having less than 50 percent moisture content to provide for the very rapid hydration process.

2.3.3 Mechanisms of Lime Stabilization

It is recognized that lime has an immediate effect on clay soils, improving its granulation and handling properties. The effect varies with the actual clay mineral present, being large with montmorillonite group clays and low to non-existent with kaolinite group clays. Lime has longer-term effects on strength, causing continuing strength improvements with time. The basic mechanisms of soil-lime interactions have been described by Eades and Grim (1960), Compendium (1987), IRC (1973a) and Hausmann (1990). The basic mechanisms that have been identified in soil-lime interaction are base exchange (ion exchange), flocculation, cementation and carbonation. These mechanisms are briefly presented in the following several sections.

2.3.3.1 Base Exchange and Flocculation

Clay particles are usually negatively charged and they contain adsorbed exchangeable cations of sodium, magnesium, potassium or hydrogen on the surface. The strong positively charged cations of calcium present in lime replace the weaker ions of sodium, magnesium, potassium or hydrogen present on the clay surface and this base-exchange results in a predominance of positively charged calcium ions on the surface of clay particles. This reaction is usually completed within a few days of the mixing.

This change in the cation exchange complex affects the way the structural components of the clay minerals are connected together. Lime causes clay to flocculate. The plasticity of clay (measured in terms of Atterberg limits) is reduced, making it more easily workable and potentially increasing its strength and stiffness.

2.3.3.2 Cementation

Cementation is the main contributor to the strength of the stabilized soil. The higher the surface area of the soil, the more effective is this process. If lime is added in excess of the lime fixation point, complex chemical reactions similar to pozzolanic reactions are known to take place between lime and the clay minerals in the soil. These reaction products are cementitious. The aluminous and siliceous materials in clayey soil have no cementitious value by themselves but react with calcium hydroxide in the presence of water to form cementitious compounds according to the following equations:

$$Ca^{++} + \ (OH)^{-} + SiO \ (Soil\ silica) \rightarrow CSH \qquad (6)$$
$$Ca^{++} + \ (OH)^{-} + Al\ O \ \ (Soil\ alumina) \rightarrow CAH \qquad (7)$$

In equations (6) and (7), CSH and CAH are cementitious products. The above reactions represented by Equations (6) and (7) are slow and long-term in nature. Long-term chemical reactions of lime with certain clay minerals (silicate and aluminate) of soil in presence of water is referred to pozzolanic reaction in lime stabilization. Moreover, these reactions are more effective when the soil-lime mixture is adequately compacted.

Cementation is, however, limited by the amount of available silica and alumina. Increasing the quantity of lime added would increase strength only up to the point where all the silica and alumina of the clay is used up; adding too much lime can actually be counterproductive. This contrasts with cement stabilization, where strength continues to improve with the amount of admixture. Cementation on the surface of clay lumps causes a rapid initial strength gain, but further diffusion of the lime in the soil will bring about continued improvement in the longer term, measured in weeks or months.

2.3.3.3 Carbonation

As lime absorbs carbon dioxide from the air, calcium carbonate ($CaCO_3$) is formed. These carbonates are relatively weak cementing agent (Hausmann, 1990). This reaction is the slowest of all the reactions involved in a soil-lime system and as in pozzolanic reaction, requires that the mixture must be thoroughly compacted. Carbonation may be beneficial where lime is plentiful; the $CaCO_3$ formed will not react any further with the soil.

2.3.4 Factors Governing the Characteristics of Soil-Lime Mix

Like cement-treated soil mixtures, properties of lime-treated soils are influenced by several factors. These factors are broadly classified as material factors and production factors. Material factors deal with the composition of the untreated soil and its response to lime. The production factors include the quality of water, lime, the uniformity of mixing and curing. The factors influencing the properties of lime-treated soil are described in the following several sections.

2.3.4.1 Soil Characteristics

2.3.4.1.1 Type of Soil

For lime to be effective there must be within the soil, clay particles or other pozzolanic materials that are reactive with the lime. Thompson (1966a) stated that the extent of improvement of the engineering characteristics of soil depends largely upon the soil type. The gain in strength of a soil lime system is mainly due to the pozzalanic reaction i.e. the long-term reaction between lime and certain clay minerals (silicate and aluminates) in the presence of water. He also noted that soils having larger amount of clay fraction and less amount of organic matter are very effective to lime stabilization. In general the more plastic the clay fines and the higher the clay content, the larger will be the lime content to produce a specific strength gain or other effect. On the other hand, the amount of bonding achievable with lime can be limited by the amount of reactive material.

For lime stabilization to be successful, the clay content of the soil should not be less than 20% and the sum of the silt and clay fractions should preferably exceed 35%, which is normally the case when the plasticity index of the soil is greater than 10 (Broms, 1986). Ingles and Metcalf (1972) did not recommend crushed rock and sands for use in lime stabilization.

NASSRA (1970) stated that highly plastic soils are more effective to gain strength. NASSRA (1970) pointed out that soil having plasticity index in the range of 10 to over 50 are suitable for lime stabilization. Soils with plasticity index lower than 10 do not react readily with lime, although there are some few exceptions. Ingles and Metcalf (1972) studied the effect of the unconfined compressive strength on different types of soil stabilized using lime. It was found that the strength of Lime Stabilized Silty Clay is higher than the other types of soil.

Yu Kuen (1975) stated that in general, highly plastic soils are more effective than other types of soil when stabilized with lime. Compendium (1987) stated that lime is very effective in stabilizing the clay soils with a substantial portion of the coarse grained soil. Rodriguez et al. (1988) noted that the maximum effect of lime is on clayey gravel soil. Sometimes, the strength increase due to lime stabilization on these types of soil is such that the stabilized soil becomes stronger than those that would be obtained with cement. Rodriguez et al. (1988) also reported that lime has been more frequently used with plastic clays, which become more workable and easy to compact.

Locat et al. (1990) studied the effect of four types of soil of Canada stabilized with lime. He observed that the unconfined compressive strength of the silty clay soil is higher than the other types of soil. Figure 2.12 shows the variations of unconfined compressive strengths with lime contents for four types of soil. It has been found that the maximum strength is gained by the soil with higher clay content.

Serajuddin (1992) reported the results of three types of lime treated soil of the South West region of Bangladesh. Silt and clay types of soil were used in the investigation. The results of the investigation are shown in Figure 2.13. It has been found that silty soil has much lower unconfined compressive strength than the clay types of soil.

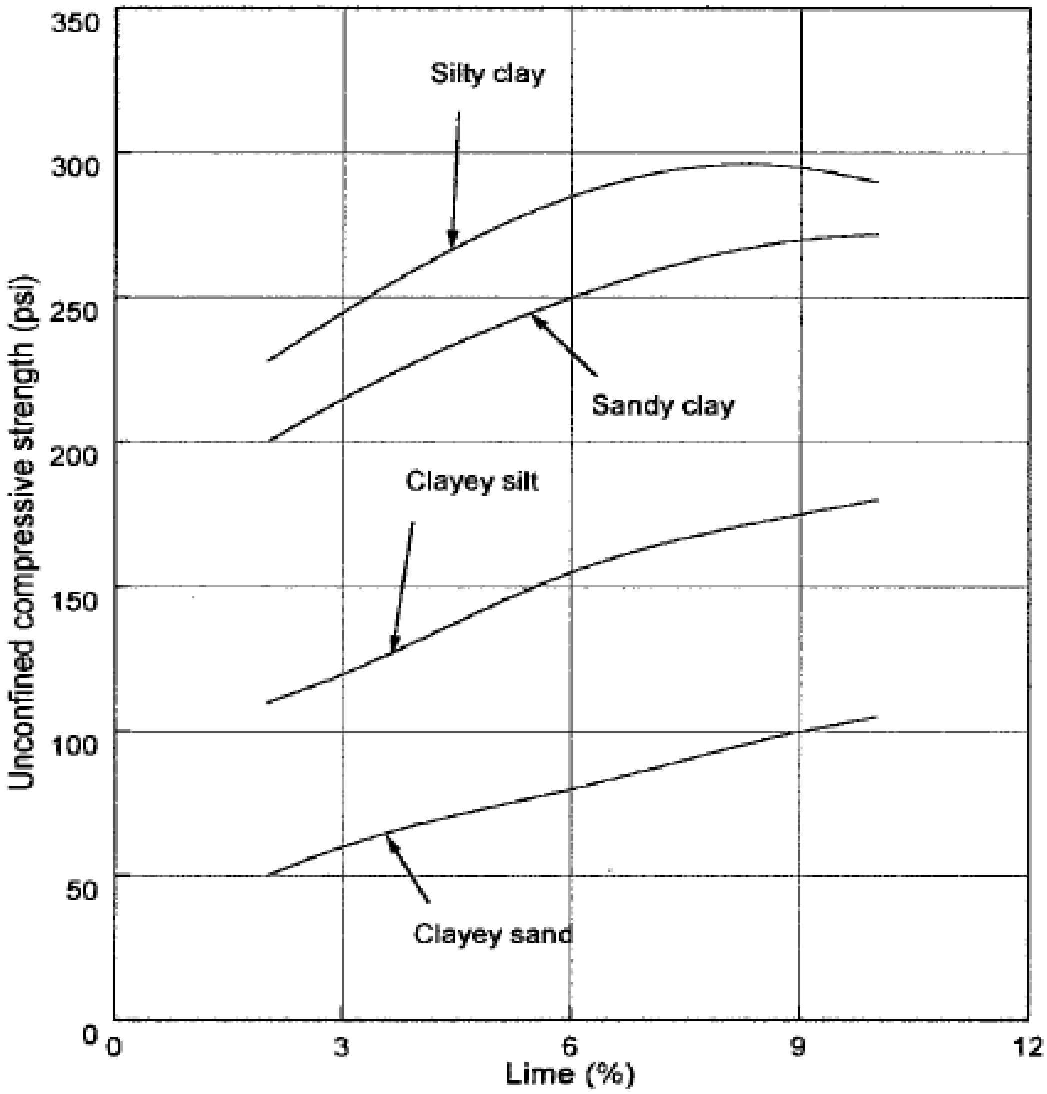

Figure 2.12: Variations of unconfined compressive strengths (q_u) with lime contents for various types of soil (reproduced after Locat et al., 1990)

The pH value of the soil, which indicates its acidity or alkalinity, is of great importance to lime stabilization. Ho and Handy (1963) have shown that for montmorillonite clays, no lime reaction occurs at pH less than 11.0. The presence of significant amounts of sulphate diminishes the effectiveness of lime. The Indian Road Congress, IRC (1976) specifications also require that where the sulphate content is in excess of 0.2 percent, special studies would be needed to determine the efficacy of lime-treatment.

2.3.4.1.2 Organic Matter Present in the Soil

One of the important factors that inhibit lime-soil reaction is the organic content. One of the possible reasons is that organic matter has a high base exchange capacity and when lime is added to such soils, some of the Ca^{++} ions are used to satisfy the exchange capacity of the organic matter, thus depriving the clay minerals of calcium ions for pozzolanic reactions. Ingles and Metcalf (1972) reported that organic soils should not be used in lime stabilization. However, IRC (1973a) recommended a maximum limit of 2% organic content for lime stabilization.

NASSARA (1970) stated that the presence of organic matter in the soil reduces the strength of the stabilized soil. He pointed that soil containing more than 3% of organic matter is very harmful to the strength development of the stabilized soil.

Arman and Muhfakh (1972) studied the effect of the percent of organic matter on the unconfined compressive strength of the lime stabilized soil. It has been found that the presence of organic matter in the soil reduces the strength of the stabilized soil to a large extent. As the organic contents in the soil increase, unconfined compressive strengths continue to decrease as shown in Figure 2.14. Holm et al. (1983) also stated that the effect of lime decreases with increasing organic content. The strength increase of lime stabilized organic soil is very low. According to them, one of the possible reasons is that organic matter has high base exchange capacity. When lime is added to organic soils most of the Ca^{++} ions are used to satisfy the exchange capacity of organic matter, thus depriving the clay minerals of calcium ions for pozzalanic action. Even a small amount of organic content can have a large effect on strength.

2.3.4.2 Lime Content

The strength of soil-lime mix is determined to a great extent by the quantity of lime added. Small quantities of lime, 1 to 2 percent, help in the immediate effects caused by the base-exchange and flocculation. The effect of soil-lime stabilization in increasing the strength of the mixture begins to be felt as the lime content is further increased and this is due to pozzolanic reactions resulting in the production of cementitious compounds. It is also observed that this strength gain is time-dependent and efficiencies in strength gain due to varying lime percentages are more marked for longer curing periods.

Ingles and Metcalf (1972) suggested that the addition of up to 3% of lime would modify well graded clay gravels, while 2% to 4% were required for the stabilization of silty clay, and 3% to 8% were proposed for stabilization of heavy and very heavy clays. Ingles and Metcalf (1972) further suggested that a useful guide is to allow 1% of lime (by weight of dry soil) for each 10% of clay in the soil.

Hausmann (1990) stated that the practical lime content for lime stabilization varies from 2% to 8%. Variations of the unconfined compressive strengths of the lime stabilized soil due to the variations of the lime contents as found by Molla (1997) are shown in Figure 2.15 for three regional soils of Bangladesh. It can be seen from Figure 2.15 that the unconfined compressive strengths of the lime stabilized soils increase with the increases of lime contents for all the three soil types. Optimum lime content is the lime content by which the maximum strength of the lime stabilized soil can be achieved. Researchers stated different criteria for optimum lime content. Herrin and Mitchell (1961) pointed that there appears to be no optimum lime content in the lime stabilized soil, which will produce a maximum strength of the soil under all conditions. However, it can be stated that for a particular condition of soil type and curing time, there is a corresponding lime content, which will produce maximum strength.

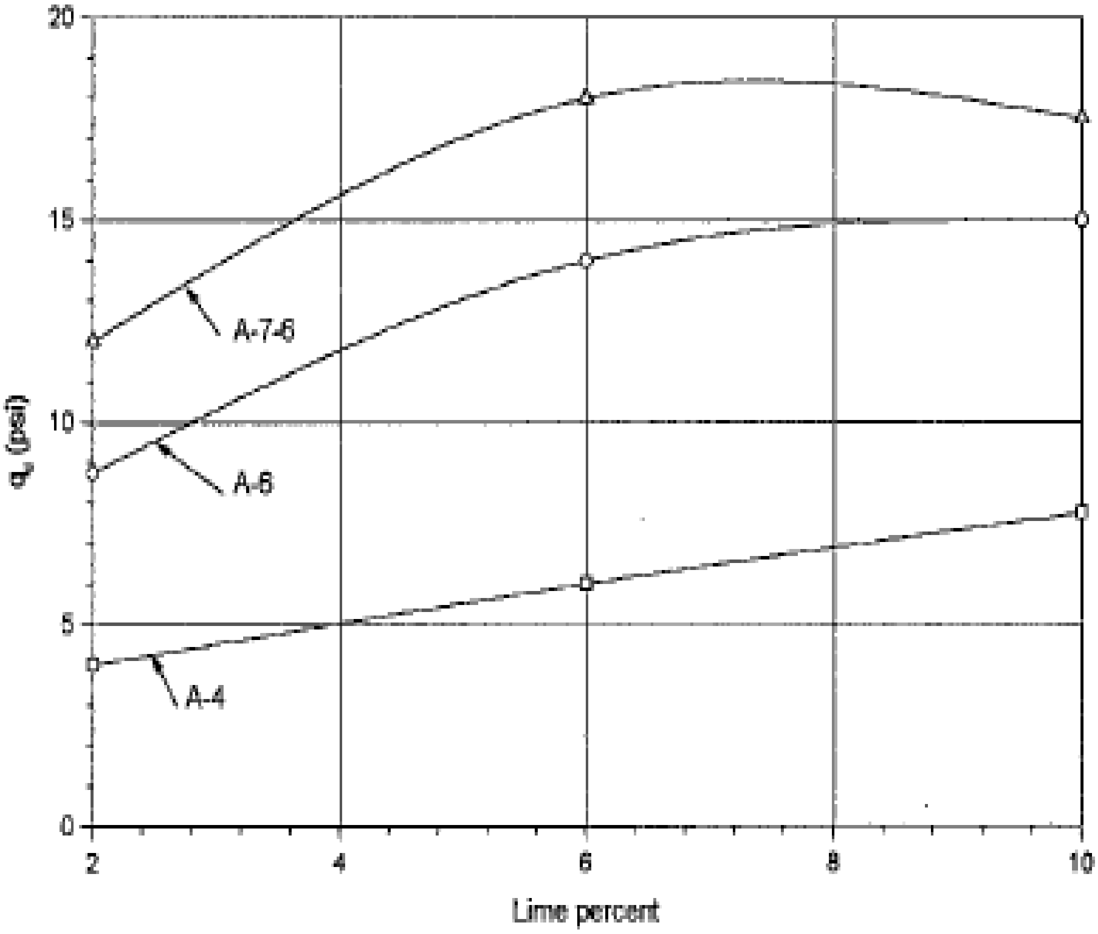

Figure 2.13: Variations of unconfined compressive strengths with lime contents for different types of soil (reproduced after Serajuddin, 1992)

Based on intensive investigation at the Iowa State University, Diamond and Kinter (1965) defined optimum lime content as one at which the percentage of lime is such that additional increments of lime will produce no appreciable increase in the plastic limit. According to them, lime content above the lime fixation point for a soil will generally contribute to the improvement of soil workability, but may not result in sufficient strength increase.

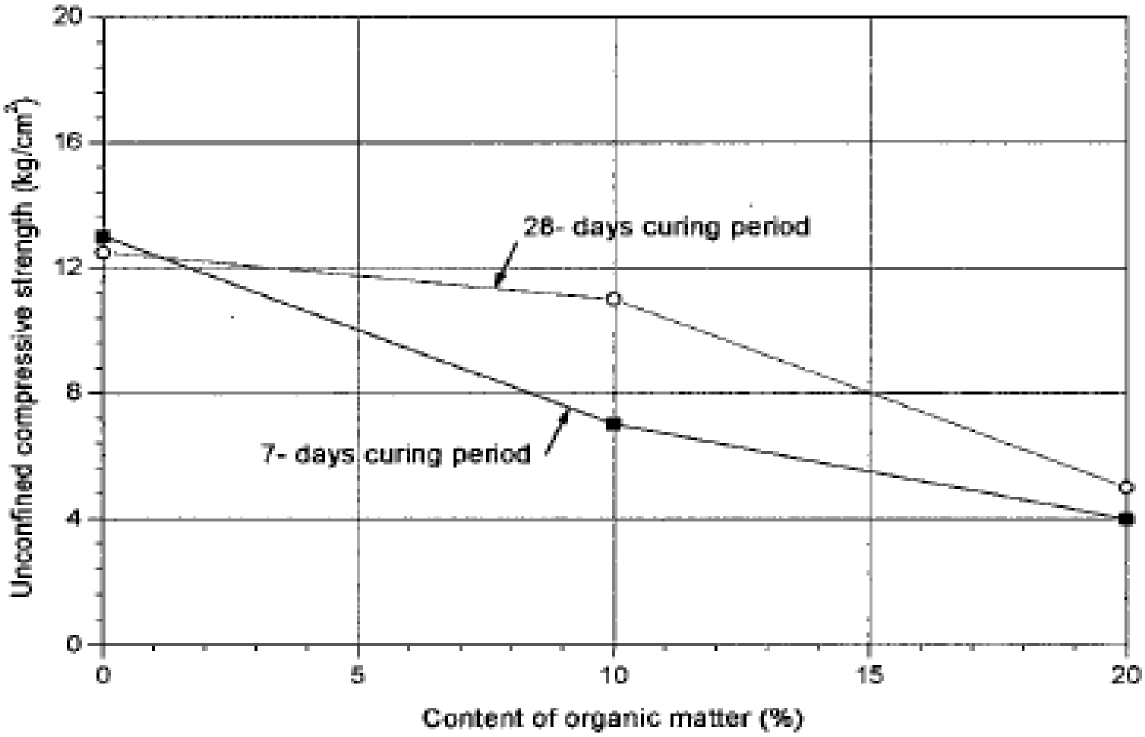

Figure 2.14: Effects of organic matter on unconfined compressive strengths of lime treated soil (reproduced after Arman and Muhfakh, 1972)

2.3.4.3 Mixing and Compaction Procedure

2.3.4.3.1 Compactive Effort

The success of lime-soil stabilization technique depends to a great extent on adequate compaction of the mixture. Compaction is considered to be necessary for bringing the clay minerals into close and intimate contact with the lime particles so that the inter-growth of crystalline reaction products is facilitated (Croft, 1964). With soil-lime mixture, the greater the compactive effort, the more is the strength attained. Taking typical data from Remus and Davidson (1961), a calcitic lime (6 percent) used with glacial till soil yielded an unconfined compressive strength (7 days curing and 24 hours immersion) of 250 psi at Standard AASHO compaction. For the same conditions, but with modified AASHO compaction, the strength increased to 525 psi.

Compendium (1987) stated that the maximum dry density normally continues to decrease as the lime content is increased. In addition, the optimum moisture content increases with increasing lime content.

Hausmann (1990) pointed that flocculation and cementation will make the soil more difficult to compact, therefore, the maximum dry density achieved with a particular compactive effort is reduced. Faisal et al. (1992) noted that the addition of lime leads to decrease in the dry density of the soil and increase in optimum moisture content, for the same compactive effort. The decrease in maximum dry density of the treated soil is the reflection of the increased resistance offered by the flocculated soil structure to that compactive effort.

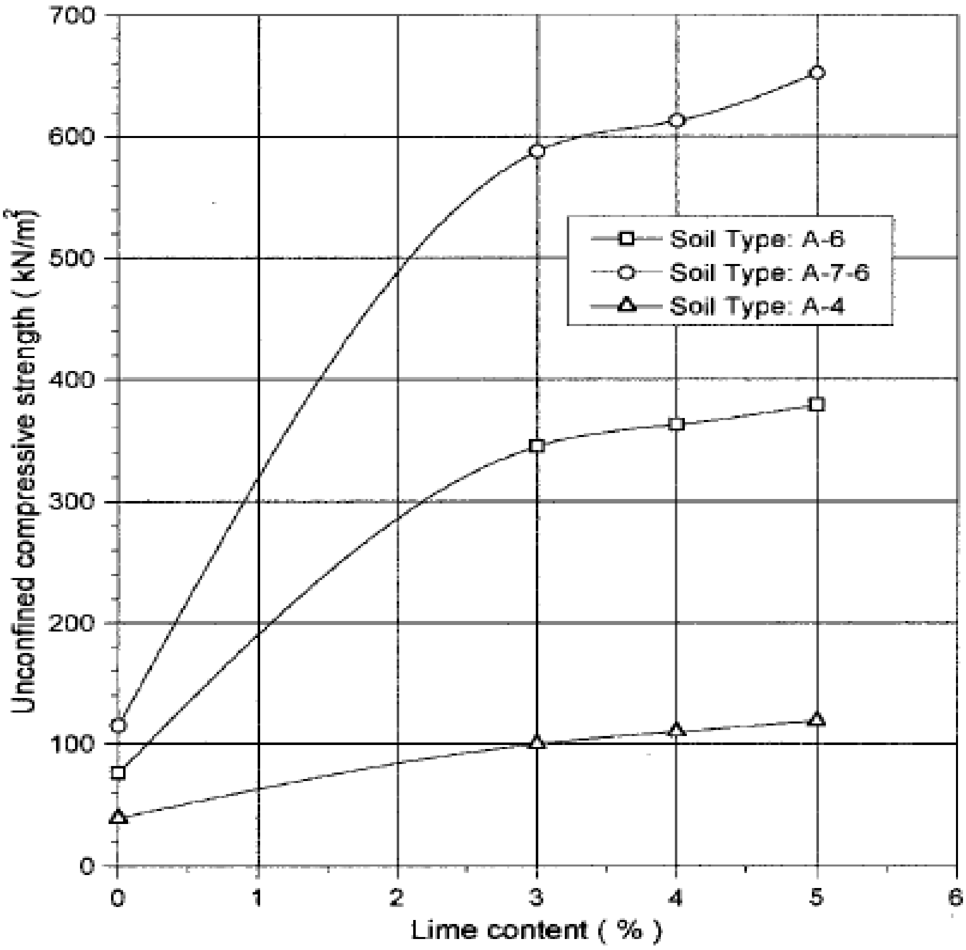

Figure 2.15: Variations of the unconfined compressive strengths of lime stabilized soil due to variations of lime contents (reproduced after Molla, 1997)

Faisal et al. (1992) also noted that the increase in optimum moisture content is probably a consequence of additional water held within the flocculated soil structure resulting from lime interaction with soil.

Dunlop (1977) observed that unconfined compressive strength of the lime stabilized soil is increased about 15% percent for modified proctor test method than the standard proctor test method. Dunlop (1977) also stated that strength of the stabilized soil is also dependent upon the uniformity of the compaction. He showed that increasing the number of blows per layer from the standard compactive effort but keeping the weight less than the standard compactive effort and reducing the falling height gives as much as 10% increase in strength.

Serajuddin (1992) reported lime stabilized soil attains higher strength and density in modified proctor test method than the standard proctor test method.

Molla (1997) investigated the effect of the amount of compaction energy on unconfined compressive strengths of three regional soils (liquid limit = 34 - 47) of Bangladesh. Molla (1997) reported that unconfined compressive strengths increase with the increases in compaction energies as shown in Figure 2.16.

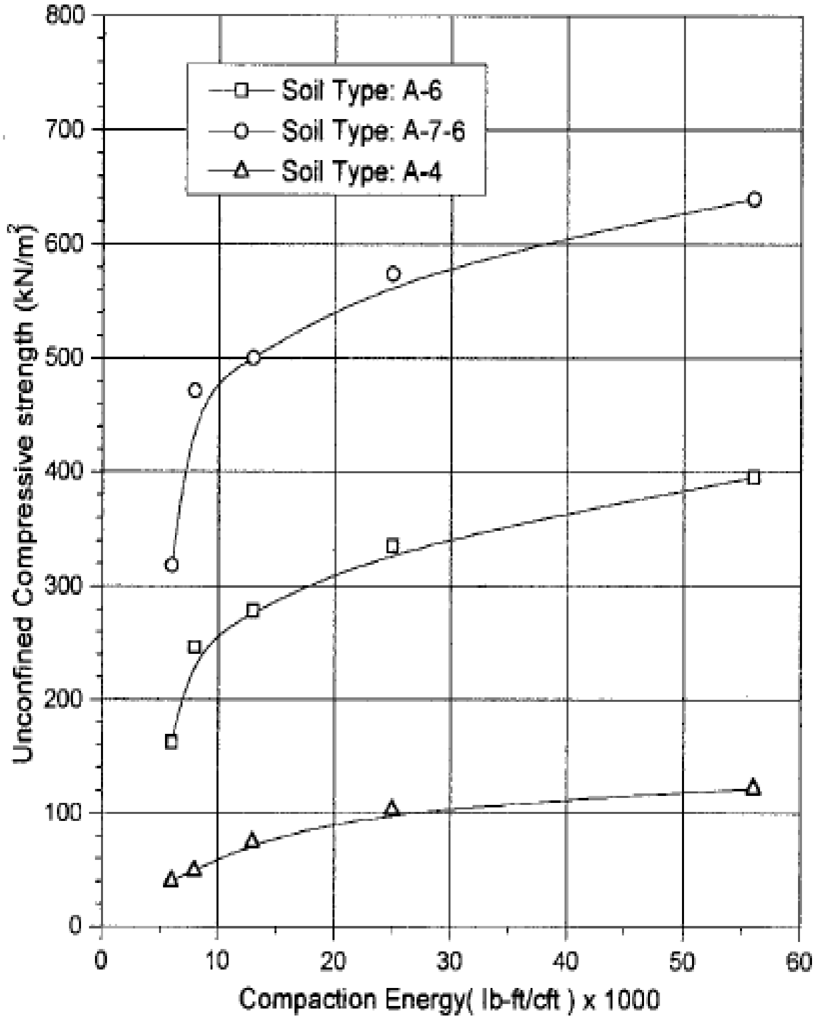

Figure 2.16: Variations of unconfined compressive strengths (q_u) at different compactive efforts for stabilized soils using 3% lime (reproduced after Molla, 1997)

2.3.4.3.2 Compaction Delay Time

Compaction delay time is the time interval between mixing of lime with soil and compaction. Mitchell and Hooper (1961) from their experiments on expansive clay reported that a delay between mixing and compaction is definitely detrimental in terms of density, swell and strength for samples under the same compactive effort. Croft (1964) also concluded that compaction should proceed immediately. The sooner the particles are brought into contact with one another, the greater will be the final strength achieved and prolonged delays will certainly be detrimental. The IRC (1973b) specifies a maximum time lag of 3 hours between mixing and compaction for the construction of roads and highways.

NAASRA (1986) suggests that if high strengths are required, then this can best be obtained by early compaction as these results in high densities. If soils are wet, a delay can be used to improve handling and compactability. Conversely, with dry soils a delay in compaction, will increase the moisture requirements.

Townsend et al. (1970) observed that the compaction delay time of 24 hours can reduce the strength of the specimen up to 30% as compared to the specimen prepared by compacting immediately after mixing.

Sastry et al. (1987) observed that for a delay period of time for two hours between mixing and compaction, there is practically no reduction in strength. But for further delay the strength of soil lime mixture continues to fall. By an independent study Sastry et al. (1987) observed the delay for 96 hours between mixing and compaction, strength of the soil lime mixture continuous to fall in the same trend.

Compendium (1987) stated that granular soil-lime mixture should be compacted as soon as possible after mixing, although delays up to two days are not detrimental, especially if the soil is not allowed to dry out. Fine grain soils can also be compacted, soon after final mixing, although delays of up to 4 days are not detrimental. Boominathan and Prasad (1992) stated that compaction delay of 24 hours can decrease the strength from 30% to 70%.

Shahjahan (2001) investigated the effect of compaction delay time on unconfined compressive strength of three regional soils of Bangladesh. He reported that unconfined compressive strengths decrease with the increases in compaction delay times. This trend is presented in Figure 2.17.

2.3.4.4 Curing Time and Curing Conditions

The shear strength of lime-treated soils increases with time in a manner similar to concrete or soil-cement mixes. The rate of increase is generally rapid at the early stage of curing time and thereafter the rate of increase in strength reduces with time. Though strength gaining occurs even after prolonged curing, the soil-lime mixtures are normally designed for a curing period of 7 to 28 days (IRC, 1976). Hilt and Davidson (1960) conducted unconfined compressive strength test on lime stabilized silty clays and found that the rate of strength gain is relatively constant up to 150 days, after which the rate slowed.

Ingles and Metcalf (1972) also studied the effect of time on the unconfined compressive strength. The variations of strength for the different curing ages as followed by Ingles and Metcalf (1972) are presented in Figure 2.18. From Figure 2.18, it can be seen that strength gain of the lime stabilized soil is highly dependent upon the soil type. For some soils the rates of increase in strengths with curing times are high but for some soils the rates are slow.

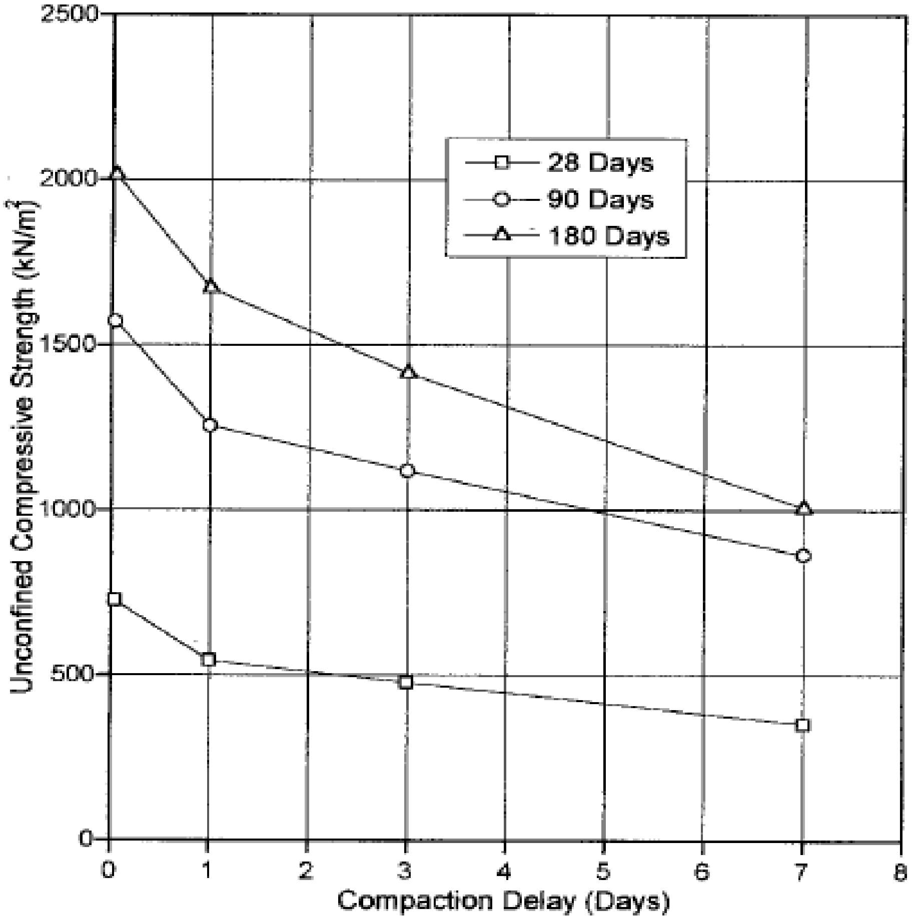

Figure 2.17: Variations of unconfined compressive strengths (q_u) with compaction delay times (Soil type-ML/CL) (reproduced after Shahjahan, 2001)

The temperature at which soil-lime mixtures are cured has a profound effect on the strength characteristics (IRC, 1976; Broms, 1986). Low temperatures are not suitable for the chemical reactions that are necessary for the cementitious actions. The chemical reactions in the soil favored by a high temperature. In fact, one of the limitations of soil-lime stabilization is the climatic factor. It is found that reactions are not effective at temperatures below 50°F and therefore under such circumstances, soil-lime stabilization is not desirable (IRC, 1976). The rate of strength gain is temperature sensitive and there are some evidences that the physical form of the cementitious products is sensitive to curing temperatures (Ingles and Metcalf, 1972; Bell, 1993). The effects of curing temperatures and times on unconfined compressive strengths of a plastic clayey soil stabilized with 5% lime are shown in Figure 2.19. It can be seen from Figure 2.19 that for a particular curing age unconfined compressive strengths increase considerably with curing temperatures and that at a particular temperature strengths increase with increasing curing ages.

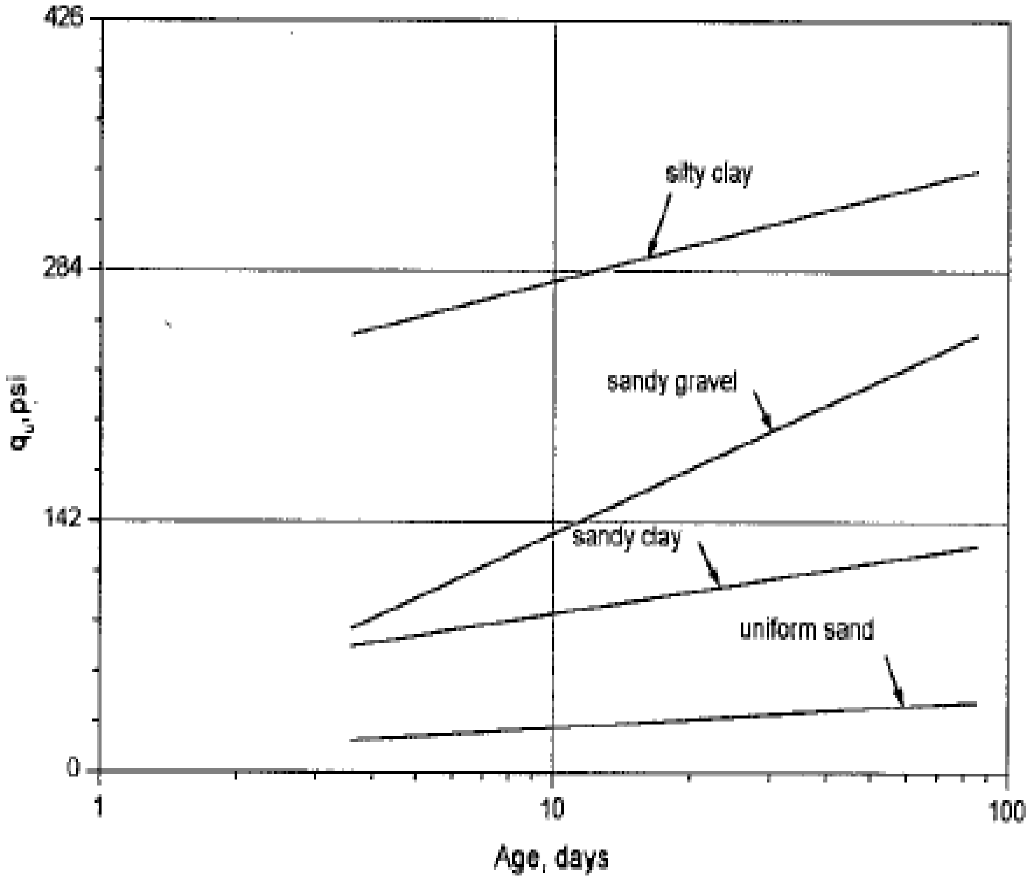

Figure 2.18: Effects of curing ages on unconfined compressive strengths (q_u) for various types of soils stabilized with 5% lime (reproduced after Ingles and Metcalf, 1972)

2.3.5 Properties of Lime Stabilized Soil

The main benefits of lime stabilization of clays are improved workability, increased strength, and volume stability. The properties of soil-lime mix have been summarized by a number of investigators (Ingles and Metcalf, 1972; IRC, 1976; Mitchell, 1981; Kezdi, 1979; NAASRA, 1986; TRB, 1987; Bell, 1993). In the following several sections the various physical and engineering properties of lime stabilized soils are reviewed.

2.3.5.1 Unconfined Compressive Strength

The unconfined compressive strength of soil-lime mix increases with increasing lime content. The rate of gain of compressive strength of soil-lime mix in the initial stages (first few days) is considerably less than that for cement stabilized materials. Lime stabilized material continues to gain strength with time provided curing is sustained.

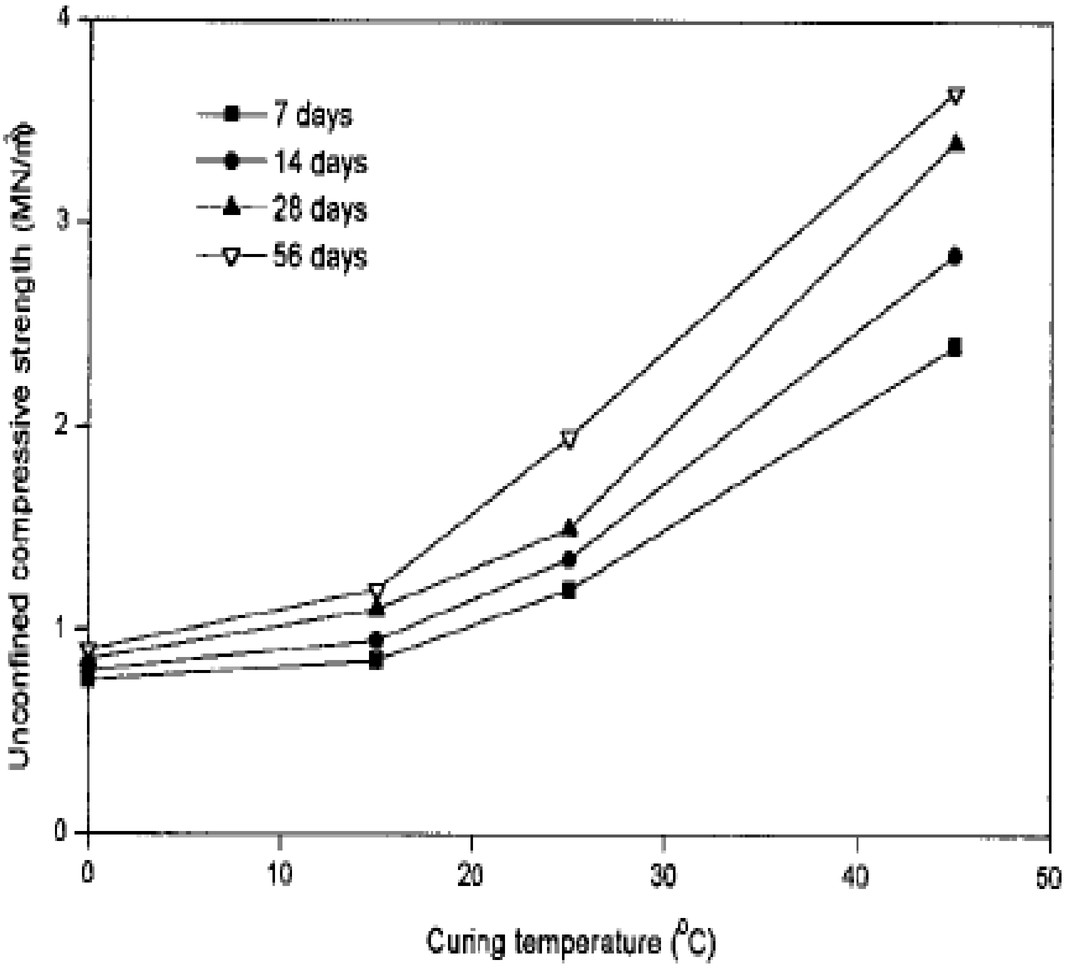

Figure 2.19: Effects of curing temperatures and curing ages on unconfined compressive strengths of a clay of high plasticity stabilized with 5% lime (reproduced after Bell, 1988)

Ahmed (1984) reported the effects of lime contents and curing ages on unconfined compression strengths for sandy silt and silty clay samples (1.4 in. diameter by 2.8 in. high) treated with various lime contents (0.5% to 5%). Typical results for the silty clay samples are shown in Figure 2.20, which shows that unconfined compressive strengths increase with the increases in lime contents and curing ages. Serajuddin and Azmal (1991) and Serajuddin (1992) also reported the effect of lime content and curing age on unconfined compressive strength of samples (50 mm diameter and 100 mm high) of regional alluvial soils of Bangladesh. Samples were treated with 5%, 7.5% and 10% slaked lime. Typical results showed that unconfined compressive strength of lime-treated samples increase with the increase in curing age and lime content. Hossain (1991) also found increases in unconfined compressive strengths with the increases in lime contents and curing ages for two regional soils of Bangladesh.

Rajbongshi (1997) also investigated the effects of lime contents and curing ages on unconfined compressive strengths of large diameter samples (2.8 in. diameter by 5.6 in. high) of a coastal soil. Rajbongshi (1997) reported that unconfined compressive strengths of lime-treated samples increase with the increases in lime contents and curing ages as shown in Figure 2.21. Shahjahan (2001) found that unconfined compressive strengths of lime-treated samples increased with the increases in lime contents and curing ages for three regional soils of Bangladesh.

Rajbongshi (1997) investigated the rate of strength gain with curing time in terms of the parameter termed as strength development index (SDI) as proposed by Uddin (1995). SDI is defined by the following expression (Uddin, 1995):

$$\text{SDI} = \frac{Strength\ of\ stabilized\ sample - Strength\ of\ untreated\ sample}{Strength\ of\ untreated\ soil} \qquad (2.16)$$

Plotting of SDIs with curing ages of samples of a lime treated coastal soil is shown in Figure 2.22. Figure 2.22 shows that the values of SDIs increase with increasing curing times and lime contents as well. Figure 2.22 clearly shows the relative degrees of strength gains resulted due to increasing lime contents and curing ages. As can be seen from Figure 2.22 that the strengths gaining for samples treated with 7% lime is relatively much higher than those of samples treated with 3% and 5% lime.

Rajbongshi (1997) and Molla (1997) investigated the effects of moulding moisture contents on unconfined compressive strengths of lime-treated samples. Unconfined compressive strengths of samples were found to increase with increasing moulding moisture contents as shown in Figure 2.23. Rajbongshi (1997) reported that at a particular curing age the values of unconfined compressive strengths of samples compacted at wet side of optimum moisture content are higher than the values of unconfined compressive strengths of samples compacted at optimum or dry side of optimum moisture content as shown in Figure 2.24. The values of unconfined compressive strengths of samples compacted at dry side of optimum moisture content has been found to the least.

2.3.5.2 Shear Strength

Shear strength of the soil increases due to the addition of lime to it. Assarson et al. (1974) stated that the increase of strength is lowest immediately after mixing of lime with soil but after 28 days the increase in strength can be reached up to 30 times to the initial strength. They also found that the increase of shear strength due to stabilization is dependent upon lime content and other factors.

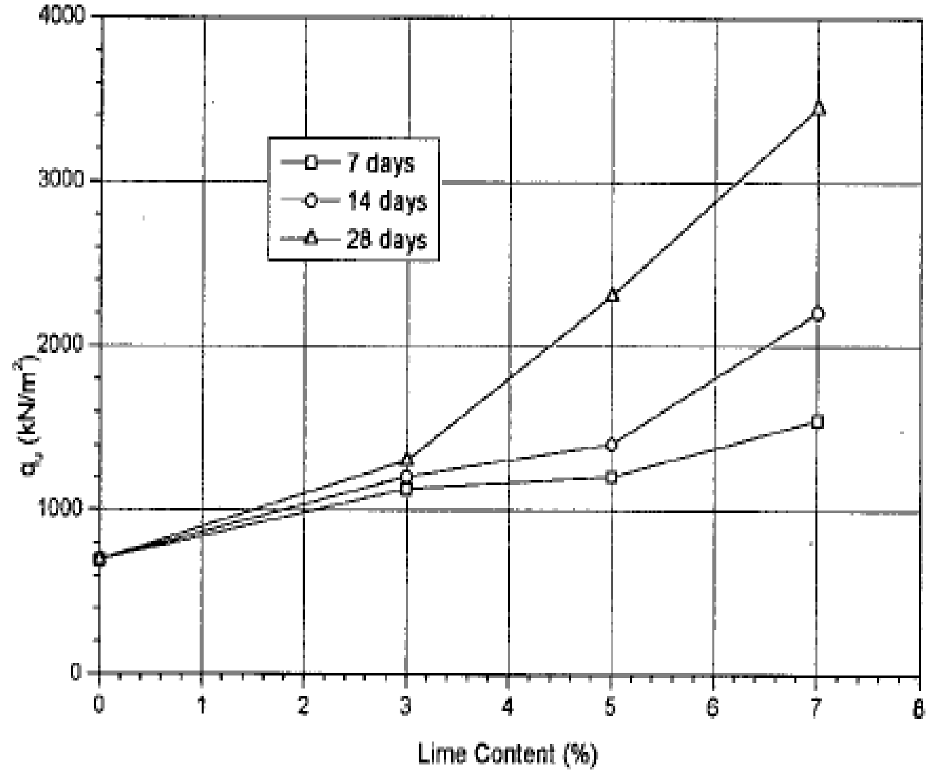

Figure 2.20: Effects of lime contents on unconfined compressive strengths (q_u) of a coastal soil at different curing ages (reproduced after Rajbongshi, 1997)

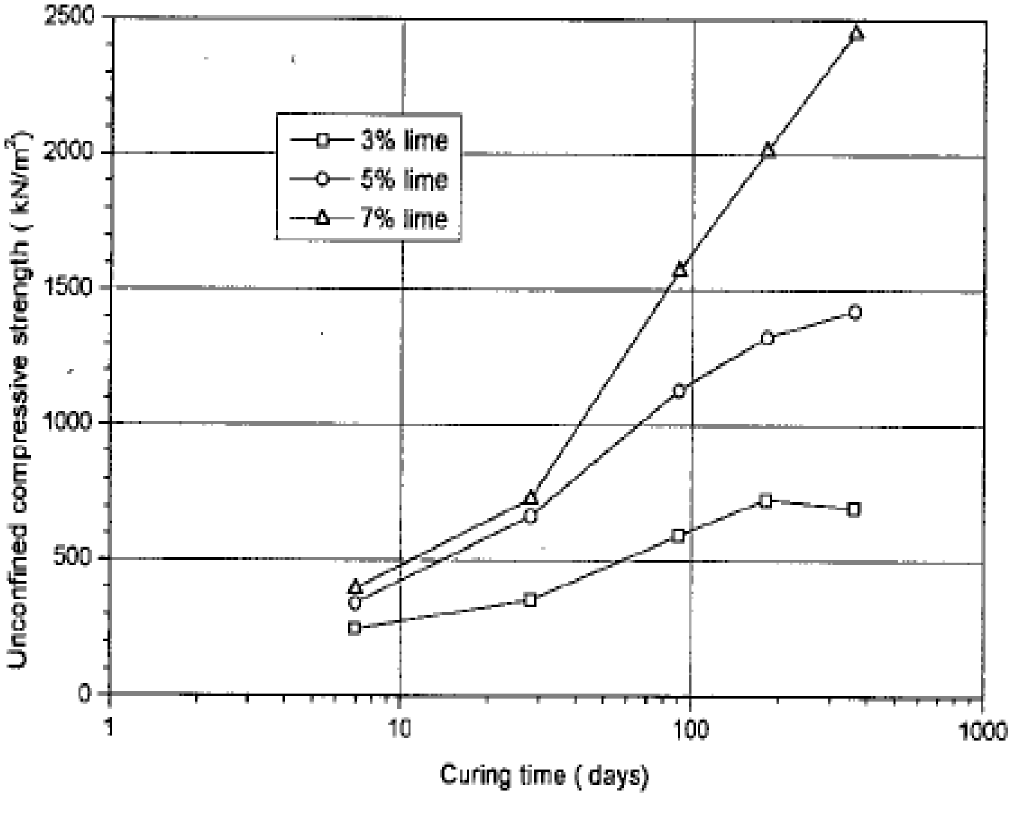

Figure 2.21: Effects of curing ages on unconfined compressive strengths (q_u) of a soil (Type-ML/CL) at different lime contents (reproduced after Shahjahan, 2001)

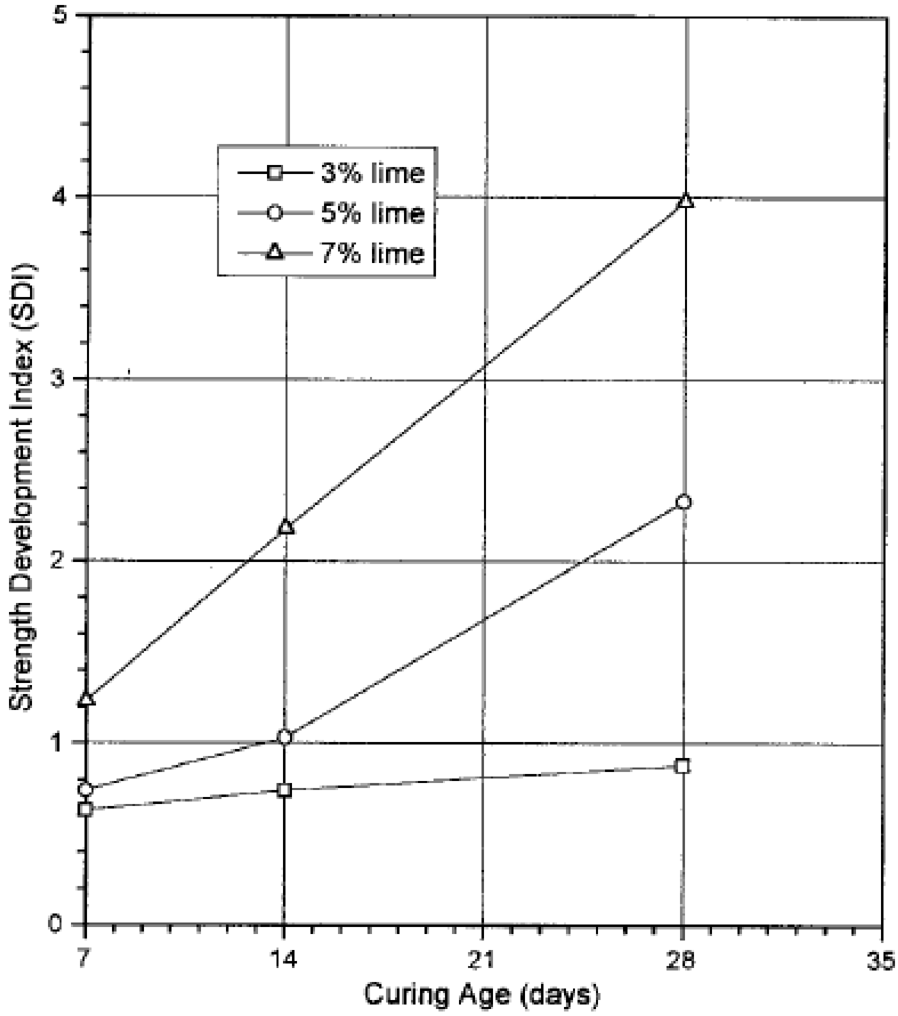

Figure 2.22: SDIs versus curing ages curves for samples of a lime-treated coastal soil (reproduced after Rajbongshi, 1997)

Yu Kuen (l975) stated that the part of the initial strength increase in stabilized soil is due to the formation of crystalline calcium hydroxide (or gel phase), which posses cementing properties. Kezdi (1979) noted that the gel phase can be clearly discerned through microscope though its chemical and crystalline composition could not be determined experimentally. Among the strength parameters of the soil (c and Φ), the increase in cohesion may be due to formation of cementitious products resulting from pozzolanic action and the increase of the angle of internal friction may be the effect of aggregation which results in greater interlocking and rough surface.

Broms and Bomans (1977) noted that the ultimate strengths of lime stabilized soils are not uniform, even when the mixing of lime with clay has been done very carefully. Broms (1984) pointed that the physical and chemical reactions brought about by lime stabilized soil result to a corresponding increase in shear strength for the treated soil mass. The shear strength of clay stabilized soil with lime will normally be higher than that of the undisturbed clay for about one or two hours after mixing. Thereafter, the shear strength of stabilized soil gradually increases with time through pozzolanic reactions, which take place for larger period. Broms (1984) pointed that the carbonation also results when lime reacts with carbon dioxide present in the soil and air. However, the strength of calcium carbonate thus formed is low. The calcium carbonate has been to retard pozzolanic reaction.

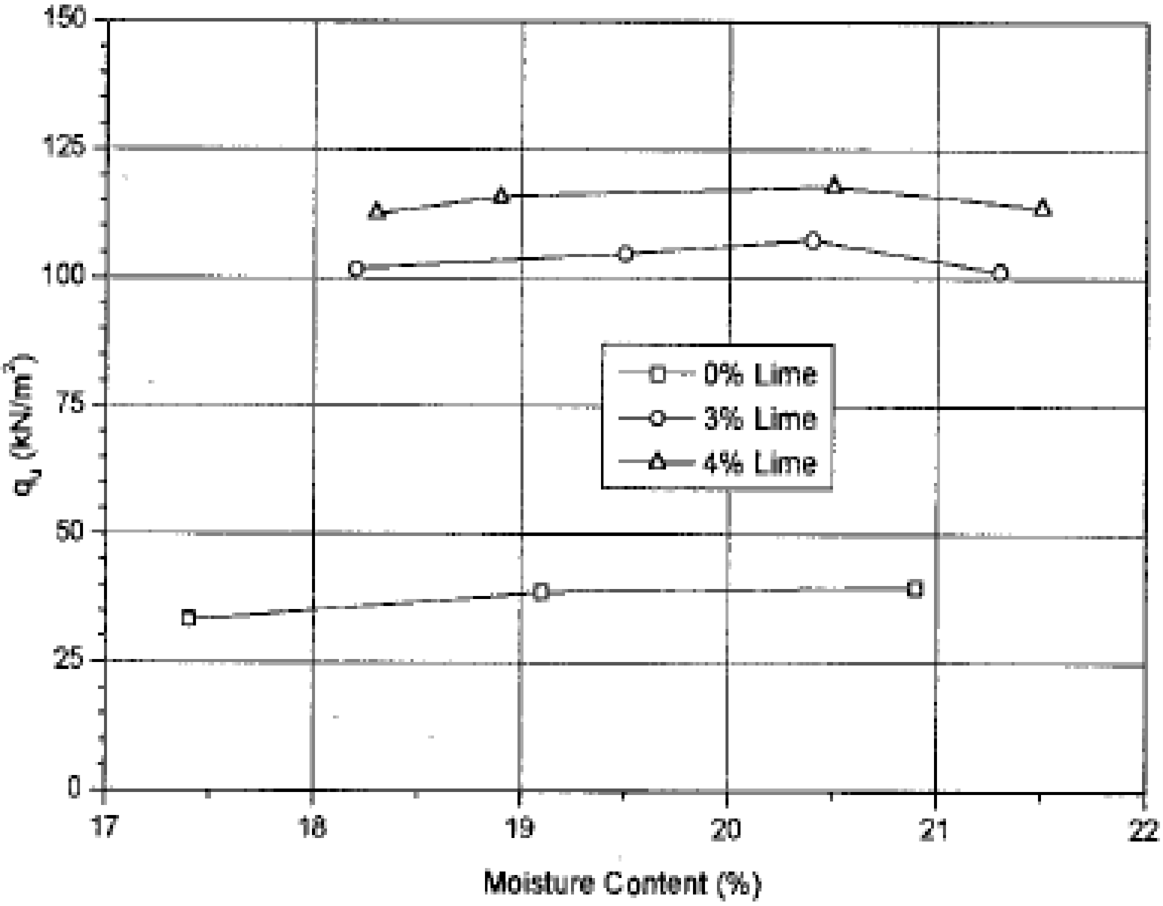

Figure 2.23: Variations of unconfined compressive strengths (q_u) with moulding moisture contents for a lime-treated silty clay soil (reproduced after Molla, 1997)

2.3.5.3 Plasticity and Shrinkage Properties

Substantial changes in the plasticity properties are produced by lime treatment. The liquid limit generally reduces with increasing quantity of lime. This observation is by and large true for clayey soils. In general, liquid limit decreases in the more plastic soils, and increases in the less plastic soils (IRC, 1976).

Irrespective of the reduction or increase in the liquid limit of the mixture, the plastic limit increases with the addition of greater percentages of lime, whether the specimens are tested immediately or after a lapse of time. The plastic limit increases with the addition of lime up to some limiting lime content and any increase thereafter causes insignificant or no increase (Mateous, 1964). As a result of the general decrease in liquid limit and a good rise in the plastic limit, the plasticity index drops down very considerably and in many cases the soil may become nonplastic (Mateous, 1964). Generally, soil with high clay content or soil exhibiting high initial plasticity index require greater quantity of lime for achieving the nonplastic condition, if it can be achieved at all. The amount of reduction in the plasticity index varies with the quantity and type of lime and also type of soil (IRC, 1976).

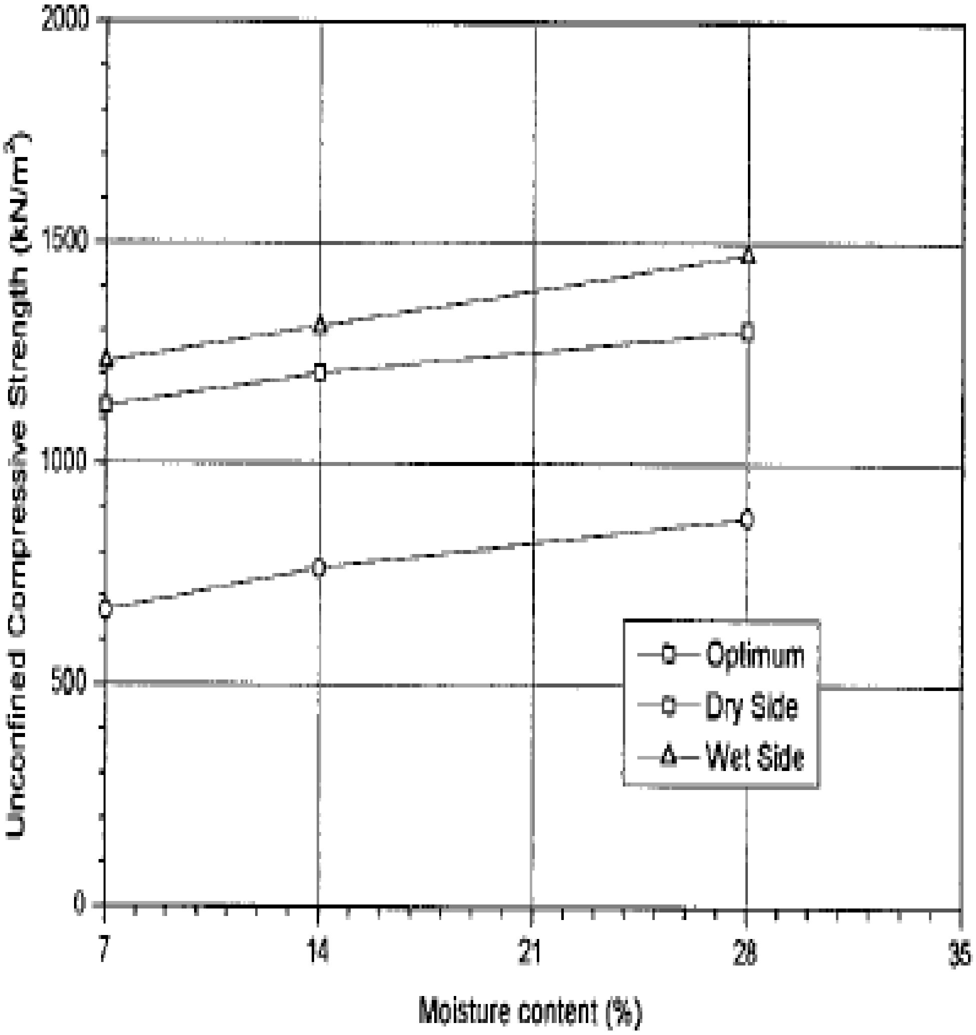

Figure 2.24: Variations of unconfined compressive strengths with different moulding water contents at a specific curing age for a coastal soil treated with 3% lime (reproduced after Rajbongshi, 1997)

Holtz (1969) reported the effects of lime on plastic characteristics of four expansive montmorillonitic clays. Holtz (1969) found that lime drastically reduces liquid limit and plasticity index and drastically raises the shrinkage limit of montmorillonitic clays.

Ahmed (1984) investigated the effect of increasing lime content on the liquid limit, plastic limit and plasticity index of regional soils of Bangladesh. Ahmed (1984) found an increase in plastic limit while liquid limit and the plasticity index reduced with increasing addition of lime. Hossain (1991), however, found increase in liquid limits and plastic limits while plasticity indices reduced (became nonplastic) with increasing of lime for two regional soils (LL = 25 and 42, PI = 12 and 20) of Bangladesh. Rajbongshi (1997) also investigated the effects of increasing lime content on the liquid limit, plastic limit, plasticity index and shrinkage limit of a coastal soil (LL = 44, PI = 19) of Bangladesh. Rajbongshi (1997) found an increase in plastic limit and shrinkage limit while liquid limit and the plasticity index reduced with increasing addition of lime, as shown in Figure 2.25. The linear shrinkage of a clayey soil is also affected by addition of lime. Linear shrinkage reduces as the lime content increases (IRC, 1976). Typical results showing the influence of linear shrinkage are presented in Figure 2.26. It can be seen from Figure 2.26 that compared with the silty clay soil, the reduction in linear shrinkage with the increase in lime content in the heavy clay is much higher.

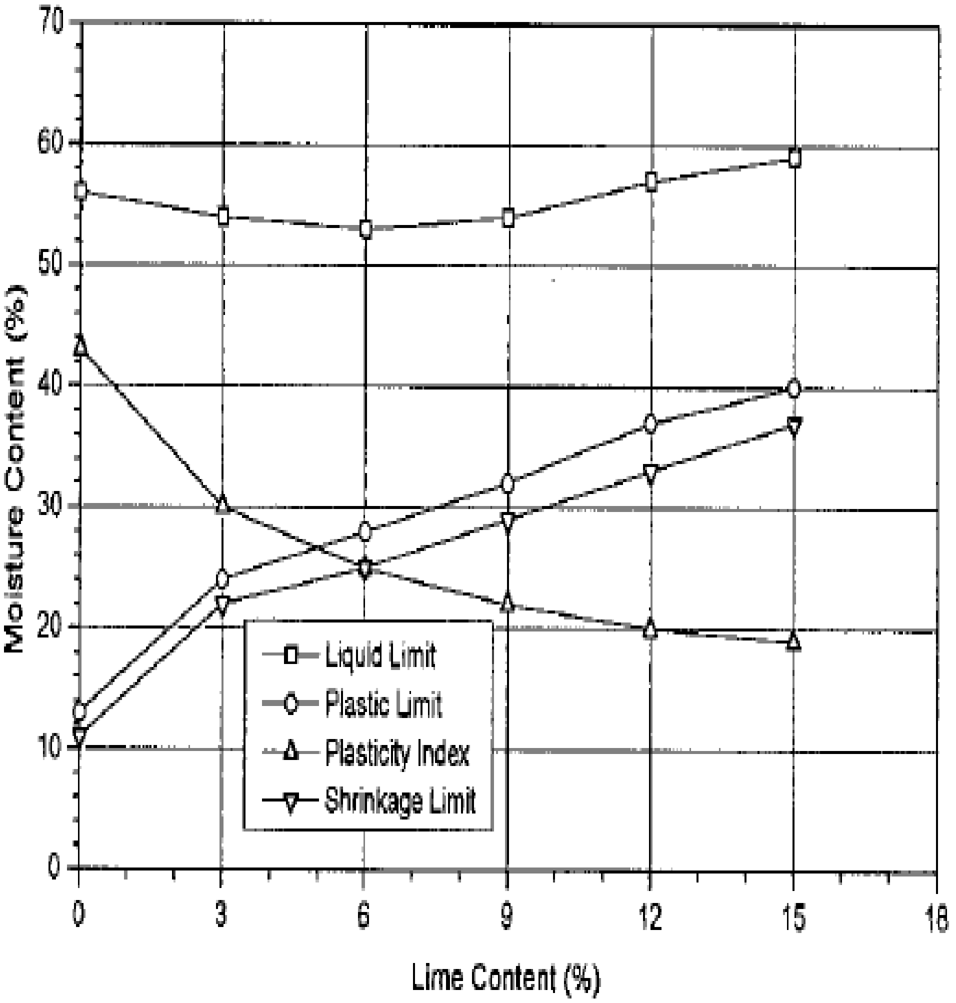

Figure 2.25: Effects of lime contents on Atterberg limits and shrinkage limits of an expansive soil (reproduced after Hossain, 2001)

2.3.5.4 Moisture-Density Relations

The addition of lime to clayey soils increases the optimum moisture content and reduces the maximum dry density for the same compactive effort. This effect is shown in Figure 2.27. The significance of these changes depends upon the amount of lime added and the amount of clay minerals present. Flocculation and cementation make the soil more difficult to compact and therefore, the maximum dry density achieved with a particular compactive effort is reduced. As lime treatment flattens the compaction curve, a given percentage of the prescribed density can be achieved over a much wider range of moisture contents so that relaxed moisture control specifications are possible. Due to increase in optimum moisture content, lime stabilization provides additional advantage when dealing with wet soils. NAASRA (1986), TRB (1987), Hausmann (1990) and Bell (1993) also reported reduction in maximum dry density due to lime stabilization.

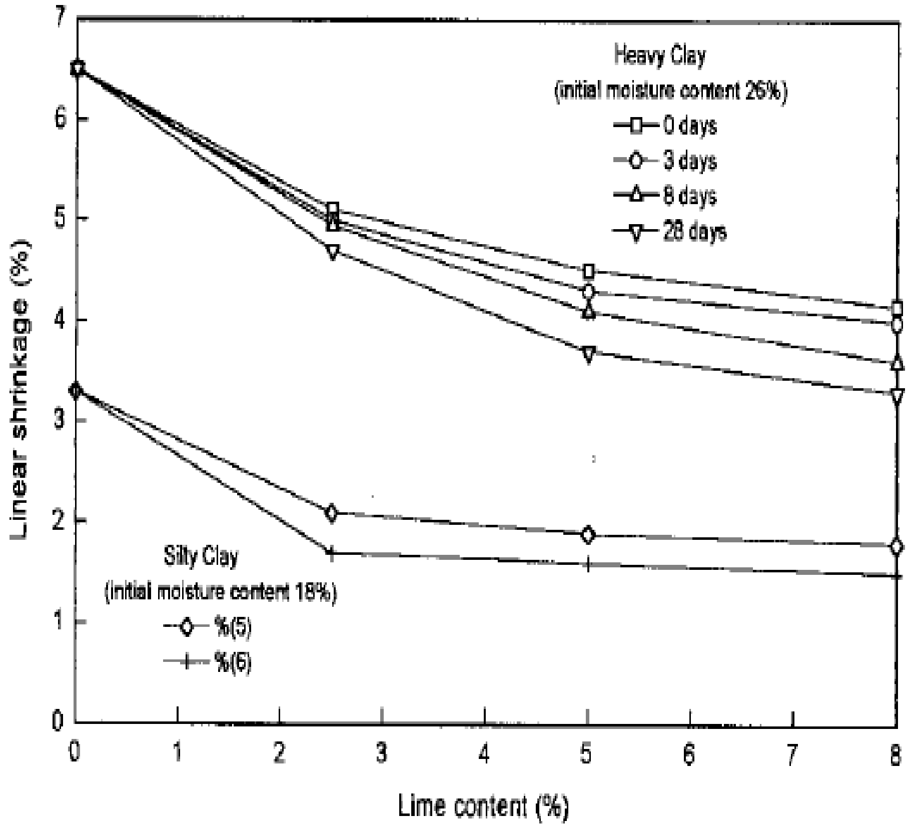

Figure 2.26: Effects of lime contents on linear shrinkages of clays (reproduced after Bell, 1988)

Ahmed (1984), Rajbongshi (1997) and Molla (1997) reported the effects of lime treatments on the maximum dry densities and optimum moisture contents of regional and coastal soils of Bangladesh. It has been reported by Ahmed (1984) that the maximum dry densities of two sandy silt and silty clay soils reduced as lime content increased. Rajbongshi (1997) and Molla (1997) reported that increment of lime content increases the optimum moisture content and reduces the maximum dry density. The reductions of maximum dry densities with increases of lime contents for a coastal soil are shown in Figure 2.28. Serajuddin and Azmal (1991) also found that compared with untreated sample, the maximum dry densities of lime-treated samples of two fine-grained regional soils reduced while optimum moisture contents slightly increased.

2.3.5.5. Failure Strain

Wang et al. (2018) studied on the dredged marine soils whose liquid and plastic limit were determined by the percussion-cup and rolling thread method according to NF P 94–051 (Association Française de Normalisation 1993) and NF P 94–052-1 (Association Française de Normalisation 1995) respectively.

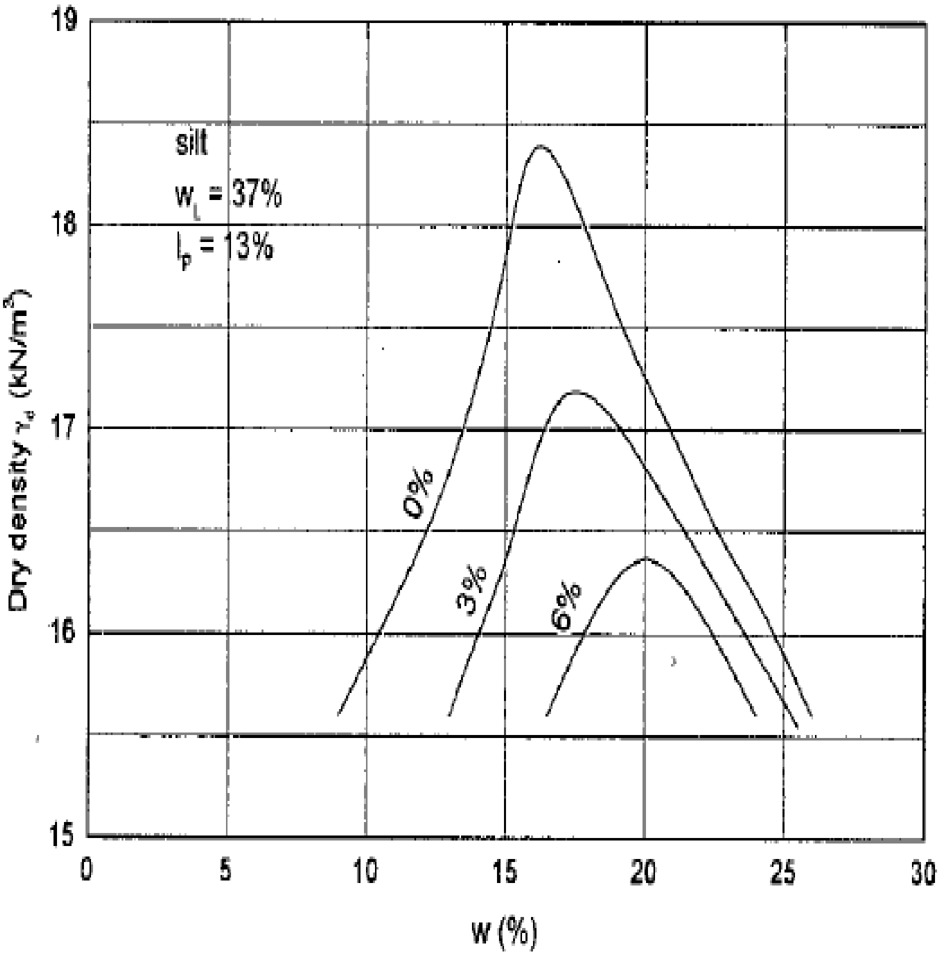

Figure 2.27: Effects of lime contents on maximum dry densities and optimum moisture contents of a lime-treated silt (reproduced after Kezdi, 1979)

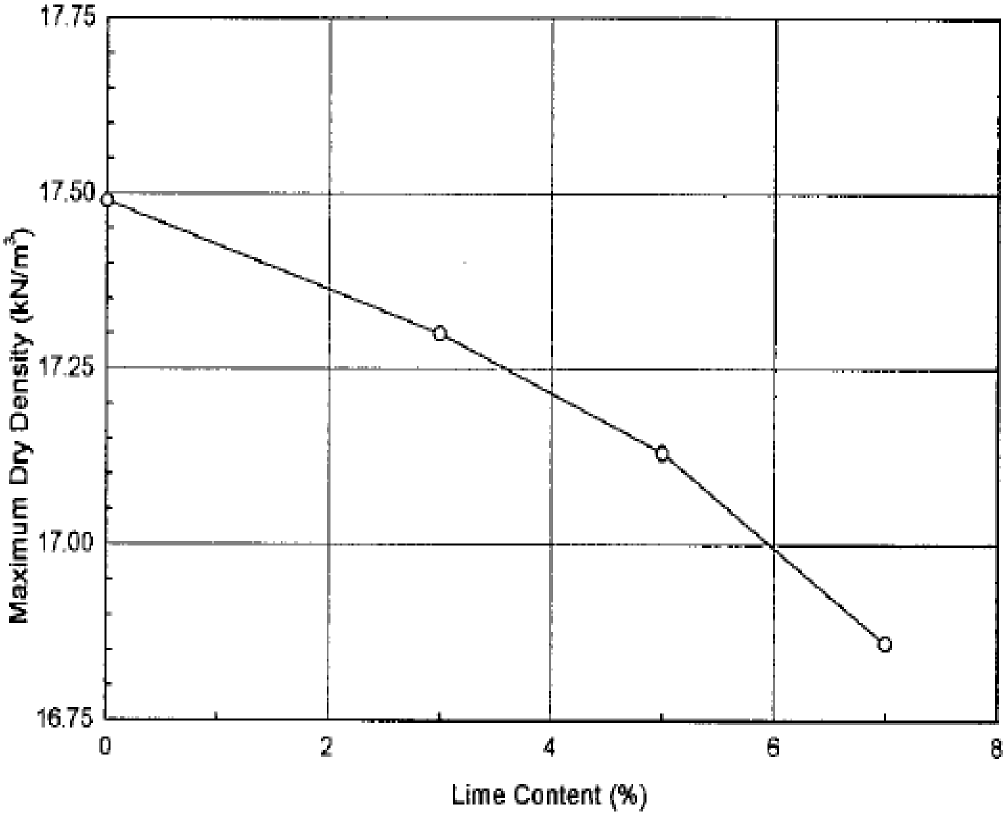

Figure 2.28: Effects of lime contents on maximum dry densities of a lime-treated coastal soil (reproduced after Rajbongshi, 1997)

According to the said two methods the liquid limit and plastic limit of the selected marine soils are 76.1% and 35.3% respectively. In his study he found that the failure strain (ε_f) values decrease from 2.13% to 1.81% with increases of lime contents from 3% to 9% in the soil-lime mixtures when these mixtures were subjected to thawing-freezing damage. He also found that as lime contents increase, ε_f values vary from 1.98% for soil-lime mixture (3% lime and 97% soil), via 1.62% for soil-lime mixture (6% lime and 94% soil), to 1.57% for soil-lime mixture (9% cement and 91% soil) when these mixtures were subjected to water immersion aging.

2.3.6 Applications of Lime Stabilization

The principal use of the addition of lime to soil is for subgrade and sub-base stabilization and as a construction convenience on wet sites where lime is used to dry out the soil. As far as lime stabilization for roadways is concerned, stabilization is brought about by the addition of between 3% and 6% lime (by dry weight of soil). When lime stabilization has been used to upgrade heavy clay soils to sub-base material quality or to upgrade plastic gravels to base course quality, an unconfined compressive strength of 1723 kN/m^2 at seven days, and a CBR of at least 80 are required, although values of unconfined compressive strength of 1034 kN/m^2 to 3102 kN/m^2 at seven days are also proposed (Ingles and Metcalf, 1972).

Lime is effective in modifying excessive plastic properties of sub-base and base course materials. Those that have plasticity indices and/or fines contents above the normally accepted level for the desired usage can usually be modified with lime. Such modification of base courses is a widely accepted and successful practice. At low lime contents (less than 2 to 3 percent) the risk of undesirable shrinkage cracking is low. Lime is usually used to modify rather than bind soils.

Lime has no application in cohesion less sands and gravels regardless of particle size distribution. Fine and clayey gravels, clayey sands and silty sands may remain excessively friable and unsuitable for base course usage when stabilized with lime. Lime stabilization is used in embankment construction for roads, railways, earth dams and levees to enhance the shear strength of the soil. In retaining structures it is used primarily to increase the resistance to water, either external or internal. Lime has also been used to stabilize low-angled slopes.

CHAPTER 3
EXPERIMENTAL INVESTIGATIONS

3.1 General

The investigations in the laboratory which were conducted on the untreated and stabilized samples of the soil collected from Mouchak in Kaliakoir Upazilla of Gazipur District (the north latitude and east longitude of the soil collection point are about 24.027868^0 and 90.299284^0 respectively) are discussed in details in this chapter.

3.2 Sampling and Collection of Soil Samples

The present investigations are carried out on disturbed soils collected from Mouchak in Kaliakoir Upazilla of Gazipur District. The colour of the collected sample was reddish brown. Approximately 2~3 feet top soil of an area of about 1 square meter was removed. Then disturbed sample of soil was collected from excavated pit in several large plastic bags. Water table was below excavated pit. Proper care was taken to remove any loose material, debris, coarse aggregates and vegetation from soil. All samples were transported to the Geotechnical Engineering Laboratory of Bangladesh University of Engineering and Technology, Dhaka. The natural moisture content of sample was 15.2%.

3.3 Laboratory Testing Programme

In order to examine the physical, index and engineering characteristics of the untreated soil and soil stabilized with cement and lime, a comprehensive laboratory investigation programme was undertaken. Ordinary Portland cement (Type-I) and air-slaked lime were used as additives for stabilization. The soil sample was mixed with slaked lime and cement in different proportions. In the soil-lime mixtures lime's contents were 2%, 5% and 8% of the oven dried (at 105^0C) weight of soil. In the soil-cement mixtures cement's contents were also 2%, 5% and 8% of the oven dried (at 105^0C) weight of soil.

The sample collected from the field was disturbed sample. This sample was air dried for about five days and then the soil lumps were broken carefully with a wooden hammer so as to avoid breakage of soil's individual particle. The required quantities of soil were then sieved through sieve No. 4 (opening size is 4.76 mm).

Information about laboratory testing programme showing the tests carried out, type of samples tested, binder content, curing period and number of tests performed are presented in Table 3.1.

Table 3.1: Detailed list of laboratory tests performed

Type of Test	Sample Type	Binder Content (Percent by Weight of Oven Dried (at 105^0C) Soil)	Curing Period (Days)	No. of Each Test
Liquid Limit, Plastic Limit, Shrinkage Limit, Linear Shrinkage Limit, and Standard Proctor Compaction Test.	Untreated soil	-	-	1
	Soil-lime mixture	2 % lime	-	1
		5 % lime	-	1
		8 % lime	-	1
	Soil-cement mixture	2 % cement	-	1
		5 % cement	-	1
		8 % cement	-	1
Direct Shear Test	Soil-lime mixture	5 % lime	7	1
			28	1
			56	1
	Soil-cement mixture	5 % cement	7	1
			14	1
			28	1
			56	1
	Untreated soil	-	-	1
Unconfined Compression Strength Test	Soil-lime mixture	2 % lime	7	1
			14	1
			28	1
			56	1
		5 % lime	7	1
			14	1
			28	1
			56	1
		8 % lime	7	1
			14	1
			28	1
			56	1
	Soil-cement mixture	2 % cement	7	1
			14	1
			28	1
			56	1
		5 % cement	7	1
			14	1
			28	1

Type of Test	Sample Type	Binder Content (Percent by Weight of Oven Dried (at 105^0C) Soil)	Curing Period (Days)	No. of Each Test
Unconfined Compression Strength Test	Soil-cement mixture	5% cement	56	1
		8 % cement	7	1
			14	1
			28	1
			56	1
	Untreated soil	-	-	1

3.4 Properties of Binders Used for Soil Stabilization

For this research, Ordinary Portland Cement (Type-I) and slaked lime were used for the stabilization of soil. For the determination of normal consistency of cement paste, setting times (initial and final setting times) of cement paste and compressive strength of 50 mm (2 inch) cubic specimens, the standard test procedures outlined in ASTM C187, Cl91 and C109 were followed respectively. The results of these tests are shown in Table 3.2. In this research, hydrated lime (i.e., slaked lime), which is commercially available in the market, was used for the stabilization with lime.

Table 3.2: Test results of Ordinary Portland Cement (Type-I)

Properties		Results
Amount of water for Normal Consistency		23.5%
Setting time	Initial setting time	130 minutes
	Final setting time	330 minutes
Compressive strength	3 days	25270 kilo newton per square meter
	7 days	33870 kilo newton per square meter
	28 days	42890 kilo newton per square meter

A plot of compressive strength vs. curing period curve of the cement used for soil stabilization is given as follows:

3.5 Liquid Limit, Plastic Limit, Shrinkage Limit and Linear Shrinkage Tests

Liquid limits, plastic limits, plasticity indices and shrinkage characteristics including shrinkage limits and linear shrinkages of untreated samples (0% lime or cement content), lime-soil mixtures (lime content was 2%, 5% and 8% of the oven dried (at 105^0C) weight of soil) and cement-soil mixtures (cement content was 2%, 5% and 8% of the oven dried (at 105^0C) weight of soil) were determined.

Liquid limit and plastic limit tests of the said stabilized samples were carried out on air-dried pulverised samples. The required quantities of soil were pulverised in such a way that all the pulverised soil pass through sieve No. 40 (sieve opening is 0.425 mm). After mixing of water in the untreated soil and said soil-lime and soil-cement mixtures, they were kept at least 16 hours in covered condition to bring uniform moisture content in soils. Liquid limit, plastic limit and plasticity index of the untreated and said treated samples were determined following the standard procedure outlined in ASTM D4318 – 10. Shrinkage limit was determined in accordance with the procedure specified in ASTM D427-9804. Linear shrinkage of the untreated and said cement and lime treated samples were determined following the procedure outlined in BS 1377.

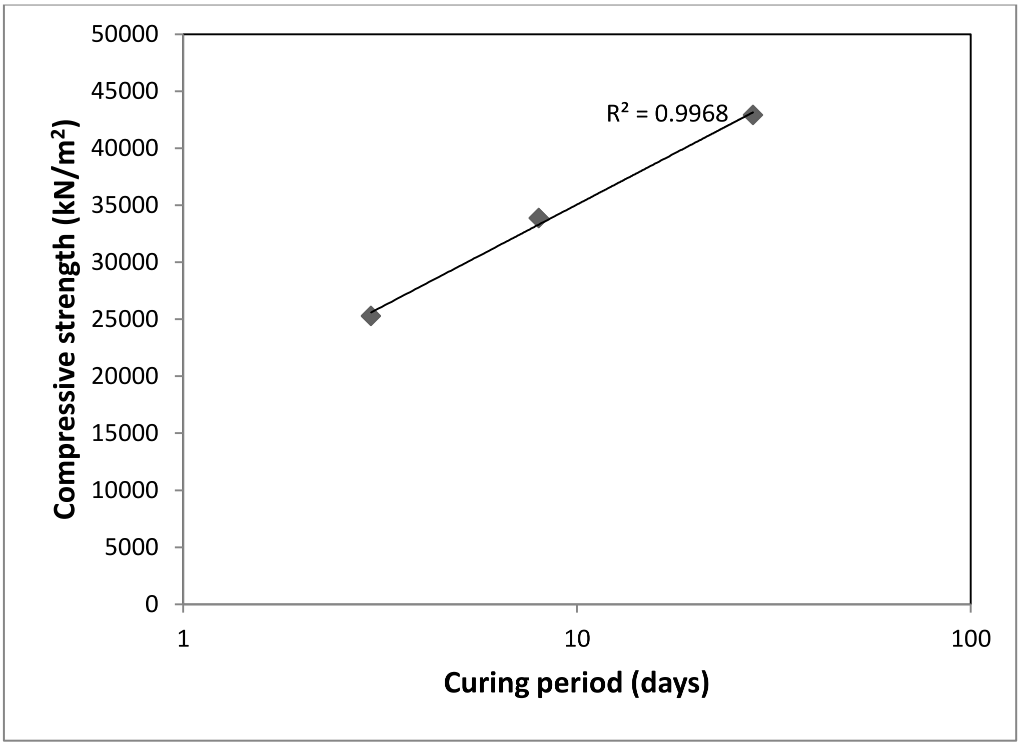

Figure 3.1: A plot of compressive strength of the cement used for soil stabilization vs. curing period curve

3.6 Proctor Test

The moisture content versus dry density relationships of the untreated sample and treated samples were investigated by carrying out standard proctor tests. These tests were performed according to the standard procedure outlined by ASTM D698 – 12. The required quantities of soil were pulverised in such a way that all the pulverised soil pass through sieve No. 4. For compaction of the moist samples, a cylindrical mould of 4 inch (l01.6mm) inside diameter and of volume 0.0333 cubic foot was used. A series of moist samples of varying moisture contents were compacted in three layers of approximately equal height. Each layer was compacted by 25 blows from a rammer of weight 5.5 lbf (2.495 kg) and falling from a free height of 12 inch. The amount of material used was such that the third compacted layer was slightly above the top of the mould but not exceeding 6 mm. During compaction, the mould was placed on an uniform rigid foundation. Moisture content and dry density determinations were made on each compacted sample of said treated and untreated soil. Finally, optimum moisture content and maximum dry density for the said treated mixtures and untreated soil were determined.

3.7 Unconfined Compression Strength Test

3.7.1 Preparation and Mixing of Soils

Untreated soil was first air-dried for 5 days. Then the soil aggregates were broken carefully with a wooden hammer in order to avoid reducing the natural size of the individual particles. The required quantities of soil were pulverised in such a way that all the pulverised soil pass through sieve No. 4 (4.76 mm). Representative soil sample of required quantity (1800 gram) was taken to prepare each untreated and treated sample of desired dry density, i.e., the maximum dry density obtained in the standard proctor test. Moisture content of air dried soil sample was determined. Ordinary Portland Cement (Type-I) was mixed uniformly with soil in amount of 2%, 5% and 8% of the oven dried (at 105^0C) weight of soil individually. Similarly, lime was mixed uniformly with soil in amount of 2%, 5% and 8% of the oven dried (at 105^0C) weight of soil individually. Soils were mixed by hand with cement and lime respectively. This mixing was carried out on a steel tray. The uniformly mixed soil-binder mixtures were spread on the steel tray. The spread area of the soil-binder mixtures was about 2 square foot. Water was sprayed on the spread soil-binder mixtures. The amount of sprayed water was determined in such a way that the water content of the compacted soil reaches to its respective optimum moisture content which was obtained from respective standard proctor test. After spraying water on the spread soil-binder mixtures they were uniformly mixed again with hand.

3.7.2 Mould for Compression Test

The mould which was used for compacting untreated soil, said soil-cement and soil-lime mixtures was fabricated using locally available seamless pipe of mild steel. The mould complies with the requirements of standard steel cylindrical mould with necessary accessories as outlined in ASTM D698-12. The mould was fabricated for the preparation of standard proctor test samples of soil-cement and soil-lime in the laboratory under accurate control of quantities of materials and test conditions.

3.7.3 Compaction and Preparation of Samples

Standard proctor test samples of untreated and treated soils were prepared with the cylinder of size with 4 inch (101.4 mm) in diameter by 4.584 inch (116.4 mm) in height. After completing the mixing of the soil-binder-water, about 50 minute was delayed for softening of the sample. After softening of the mixtures they were compacted according to ASTM D 698-12. The samples were then ejected from the mould using a hydraulic ejector. The compacted dry densities of the samples were nearly equal to their respective maximum dry density achieved in the standard proctor test performed according to the standard procedures outlined in ASTM D698-12. The dry densities of different compacted samples with corresponding maximum dry densities obtained from corresponding standard proctor tests, and moisture contents after preparation of the samples with corresponding optimum moisture contents obtained from standard proctor tests are mentioned in the following Table 3.3.

Table 3.3: Different dry densities with corresponding maximum dry densities, and moisture contents after preparation of the samples with corresponding optimum moisture contents for different soil-binder mixtures for unconfined compressive strength tests

Binder Content	**Curing Period (Days)**	**Dry Density Achieved (kN/m^3)**	**Maximum Dry Density (kN/m^3)**	**Moisture Content (%)**	**Optimum Moisture Content (%)**
2% lime	7	15.89	15.88	18.9	21.9
5% lime		15.57	15.58	20.7	22
8% lime		14.9	14.98	21.4	22.2
2% lime	14	16.11	15.88	19.6	21.9
5% lime		15.48	15.58	20	22
8% lime		14.65	14.98	20.7	22.2
2% lime	28	15.29	15.88	21	21.9
5% lime		16.17	15.58	20.1	22
8% lime		14.59	14.98	22.2	22.2
2% lime	56	15.35	15.88	19.4	21.9
5% lime		15.14	15.58	20	22
8% lime		15.03	14.98	21.00	22.2
0% lime and cement	0	16.87	15.96	17.6	20.6
2% cement	7	15.6	15.67	23	22.7
5% cement		14.7	15.55	22.6	23
8% cement		14.9	15.64	22.5	22.9
2% cement	14	15.5	15.67	22.6	22.7
5% cement		15.5	15.55	22.9	23
8% cement		15.4	15.64	22.1	22.9

Binder Content	Curing Period (Days)	Dry Density Achieved (kN/m^3)	Maximum Dry Density (kN/m^3)	Moisture Content (%)	Optimum Moisture Content (%)
2% cement	28	15.3	15.67	22.3	22.7
5% cement		14.6	15.55	23.1	23
8% cement		15.3	15.64	22.7	22.9
2% cement	56	15.0	15.67	22.4	22.7
5% cement		15.4	15.55	23.2	23
8% cement		15.1	15.64	22.9	22.9

After ejecting the sample from mould it was shaped into a cylinder of diameter=1.5 inch and height=3 inch with the help of a piano wire. Total 25 numbers of samples were prepared.

3.7.4 Curing of Samples

As soon as the samples were prepared for Unconfined Compression Strength Test, they were then kept in a desiccator at normal room temperature (19^0 C-25^0 C). There was maximum 0.5% change in moisture content between starting and completing curing period. The samples were never cured with direct water spray or under submerged condition. The samples were always protected from free water for the specified moist curing periods of 7, 14, 28 and 56 days. It may be mentioned that the soil sample which was prepared without adding cement or lime, i.e., the untreated sample was not cured.

3.7.5 Determination of Unconfined Compressive Strength

The samples were placed in the compression-testing machine directly after removal from the moist desiccator at different ages. A strain gauge was used to monitor deformation during the application of load. Each sample was tested under strain-controlled condition. During the progress of test, load was applied continuously and without shock at a deformation rate of approximately 0.02 mm per second. The total load and the corresponding deformation at failure were recorded. These tests were carried on following ASTM D2166/D2166M-13. The untreated samples were tested in compression immediately after preparation.

3.8 Direct Shear Test

3.8.1 Preparation and Mixing of Soils

Untreated soil was first air-dried for 5 days. Then the soil aggregates were broken carefully with a wooden hammer in order to avoid reducing the natural size of the individual particles. The required quantity of soil was pulverised in such a way that all the pulverised soil pass through sieve No. 4 (sieve opening size 4.76 mm). Representative soil sample of required quantity (1800 gram) was taken to prepare test sample of desired dry density, i.e., 95% of maximum dry density obtained in the corresponding standard proctor test. Moisture content of air-dried soil sample was determined. Ordinary Portland cement (Type-I) was mixed uniformly with soil in amount of 5% of the oven dried (at 105^0C) weight of soil. Similarly, lime was mixed uniformly with soil in amount of 5% of the oven dried (at 105^0C) weight of soil. Soil was mixed by hand with cement and lime respectively. This mixing was carried out on a steel tray. The uniformly mixed soil-binder mixtures were spread on the steel tray. The spread area of the soil-binder mixtures was about 2 square foot. Water was sprayed on the spread soil-binder mixtures. The amount of sprayed water was determined in such a way that the water content (the wet side of Optimum Moisture Content) of the compacted soil reaches to such value that 95% of Maximum Dry density which was obtained from respective Standard Proctor Test is achieved. After spraying water on the spread soil-binder mixtures they were uniformly mixed again with hand.

3.8.2 Mould for Compaction of the Samples

The description of mould for compaction of the samples for direct shear test is same as mentioned in the 3.7.2.

3.8.3 Compaction and Preparation of Samples

The compaction procedure of the different samples which were prepared as mentioned in 3.8.1 is same as mentioned in 3.7.3. The dry densities of different compacted samples with corresponding 95% dry densities of the maximum dry densities obtained from corresponding standard proctor tests, and moisture contents after preparation of the samples with corresponding moisture contents (on the wet side of optimum moisture content) required for achieving 95% of the maximum dry densities are mentioned in the following Table 3.4.

After ejecting the sample from mould it was cut into three equal pieces of 4 inch diameter. Then the direct shear ring was pushed into each piece which was cut. Then top and bottom surfaces of direct shear ring were leveled with a knife. Total 25 numbers of samples were prepared.

3.8.4 Curing of Samples

Curing of the samples prepared according to 3.8.1 and 3.8.3 was done in the same procedure mentioned in 3.7.4.

3.8.5 Determination of Shear Parameters

The stabilized samples were placed in the shear box after removal from the moist desiccator at corresponding ages. Shear Box was placed in position in the shear device. The shear box was filled with water. Desired reasonable normal load was applied on the sample (applied normal load is mentioned in Table 3.4). Maximum duration for consolidation by normal load was 90 minute. The deformation due to consolidation due to applied normal load at different time was recorded. Each sample was tested under shear displacement-controlled condition. During the progress of test, shear displacement was applied continuously and without shock at a deformation rate of approximately 0.16 mm per minute. The shear displacements with corresponding shear loads and vertical expansions or deformations were recorded. The test was carried on following ASTM D6528-17.

Table 3.4: Different data regarding samples of consolidated undrained direct shear tests

Binder Content	**Normal Load (kg)**	**Curing Period (Days)**	**Dry Density Achieved (kN/m^3)**	**95% of Maximum Dry Density (kN/m^3)**	**Moisture Content (%)**	**Moisture Content for Achieving 95% Dry Density at Wet Side of Optimum Moisture Content(%)**
5% lime	8	7	15.1	14.8	25.1	25.5
	16		15.1		22.1	
	32		14.6		26	
	10	14	14.2		24.5	25.5
	20		14.9		24.6	
	40		15.1		24.6	
	10	28	15.1		25.1	25.5
	20		14.7		25.6	
	40		15.3		25.4	
	10	56	15.5		25.4	25.5
	20		15.6		25.5	
	40		15.4		26.0	25.5
5% cement	10	7	14.6	14.8	26.1	26.9
	20		15.1		26.5	
	40		15.1		27.0	

Binder Content	Normal Load (kg)	Curing Period (Days)	Dry Density Achieved (kN/m^3)	95% of Maximum Dry Density (kN/m^3)	Moisture Content (%)	Moisture Content for Achieving 95% Dry Density at Wet Side of Optimum Moisture Content(%)
5% cement	10	14	14.9	14.8	25.8	26.9
	20		14.7		26.1	
	40		14.7		25.6	
	10	28	14.5		26.2	26.9
	20		15.3		26.8	
	40		14.3		27.0	
	10	56	14.5		25.9	26.9
	20		15.1		26.6	
	40		15.1		27.2	
Untreated	10	-	13.7	15.2	24.0	24.8
	20		14.6		24.7	
	40		13.7		24.9	

CHAPTER 4
RESULTS AND DISCUSSIONS

4.1 Introduction

The findings of the laboratory investigations on the characteristics of untreated and stabilized samples of the selected regional soil are presented and discussed in the following sections of this chapter. These results demonstrate the effect of additives, e.g., cement and lime on the several physical and engineering properties of the samples investigated.

4.2 Properties of Untreated Soil

The liquid limit, plastic limit, plasticity index, shrinkage limit, linear shrinkage, optimum moisture content, maximum dry density, unconfined compressive strength, axial failure strain, initial tangent modulus, undrained consolidated cohesion and undrained consolidated angle of internal friction values of untreated soil are shown in Table 4.1.

Table 4.1: Different properties of untreated soil

Property	Unit	Amount
Liquid limit	%	42
Plastic limit	%	17
Plasticity index	%	25
Shrinkage limit	%	16
Linear shrinkage	%	11.5
Optimum moisture content	%	20.6
Maximum dry density	kN/m^3	16
Unconfined compressive strength	kN/m^2	785
Axial failure strain from Unconfined Compressive Strength Test	%	2.8
Initial tangent modulus from Unconfined Compressive Strength Test	kN/m^2	61300
Undrained consolidated cohesion	kN/m^2	6.1
Undrained consolidated angle of internal friction	degree	24

The soil used in this research work can be classified according to Unified Soil Classification System as CL. On the other hand according to the works done by Hossain, soils with linear shrinkage in the range of 10% -14% are medium expansive (Hossain, 1983). The linear shrinkage of the soil used in this research work is 11.5%. So, the soil used in this research work is medium expansive because it’s linear shrinkage value is 11.5%.

4.3 Physical and Engineering Properties of Cement-Treated Soils

In the following sections i.e. sections 4.3.1 to 4.3.4.3, physical and engineering characteristics comprising plasticity and shrinkage properties, moisture-density relations, unconfined compressive strengths, axial failure strains, initial tangent moduli, consolidated undrained cohesions, consolidated undrained angles of internal frictions and changes in heights of samples with shear displacements in direct shear tests of untreated and cement-treated samples of the selected medium expansive regional soil are presented and discussed.

4.3.1 Plasticity and Shrinkage Characteristics

The values of plasticity and shrinkage properties of the untreated and cement-treated soil samples are shown in Table 4.2.

Table 4.2: Variations of plastic limits, liquid limits, shrinkage limits, plasticity indices, percentages of linear shrinkages with respect to different cement contents in the cement stabilized soil samples

Cement Content (%)	Plastic Limit	Liquid Limit	Shrinkage Limit	Plasticity Index	Percentage of Linear Shrinkage
0	17	42	16	25	11.5
2	19	44	23	25	10
5	26	52	33	26	11
8	31	57	42	26	9

It can be seen from Table 4.1 that compared with the untreated sample; plastic limits, liquid limits and shrinkage limits of the soil-cement mixtures increase while plasticity indices of the soil-cement mixtures do not virtually change. The said table also shows that the percentages of linear shrinkages of cement treated samples are lower than untreated sample.

The variations of plastic limits, liquid limits, shrinkage limits, plasticity indices and percentages of linear shrinkages with respect to different cement contents are shown in Figure 4.1, Figure 4.2, Figure 4.3, Figure 4.4 and Figure 4.5 respectively. Figure 4.1, Figure 4.2 and Figure 4.3 show that the plastic limits, liquid limits and shrinkage limits increase with increasing cement contents in the soil-cement mixtures. Figure 4.4 shows that the plasticity indices do not change significantly with change of cement contents in the soil-cement mixtures. Figure 4.5 shows that the percentages of linear shrinkages change from 11.5 for 0% cement content via 10 for 2% cement content and 11 for 5% cement content to 9 for 8% cement content. Since changes in percentages of linear shrinkages due to cement treatment is insignificant so, change in expansive characteristics of the selected medium expansive soil is also insignificant.

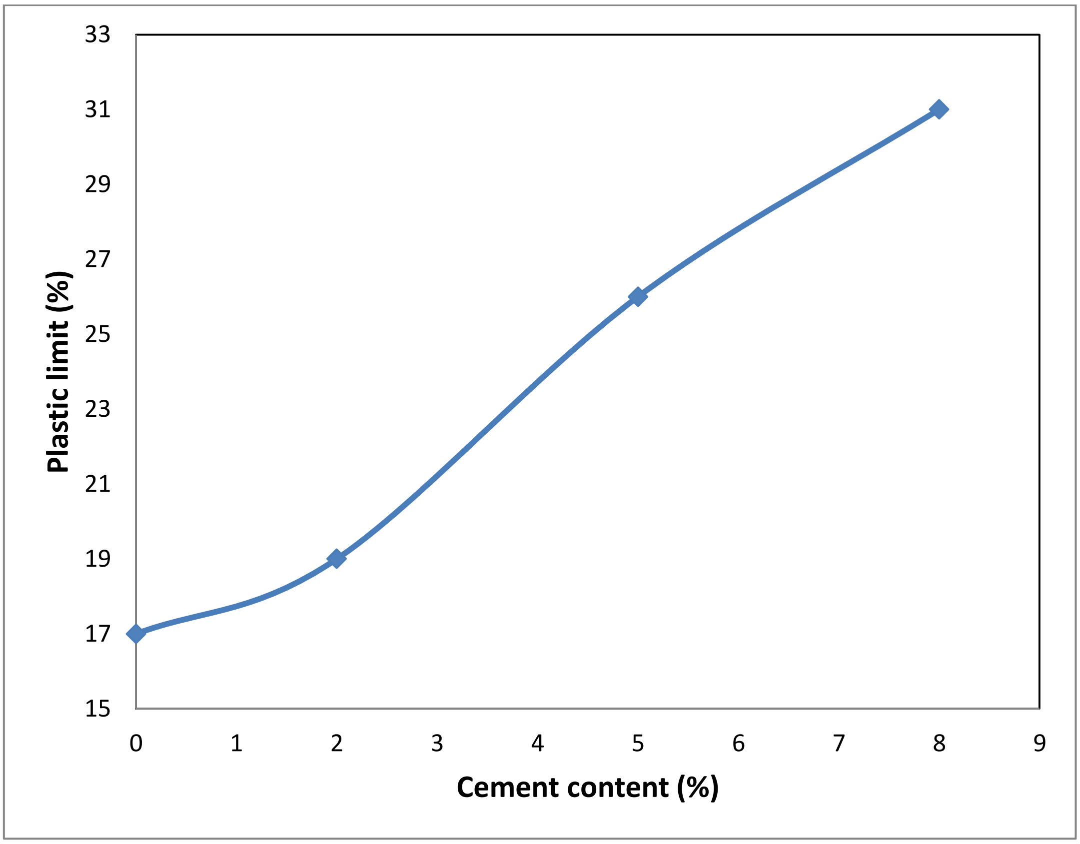

Figure 4.1: Variation of plastic limits with different cement contents

The increments of liquid limits and plastic limits with increasing cement contents in the soil-cement mixtures are also mentioned in the works of Hasan (2002) although the rates of increments are higher for the case of the present study. Hasan (2002) found that with the increases of cement contents in a typical soil, both liquid limits and plastic limits increase while plasticity indices decrease. However, Rajbongshi (1997) found reductions in liquid limits and plasticity indices, and increases in plastic limits with increasing cement contents for a typical soil. The trend of plasticity indices and shrinkage limits of the present study is also contradictory to those of observed by Rajbongshi (1997) and Hasan (2002). The decreases of percentages of linear shrinkages observed in the current research work are not sequential as observed in the works of Rajbongshi (1997) and Hasan (2002).

For a typical soil, reductions in shrinkage limits and percentages of linear shrinkages with increased cement content have been reported by Rajbongshi (1997) and Hasan (2002). On the contrary of the said investigations, present study shows increases of shrinkage limits with increases of cement contents in soil-cement mixtures. The changes of the percentages of linear shrinkages of the selected soil-cement mixtures also do not show the same trend of the mentioned two studies.

4.3.2 Moisture-Density Relations

The moisture-density relations of untreated and cement-treated samples of the selected medium expansive regional clay are shown in Figure 4.6. From the relations presented in Figure 4.6, the maximum dry densities and optimum moisture contents of the different soil-cement mixtures have been determined which are presented in Table 4.3.

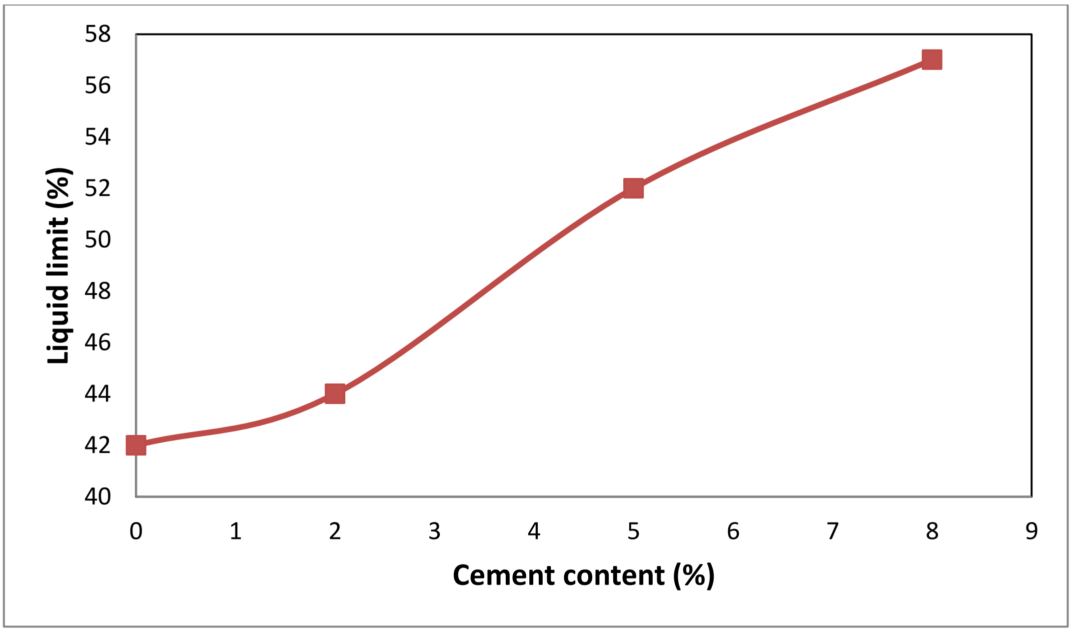

Figure 4.2: Variation of liquid limits with different cement contents

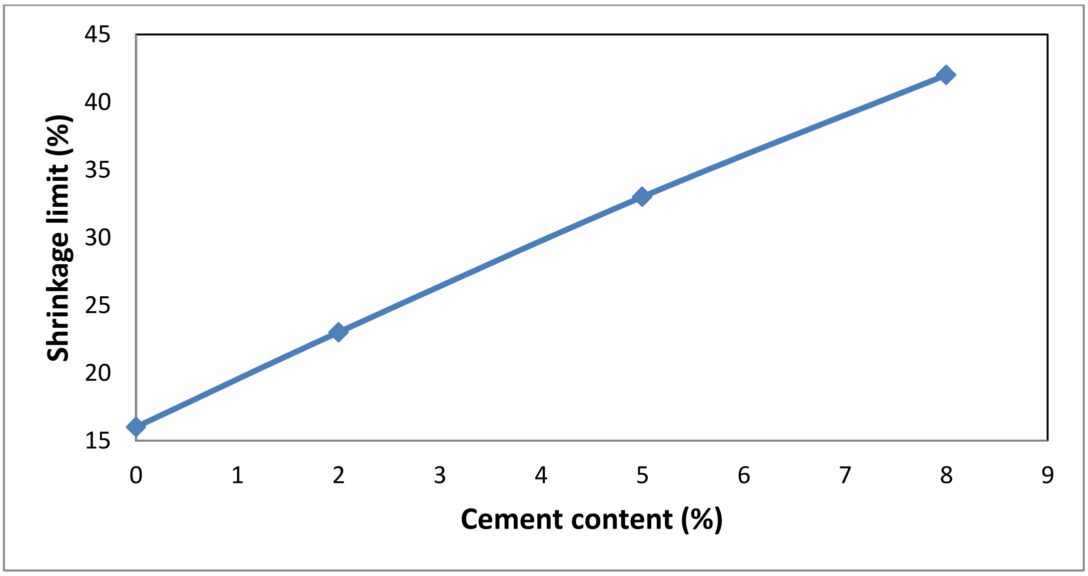

Figure 4.3: Variation of shrinkage limits with different cement contents

It can be seen from Table 4.3 that for the selected regional clay, values of maximum dry densities decrease up to 5% cement content and then no change occur up to 8% cement content while optimum moisture contents increase up to 5% cement content and then no change occur up to 8% cement content. The changes in maximum dry densities and optimum moisture contents with respect to increases in cement contents for the selected medium expansive regional clay are shown in Figure 4.7 and 4.8 respectively.

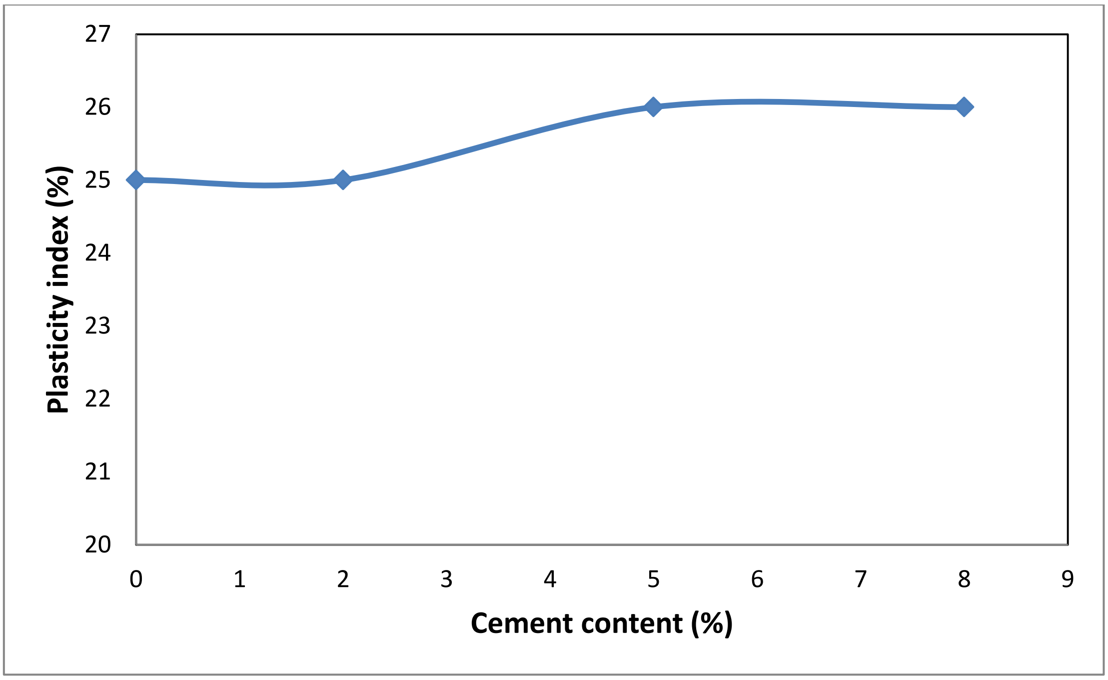

Figure 4.4: Variation of plasticity indices with different cement contents

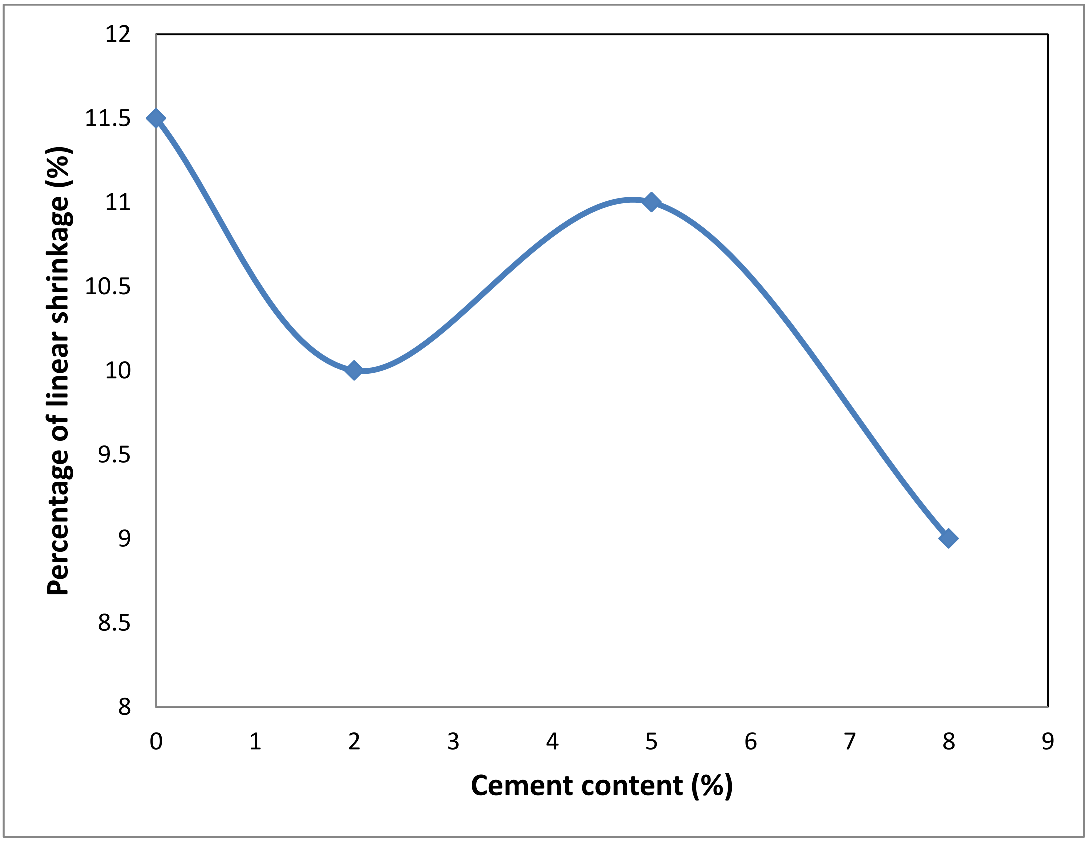

Figure 4.5: Variation of percentages of linear shrinkages with different cement contents

Ahmed (1984) found that for an inorganic clay with low plasticity, the maximum dry density reduced for increase in cement content up to 5% and then the maximum dry density increased with further increase in cement content. On the other hand, Rajbongshi (1997) and Hasan (2002) found increase of maximum dry density with increment of cement content while optimum moisture content decreases with increase of cement content.

4.3.3 Results of Unconfined Compression Strength Tests

Table 4.4 shows a summary of the unconfined compression test results for the selected clay-cement mixtures. In Table 4.4, the values of unconfined compressive strength, related axial strain at failure and related initial tangent modulus for the untreated samples and samples treated with different cement contents (2%, 5% and 8%) which were cured for 7, 14, 28 and 56 days are presented. The effects of cement contents on unconfined compression strengths, related failure strains and related initial tangent moduli are interpreted in sections 4.3.3.1 to 4.3.3.4.

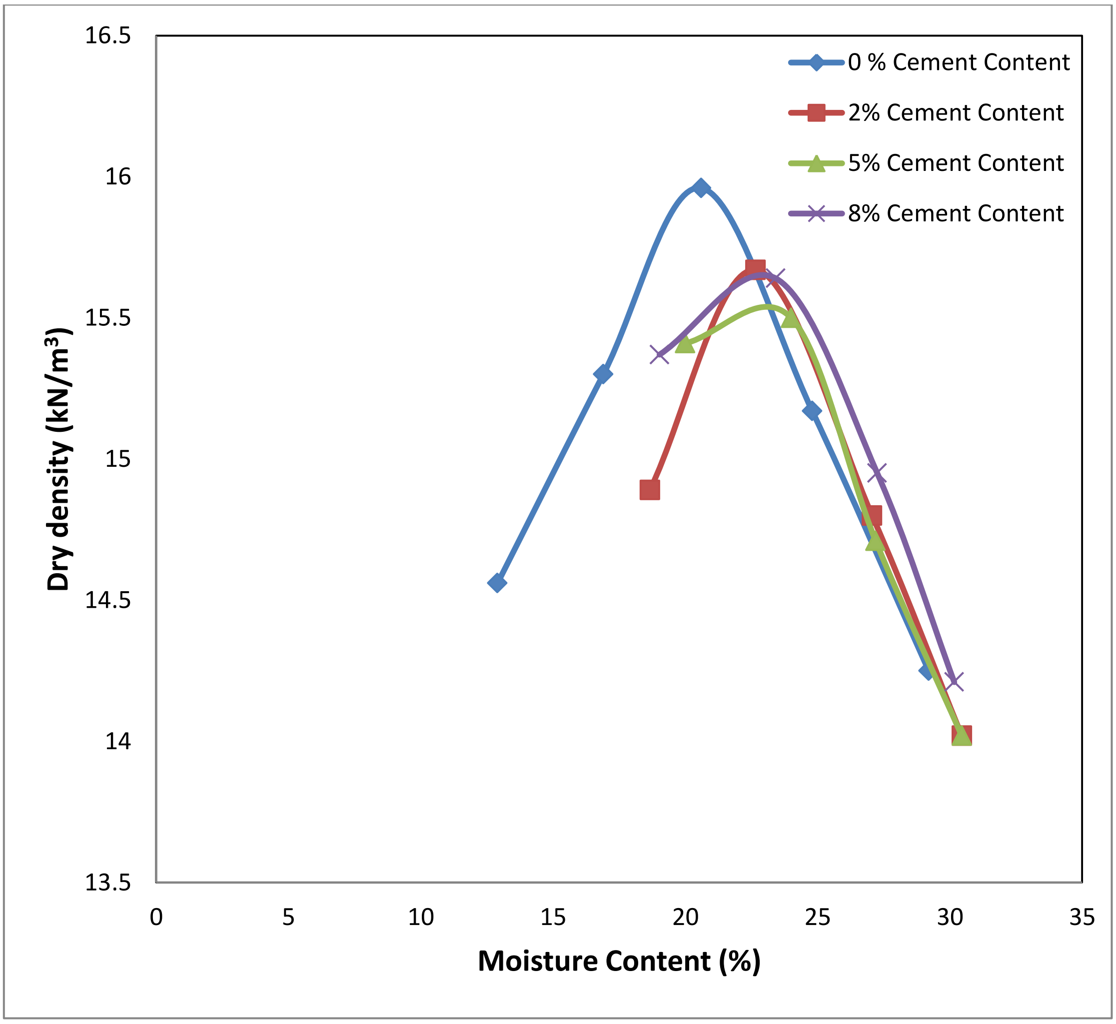

Figure 4.6: Variations of dry densities with different moisture contents in the selected soil-cement mixtures

Table 4.3: Variations of optimum moisture contents and maximum dry densities with respect to different cement contents in the soil-cement mixtures

Cement Content (%)	Optimum Moisture Content (%)	Maximum Dry Density (kN/m^3)
0	20.6	16
2	22.7	15.7
5	23	15.6
8	23	15.6

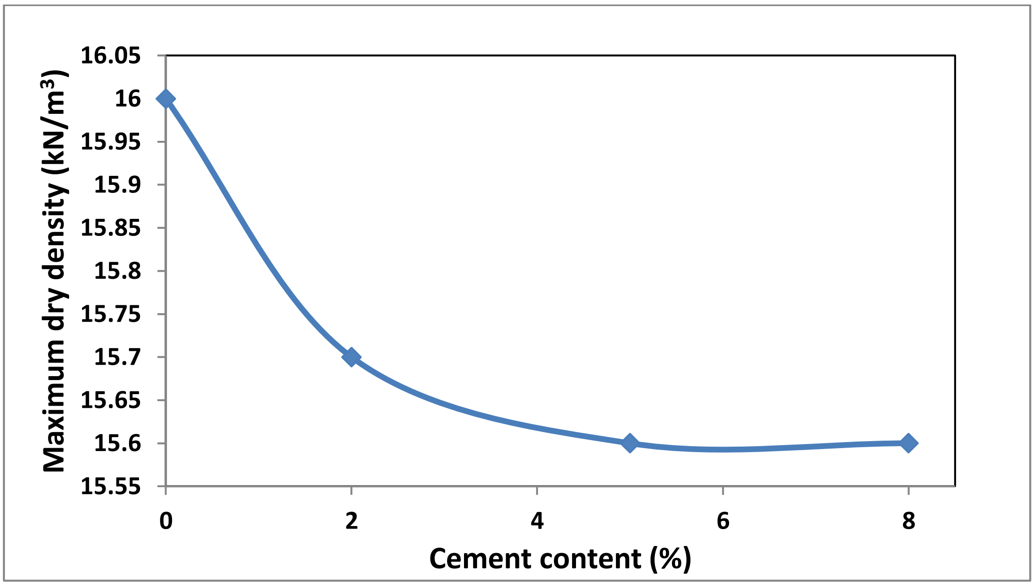

Figure 4.7: The variation of maximum dry densities with different binder contents in the soil-cement mixtures

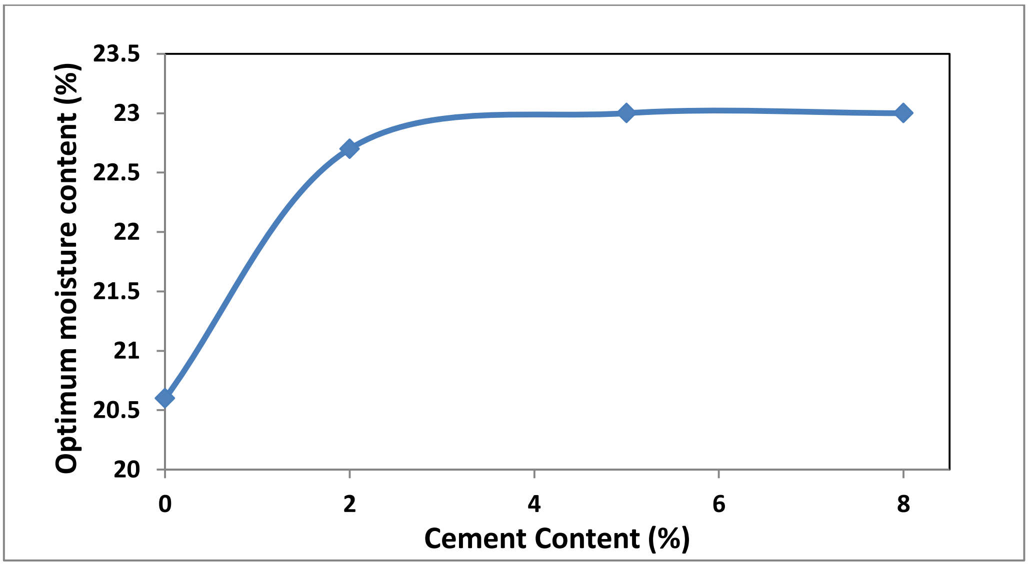

Figure 4.8: Variation of optimum moisture contents with different cement contents in soil-cement mixtures

4.3.3.1 Unconfined Compressive Strength

It can be seen from Table 4.4 that for the selected clay-cement mixtures compared with the untreated sample, the values of unconfined compressive strength of the treated samples increase significantly, depending on the cement contents and curing ages (exception for 2% cement content).

Table 4.4: Variations of unconfined compressive strengths, axial strains at failures and initial tangent moduli with respect to curing periods and percentages of cement

Curing Period	Percentage of Cement	Unconfined Compression Strength (kN/m^2)	Axial Strain at Failure (%)	Initial Tangent Modulus (kN/m^2)
0 days	0	785	2.8	61300
7 days	2	755	1.11	102200
	5	1145	0.59	275300
	8	2175	0.82	287300
14 days	2	805	0.64	117900
	5	1540	0.73	287300
	8	1540	1.11	153200
28 days	2	845	1.2	60200
	5	1305	0.9	175500
	8	2195	0.84	324900
56 days	2	530	0.59	162800
	5	1305	0.57	526700
	8	1430	0.94	175500

While comparing curing period of 56 days with curing period of 28 days, the unconfined compressive strengths decrease instead of increase for the cement content of 2%. For 7 days, 14 days and 28 days, the changes in unconfined compressive strengths with respect to preceding curing period (i.e. 14 days compared with 7days and 28 days compared with 14 days) are not significant for 2% cement content. For 5% cement content, the unconfined compressive strength of the selected clay-cement mixture increase after 14 days curing period while compared with respect to preceding curing period (i.e. comparing 14 days with 7 days). For 5% cement content after 28 days curing period the unconfined compressive strength decrease instead of increase in comparison to 14 days curing period. For the same cement content the unconfined compressive strength remains unchanged after curing for 56 days while comparing with respect to 28 days curing period. For 8% cement content, the unconfined compressive strength of the selected clay-cement mixture increases after 7 days curing while compared with respect to untreated and uncured clay. For the same cement content, after 14 days curing period the unconfined compressive strength decrease instead of increase in comparison to 7 days curing period. For the same cement content, after curing for 28 days the unconfined compressive strength increase in comparison to 14 days curing period. Again for the same cement content, after curing for 56 days the unconfined compressive strength decrease in comparison to 28 days curing period.

For 7 days curing period unconfined compressive strengths vary from 755 kN/m^2 for 2% cement content via 1145 kN/m^2 for 5% cement content to 2175 kN/m^2 for 8% cement content. For 14 days curing period unconfined compressive strengths vary from 805 kN/m^2 for 2% cement content via 1540 kN/m^2 for 5% cement content to 1540 kN/m^2 for 8% cement content. For 28 days curing period unconfined compressive strengths vary from 845 kN/m^2 for 2% cement content via 1305 kN/m^2 for 5% cement content to 2195 kN/m^2 for 8% cement content. For 56 days curing period unconfined compressive strengths vary from 530 kN/m^2 for 2% cement content via 1305 kN/m^2 for 5% cement content to 1430 kN/m^2 for 8% cement content. The variations of unconfined compressive strengths with respect to various cement contents for different curing periods is shown in Figure 4.9.

Maximum increase of unconfined compressive strength was obtained for 8% cement content after curing for 28 days but this value decrease after 56 days curing. After curing period of 56 days the maximum value of unconfined compressive strength is obtained for 8% cement content in comparison to other cement contents. Hence, further study is required to obtain the effects of more cement contents and curing periods.

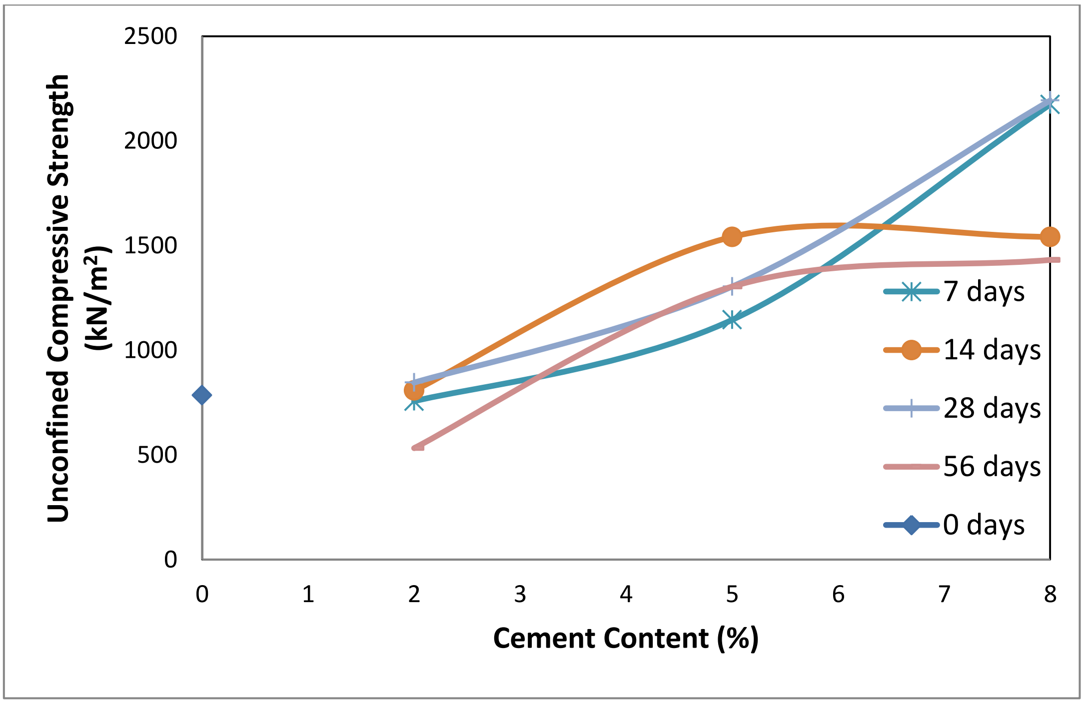

Figure 4.9: Variations of unconfined compressive strengths with respect to different cement contents for different curing periods

On the contrary of the results of our present study; Ahmed (1984), Rajbongshi (1997) and Hasan (2002) reported increments of unconfined compressive strengths with respect to preceding cement contents for different curing periods or with respect to preceding curing periods for different cement contents. The author of the present project report thinks that the reason of decreasing unconfined compressive strengths of the selected clay-cement mixtures instead of increasing is breaking of $C_3S_2H_x$ into CSH and $Ca(OH)_2$ at low pH values (pH<12). The cementation strength of $C_3S_2H_x$ is much stronger than CSH. So, the strength got reduction.

4.3.3.2 Comparison among Unconfined Compressive Strengths of Present Study and Other Studies

Figure 4.10, Figure 4.11 and Figure 4.12 show the relationships among different unconfined compressive strengths and different cement contents for different types of soil and curing periods. The said relationships for 7, 14 and 28 days curing periods are shown in Figure 4.10, Figure 4.11 and Figure 4.12 respectively. Table 4.5 shows liquid limits, plastic limits, soil symbols and percentages of particles passing through 200 No. sieve for different types of soil mentioned in the said three figures.

Table 4.5: Values of percentage of soil passing through No. 200 sieve, liquid limits and plastic limits with corresponding researchers and soil symbols mentioned in Figure 4.10, Figure 4.11 and Figure 4.12

Researcher	**Soil Symbol**	**Percentage of Soil Passing through No. 200 Sieve**	**Liquid Limit**	**Plastic Limit**
Present investigation	CL	-	42	17
Ahmed (1984)	ML1	63	-	-
Ahmed (1984)	ML2	96	40	30
Ahmed (1984)	CL	82	43	22
Hasan (2002)	ML	92	41	29
Hasan (2002)	CH	98	52	23
Hossain (1986)	ML3	95	-	-
Hossain (1986)	ML4	98	33	27.5
Rajbongshi (1997)	ML	68	30	23
Rajbongshi (1997)	CL	94	44	25

Figure 4.10 shows that for 7 days curing period, the unconfined compressive strengths of different soil types increased with increment of cement content as like present investigation except the four cases of Ahmed (1984). However, the rates and magnitudes of increments are not same for all soils.

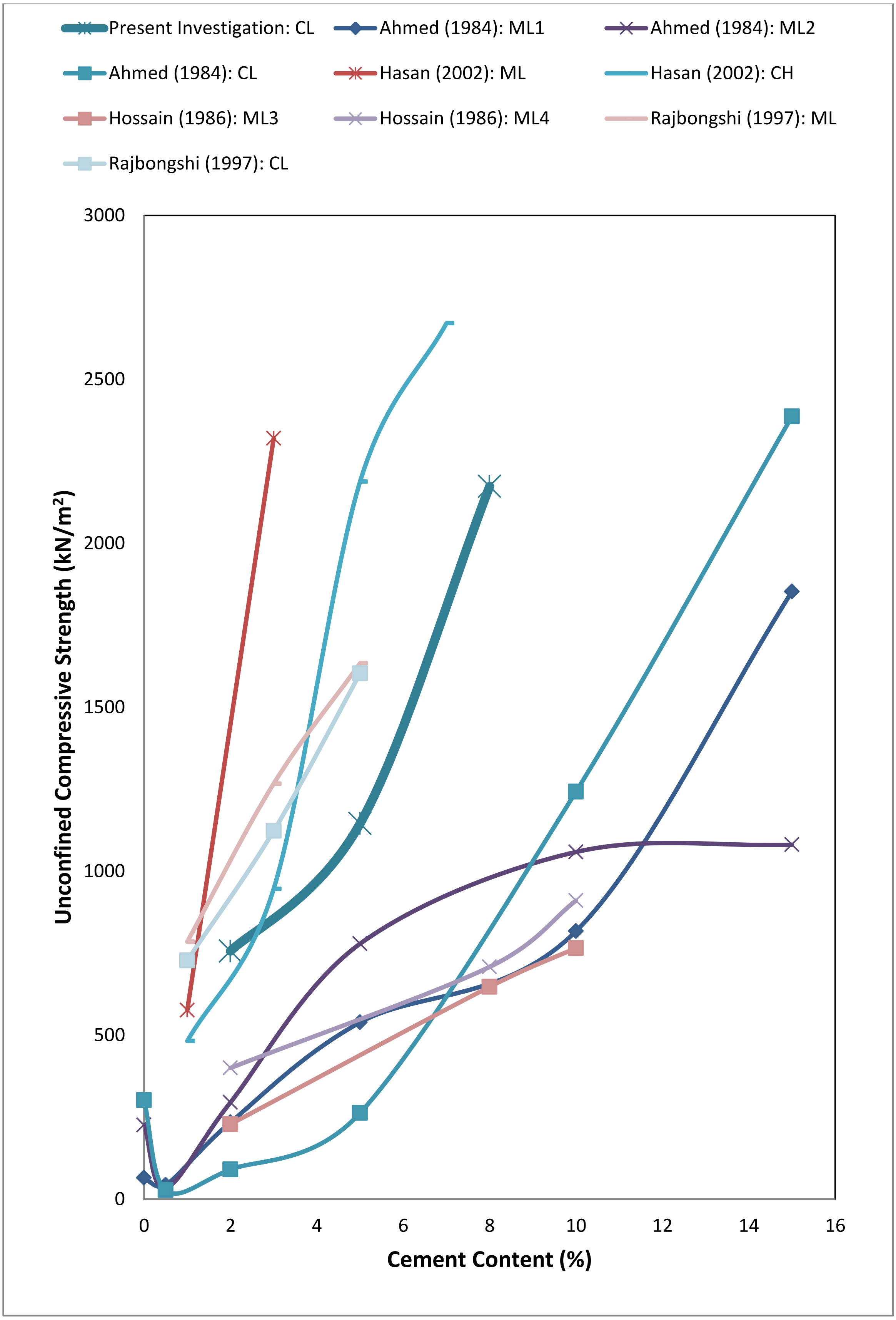

Figure 4.10: Variations of unconfined compressive strengths with respect to cement contents for different types of soil and 7 days curing period

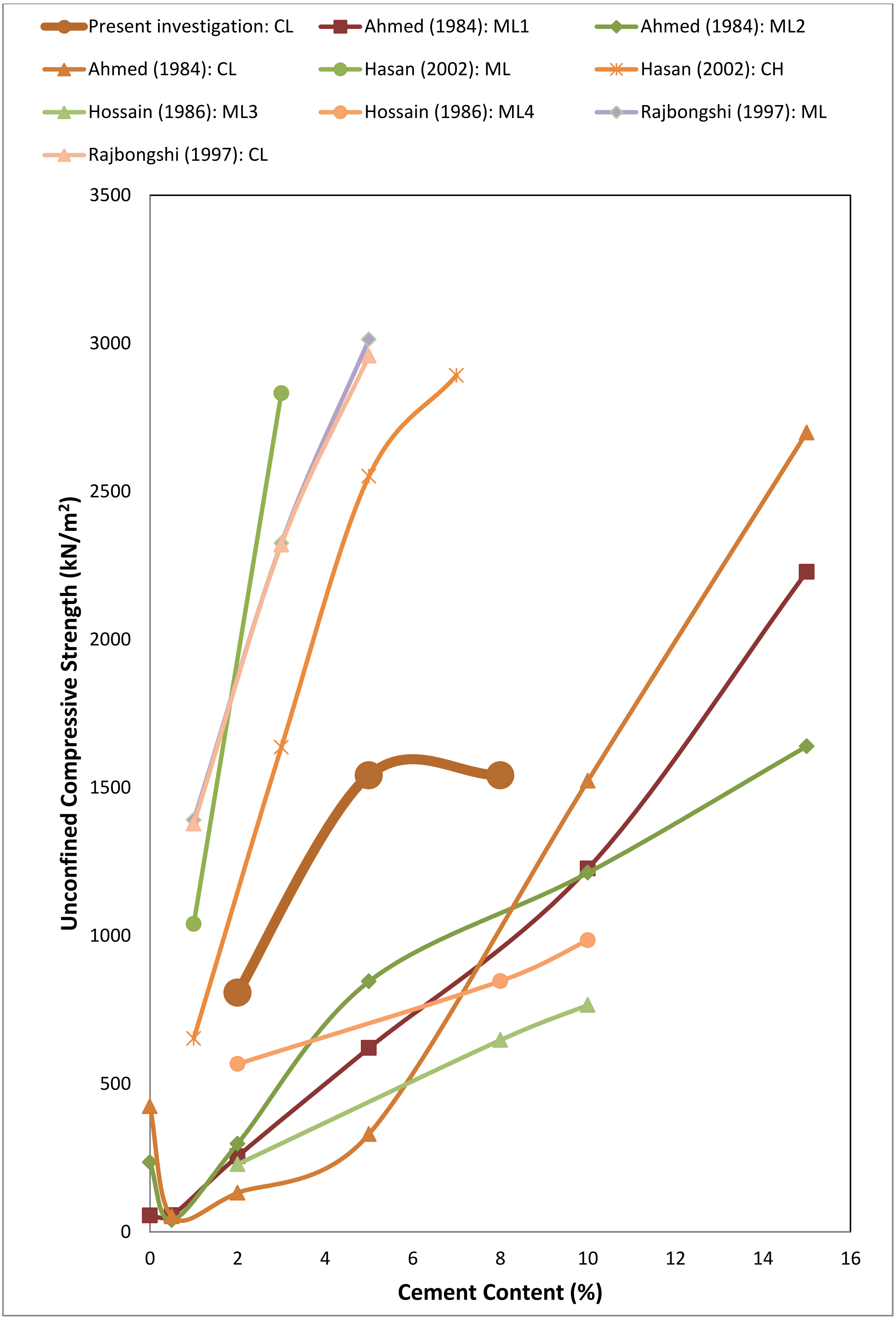

Figure 4.11: Variations of unconfined compressive strengths with respect to cement contents for different types of soil and 14 days curing period

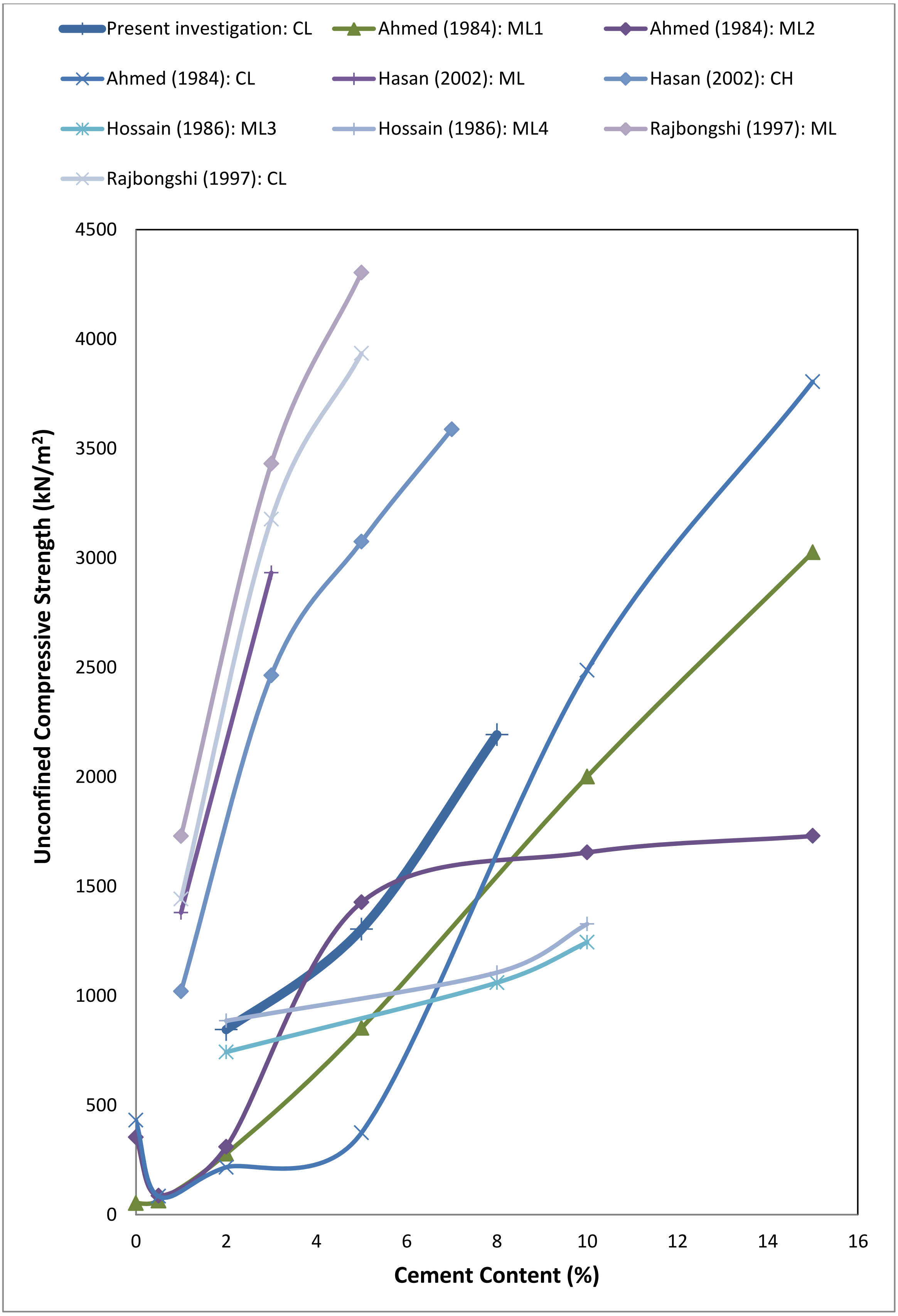

Figure 4.12: Variations of unconfined compressive strengths with respect to cement contents for different types of soil and 28 days curing period

The phenomenon of decreasing of the unconfined compressive strengths for 0.5% cement contents instead of increasing for the three soil types of Ahmed (1984) for 7 days curing period is shown in Figure 4.10. The investigation on the effects of 0.5% cement contents in cement-soil mixtures is beyond the scope of present investigation. The fourth case of exception is insignificant increase of unconfined compressive strength of ML2 of Ahmed (1984) for 7 days curing period. In the said fourth case, insignificant change of unconfined compressive strength is observed for changing cement content from 10% to 15% for 7 days curing period.

The investigation on the effects of changing cement content from 10% to 15% is also beyond the scope of present investigation but the said trend of insignificant change is observed in the present investigation for changing cement content from 5% to 8% for 14 days curing period (Figure 4.11).

In the present research, for changing cement content from 2% to 5% for 14 days curing period, the unconfined compressive strength increase with increment of cement content showing almost similar trends of findings of different researchers although the rates and magnitudes of increments are not same. For 14 days curing period, the case of insignificant change of unconfined compressive strength for changing cement content from 5% to 8% is not observed in the findings of mentioned researchers in Figure 4.11.

Figure 4.12 shows that for 28 days curing period, the unconfined compressive strengths increase with increasing cement content for different soil types as like present investigation except the two cases of Ahmed (1984) although the rates and magnitudes of increments are different for different types of soils. The phenomenon of decreasing of the unconfined compressive strengths for 0.5% cement contents instead of increasing for ML2 and CL soil types of Ahmed's (98) investigation for 8 days curing period is shown in Figure 4.12. However, the investigation on the effects of 0.5% cement contents in cement-soil mixtures is beyond the scope of present investigation.

4.3.3.3 Failure Strains in Unconfined Compression Strength Tests

Table 4.4 shows the variations of failure strains in unconfined compressive strength tests with curing periods and cement contents. For 7 days curing period, the said failure strains vary from 1.11% for 2% cement content via 0.59% for 5% cement content to 0.82% for 8% cement content. For 14 days curing period, the said failure strains vary from 0.64% for 2% cement content via 0.73% for 5% cement content to 1.11% for 8% cement content. For 28 days curing period, the said failure strains vary from 1.2% for 2% cement content via 0.9% for 5% cement content to 0.84% for 8% cement content. For 56 days curing period, the said failure strains vary from 0.59% for 2% cement content via 0.57% for 5% cement content to 0.94% for 8% cement content. The variations of the said failure strains with respect to different cement contents are shown in the Figure 4.13.

For 7 days and 56 days curing periods the relationships between failure strains and cement contents show decrement and then increment. For 14 days curing period the relationship between failure strains and cement contents show increment only while for 28 days curing period the relationship between failure strains and cement contents show decrement only. However, Rajbongshi (1997) found decreases in failure strains due to increases in cement contents. The highest decrease of failure strain was obtained for 5% cement content after curing period of 56 days. The lowest decrease of failure strain was obtained for 2% cement content after curing period of 28 days. Since, change in result is continuous up to 8% cement content and 56 days curing period so, further study should be done to investigate the effects of more curing periods and cement contents.

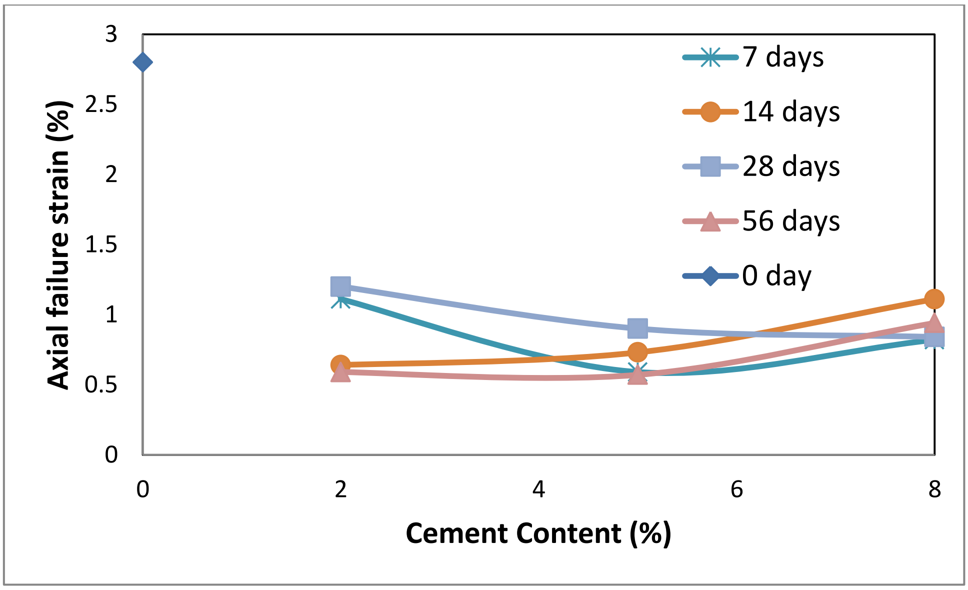

Figure 4.13: Variations of axial failure strains with respect to different cement contents for different curing periods

4.3.3.4 Initial Tangent Moduli from Unconfined Compression Strength Tests

Table 4.4 shows the variation of initial tangent moduli in unconfined compressive strength tests with respect to different cement contents. For 7 days curing period, the said initial tangent moduli vary from 102200 kN/m^2 for 2% cement content via 275300 kN/m^2 for 5% cement content to 287300 kN/m^2 for 8% cement content. For 14 days curing period, the said initial tangent moduli vary from 117900 kN/m^2 for 2% cement content via 287300 kN/m^2 for 5% cement content to 153200 for 8% cement content.

For 28 days curing period, the said initial tangent moduli vary from 60200 kN/m^2 for 2% cement content via 175500 kN/m^2 for 5% cement content to 324900 kN/m^2 for 8% cement content. For 56 days curing period, the said initial tangent moduli vary from 162800 kN/m^2 for 2% cement content via 526700 kN/m^2 for 5% cement content to 175500 kN/m^2 for 8% cement content. The variations of the said initial tangent moduli with respect to different cement contents are shown in the Figure 4.14.

The highest value of initial tangent modulus in the scheme of the current research work was obtained for 5% cement content after curing period of 56 days. The lowest value of initial tangent modulus was obtained for 2% cement content after curing period of 28 days. Since, change in result is continuous up to 8% cement content and 56 days curing period so, further study should be done to investigate the effects of more curing periods and cement contents.

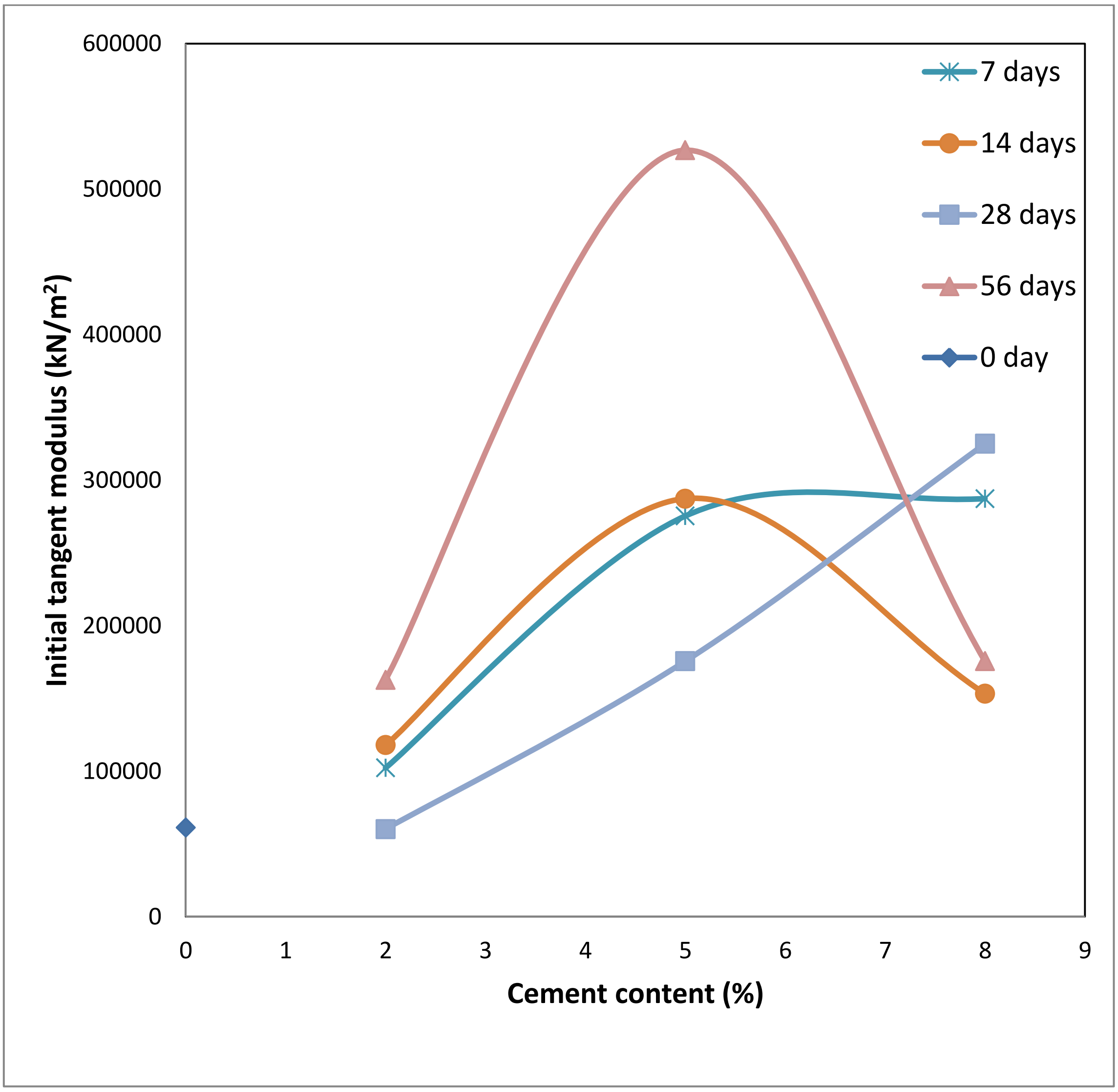

Figure 4.14: Variation of initial tangent modulus determined with unconfined compression tests with respect to different cement contents for different curing periods

4.3.4 Consolidated Undrained Cohesions, Consolidated Undrained Angles of Internal Frictions and Changes in Heights with Corresponding Shear Displacements Obtained from Direct Shear Tests

Table 4.6 shows the values of consolidated undrained cohesions and consolidated undrained angles of internal frictions for the untreated samples and samples treated with 5% cement which were cured for 7, 14, 28 and 56 days respectively. The effects of curing periods on consolidated undrained cohesions and consolidated undrained angles of internal frictions and relationships among shear displacements and corresponding changes in heights of specimens etc. are interpreted in sections 4.3.4.1 to 4.3.4.3 respectively.

4.3.4.1 Effect of Curing Periods on Consolidated Undrained Cohesions

Figure 4.15 shows the variation of the consolidated undrained cohesions (which were obtained from consolidated undrained direct shear tests) with respect to different curing periods for 5% cement content in the samples. This figure illustrates that the consolidated undrained cohesion values vary from 61.3 kN/m^2 for 7 days curing period via 130.2 kN/m^2 for 14 days curing period and 139.8 kN/m^2 for 28 days curing period to 183.9 kN/m^2 for 56 days curing period. It is observed that the values of consolidated undrained cohesion increase with the increase of curing period. For 5% cement content, the said cohesion values increase rapidly up to 14 days curing period, then slowly up to 28 days and then again rapidly up to 56 days. The value of consolidated undrained cohesion obtained from the said test is much lower for untreated selected clay than the selected clay-cement mixtures.

Table 4.6: Effects of curing periods on consolidated undrained cohesions and consolidated undrained angles of internal frictions while the cement content in the samples is 5%

Cement Content	**Curing Period (Days)**	**Consolidated Undrained Cohesion (kN/m^2)**	**Consolidated Undrained Angle of Internal Friction (Degree)**
5%	7	61.3	23
	14	130.2	14
	28	139.8	13
	56	183.9	6
0%	0	6.1	24

4.3.4.2 Effect of Curing Periods on Consolidated Undrained Angles of Internal Frictions

Figure 4.16 shows the variation of the consolidated undrained angles of internal frictions (which were obtained from consolidated undrained direct shear tests) with respect to different curing periods for 5% cement content in the samples. This figure illustrates that the angles of internal frictions values vary from 23^0 for 7 days curing period via 14^0 for 14 days curing period and 13^0 for 28 days curing period to 6^0 for 56 days curing period. It is observed that the values of angles of internal frictions decrease with the increase of curing periods. For 5% cement content in the samples, the consolidated undrained angles of internal frictions values decrease rapidly up to 14 days curing period, then slowly up to 28 days and then again rapidly up to 56 days. The value of consolidated undrained angle of internal friction obtained from the said test is higher for untreated selected clay than the selected clay-cement mixtures. By matching Figure 4.15 and Figure 4.16 it is observed that the values of consolidated undrained angles of internal frictions decrease when values of consolidated undrained cohesions increase and vice-versa.

4.3.4.3 Relationships among Shear Displacements and Corresponding Changes in Heights of Specimens

Figure 4.17 to Figure 4.20 show the relationships between shear displacements and corresponding changes in heights of specimens for 7 days, 14 days, 28 days and 56 days curing periods respectively, for different normal loads and for 5% cement content in the samples. The said relationships do not show a typical trend. Figure A.6 to Figure A.9 in Appendix-A show that the cement treated clay samples show nature of over consolidated clay for the applied normal loads but all the curves in Figure 4.17 to Figure 4.20 do not show nature of over consolidated clay. From both Figure A.1 and Figure C.1 it can be seen that the untreated selected clay shows nature of normally consolidated clay.

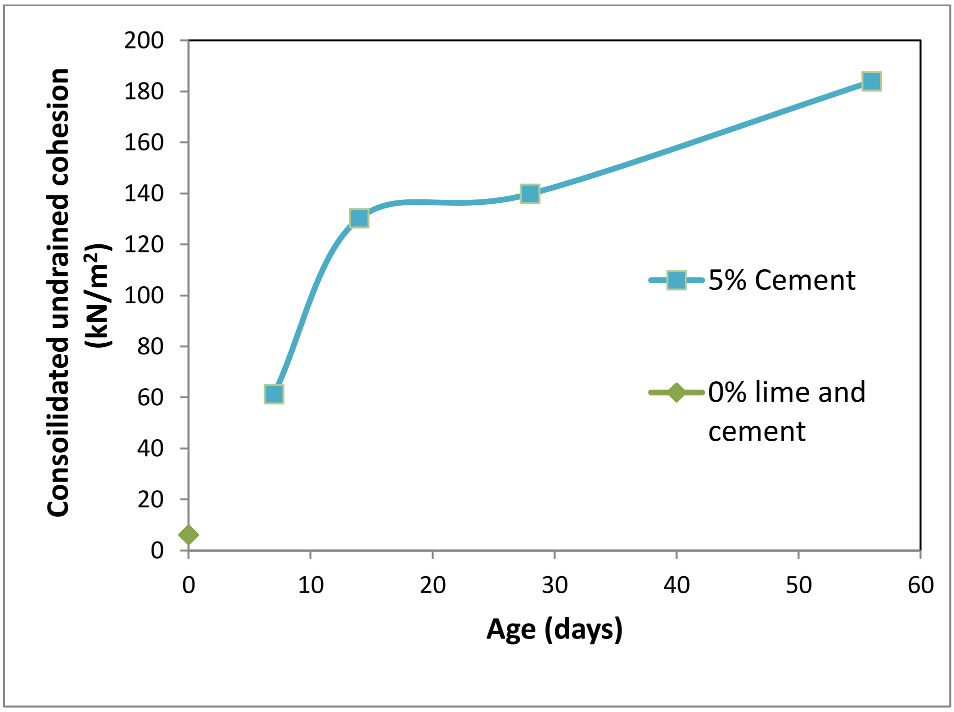

Figure 4.15: Effect of curing ages on consolidated undrained cohesions of cement treated samples

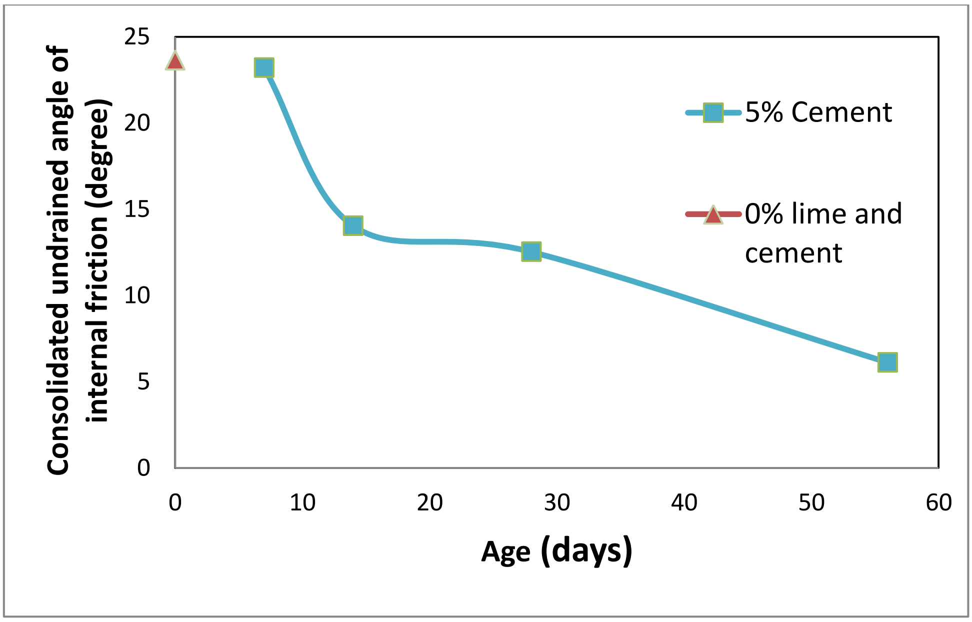

Figure 4.16: Variation of consolidated undrained angles of internal frictions of cement treated samples with respect to different curing periods

4.4 Physical and Engineering Properties of Lime-Treated Soils

In the following sections i.e. sections 4.4.1 to 4.4.4.3, physical and engineering characteristics comprising plasticity and shrinkage properties, moisture-density relations, unconfined compressive strengths, axial failure strains, initial tangent moduli, consolidated undrained cohesions, consolidated undrained angles of internal frictions and changes in heights of samples with shear displacements in direct shear tests of untreated and lime-treated samples of the selected medium expansive regional soil are presented and discussed.

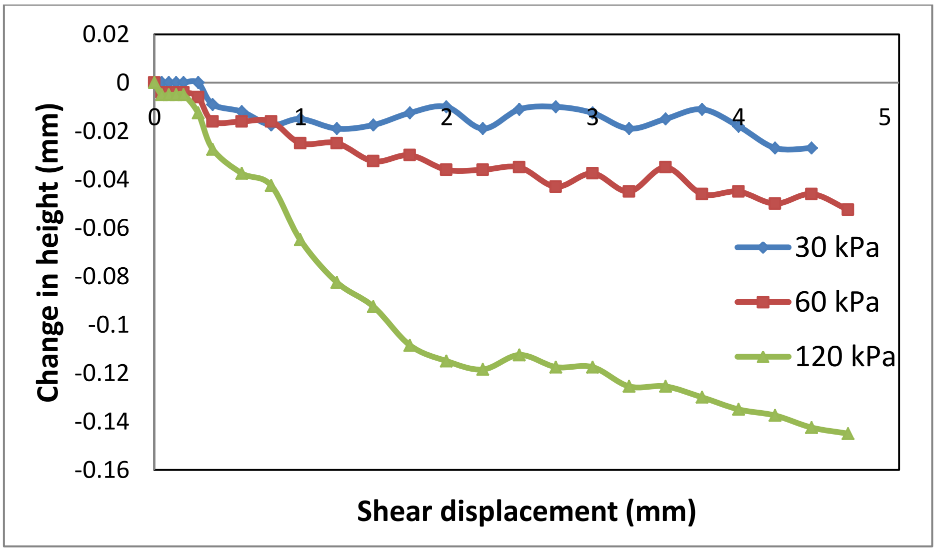

Figure 4.17: Relationships between shear displacements and corresponding changes in heights of the specimens for different normal stresses, for 5% cement content and for 7 days curing period

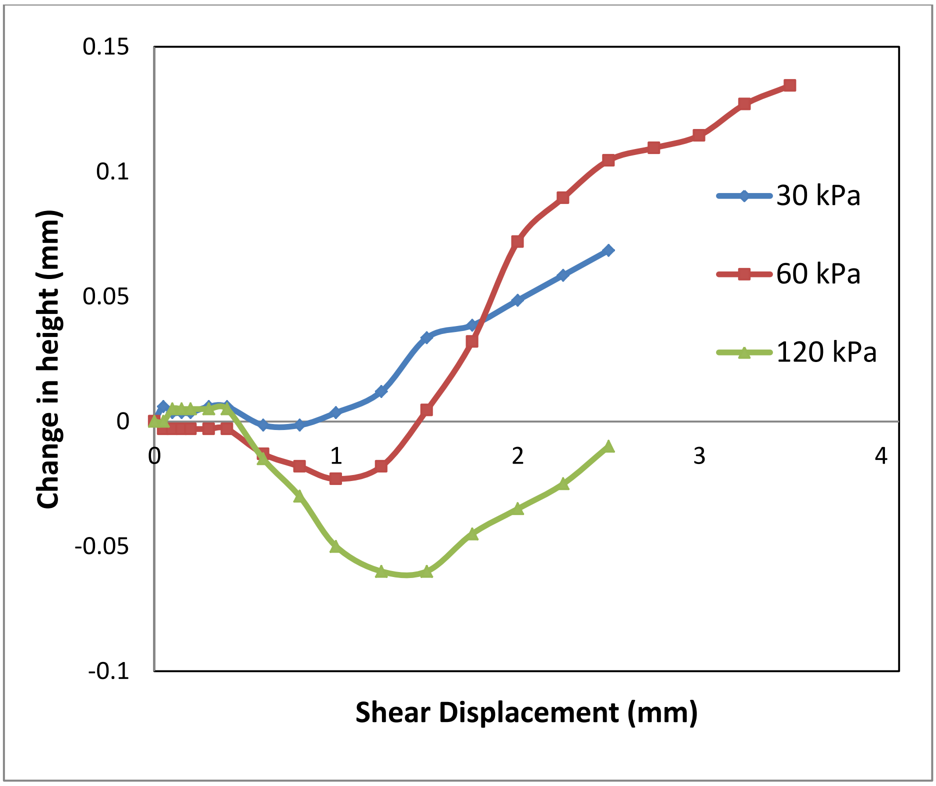

Figure 4.18: Relationships between shear displacements and corresponding changes in heights of the specimens for different normal stresses, for 5% cement content and for 14 days curing period

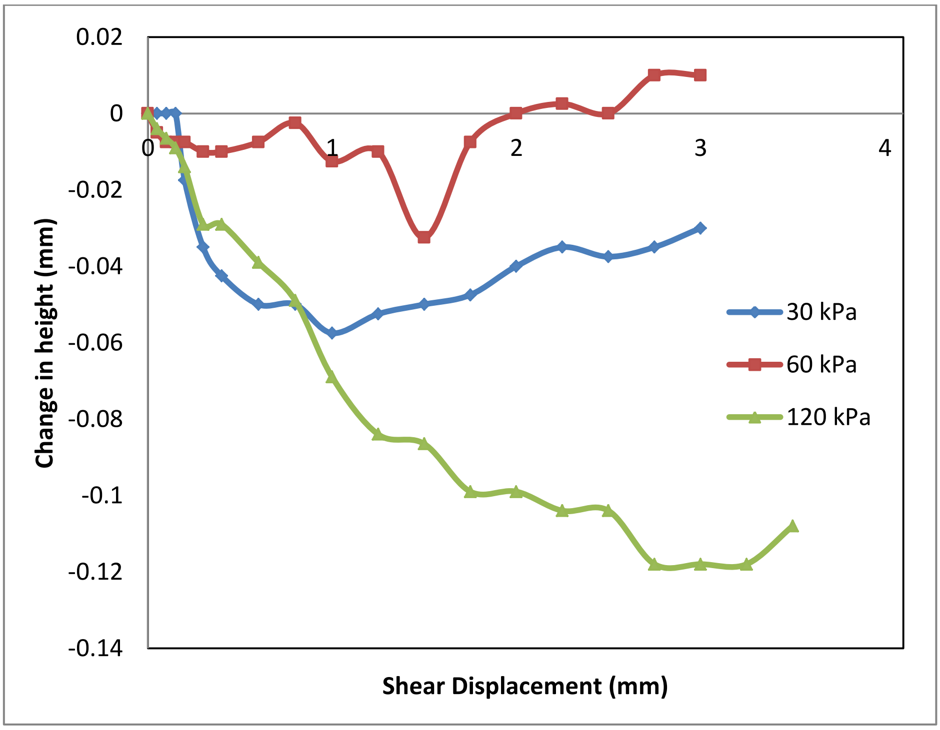

Figure 4.19: Relationships between shear displacements and corresponding changes in heights of the specimens for different normal stresses, for 5% cement content and for 28 days curing period

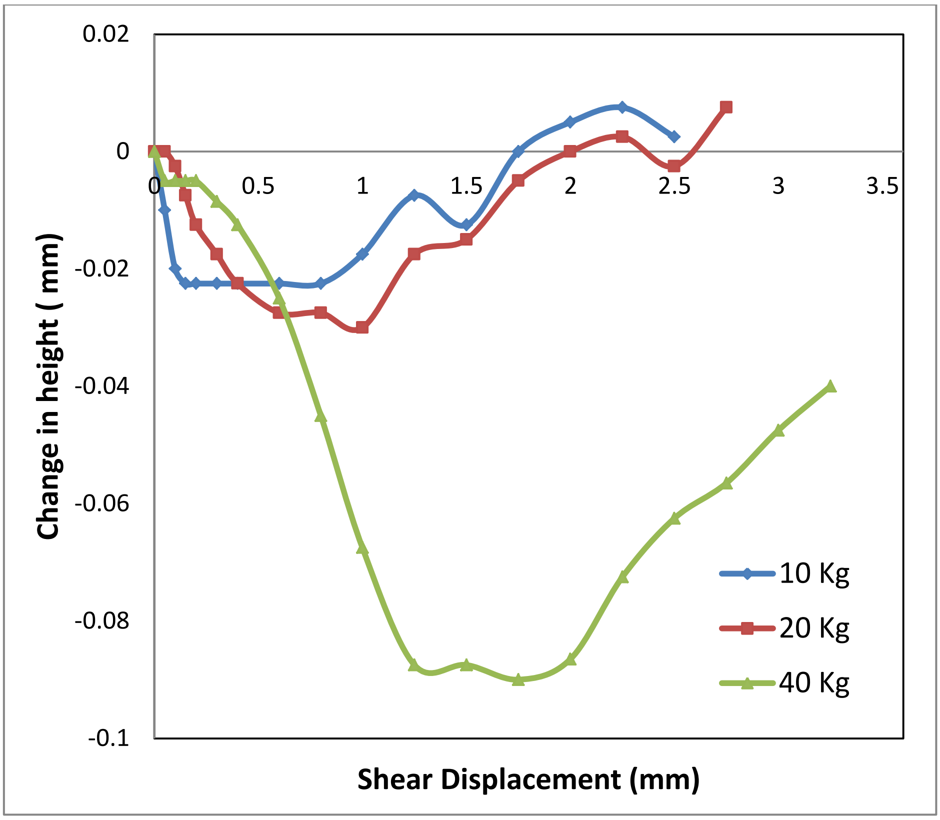

Figure 4.20: Relationships between shear displacements and corresponding changes in heights of the specimens for different normal stresses, for 5% cement content and for 56 days curing period

4.4.1 Plasticity and Shrinkage Characteristics

The values of plasticity and shrinkage properties of the untreated and lime-treated soil samples are shown in Table 4.7.

Table 4.7: Variations of plastic limits, liquid limits, shrinkage limits, plasticity indices, percentages of linear shrinkages with respect to different lime contents in the lime stabilized soil samples

Lime Content (%)	Liquid Limit	Plastic Limit	Shrinkage Limit	Plasticity Index	Percentage of Linear Shrinkage
0	42	17	16	25	11.5
2	40	16	13	24	11
5	41	16	21	25	12
8	45	18	35	27	12

It can be seen from Table 4.7 that comparing lime treated samples with the untreated samples; plastic limits, liquid limits, plasticity indices and percentages of linear shrinkages of the soil-lime mixtures do not change significantly while shrinkage limits of the soil-lime mixtures increase significantly.

The relationships of plastic limits, liquid limits, shrinkage limits, plasticity indices and percentages of linear shrinkages with different lime contents are shown in Figure 4.21, Figure 4.22, Figure 4.23, Figure 4.24 and Figure 4.25 respectively.

Figure 4.21 shows that plastic limit decrease to 16 from 17 when lime content is increased to 2% from 0%. The magnitude of plastic limits is 16 for both 2% and 5% lime contents. The plastic limit increases to 18 from 16 when lime content is increased to 8% from 5%. The obtained trend of plastic limits of selected soil-lime mixtures in the current study is not obtained in any research works studied by the author of the current project report. However, the said trend of the plastic limits will be changed if the recommended method by ASTM D4318-10 of calculating plastic limit by taking average value of the corresponding two individual plastic limits is disregarded. If a single value up to single digit after decimal is considered from each two values of a single plastic limit for the case of each lime contents then the values of plastic limits become 16.1, 16.3, 16.7 and 18.8 for 0%, 2%, 5% and 8% lime contents respectively. This type of trend of increasing magnitudes of plastic limits with increases of lime contents is mentioned in the works of Ahmed (1984) and Rajbongshi (1997).

Figure 4.22 shows that the liquid limit decreases to 40 from 42 when lime content increases to 2% from 0%. The liquid limit increases to 41 from 40 when the lime content increases to 5% from 2%. The liquid limit increases to 45 from 41 when lime content increases to 8% from 5%. This type of trend of changing liquid limits with respect to lime contents in lime-soil mixtures (soil type is silt with low plasticity) can also be shown in the works of Ahmed (1984).

Figure 4.23 shows that the shrinkage limit decreases from 16 to 13 when lime content changes from 0% to 2%. Then the shrinkage limits increase almost linearly with increases of lime contents up to 8%. The said trend of shrinkage limits of the selected soil-lime mixtures is not obtained in research works studied by the author of the current project report.

Figure 4.24 shows that the plasticity index of selected soil-lime mixture decreases to 24 from 25 when lime content is changed to 2% from 0%. The plasticity index increases to 25 from 24 when lime content is changed to 5% from 2%. The plasticity index increases to 27 from 25 when lime content is changed to 8% from 5%. This type of trend of changing plasticity indices with respect to different lime contents in lime-soil mixtures is also found in the study of Ahmed (1984).

Figure 4.25 shows that the percentages of linear shrinkages do not change significantly with increasing lime contents in the soil-lime mixtures up to 8% lime content. This type of result has not yet been observed in research works studied by the author of the current project report.

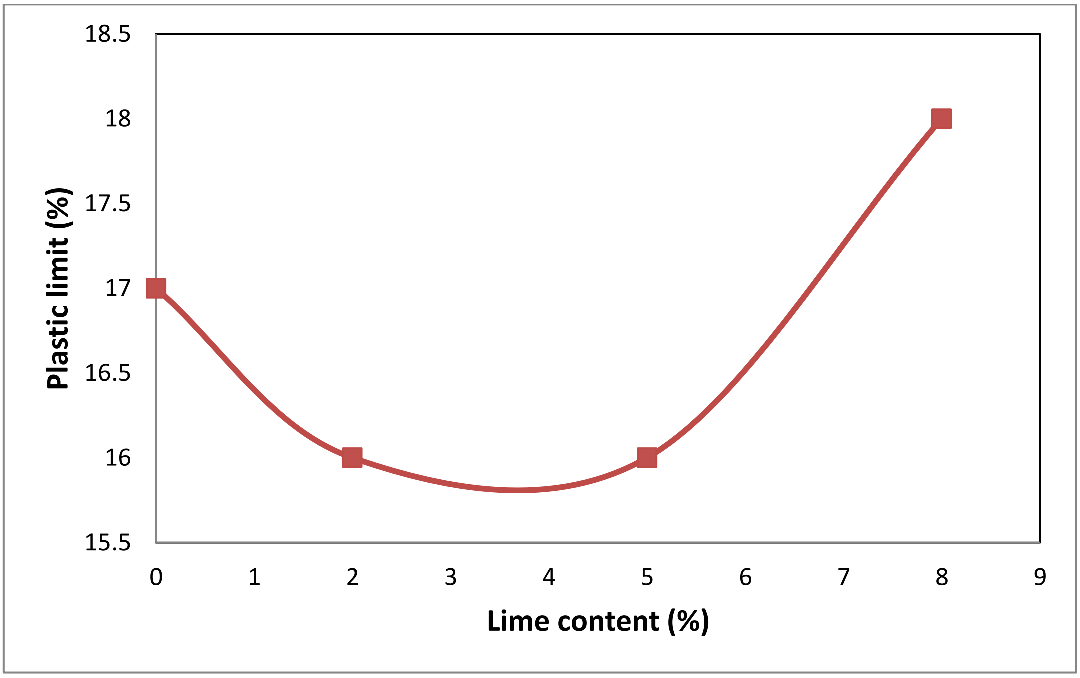

Figure 4.21: Variation of plastic limits with different lime contents

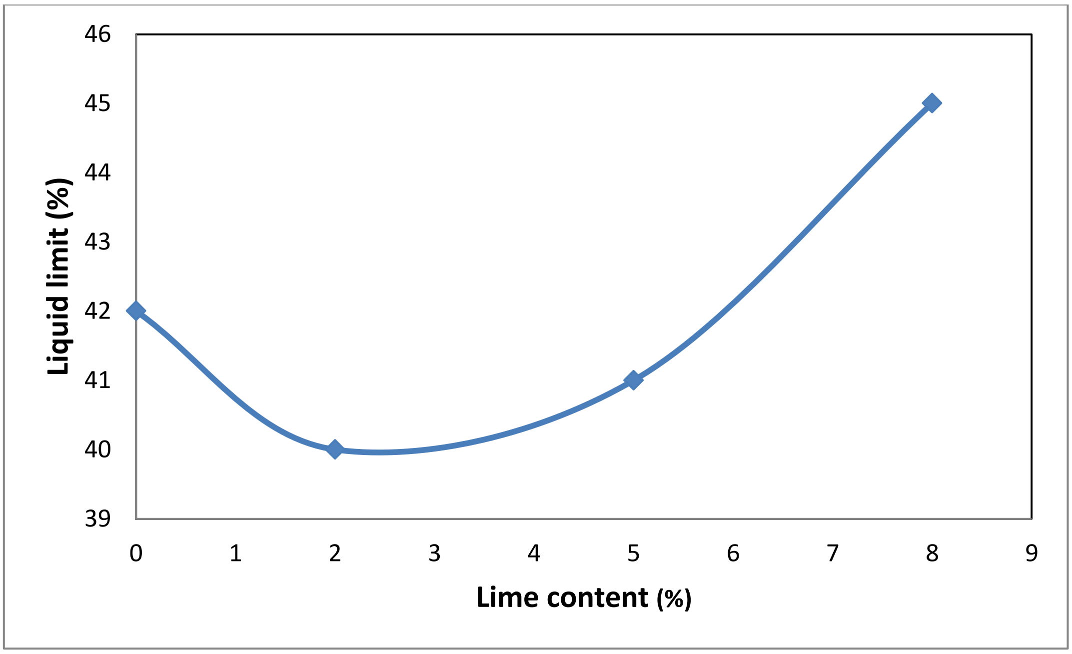

Figure 4.22: Variation of liquid limits with different lime contents

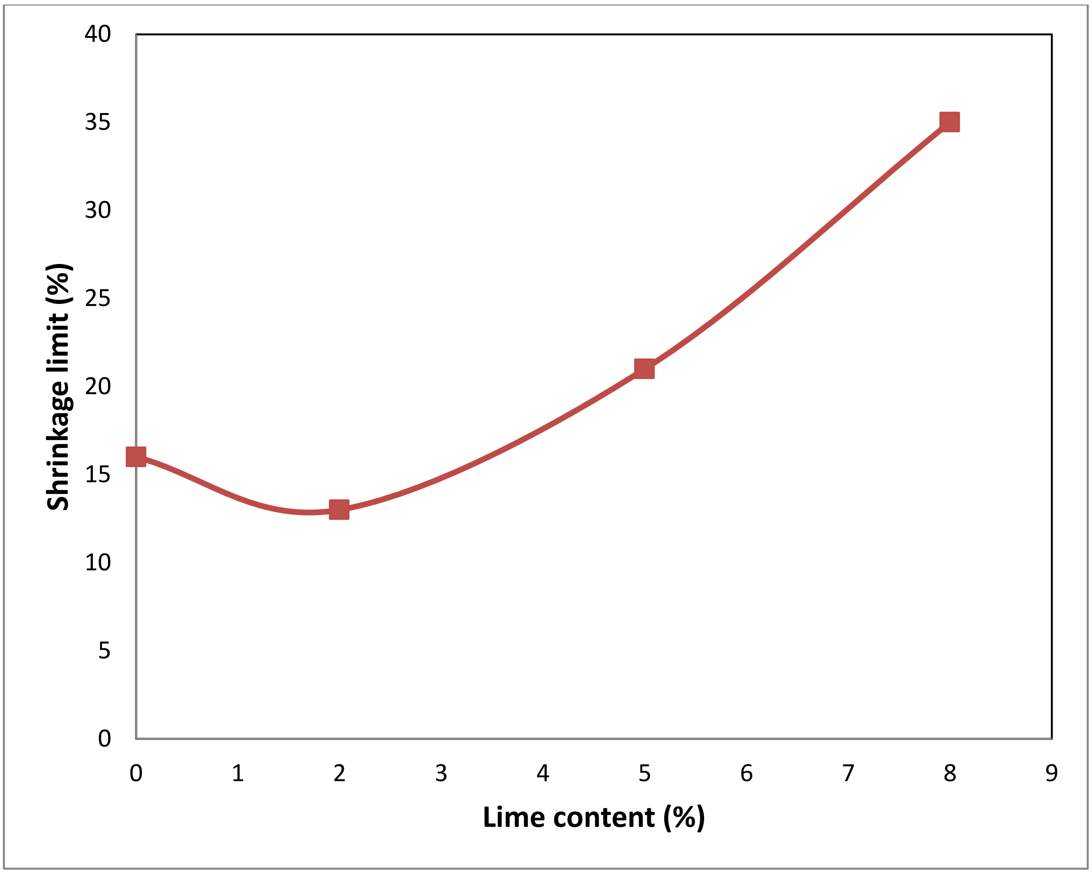

Figure 4.23: Variation of shrinkage limits with different lime contents

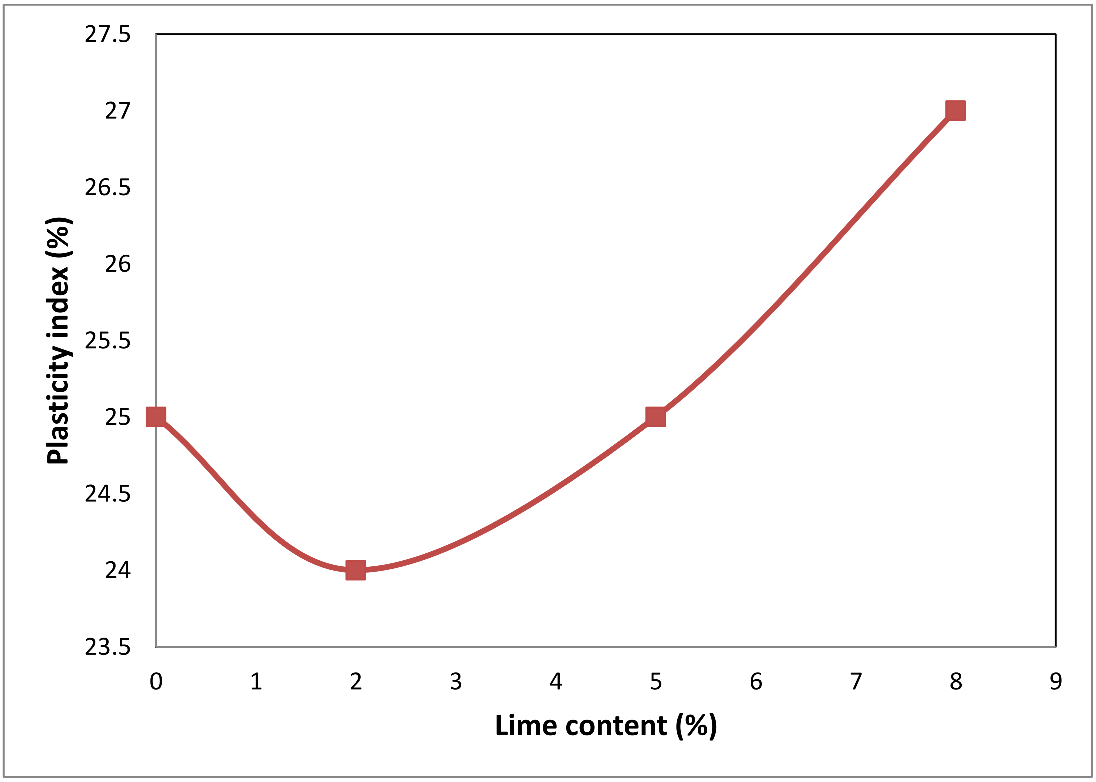

Figure 4.24: Variation of plasticity indices with different lime contents

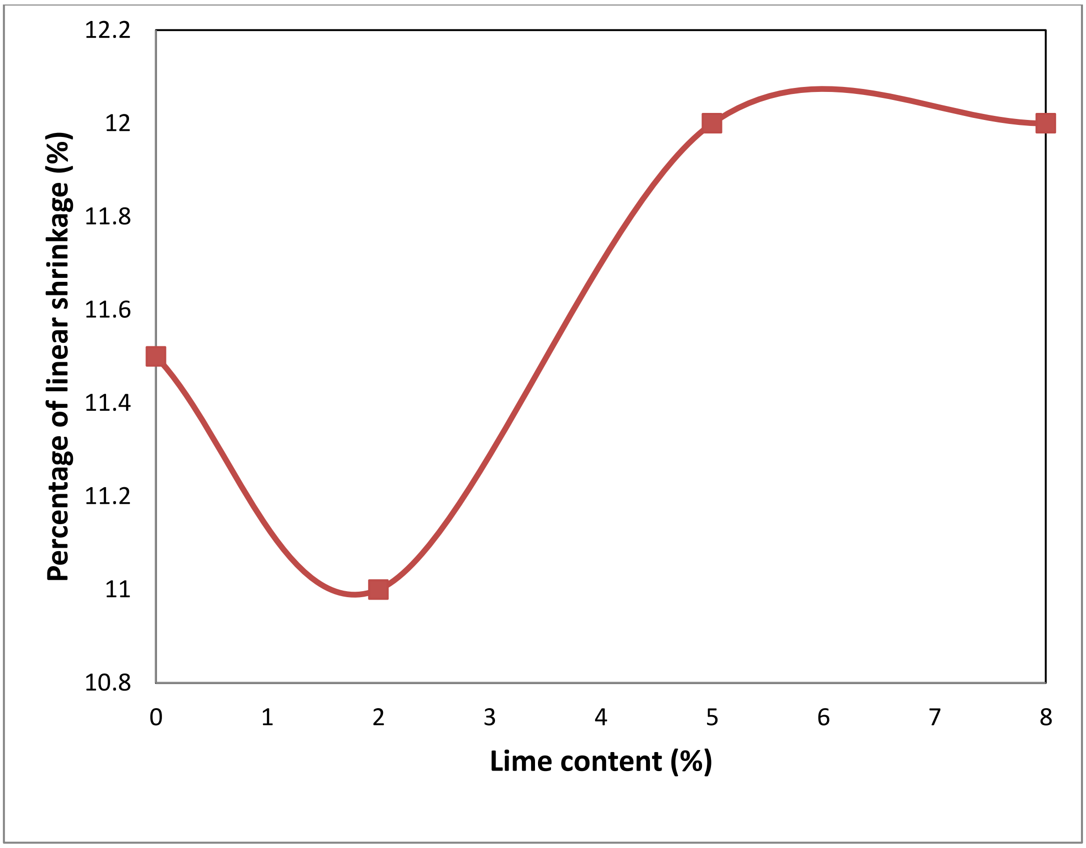

Figure 4.25: Variation of percentages of linear shrinkages with different lime contents

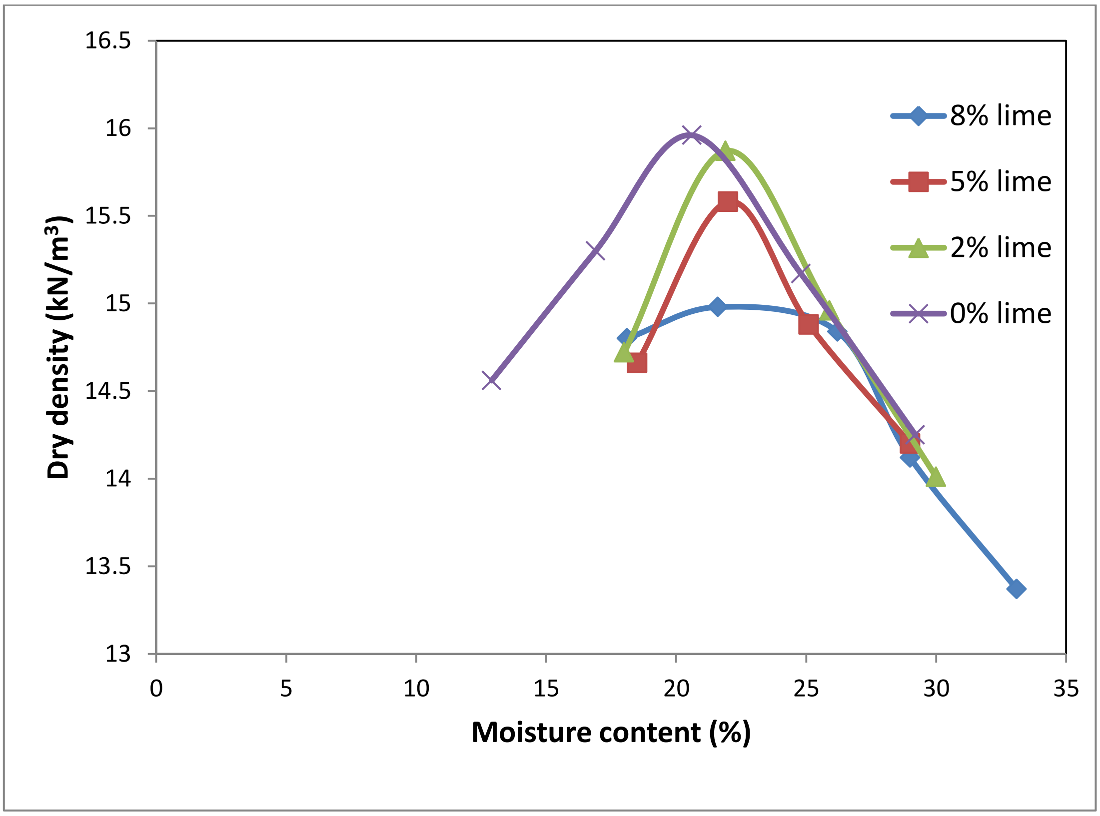

Figure 4.26: Variations of dry densities with different moisture contents in the selected soil-lime mixtures

4.4.2 Moisture-Density Relations

The moisture-density relations of untreated and lime-treated samples of the selected regional clay are shown in Figure 4.26. From the relations presented in Figure 4.26, the maximum dry densities and optimum moisture contents of the different soil-lime mixtures have been determined which are presented in Table 4.8.

Table 4.8: Variations of optimum moisture contents and maximum dry densities with respect to different lime contents in the soil-lime mixtures

Lime Content (%)	Optimum Moisture Content (%)	Maximum Dry Density (kN/m^3)
0	20.6	16
2	21.9	15.9
5	22	15.6
8	22.2	15

It can be seen from Table 4.8 that for the selected regional clay, values of maximum dry densities decrease with increment of lime content while optimum moisture contents increase with increment of lime content. The changes in maximum dry densities and optimum moisture contents with respect to increase in lime contents for the selected regional clay-lime mixtures are shown in Figure 4.27 and Figure 4.28 respectively. Figure 4.27 shows that the rates of decrements of maximum dry densities increase with increments of lime contents. Figure 4.28 shows that the slopes of change of optimum moisture contents with respect to lime contents are steeper for 0% lime content to 2% lime content than 2% lime content to 8% lime content. Rajbongshi (1997) also found increment of optimum moisture contents and decrement of maximum dry densities with increments of lime content in the samples.

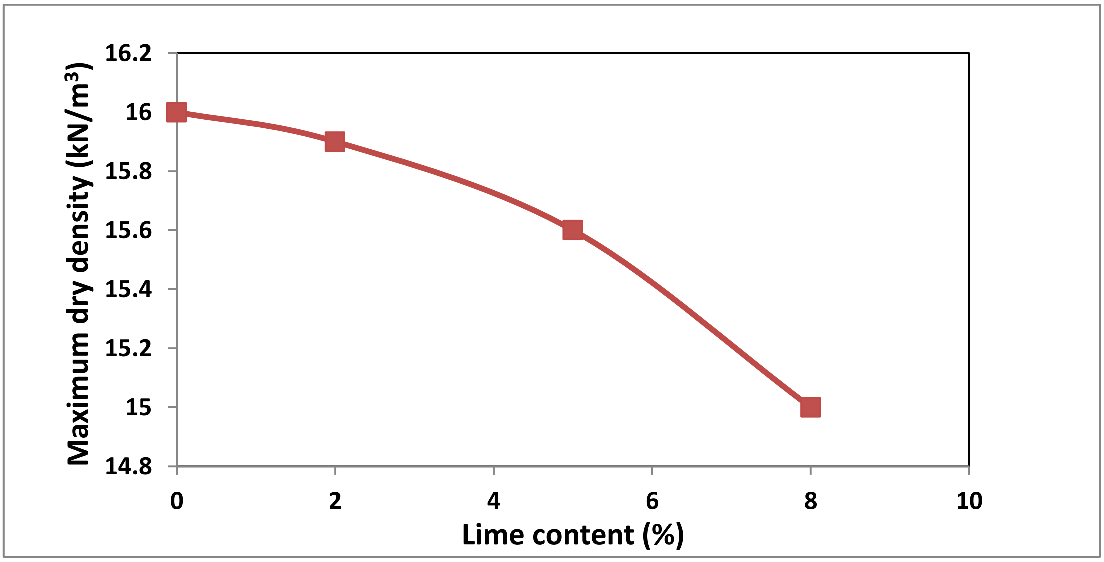

Figure 4.27: Variation of maximum dry densities with respect to different lime contents

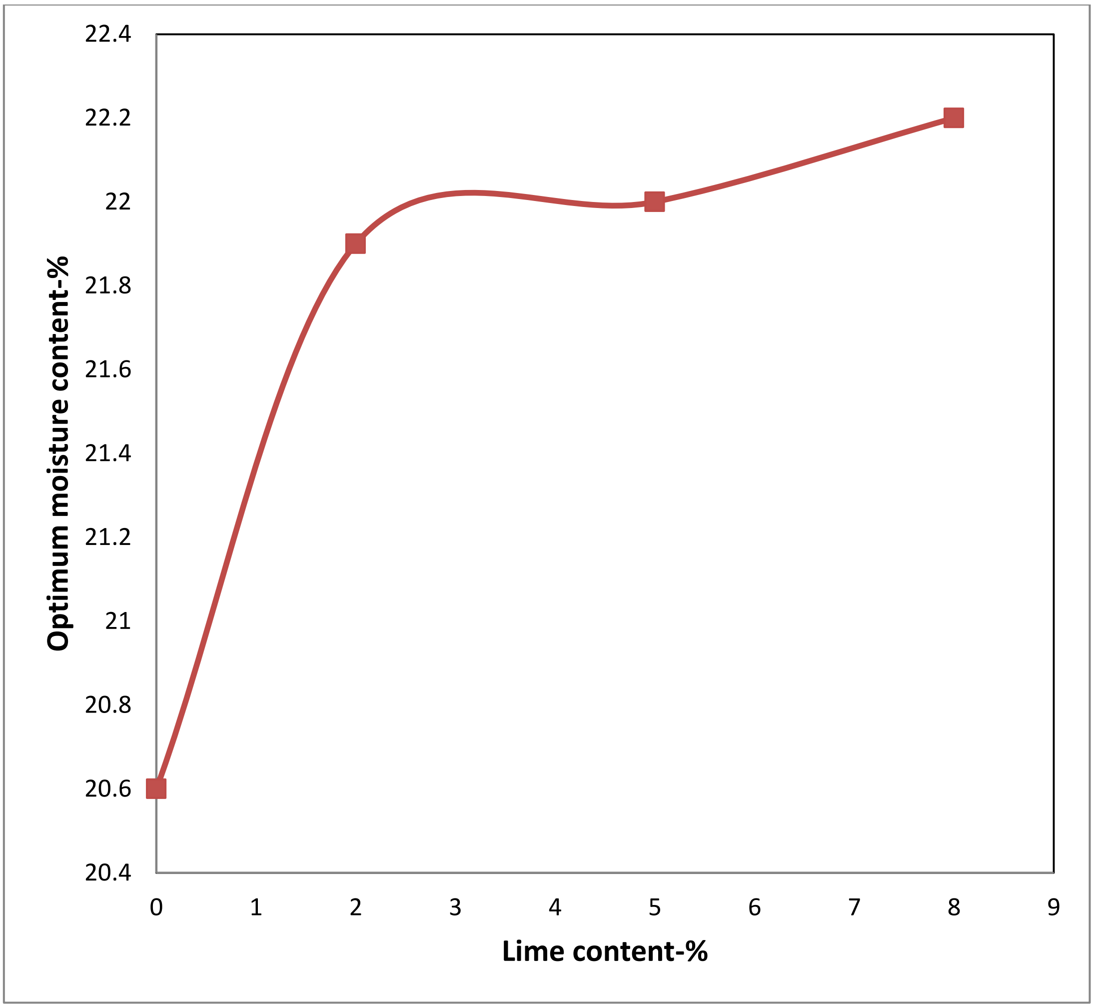

Figure 4.28: Variation of optimum moisture contents with respect to different lime contents

4.4.3 Results of Unconfined Compression Strength Tests

Table 4.9 shows a summary of the unconfined compression strength test results for the selected clay-lime mixtures. In Table 4.9, the values of unconfined compressive strengths, related axial strains at failures and related initial tangent moduli for the untreated samples and samples treated with different lime contents (2%, 5% and 8%) which were cured for 7, 14, 28 and 56 days are presented. The effects of lime contents and curing periods on unconfined compression strengths, axial failure strains and initial tangent moduli are interpreted in sections 4.4.3.1 to 4.4.3.4.

4.4.3.1 Unconfined Compressive Strength

It can be seen from Table 4.9 that for the selected clay-lime mixtures, the values of unconfined compressive strengths of the untreated and treated samples did not vary significantly (exception is 8% lime content for 28 days curing period).

Table 4.9: Variations of unconfined compressive strengths, axial strains at failures and initial tangent moduli with respect to curing periods and lime contents obtained from unconfined compressive strength tests

Curing Period	Percentage of Lime	Unconfined Compression Strength (kN/m^2)	Axial Strain at Failure (%)	Initial Tangent Modulus (kN/m^2)
0 days	0	785	2.8	61300
7 days	2	770	2.04	47900
	5	920	1.47	71100
	8	845	1.16	83000
14 days	2	615	3	45300
	5	1005	1.46	76600
	8	845	1.12	73400
28 days	2	605	3.92	62200
	5	845	2.71	95800
	8	1990	0.76	95800
56 days	2	850	4.6	49800
	5	880	1.4	72400
	8	775	1.17	87500

For 28 days curing period, the magnitude of unconfined compressive strength of lime-clay mixture containing 8% lime is about 2.5 times of the untreated sample. The reason of this higher increment of unconfined compressive strength of clay-lime mixture containing 8% lime while the curing period is 28 days cannot be predicted with in the scheme of the present project work. More studies are required to predict the real case. The variations of unconfined compressive strengths with respect to various lime contents for different curing periods are shown in Figure 4.29.

4.4.3.2 Comparison among Unconfined Compressive Strengths of Present Study and Other Studies

Figure 4.30, Figure 4.31 and Figure 4.32 show the relationships among different unconfined compressive strengths and different lime contents for different types of soil and curing periods. The said relationships for 7, 14 and 28 days curing periods are shown in Figure 4.30, Figure 4.31 and Figure 4.32 respectively. Table 4.10 shows liquid limits, plastic limits, soil symbols and percentages of particles passing through 200 No. sieve for different types of soil mentioned in the said three figures.

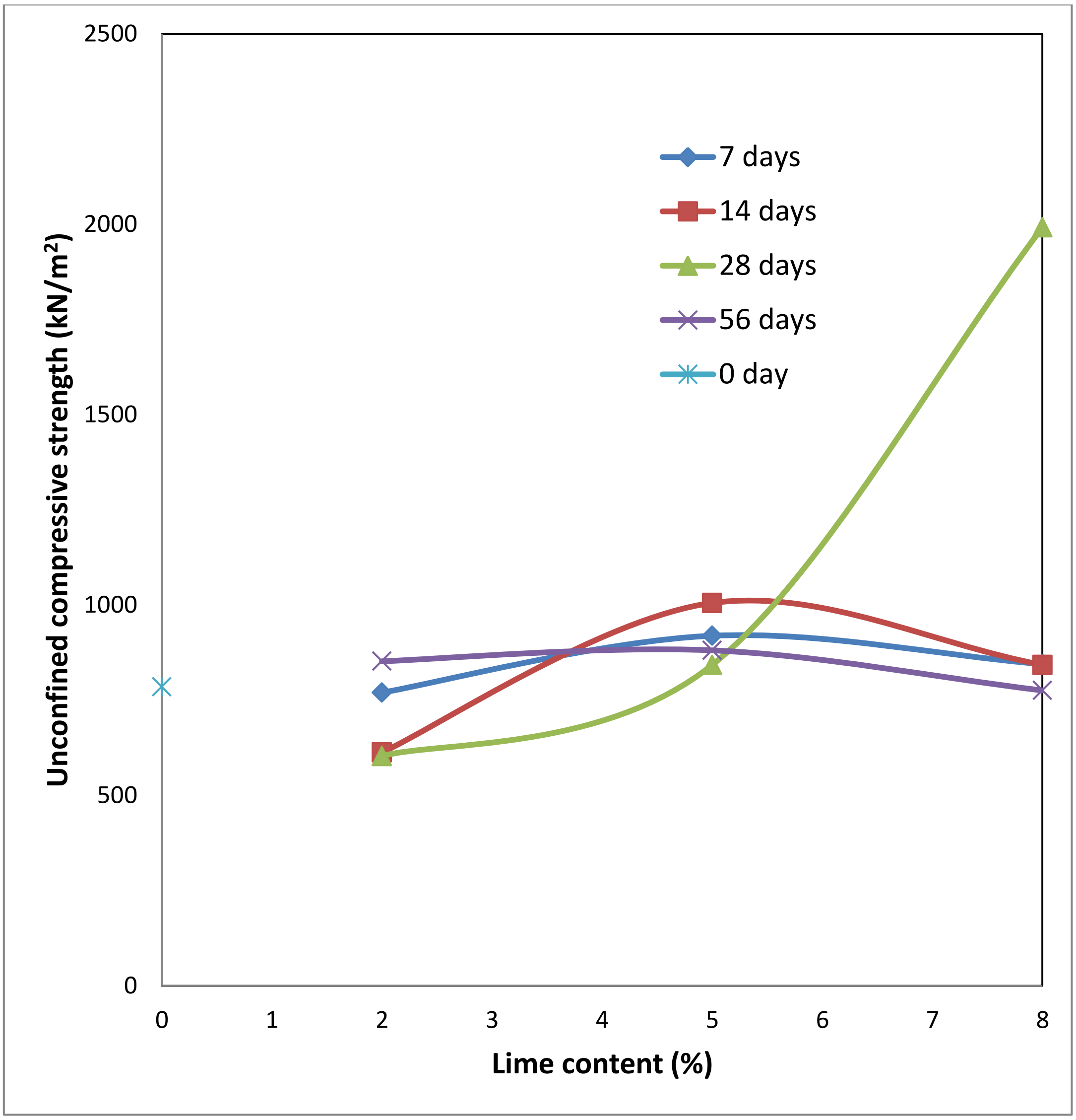

Figure 4.29: Variations of unconfined compressive strengths with respect to different lime contents for different curing periods

Table 4.10: Values of percentages of soil passing through No. 200 sieve, liquid limits and plastic limits with corresponding researchers and soil symbols mentioned in Figure 4.30, Figure 4.31 and Figure 4.32

Researcher	**Soil Symbol**	**Percentage of Soil Passing through No. 200 Sieve**	**Liquid Limit**	**Plastic Limit**
Present investigation	CL	-	42	17
Ahmed (1984)	ML1	63	-	-
Ahmed (1984)	ML2	96	40	30
Ahmed (1984)	CL	82	43	22
Hasan (2002)	ML	92	41	29
Hasan (2002)	CH	98	52	23
Shahjahan (2001)	ML/CL1	96	46	27
Shahjahan (2001)	ML/CL2	97	48	28

Researcher	Soil Symbol	Percentage of Soil Passing through No. 200 Sieve	Liquid Limit	Plastic Limit
Shahjahan (2001)	CL	84	40	23
Hossain (2001)	CH	96	56	13
Molla (1997)	CL1	88	34	21
Molla (1997)	CL2	81	47	21
Molla (1997)	ML	90	37	28
Rajbongshi (1997)	CL	94	44	25

Figure 4.30 shows that for present investigation and 7 days curing period, unconfined compressive strength increase slightly (150 kN/m^2) for increasing lime content from 2% to 5% and then decrease slightly (75 kN/m^2) for increasing lime content from 5% to 8%. This type of trend (first increasing and then decreasing) is not found for any case mentioned in the said Figure 4.30. Figure 4.30 shows that like present investigation unconfined compressive strengths of the samples of different researchers increase with increment of lime to 5% from 3% in the samples. Although the rates and magnitudes of the said increments are not same. Figure 4.30 also shows that unlike present investigation, unconfined compressive strengths of the samples of Hasan (2002), Shahjahan (2001), Hossain (2001) and Rajbongshi (1997) increase with increment of lime content to 7% from 5% in the samples.

Figure 4.31 shows that for present investigation and 14 days curing period, unconfined compressive strength increase about 390 kN/m^2 for increasing lime content from 2% to 5% in the samples and then decrease about 160 kN/m^2 for increasing lime content from 5% to 8% in the samples. This type of trend (first increasing and then decreasing) is not found for any case mentioned in the said Figure 4.31. Figure 4.31 shows that like present investigation unconfined compressive strengths of the samples of different researchers mentioned in Figure 4.31 except Ahmed (1984) for ML1 and ML2 increase with increment of lime to 5% from 3% in the samples, although the rates and magnitudes of the said increments are not same.

For ML1 and ML2 Ahmed (1984) found slight decrease instead of increase. Figure 4.31 also shows that unlike investigations of Hasan (2002), Hossain (2001) and Rajbongshi (1997); present investigation shows decrements of unconfined compressive strengths due to increment of lime content to 7% from 5% in the samples.

The investigations including present investigation which are shown in Figure 4.32 whose lime content's ranges hold the range of % to 7% of lime contents show the increments of unconfined compressive strengths with increments of lime contents within the said ranges.

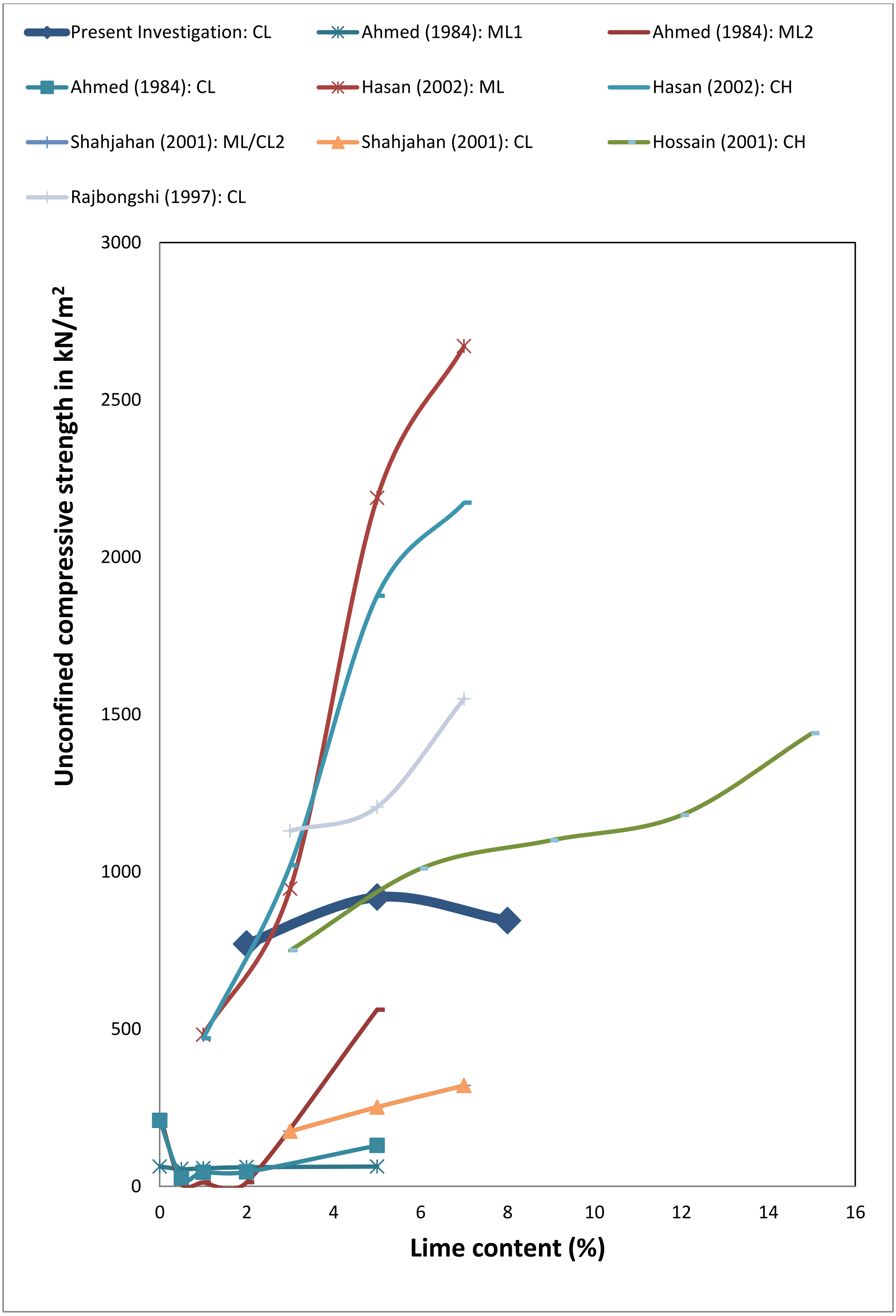

Figure 4.30: Variations of unconfined compressive strengths with respect to lime contents for different types of soil and 7 days curing period

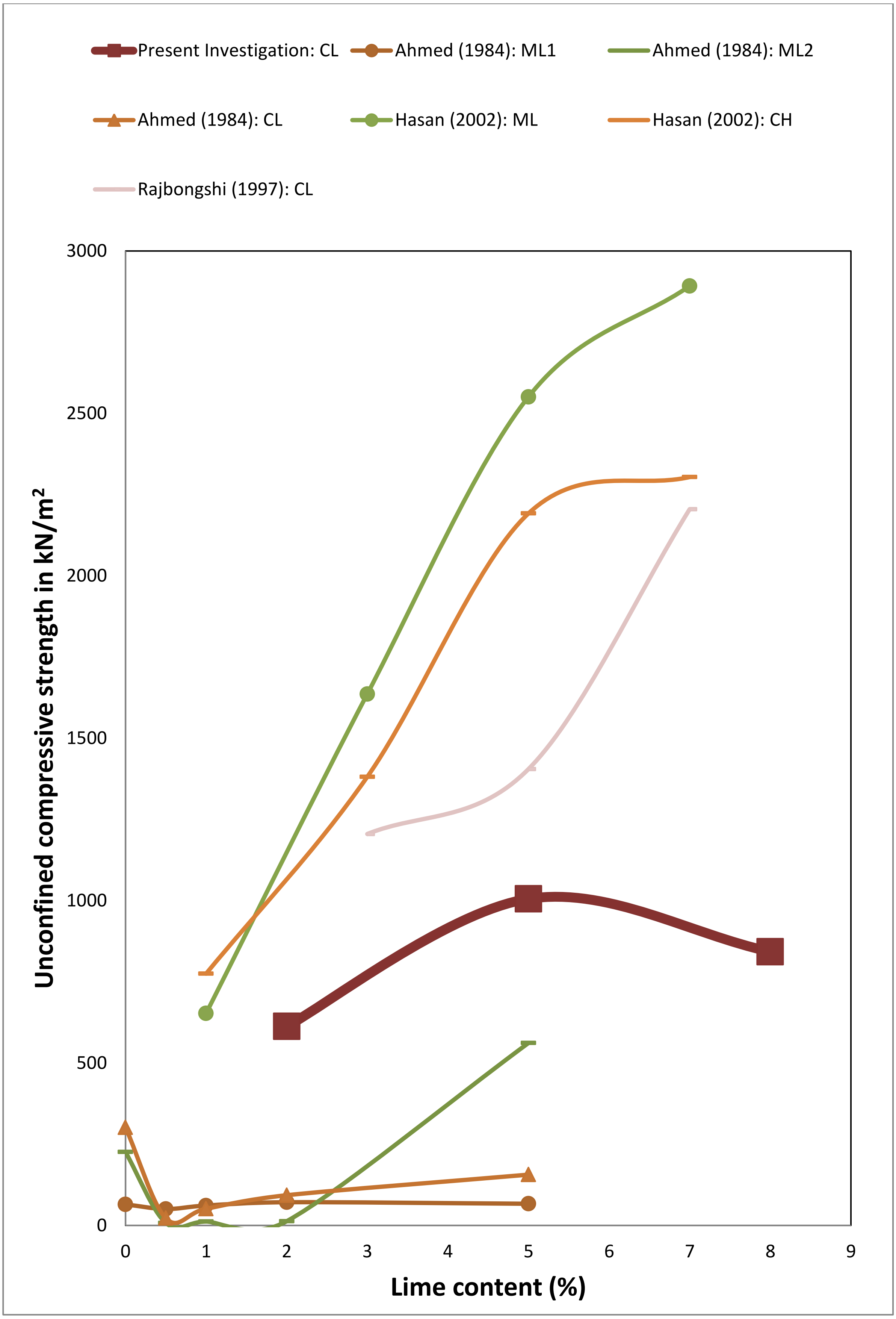

Figure 4.31: Variations of unconfined compressive strengths with respect to lime contents for different types of soil and 14 days curing period

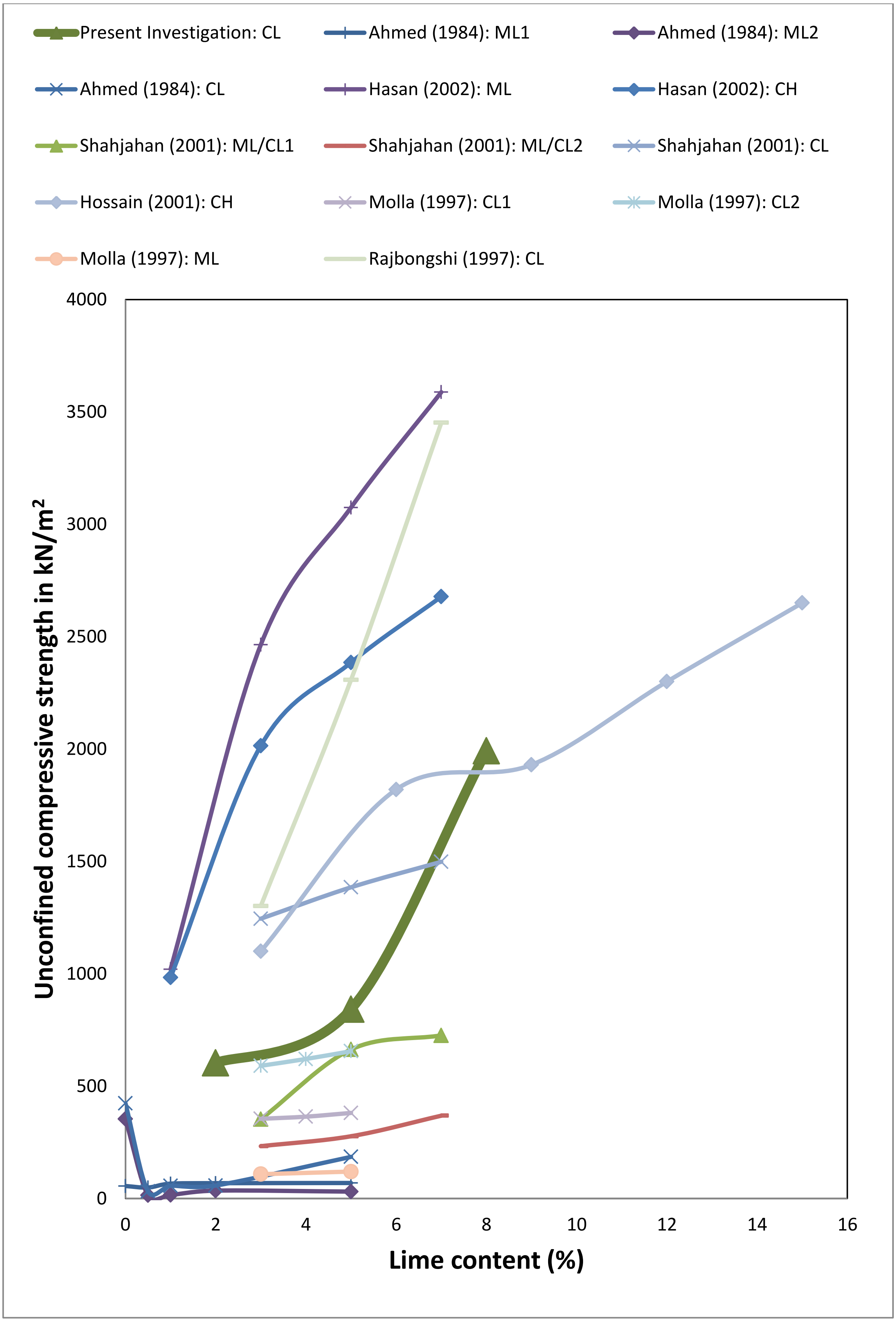

Figure 4.32: Variations of unconfined compressive strengths with respect to lime contents for different types of soil and 28 days curing period

However, the rates and magnitudes of the said increments of unconfined compressive strengths are not same for the said cases. In Figure 4.32, the three cases of Molla (1997) and one case of ML1 of Ahmed (1984) also show slight increments of unconfined compressive strengths with increments of lime contents from 3% to 5%. In the said figure, for CL of Ahmed (1984), increment of unconfined compressive strength is observed for increment of lime content from 2% to 5% while for ML2 of Ahmed (1984), slight decrease of unconfined compressive strength is observed for increment of lime content from 2% to 5%.

4.4.3.3 Axial Failure Strains from Unconfined Compression Strength Tests

Table 4.9 shows the variations of axial failure strains from unconfined compressive strength tests with respect to curing periods and lime contents. For 7, 14 and 28 days curing periods, axial failure strains in unconfined compressive strength tests decrease with increments of lime contents in the samples. According to Table 4.9, for 56 days curing period, the said failure strains also decrease with increments of lime contents in the samples but interpolation of the three known coordinates in the failure strain vs. lime content curve for 56 days curing period shows that 8% lime content in the sample produces slightly higher failure strain in comparison to 6% and 7% lime contents in the samples. However, Rajbongshi (1997) found only decreases of failure strains with increases of lime contents.

The variations of the said failure strains with respect to different lime contents for different curing periods of the present investigation are shown in the Figure 4.33. Since, change in result is continuous up to 8% lime content and 56 days curing period so, further investigations should be done to investigate the effects of more curing periods and lime contents.

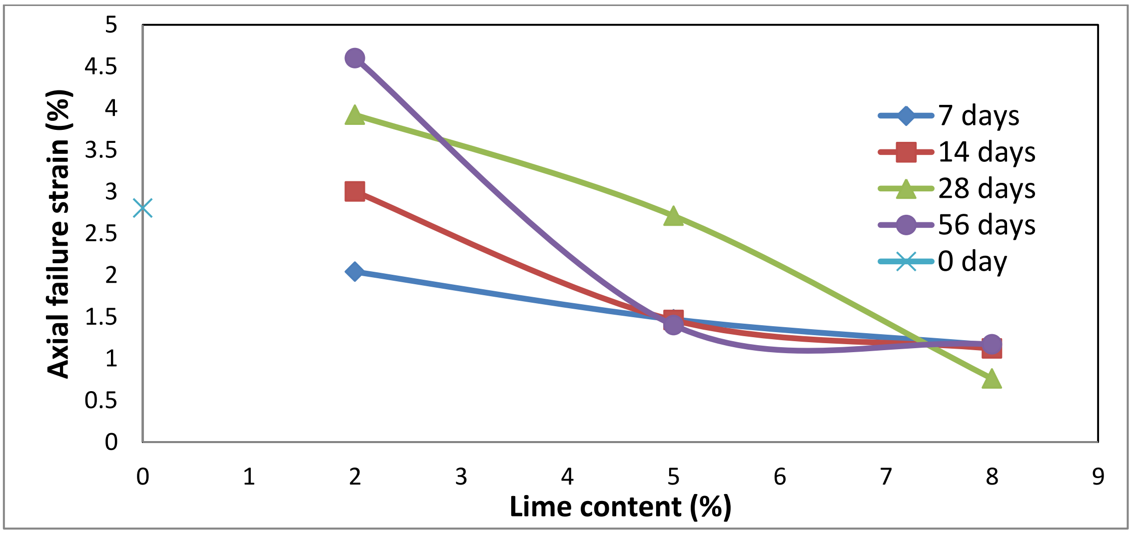

Figure 4.33: Variations of axial failure strains with respect to lime contents for different curing periods

4.4.3.4 Initial Tangent Moduli from Unconfined Compression Strength Tests

Table 4.9 shows the variations of initial tangent moduli from unconfined compressive strength tests with respect to curing periods and lime contents. For 7 days curing period, the initial tangent moduli vary from 47900 kN/m^2 for 2% lime content via 71100 kN/m^2 for 5% lime content to 83000 kN/m^2 for 8% lime content. For 14 days curing period, the initial tangent moduli vary from 45300 kN/m^2 for 2% lime content via 76600 kN/m^2 for 5% lime content to 73400 for 8% lime content. For 28 days curing period, the initial tangent moduli vary from 62200 kN/m^2 for 2% lime content via 95800 kN/m^2 for 5% lime content to 95800 kN/m^2 for 8% lime content. For 56 days curing period, the failure strains vary from 49800 kN/m^2 for 2% lime content via 72400 kN/m^2 for 5% lime content to 87500 kN/m^2 for 8% lime content.

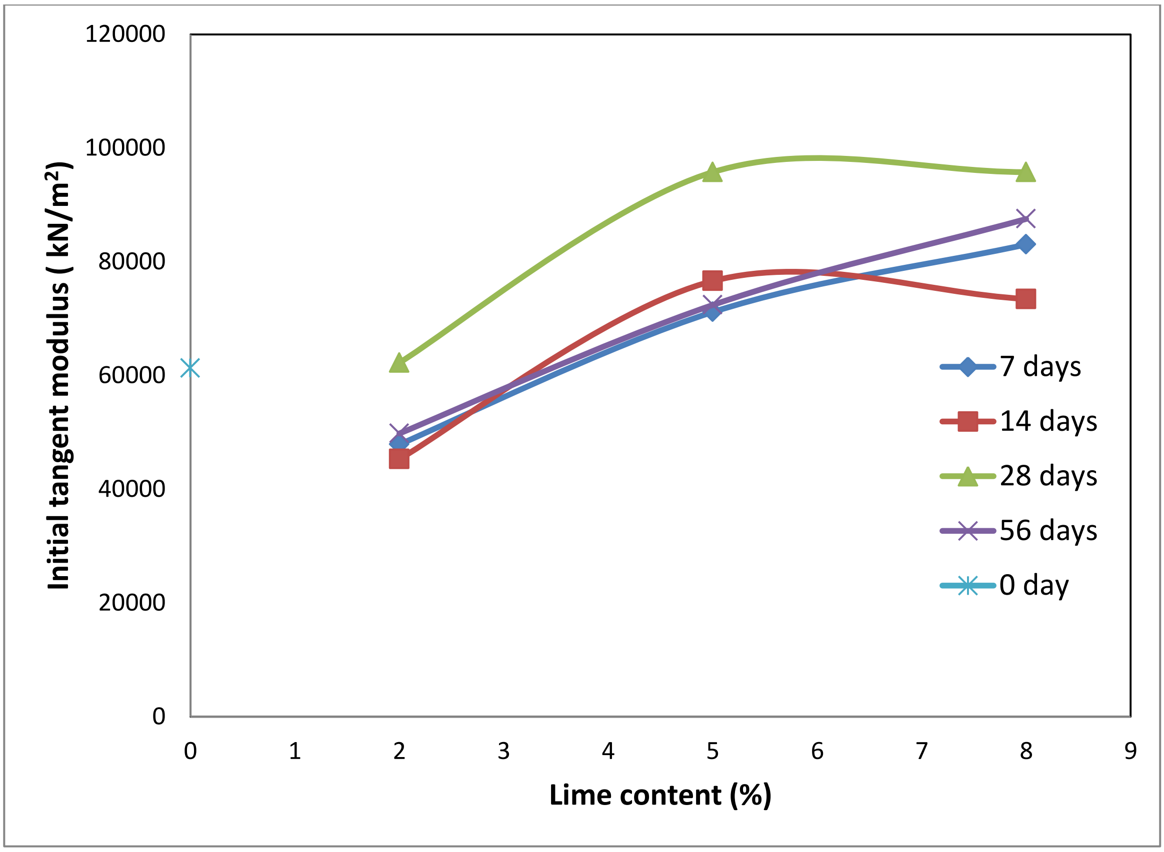

Figure 4.34: Variations of initial tangent moduli determined with unconfined compression strength tests with respect to lime contents for different curing periods

The variations of the said initial tangent moduli with respect to different lime contents are shown in the Figure 4.34. Since, change in result is continuous up to 8% lime content and 56 days curing period so, further study should be done to investigate the effects of more curing periods and lime contents.

4.4.4. Consolidated Undrained Cohesions, Consolidated Undrained Angles of Internal Frictions and Changes in Heights with Corresponding Shear Displacements Obtained From Direct Shear Tests

Table 4.11 shows the values of consolidated undrained cohesions and consolidated undrained angles of internal frictions for the untreated samples and samples treated with 5% lime which were cured for 7, 14, 28 and 56 days respectively. The effects of curing periods on consolidated undrained cohesions and consolidated undrained angles of internal frictions and relationships among shear displacements and corresponding changes in heights of specimens etc. are interpreted in sections 4.4.4.1 to 4.4.4.3 respectively.

Table 4.11: Effects of curing periods on consolidated undrained cohesions, consolidated undrained angles of internal frictions while the lime content in the samples is 5%

Lime Content	Curing Period (Days)	Consolidated Undrained Cohesion (kN/m^2)	Consolidated Undrained Angle of Internal Friction (Degree)
5%	7	35.4	22
	28	30.6	23
	56	19.2	33
0%	0	6.1	24

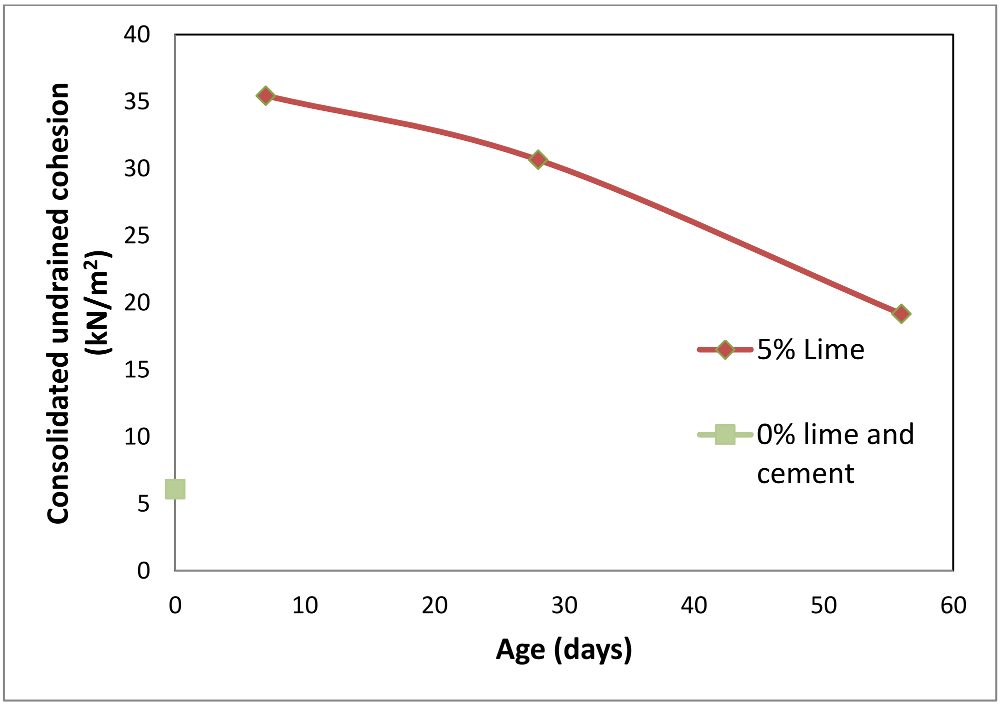

Figure 4.35: Effect of curing ages on consolidated undrained cohesions of lime stabilized samples

4.4.4.1 Effect of Curing Periods on Consolidated Undrained Cohesions

Figure 4.35 shows the variation of the consolidated undrained cohesions (which were obtained from consolidated undrained direct shear tests) with respect to different curing periods for 5% lime content. A coordinate for the untreated sample is also ploted on the said figure. This figure illustrates that the consolidated undrained cohesions values vary from 35.4 kN/m^2 for 7 days curing period via 30.6 kN/m^2 for 28 days curing period to 19.2 kN/m^2 for 56 days curing period. For the said curing periods, the values of said cohesions of lime-clay mixtures are significantly higher than untreated selected clay.

4.4.4.2 Effect of Curing Periods on Consolidated Undrained Angles of Internal Frictions

Figure 4.36 shows the variation of the consolidated undrained angles of internal frictions (which were obtained from consolidated undrained direct ahear tests) with respect to different curing periods for 5% lime content in the samples. This figure illustrates that the angles of internal frictions values vary from 22^0 for 7 days curing period via 23^0 for 28 days curing period to 33^0 for 56 days curing period. By matching Figure 4.36 with Figure 4.35 it is observed that the said angles of internal frictions increase when consolidated undrained cohesions decrease and vice-versa for the selected clay-lime mixtures. For 56 days curing period the value of said angle of internal friction is significantly higher than untreated selected clay. For 7 and 28 days curing periods the values of said angles of internal frictions are slightly lower than untreated selected clay.

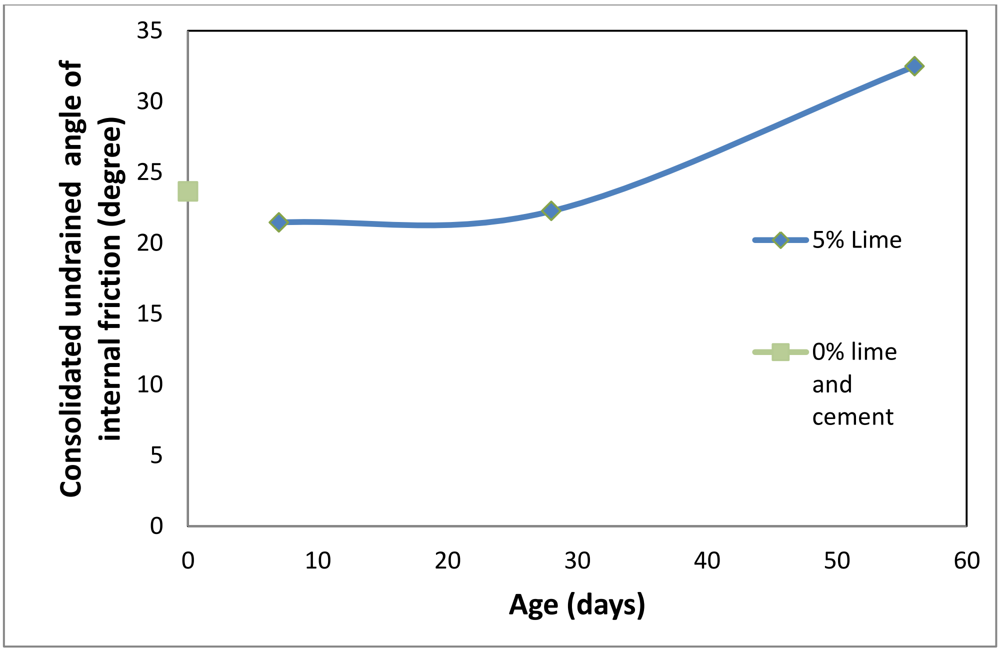

Figure 4.36: Effect of curing ages on consolidated undrained angles of internal frictions of lime stabilized samples

4.4.4.3 Relationships among Shear Displacements and Corresponding Changes in Heights of Specimens

Figure 4.37 to Figure 4.39 show the relationships among shear displacements and corresponding changes in heights of specimens for 7 days, 28 days and 56 days curing periods respectively, for different normal stresses and for 5% lime content. The said relationships do not show a typical trend. Figure A.2 to Figure A.4 in Appendix-A show that the lime treated clay show nature of over consolidated clay for the applied normal loads but all the curves in Figure 4.37 to Figure 4.39 do not show nature of over consolidated clay. From both Figure A.1 and Figure C.1 it can be seen that the untreated selected disturbed sample show nature of normally consolidated clay.

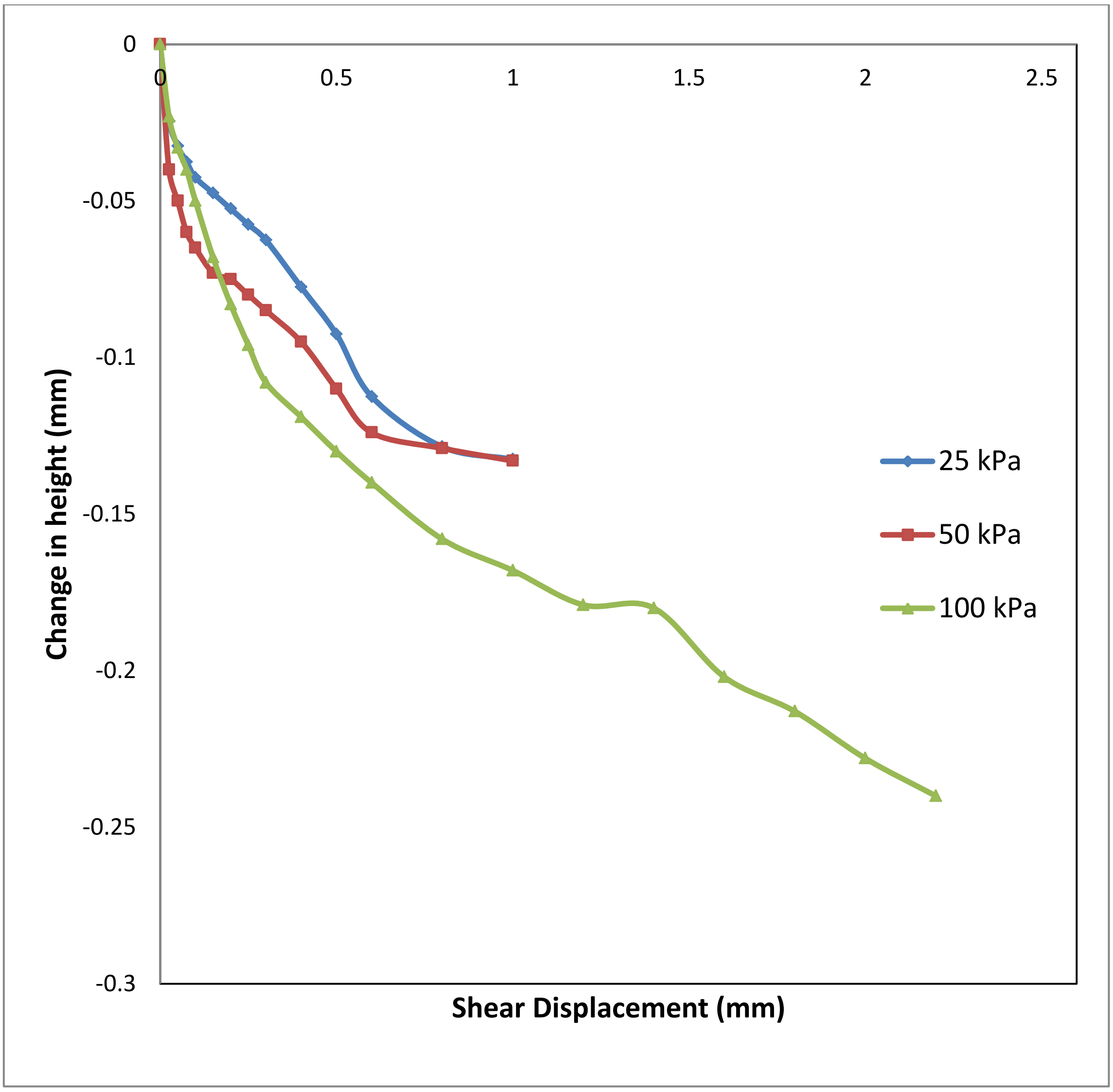

Figure 4.37: Relationships between shear displacements and corresponding changes in height of the specimens for different normal stresses, for 5% lime content and for 7 days curing period

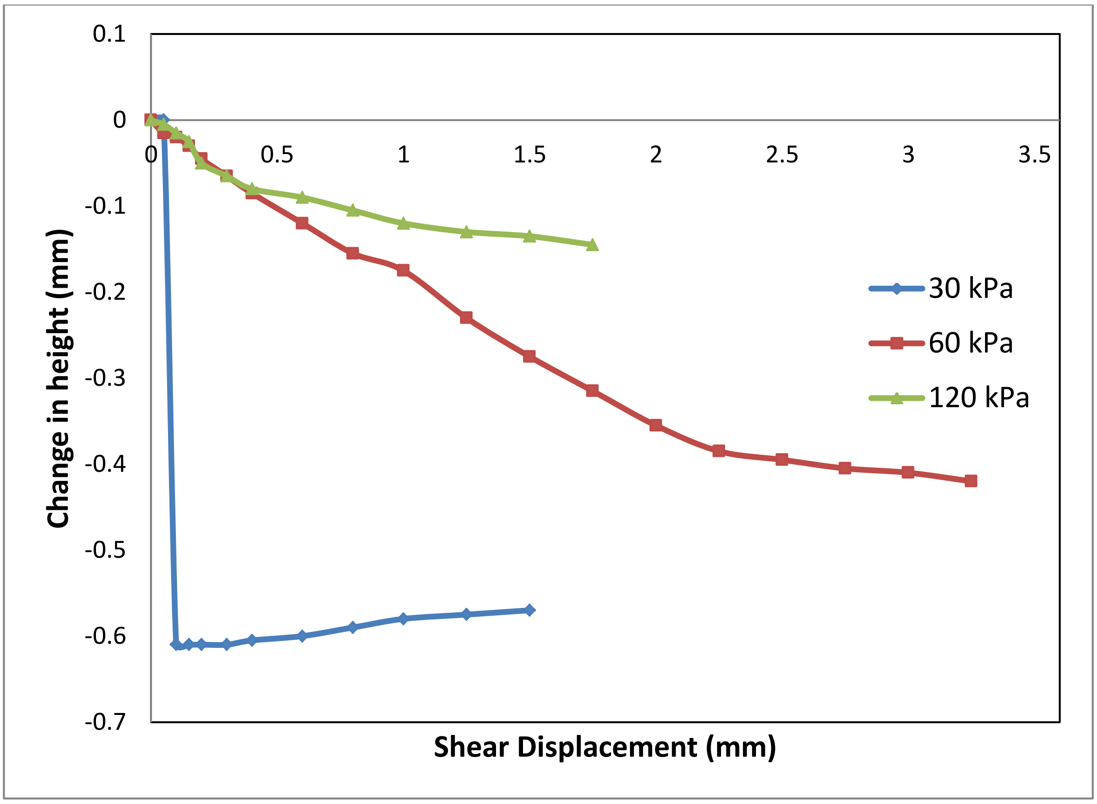

Figure 4.38: Relationships between shear displacements and corresponding changes in height of the specimens for different normal stresses, for 5% lime content and for 28 days curing period

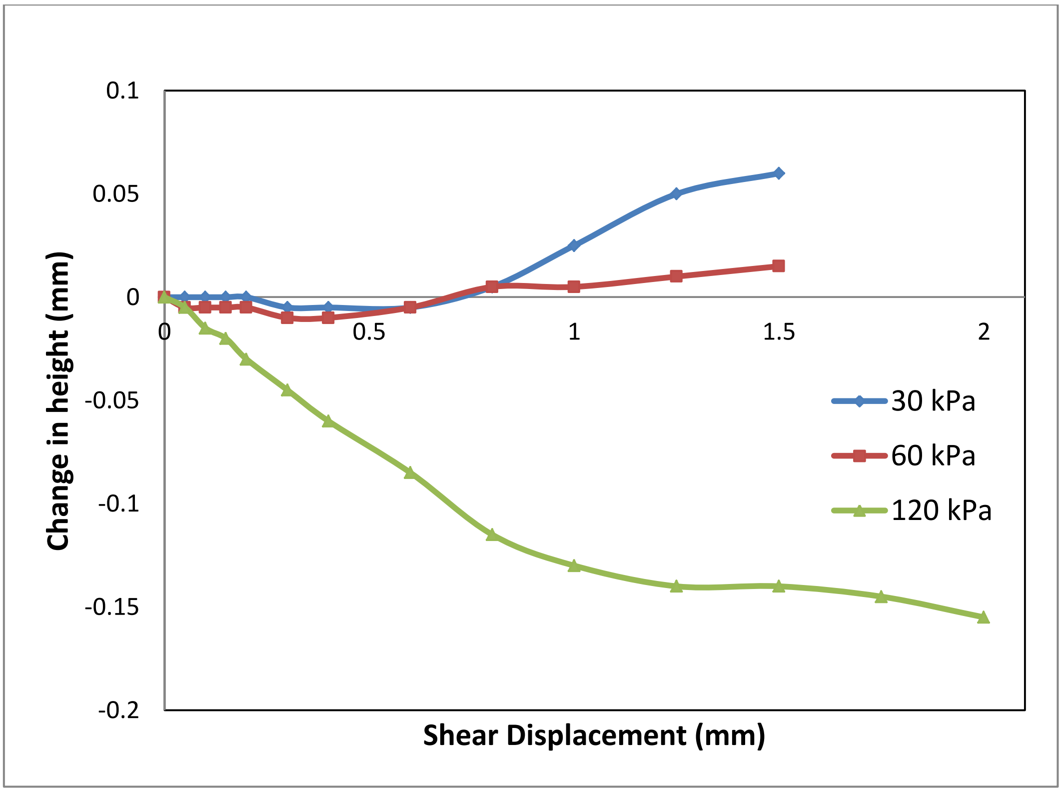

Figure 4.39: Relationships between shear displacements and corresponding changes in height of the specimens for different normal stresses, for 5% lime content and for 56 days curing period

4.5 Comparison of Properties of Cement and Lime Stabilized Samples

In the previous sections (4.3 and 4.4), the properties of cement-stabilized samples (2%, 5% and 8% cement contents) and the properties of lime-stabilized samples (2%, 5%, 8% lime contents) have been presented. In this section attempt has been made to compare the physical and engineering characteristics of cement and lime stabilized samples with 2%, 5% and 8% additives individually.

4.5.1 Index and Shrinkage Properties

A comparison of index and shrinkage properties of cement and lime treated samples is presented in Table 4.12. It can be seen from Table 4.12 that the values of liquid limits, plastic limits and shrinkage limits are much higher for cement treated samples than lime treated samples. The values of plasticity indices are almost same for both lime treatment and cement treatment. For 2% and 5% admixture contents, linear shrinkage is almost same for both cement and lime stabilized samples. For 8% admixture content, linear shrinkage for cement treated sample is lower than lime treated sample.

4.5.2 Moisture Density Relations

A comparison of optimum moisture contents and maximum dry densities of cement and lime treated soils at different percent of additives are shown in Table 4.13. Table 4.13 shows that there is no significant difference between cement and lime stabilized samples for the cases of optimum moisture contents and maximum dry densities.

Table 4.12: Comparisons of index properties and shrinkage properties of cement and lime stabilized samples

Content of Admixture (%)	Name of Admixture	Plastic Limit	Liquid Limit	Shrinkage Limit	Plasticity Index	Percentage of Linear Shrinkage
2	Cement	19	44	23	25	10
	Lime	16	40	13	24	11
5	Cement	26	52	33	26	11
	Lime	16	41	21	25	12
8	Cement	31	57	42	26	9
	Lime	18	45	35	27	12

Table 4.13: Comparisons of optimum moisture contents and maximum dry densities of cement and lime stabilized samples

Content of Admixture (%)	Name of Admixture	Optimum Moisture Content (%)	Maximum Dry Density (kN/m^3)
2	Cement	22.7	15.7
	Lime	21.9	15.9
5	Cement	23	15.6
	Lime	22	15.6
8	Cement	23	15.6
	Lime	22.2	15

4.5.3 Unconfined Compressive Strength Tests Results

The results of cement and lime treated samples obtained from unconfined compression strength tests are compared in Table 4.14. The comparisons of unconfined compressive strengths, axial deformations at failures and initial tangent moduli of cement and lime treated samples are discussed in the sections 4.5.3.1 to 4.5.3.3 respectively.

Table 4.14: Comparisons of unconfined compressive strengths, axial deformations at failures and initial tangent moduli of cement and lime treated samples

Curing Period	Name of Admixture	Percentage of Lime	Unconfined Compression Strength (kN/m^2)	Axial Strain at Failure (%)	Initial Tangent Modulus (kN/m^2)
7 days	Lime	2	770	2.04	47900
	Cement	2	755	1.11	102200
	Lime	5	920	1.47	71100
	Cement	5	1145	0.59	275300
	Lime	8	845	1.16	83000
	Cement	8	2175	0.82	287300
14 days	Lime	2	615	3	45300
	Cement	2	805	0.64	117900
	Lime	5	1005	1.46	76600
	Cement	5	1540	0.73	287300
	Lime	8	845	1.12	73400
	Cement	8	1540	1.11	153200
28 days	Lime	2	605	3.92	62200
	Cement	2	845	1.2	60200
	Lime	5	845	2.71	95800
	Cement	5	1305	0.9	175500

Curing Period	Name of Admixture	Percentage of Lime	Unconfined Compression Strength (kN/m^2)	Axial Strain at Failure (%)	Initial Tangent Modulus (kN/m^2)
28 days	Lime	8	1990	0.76	95800
	Cement	8	2195	0.84	324900
56 days	Lime	2	850	4.6	49800
	Cement	2	530	0.59	162800
	Lime	5	880	1.4	72400
	Cement	5	1305	0.57	526700
	Lime	8	775	1.17	87500
	Cement	8	1430	0.94	175500

4.5.3.1 Unconfined Compressive Strengths

It can be seen from Table 4.14 that after 56 days curing period, the unconfined compressive strength of cement treated clay was 38% lower in comparison to lime treated clay for 2% admixture content. On the other hand for the said curing period and for 5% and 8% admixture contents the unconfined compressive strengths of cement treated samples were about 48% and 85% higher respectively in comparison to lime treated samples.

4.5.3.2 Axial Deformations at Failures

It can be observed from Table 4.14 that after 56 days curing period, the axial deformations at failures of cement treated clay samples were about 87%, 59% and 20% lower in comparison to lime treated clay samples for 2%, 5% and 8% admixture contents respectively.

4.5.3.3 Initial Tangent Moduli

From Table 4.14 it can be observed that after 56 days curing period, the initial tangent moduli of cement treated clay samples were about 230%, 630% and 100% higher in comparison to lime treated clay samples for 2%, 5% and 8% admixture contents respectively.

4.5.4 Consolidated Undrained Cohesions and Consolidated Undrained Angles of Internal Frictions

The consolidated undrained cohesions and consolidated undrained angles of internal frictions of cement and lime treated samples obtained from consolidated undrained direct shear tests are compared in Table 4.15. The comparison of consolidated undrained cohesions and consolidated undrained angles of internal frictions of cement and lime treated samples are discussed in the sections 4.5.4.1 to 4.5.4.2 respectively.

Table 4.15: Comparisons of consolidated undrained cohesions and consolidated undrained angles of internal frictions of cement and lime treated samples

Admixture Content	Type of Admixture	Curing Period (Days)	Consolidated Undrained Cohesion (kN/m^2)	Consolidated Undrained Angle of Internal Friction (Degree)
5%	Lime	7	35.4	22
	Cement	7	61.3	23
	Lime	28	30.6	23
	Cement	28	139.8	13
	Lime	56	19.2	33
	Cement	56	183.9	6

4.5.4.1 Consolidated Undrained Cohesion

It can be seen from Table 4.15 that for 5% admixture content, the consolidated undrained cohesions of the selected cement-clay mixtures were about 73%, 356% and 857% higher in comparison to the selected lime-clay mixtures for 7, 28 and 56 days curing periods respectively.

4.5.4.2 Consolidated Undrained Angle of Internal Friction

It is observed from Table 4.15 that for 5% admixture content, the consolidated undrained angle of internal friction of the selected cement-clay mixture was 4.5% higher in comparison to the selected lime-clay mixture for 7 days curing period. It is also observed that for the same admixture content, the consolidated undrained angles of internal frictions of the selected cement-clay mixtures were 43% and 81% lower in comparison to the corresponding selected lime-clay mixtures.

CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

5.1 General

In this research work, investigations on effects of cement and lime stabilization on a selected soil which was collected from Mouchak in Gazipur district (the north latitude and east longitude of the soil collection point are about 24.027868^0 and 90.299284^0 respectively) have been carried out. For liquid limit tests, plastic limit tests, shrinkage limit tests, linear shrinkage limit tests, standard proctor tests and unconfined compressive strength tests; cement and lime has been used with selected soil in amount of 2%, 5% and 8% individually. For direct shear tests, amount of cement and lime in samples was 5% individually. Different physical and engineering properties of cement and lime stabilized soils have been determined in order to assess the suitability of cement and lime stabilization. The major findings and conclusions of the current research work have been separated into three sections relating to the following areas:

(i) Influences of cement stabilization on different physical and engineering properties of samples of the selected clay.

(ii) Influences of lime stabilization on the physical and engineering properties of samples of the selected clay.

(iii) Overall comments on lime and cement stabilization.

5.1.1 Cement Stabilization

The major findings and conclusions of present research on cement stabilization may be summarized as follows:

(i) The plastic limits, liquid limits and shrinkage limits of the soil-cement mixtures increase with the increases of cement contents while plasticity indices of the soil-cement mixtures do not virtually change with increases of cement contents.

(ii) For the samples, values of maximum dry densities reduce up to 5% cement content and then no change occurred up to 8% cement content while optimum moisture contents increase up to 5% cement content and then no change occurred up to 8% cement content.

(iii) Maximum increase of unconfined compressive strength was obtained for 8% cement content after curing for 28 days but this value reduced after 56 days curing.

(iv) Within the range of cement content of present research work, no specific trend of variation of axial failure strains and initial tangent moduli (obtained from unconfined compressive strength tests) is observed.

(v) It is observed that for the selected cement treated clay samples, the values of consolidated undrained cohesions increase with the increases of curing periods up to 56 days from 7 days.

(vi) It is observed that for the selected cement treated clay samples, the values of consolidated undrained angles of internal frictions decrease with the increases of curing periods up to 56 days from 7 days.

(vii) No typical trend of relationship among shear displacements and corresponding changes in heights is observed with in the scheme of present research work.

5.1.2 Lime Stabilization

The major findings and conclusions of present research on lime stabilization may be summarized as follows:

(i) It is found that plastic limit, liquid limit, shrinkage limit and plasticity index decrease slightly when lime content increase to 2% from 0%. Then plastic limits, liquid limits and plasticity indices increase slightly when lime contents increase to 8% via 5% from 2%. In the said range of 8% to 2%, shrinkage limits increase almost linearly with increases of lime contents. It is also found that the percentages of linear shrinkages do not change significantly with increasing lime contents in the soil-lime mixtures up to 8% lime content.

(ii) Within the range of present research work it is observed that values of maximum dry densities reduce with increments of lime contents in lime-soil mixtures while optimum moisture contents increase with increments of lime contents.

(iii) Within the range of present research work, the values of unconfined compressive strengths of the untreated and treated samples do not vary significantly with exception of 8% lime content after 28 days curing period.

(iv) For 7, 14 and 28 days curing periods, axial failure strains decrease with increments of lime contents in the range of 2% to 8% in the samples. For 56 days curing period axial failure strains decrease with increments of lime contents from 2% to 6% but for increments of lime contents from 6% to 8% the interpolation of data showed slight increments of axial failure strains.

(v) For 7 and 56 days curing periods initial tangent moduli increase with increments of lime contents up to 8% from 2%. For 14 and 28 days curing periods initial tangent moduli increase with increments of lime contents up to 5% from 2% and more increment of lime content up to 8% lime content than 5% lime content do not show any significant change.

(vi) Within scheme of present research work it is observed that consolidated undrained cohesions values decrease with increments of lime contents in the samples.

(vii) Within the scheme of present research work, it is observed that consolidated undrained angles of internal frictions values increase with increments of lime contents in the samples.

(viii) No typical relationship among shear displacements and corresponding changes in height is observed.

5.1.3 Overall Comments on Lime and Cement Stabilization

Based on the results of unconfined compressive strength tests, cement is better choice than lime to increase unconfined compressive strength. Most of the data of the said tests suggest the selection of cement as stabilizer instead of lime to increase initial tangent modulus and to reduce axial failure strain.

To obtain higher consolidated undrained cohesion of the selected soil, cement is also better choice than lime. For curing period of 56 days, lime is better choice than cement to obtain higher consolidated undrained angle of internal friction of the selected soil.

To increase plastic limit, liquid limit, shrinkage limit cement is better than lime for the case of selected soil. For the selected soil, both lime and cement has minor contribution to change plasticity index, percentage of linear shrinkage, optimum moisture content and maximum dry density.

5.2 Recommendations for Future Study

Several aspects of the work presented in this project report require further study. Some of the important areas of future research can be as follows:

(1) Similar investigations mentioned in this report can be carried out with soils collected from other locations of Gazipur.

(2) The physical and engineering properties of these soil samples stabilized with higher percentages of cement and lime can be investigated.

(3) The influence of longer term curing age on engineering properties of the stabilized samples can be investigated.

(4) The physical and engineering properties can be carried out by stabilizing the soils with other additives.

(5) Analysis can be carried out for stabilization of rural roads and highways.

(6) The behavior and engineering properties of the soil samples can be evaluated with more moisture density relations, freezing-thawing damage, California Bearing Ratio, flexural strength and modulus, pore pressure development, and consolidation characteristics etc.

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APPENDIX A
“SHEAR STRESS VS. SHEAR DISPLACEMENT” CURVES OBTAINED FROM CONSOLIDATED UNDRAINED DIRECT SHEAR TESTS

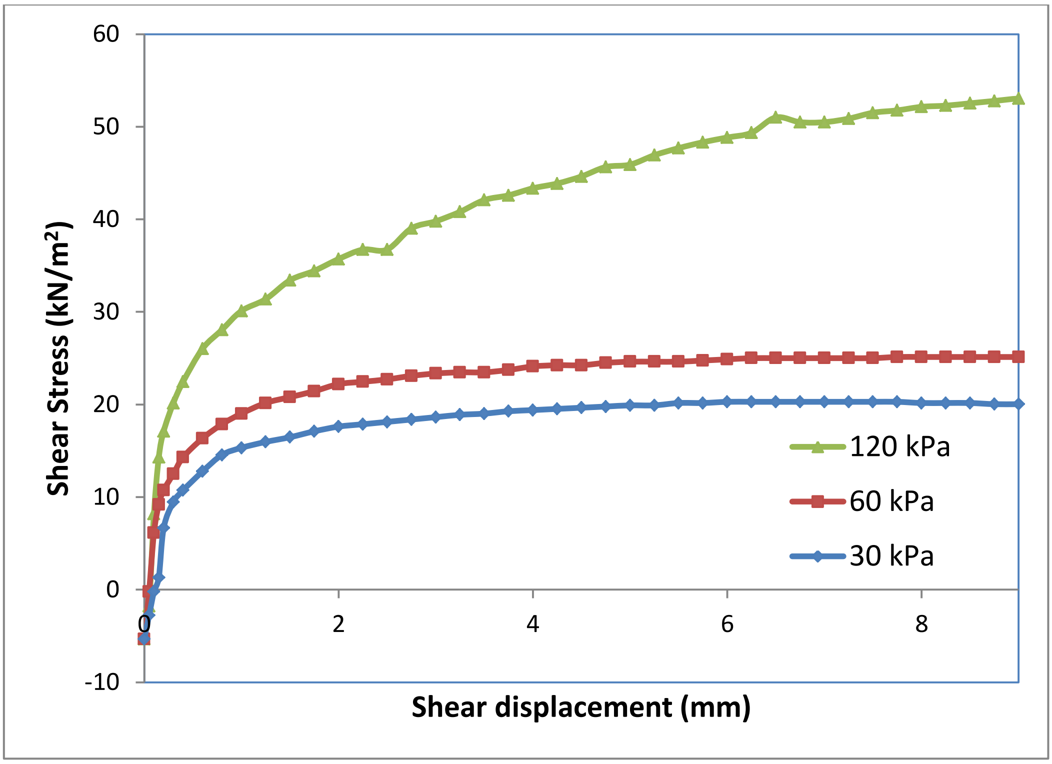

Figure A.1: ―Shear stress vs. shear displacement" curves of samples without curing and admixtures

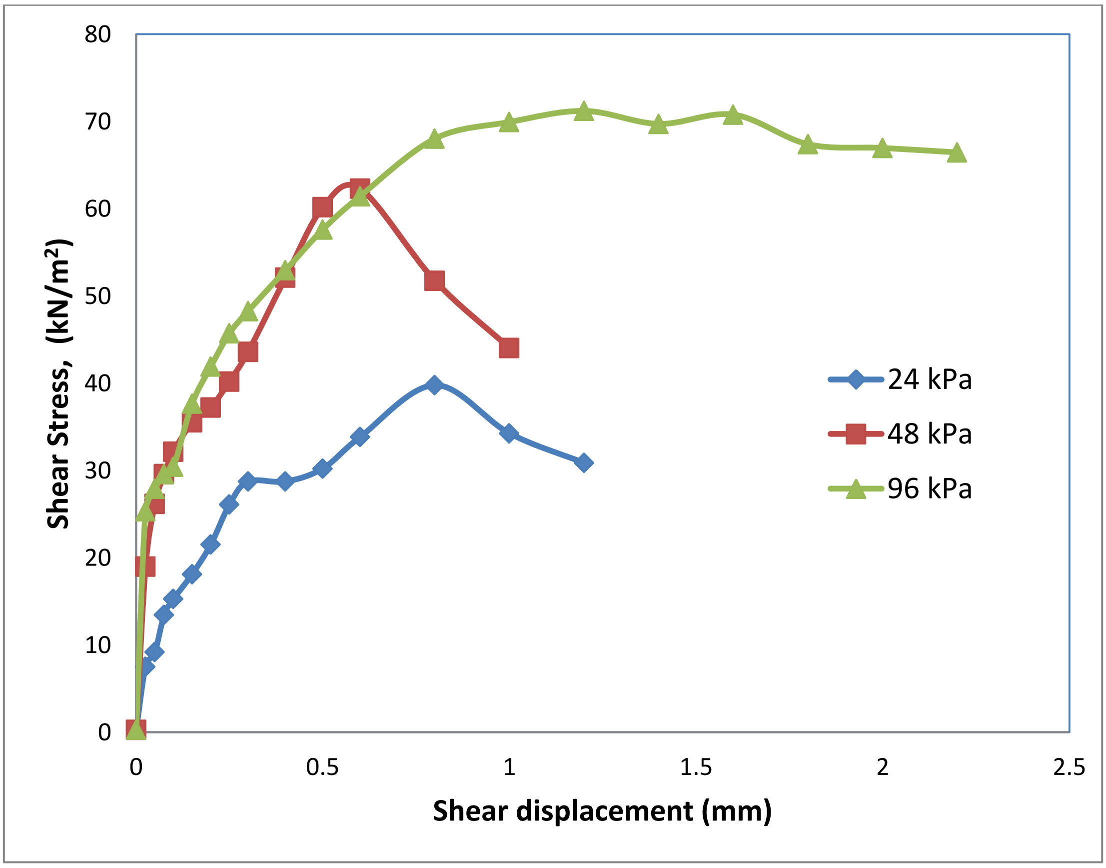

Figure A. : ―Shear stress vs. shear displacement" curves of samples with 5% lime and for tests after curing for 7 days

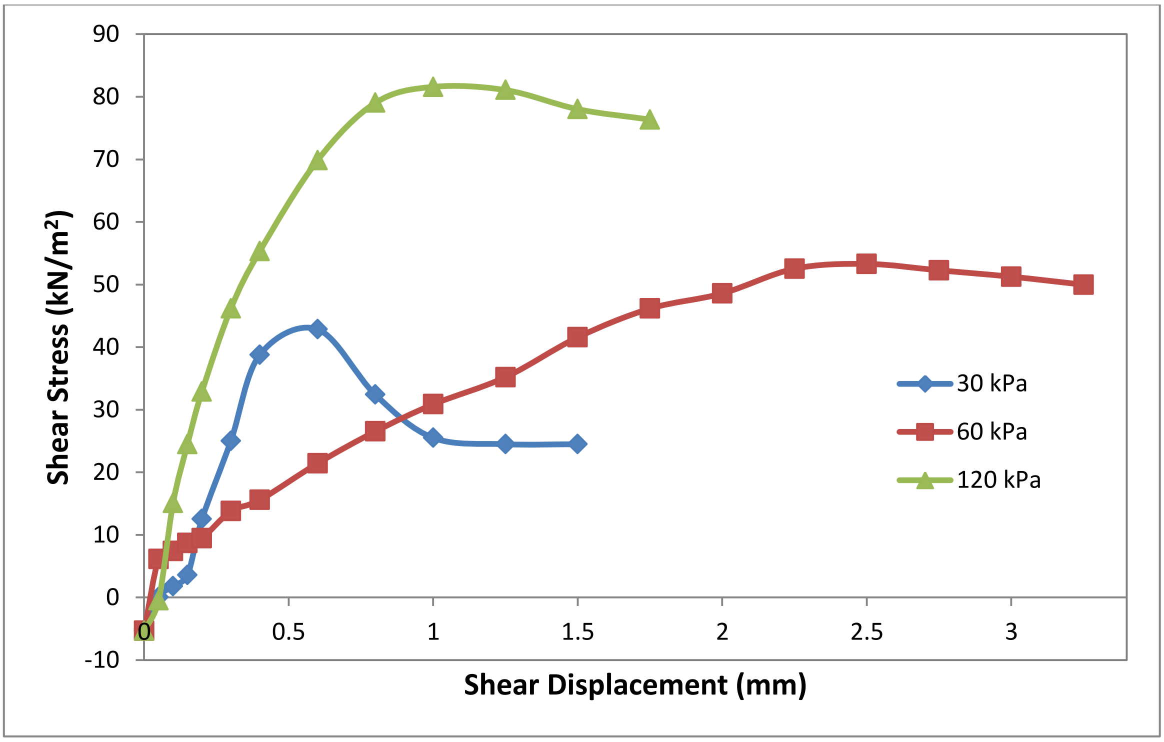

Figure A.3: ―Shear stress vs. shear displacement" curves of samples with 5% lime and for tests after curing for 28 days

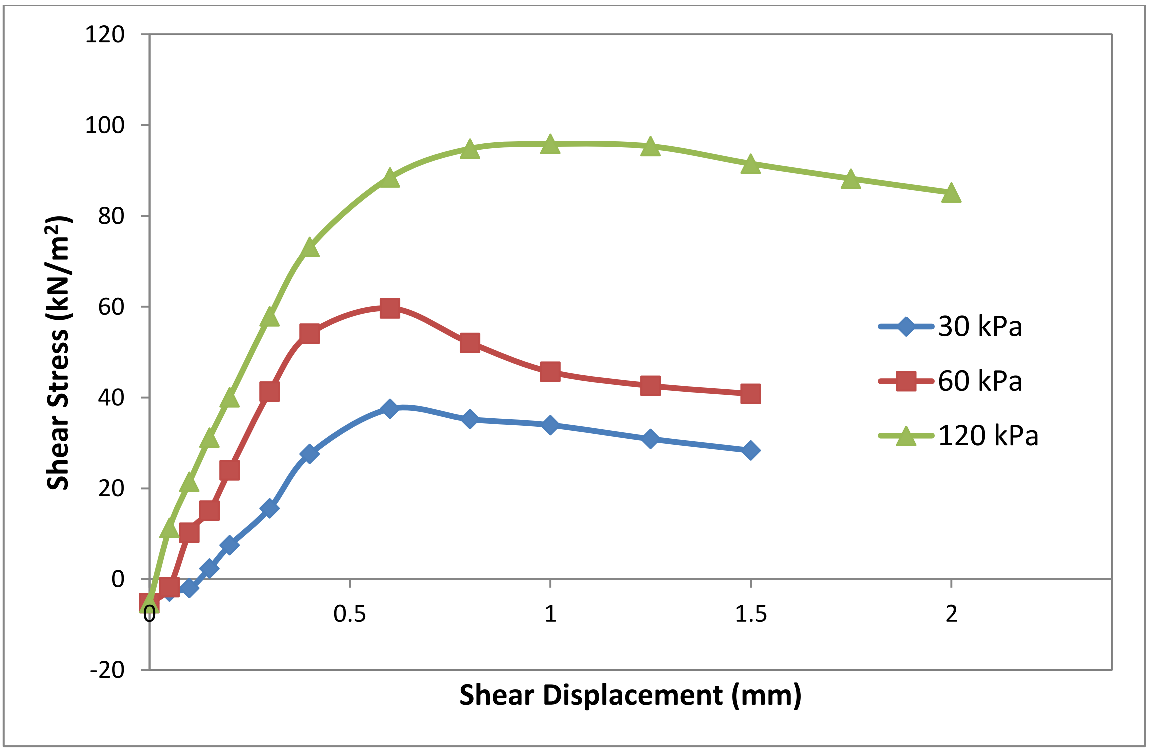

Figure A.4: ―Shear stress vs. shear displacement" curves of samples with 5% lime and for tests after curing for 56 days

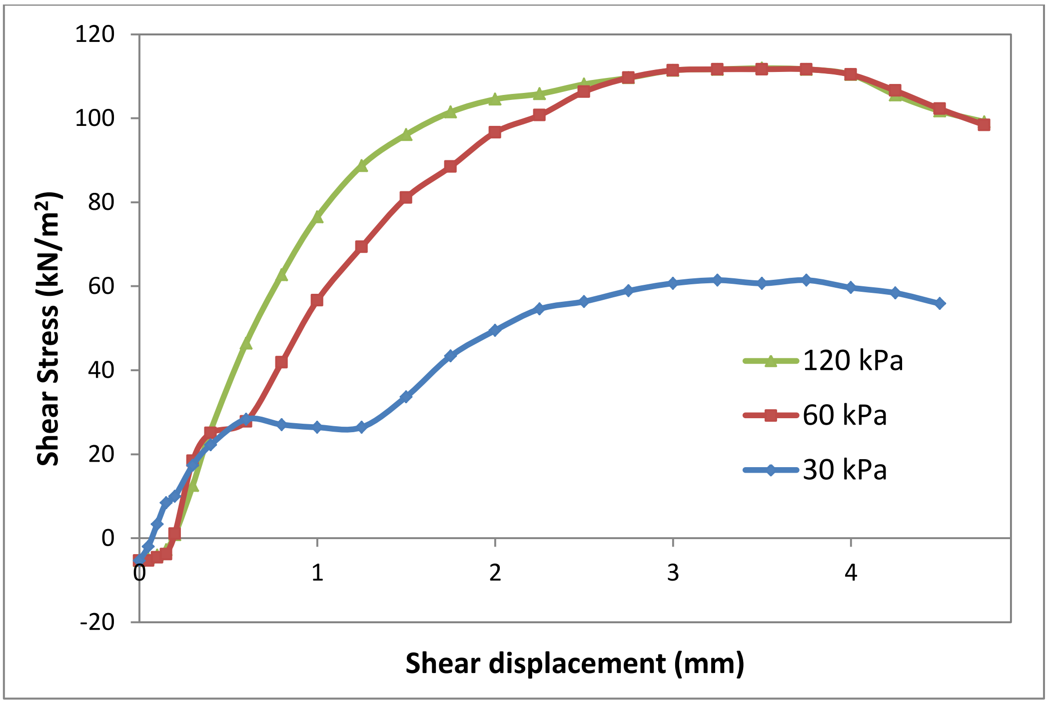

Figure A.5: ―Shear stress vs. shear displacement" curves of samples with 5% cement and for tests after curing for 7 days

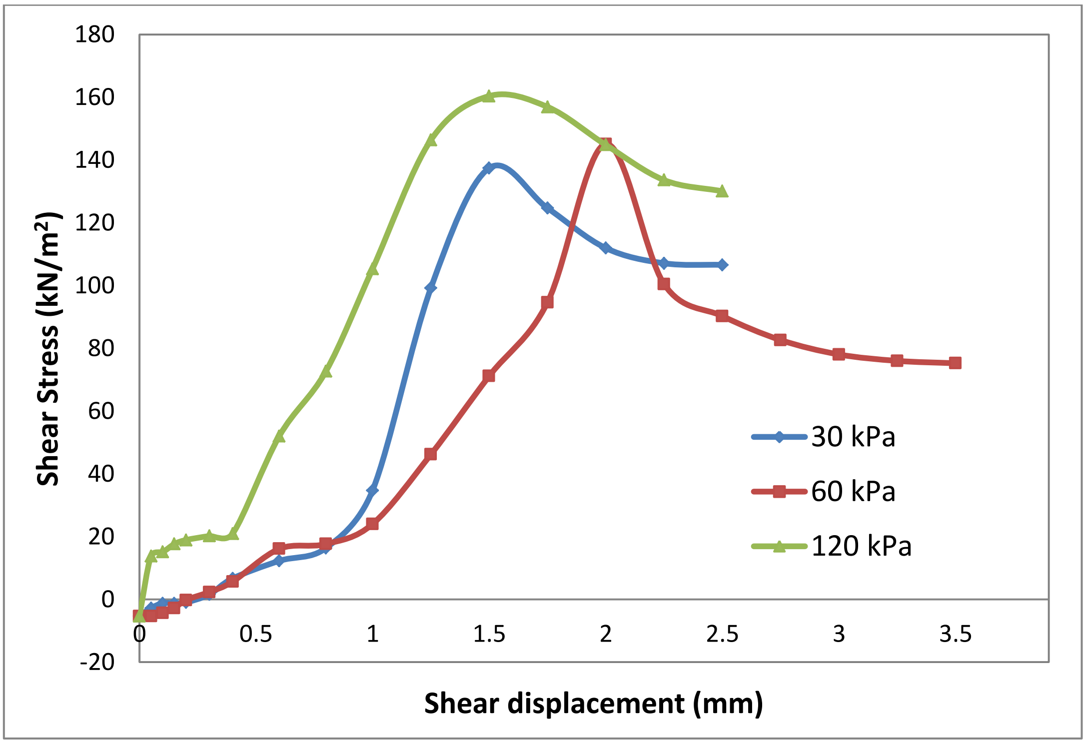

Figure A.6: ―Shear stress vs. shear displacement" curves of samples with 5% cement and for tests after curing for 14 days

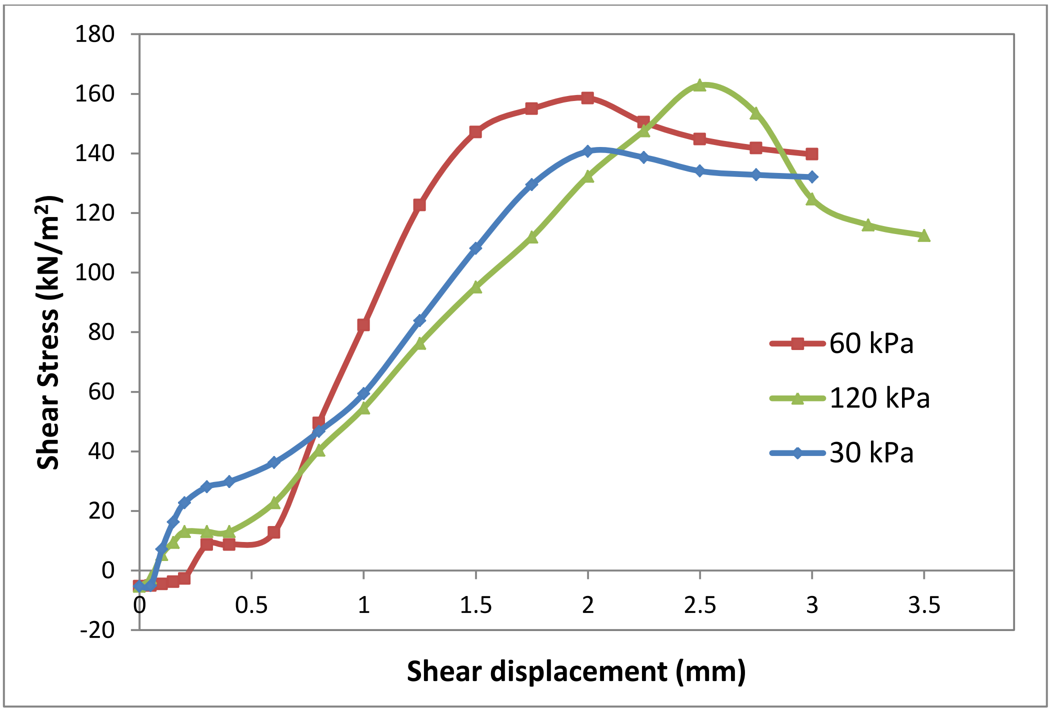

Figure A.7: ―Shear stress vs. shear displacement" curves of samples with 5% cement and for tests after curing for 28 days

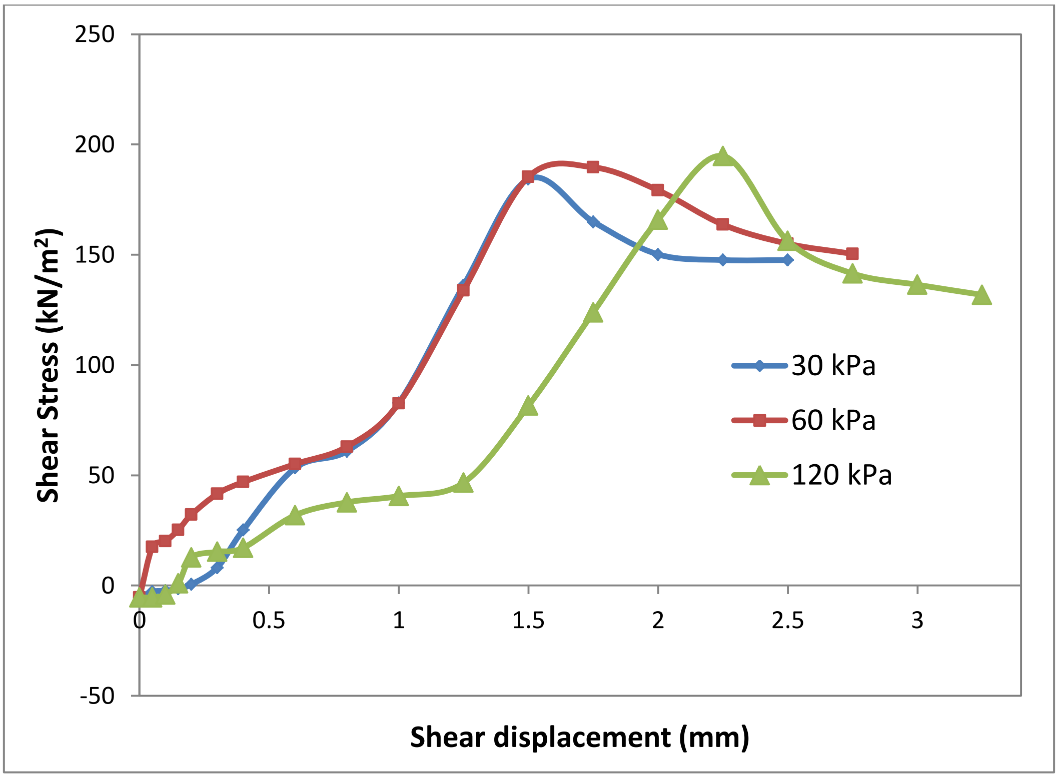

Figure A.8: ―Shear stress vs. shear displacement" curves of samples with 5% cement and for tests after curing for 56 days

APPENDIX B
RELATIONSHIP BETWEEN SHEAR STRESS AND NORMAL STRESS OBTAINED FROM CONSOLIDATED UNDRAINED DIRECT SHEAR TESTS

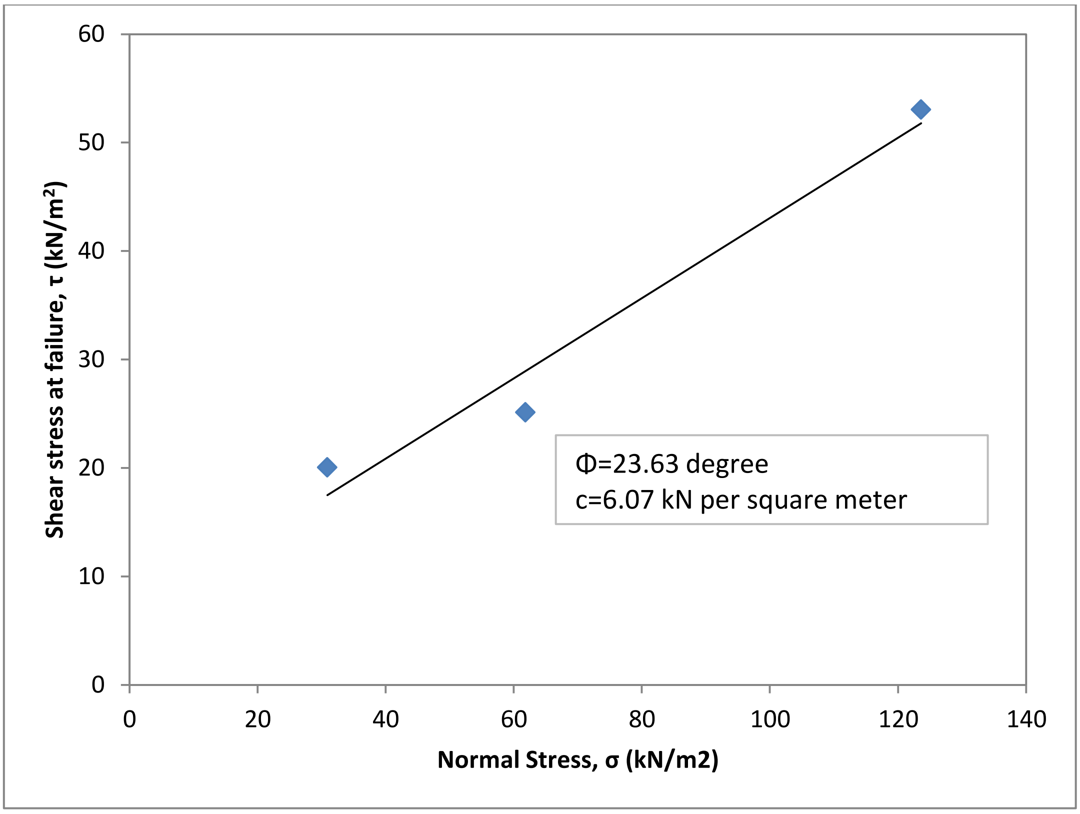

Figure B.1: Relationship between shear stress at failure and normal stress for samples with no lime and cement and for tests after no curing

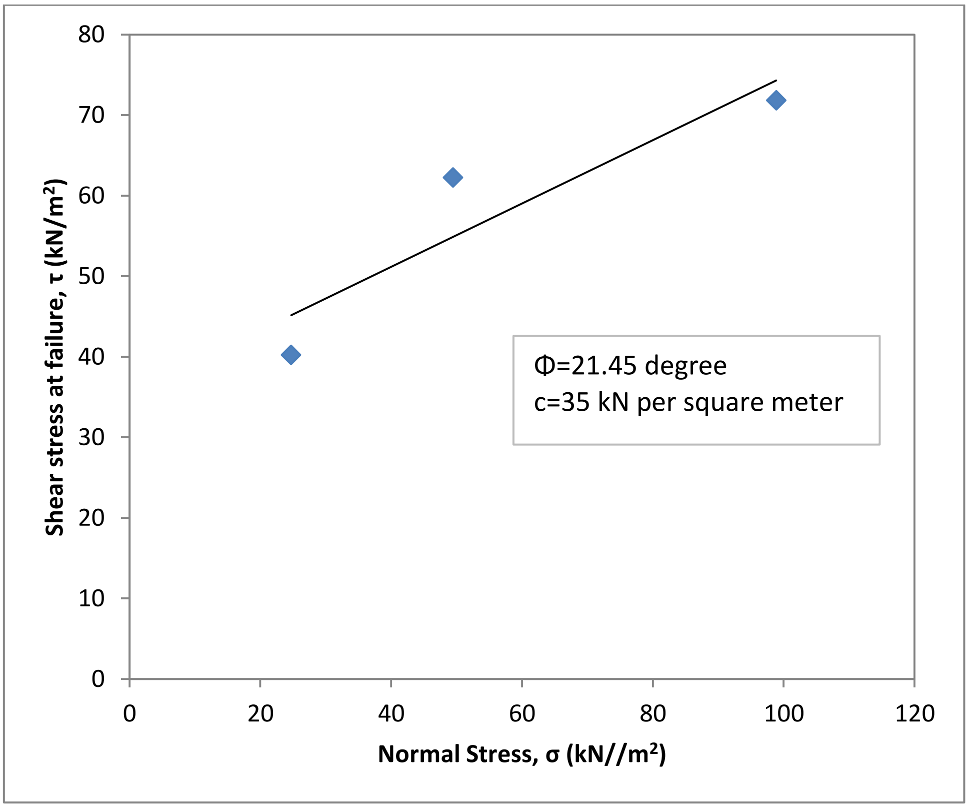

Figure B.2: Relationship between shear stress at failure and normal stress for samples with 5% lime and for tests after curing for 7 days

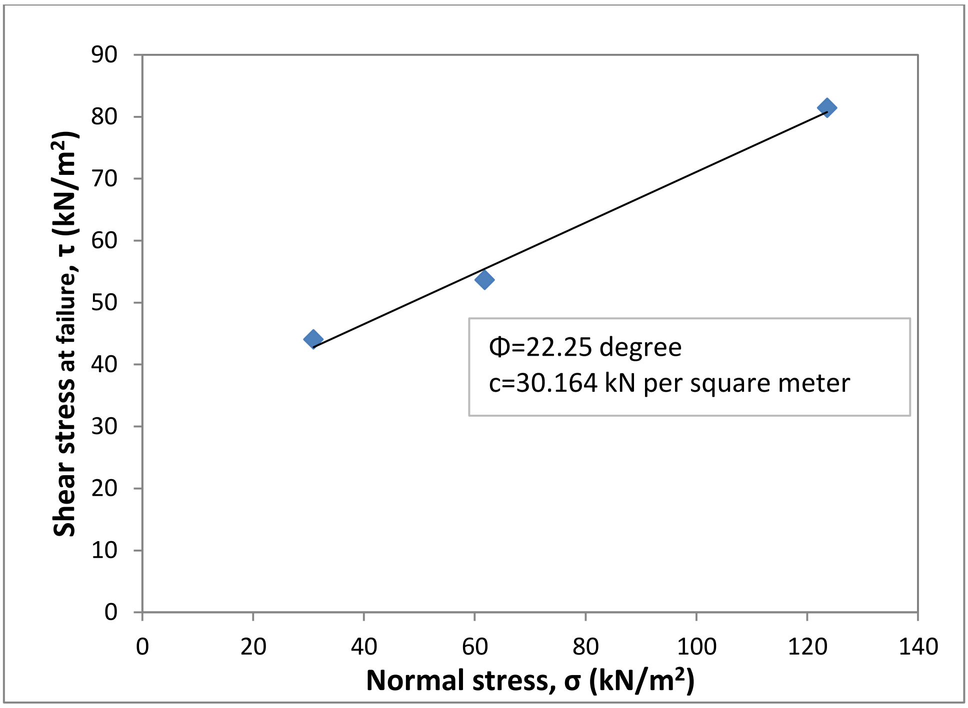

Figure B.3: Relationship between shear stress at failure and normal stress for samples with 5% lime and for tests after curing for 28 days

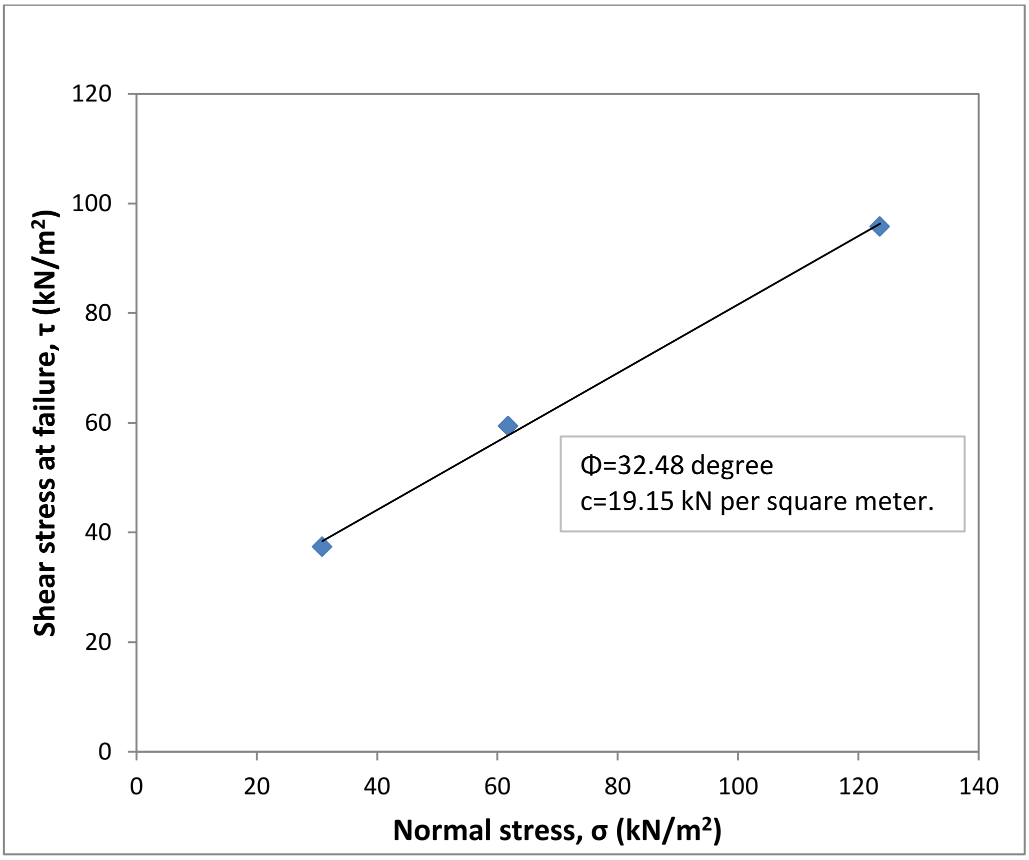

Figure B.4: Relationship between shear stress at failure and normal stress for samples with 5% lime and for tests after curing for 56 days

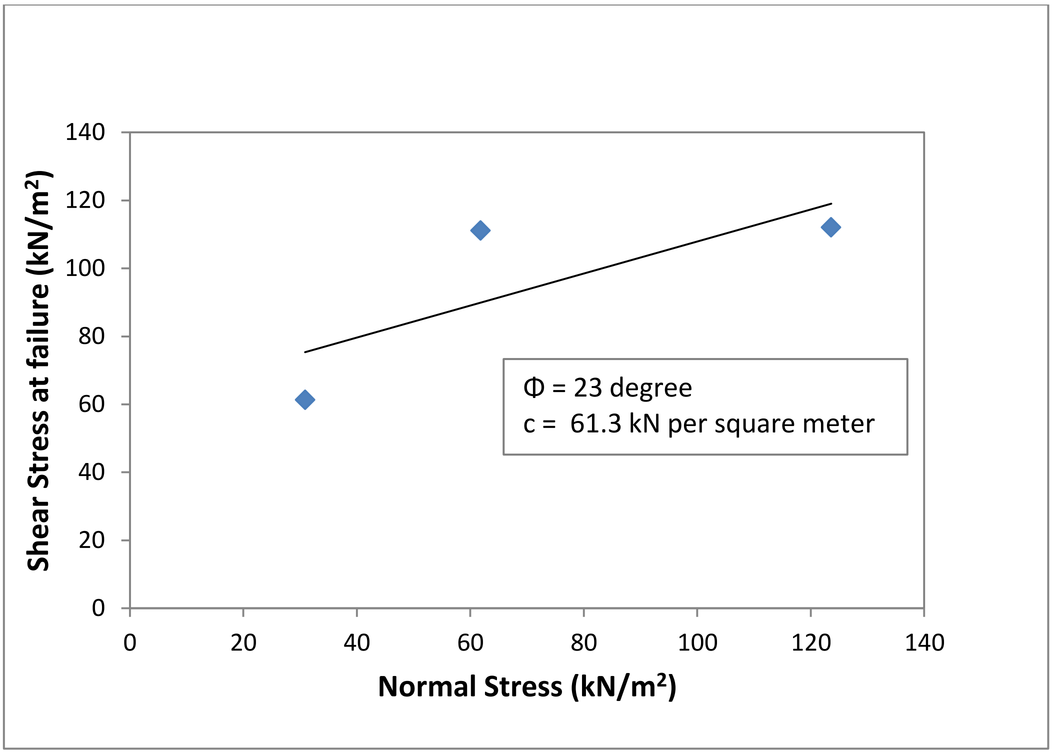

Figure B.5: Relationship between shear stress at failure and normal stress for samples with 5% cement and for tests after curing for 7 days

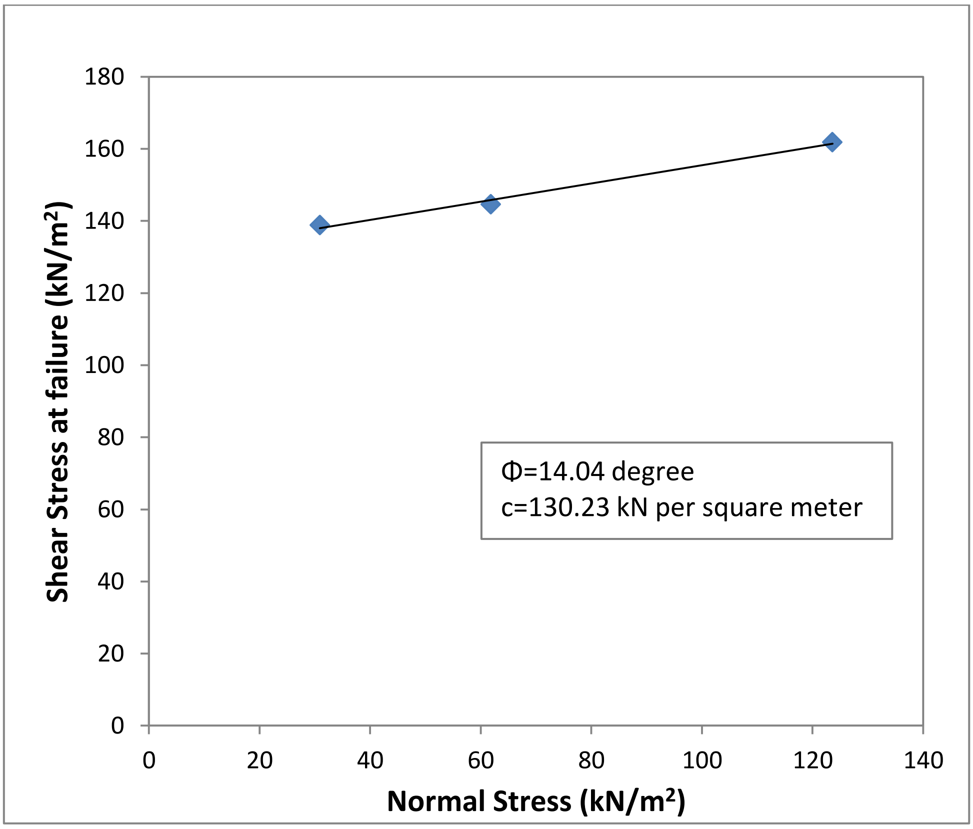

Figure B.6: Relationship between shear stress at failure and normal stress for samples with 5% cement and for tests after curing for 14 days

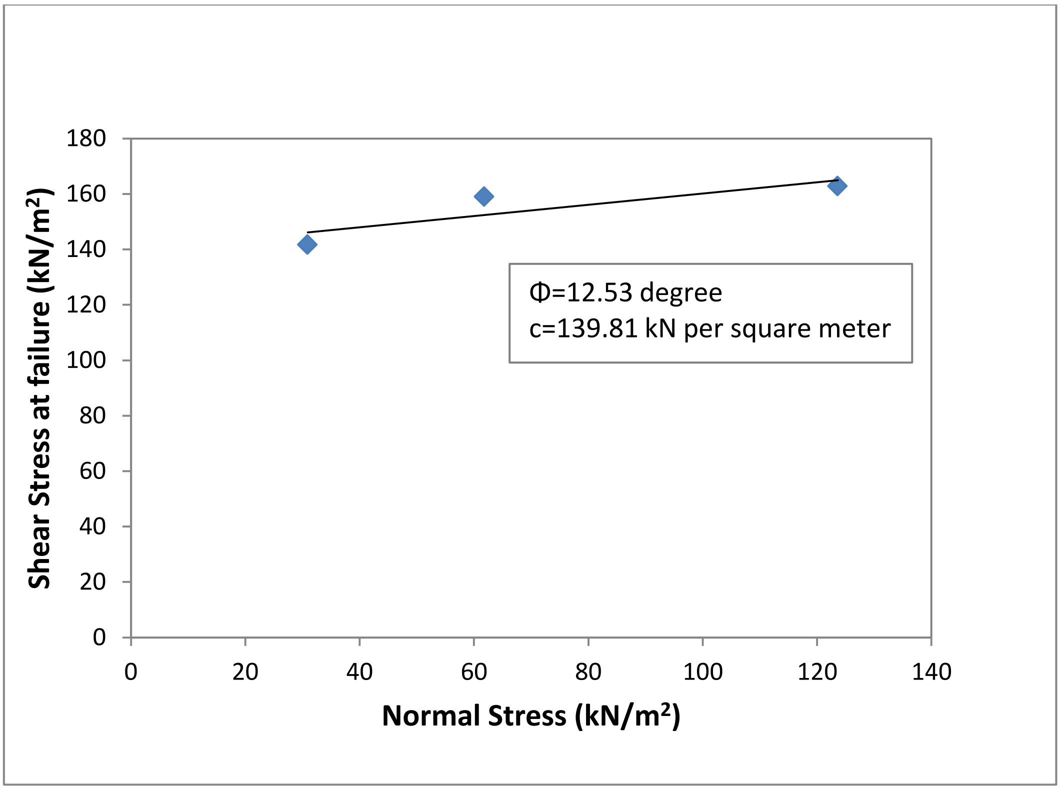

Figure B.7: Relationship between shear stress at failure and normal stress for samples with 5% cement and for tests after curing for 28 days

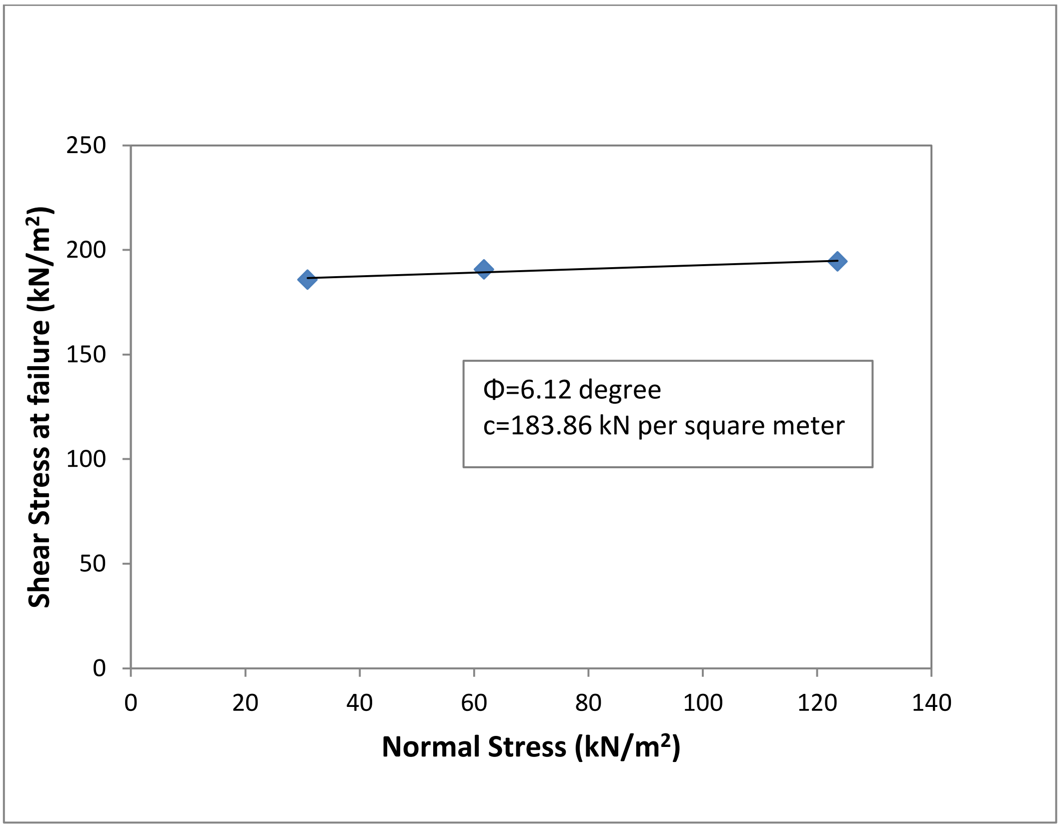

Figure B.8: Relationship between shear stress at failure and normal stress for samples with 5% cement and for tests after curing for 56 days

APPENDIX C

RELATIONSHIPS BETWEEN SHEAR DISPLACEMENTS AND CORRESPONDING CHANGES IN HEIGHTS OF THE SPECIMENS FOR NO CURING, NO ADMIXTURE AND FOR DIFFERENT NORMAL STRESSES.

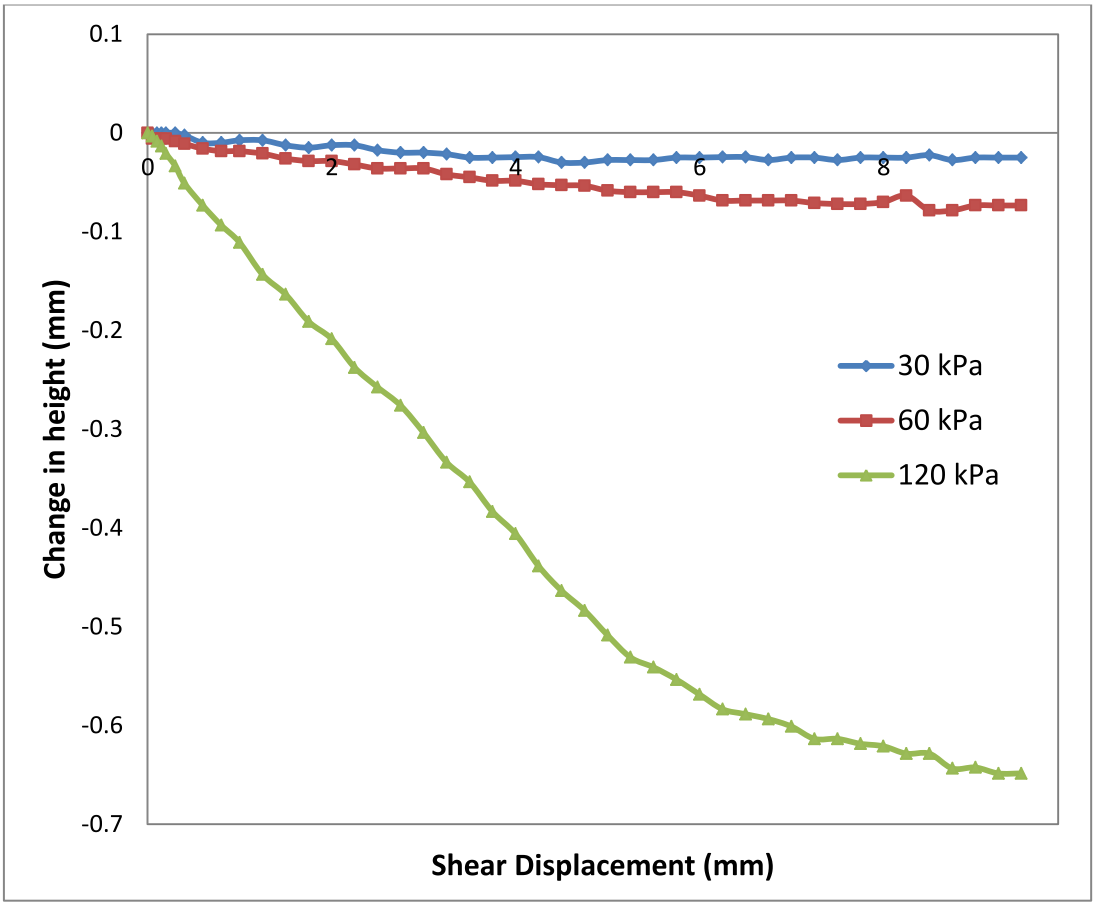

Figure C.1: Relationships between shear displacements and corresponding changes in heights of the specimens for no curing, for no admixture and for different normal stresses