

GEO-ENGINEERING PROPERTIES OF LIME TREATED PLASTIC SOILS

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By

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CERTIFICATE

This is to certify that the project entitled “*Geo-engineering properties of lime treated plastic soils*” submitted by *Subhashree Samantasinghar (Roll No.- 212CE1480)* in partial fulfillment of the requirements for the award of Master of Technology in Civil Engineering (Geotechnical engineering) during session 2012-14 at National Institute of Technology, Rourkela is an authentic work carried out by her under my supervision and guidance.

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I bow to the Devine power, who led me all through.

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ABSTRACT

For a long time, we are facing problems like failures of small and big structures. The biggest problem behind this is swelling soils. This is very unstable soil. Its property varies from hard to soft and dry to wet. It exhibits swelling and shrinkage with different water content. As a result, many structures usually face excessive settlement and differential movements, which causes damage to foundation systems and other structural elements. We are aware about this situation for a long time, but unable to make improvements due to absence of technologies till now.

Expansive soils are found in many parts of the world like Burma, South Africa, Western USA, Cuba, Spain, Russia and Indonesia, etc. In India it is found in Rajasthan, Tamilnadu, Madhya Pradesh, Maharashtra, Gujarat and Orissa.

Plastic soils undergo swelling and shrinkage causes severe distress and damage to the structure overlaying. Lime has been used as a soil stabilizer from Roman times. Through physico-chemical modifications, lime can control the plasticity, swelling and shrinkage of soil effectively. Also, lime can stabilize soil through cementation which increases strength and stiffness remarkably. This soil modification using lime depends on the type of soil and its mineralogy, lime content and compaction condition with curing period. The stabilizing effect of lime has been studied by a number of researchers. The minerals present in the soil has found to affect the engineering properties of stabilized material and the optimum lime content.

In this work the plasticity characteristics, swell shrinkage properties, compaction characteristics and strength properties of plastic soils with a wide plasticity range treated with lime has been evaluated experimentally. Based on the unconfined compressive strength the optimum lime content of soils has been found out. For this purpose, commercial high plastic clay (i.e. bentonite) having liquid limit of 340% is mixed with different proportions to a residual soil having liquid limit 34% and four different soils were synthesized over a wide range of plasticity. The physical properties of raw soils were found out. The four soils were amended with different lime content and the index properties as well as engineering properties have been studied by conducting relevant experiments conforming to Indian standard code of practice. Emphasis was

made to understand the plasticity, swell and shrink characteristics, compaction characteristics and strength properties of virgin soils and soils amended with different proportions of lime. Further, the effect of curing period and molding moisture content on unconfined compressive strength of soils treated with different proportions of lime were evaluated.

Based on the experimental results it is observed that the liquid limit and plasticity reduces as lime content increases, swell and shrink characteristics decreases with lime content. Also, lime has a significant effect on strength. Strength of soil increases with lime, but beyond a certain point, even after increasing lime there is a reduction in strength. High compressive strength is achieved at the longer curing period. The compressive strength is more on the dry side of optimum moisture content than wet side, for untreated soil, but when lime is added to the plastic soils the compressive strength changes depending upon the lime content, moisture content and curing period.

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NOTATIONS AND ABBREVIATIONS

Cc	Co-efficient of curvature
CH	Clay of high plasticity
CI	Clay of medium plasticity
CL	Clay of low plasticity
Cu	Co-efficient of uniformity
ES	Expansive soil
FSI	Free swell index
G	Specific gravity
I _P	Plasticity index
I _S	Shrinkage index
LSI	Linear shrinkage
MDD	Maximum dry density
OMC	Optimum moisture content
RS	Residual soil
UCS	Unconfined compressive strength
W _L	Liquid limit
W _P	Plastic limit

CHAPTER 1

INTRODUCTION

1.1 INTRODUCTION

Soil is one of the most commonly encountered materials in civil engineering. All the structures except some, which are founded on solid rock, rest ultimately on soil. Geotechnical engineers all over the world face huge issues, when structures founded on the soil which is expansive in nature. This expansiveness is imparted to such soils when they contain clay minerals like montmorillonite, illite, kaolinite etc. in considerable amount. Due to the clay minerals, the swelling soils expand on wetting and subjected to shrinkage on drying. These soils are commonly unsaturated.

The problem of instability of structures made on such soil is mainly due to lifting up of the structures on heaving of soil mass under the foundation on saturation during rainy season and settlement as a result of shrinkage during summer season. Due to this cavity formed, leading to loss of contact between the soil and structures at some points. This successively results in splitting of structure and failure due to loss of shear strength or unequal settlement.

On the contrary, during rainy season the foundation soil swells on imbibitions of water and it is restrained by the foundation. As a result, an upward swelling pressure is exerted by the soil on the foundation. As this pressure is not uniform everywhere, the net downward pressure becomes uneven. Similarly, during the summer season the soil shrinks and this phenomena is not uniform throughout the soil below the foundation. This additionally results unequal settlement, leading to progressive failure of structures.

Investigation into the properties of lime treated expansive soils would assess the suitability of using lime as stabilizer to reduce swelling of expansive soils. This paper presents the effect of lime stabilization on engineering properties of a lime treat

d expansive soil. Soil engineering tests like Atterberg limits, differential free swell test, shrinkage test, strength test were conducted on virgin soil and soil-lime mixture.

1.2 EXPANSIVE SOILS

Expansive soils creates greatest hazard in arid regions. Expansive soils contain clays and fine silts swells and shrinks as their moisture content changes. These expansive soils created problems for the structures, mainly lightweight structures and the structures most commonly damaged are small buildings, roadways, pipelines and irrigation canals.

The moisture in the soils shows variations due to climatic changes, change in water table, watering of gardens and lawn, presence of trees and shrubs and leakage from water and drainage pipes.

1.2.1 Origin and occurrence of expansive soils

Clay mineral is that the key component that passes on swelling characteristics to any standard non-swelling soils. Montmorillonite has the utmost swelling potential among many varieties of clay minerals. The origin of such soil is sub aqueous decomposition of blast rocks, or weathering in situ formation of vital clay mineral takes place under alkaline environments. If there is an adequate supply of magnesium of ferric or ferrous oxides and alkaline environments along with adequate silica and aluminum attributable to weathering condition, it will favor the formation of Montmorillonite. The depth is not deeper at the place of formation for clay with the parent rock beneath. The alluvium deposits may be much deeper in low lying areas, where these soils are transported and deposited.

1.2.2 Problems associated with expansive clay

For all types of engineering constructions over expansive soils are not appropriate since they create issues. However, owing to persistence of those kinds of soils in several parts of India, different irrigation projects are required to be developed on these deposits. Furthermore, examples of similar issues have also been recognized in many other parts of the globe. Structures found in these soils are subjected to differential deflections that cause distress on swelling clays and produce dangerous damage to the buildings and superstructures. Reduction of water content causes shrinkage by the evaporation of vegetation, whereas a continuous increase in moisture

content causes heave in expansive soil like bentonite. The rise of the water table has got a notable impact on the movement of foundation on expansive soils

Whether a mass of clay has been compacted by nature or by artificial means, it is unlikely to expand as much vertically as horizontally. Experiments have shown that the compacted clay soils exhibit greater value of swelling in the horizontal direction than in vertical direction. So magnitude of difference in swelling soil being very low, the vertical swelling pressure is calculated to uplift forces on structure. Due to evaporation in dry season the surface gets reduced surrounding a building which was erected on clayey layer, but a very little evaporation under the structure. Thus there will be uneven settlement at plinth level, causing danger to structures.

If a structure is constructed around dry season with foundation laying inside the unstable zone, the foundation base experiences swelling pressure because of partly saturated soil starts intake of water during rainy season. This swelling pressure has developed as a result of its constraint offered by the foundation for free swelling. If imposed pressure on the foundation is less than the swelling pressure by its structure, the structure is probably going to induce lifted up locally, which will result in cracks within the superstructure and on the other hand if imposed bearing pressure is larger than the swelling pressure than structure will be safe. If a structure is constructed during the wet season, it will experience settlement, depending on the dry season, whether the bearing pressure is low or high. The imposed bearing pressure ought to be within the allowable bearing pressure within the wet season for the soil. Thus, for higher practice the structure has to be constructed during dry season and should be finished before the wet season.

1.2.3 Clay mineralogy

Clay minerals, an important group of sheet silicate family of minerals, which are distinguished by layered structures composed of polymeric sheets of SiO_4 tetrahedra linked to sheets of $(\text{Al}, \text{Mg}, \text{Fe})(\text{O}, \text{OH})_6$ octahedra. The geotechnical importance of clay minerals stems from their ubiquity in soils and sediments, high specific surface area, and ion exchange capacities. Clay minerals tend to dominate the surface chemistry of soils and sediments. The clay minerals form a difficult group of minerals to study due to their small size, variable structural composition, and relative slow kinetics of formation and alteration.

1.2.3.1 Structure of clay minerals

Clay minerals are primarily the end product produced by chemical weathering of feldspathic rock. Chemically, these minerals are basically hydrous aluminum silicates, although often the aluminum atoms are replaced with atoms of 5other elements, such as magnesium, iron, potassium, or sodium (Duncan, 1992). The atomic structure of a clay mineral is very complex, and consists of a variety of combinations and arrangements of two basic building blocks called the silica tetrahedron and the alumina octahedron as shown in Figure 1.1.

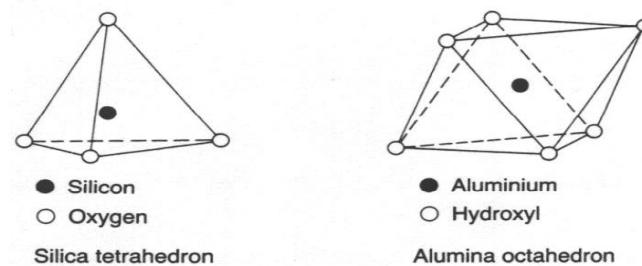


Figure 1.1 Basic units of clay minerals (Craig, 1993)

The various building blocks those make a clay mineral are arranged in orderly sheets are shown in Figure 1.2 (a) and Figure 2 (b). The particular arrangement and chemical compositions of these building blocks determines the type of clay mineral and its general characteristics.

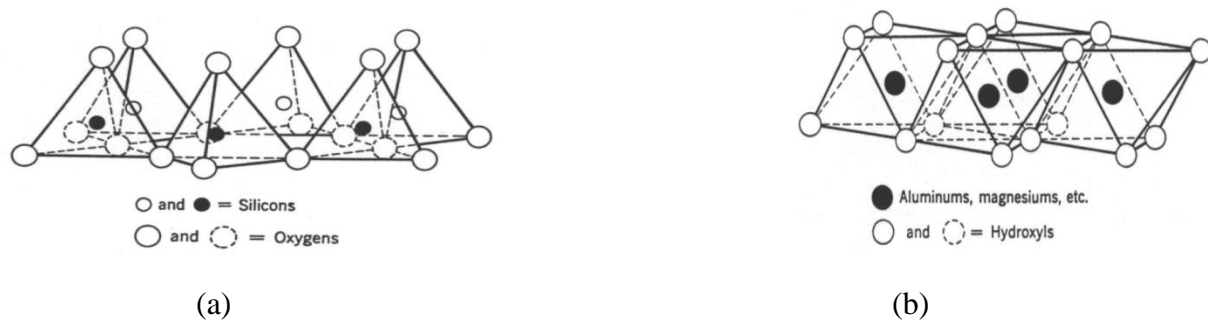


Figure 1.2 (a) Silica sheet and (b) Octahedral sheet (Mitchell, 1993)

1.2.3.2 Major clay groups

Clay minerals are grouped according to chemistry, and its interaction with water. The three main groups of clay are:

- a) kaolinite,
- b) illite, and
- c) montmorillonite.

a) Kaolinite

The kaolinite group of clay in which the mineral kaolinite $[Al_4 Si_4 O_{10} (OH)_8]$ is the principal member, are the most common of all clays. Figure 1.3 shows kaolinite mineral composed of two sheets, one consisting of silica tetrahedrons and the other one consisting alumina octahedrons. These sheets are very strongly bonded together. Kaolinite, therefore, is very stable and it has little tendency to change in volume when exposed to water or to drought. Kaolinite contains no interlayer water because of the way the sheets fit together. However, it has the ability to absorb sufficient water to develop plasticity.

The lateral dimensions of kaolinite platelets are about 1000-20,000 Å with thickness of 100-1000 Å. The specific surface of montmorillonite is about $15m^2/g$.

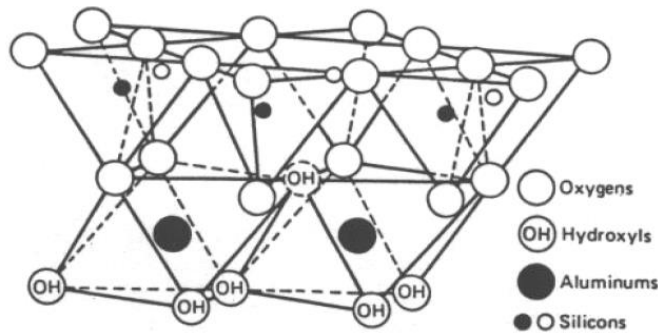


Figure 1.3 Structure of Kaolinite (Mitchell, 1993)

b) Illite

The illite group of clays does not have a principal mineral and this name refers to a group of mica like clay minerals. The basic structural unit of illite clay is composed of two silica tetrahedral sheets with a central octahedral sheet as shown in Figure 1.4. where potassium is the primary element within the central sheet. The negative charge to balance the potassium ions comes from the substitution of aluminium for some silicon in tetrahedral sheets. Illite particles have lateral dimensions of 1000-5000 Å and thickness of 50-500 Å. The specific surface of montmorillonite is about $90m^2/g$.

Illite shows more plasticity than kaolinite, and it has little tendency to change in volume when exposed to a change in moisture content unless there is a deficiency in potassium, in which case the illite particle will exhibit an increased tendency for volume change.

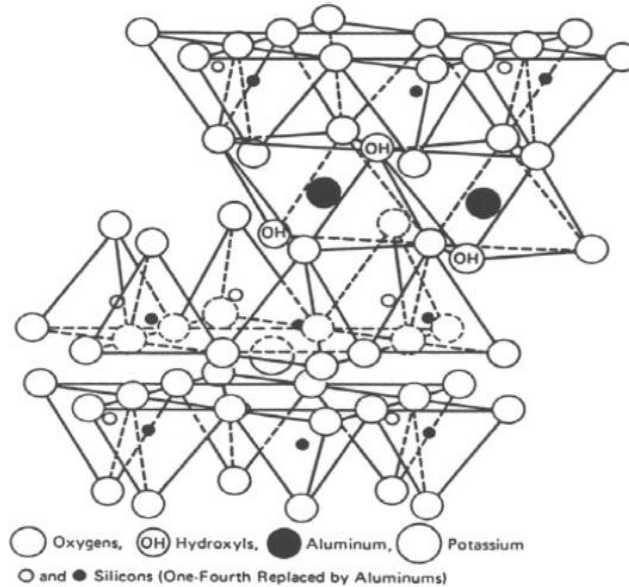


Figure 1.4 Structure of Illite (Mitchell, 1993)

c) Montmorillonite

Montmorillonite $[\text{Si}_8 \text{Al}_4 \text{O}_{20} (\text{OH})_4 \cdot n\text{H}_2\text{O}]$ is clay mineral group which has expansive structures, and also the name of the principal mineral of the clay group. The structure of montmorillonite consists of an alumina sheet placed between two silica sheets form a weak bond, three sheet layers and illustrated in Figure 1.5. This mineral exhibits considerable variation in characteristics because of the interchange between elements within each sheet. Considering iron or aluminum, for example, may replace the aluminum in the alumina sheet, and aluminum replaces some of the silicon in the silica sheet. Montmorillonite has lateral dimension of 1000-5000 Å with thickness of 10-50 Å. The specific surface of montmorillonite is about $800\text{m}^2/\text{g}$.

This mineral shows highly undesirable characteristics by undergoing considerable change in volume when soil comes in contact with moisture or when water goes from the soil mass. This characteristic may lead to the very serious issue of heaving or of settlement.

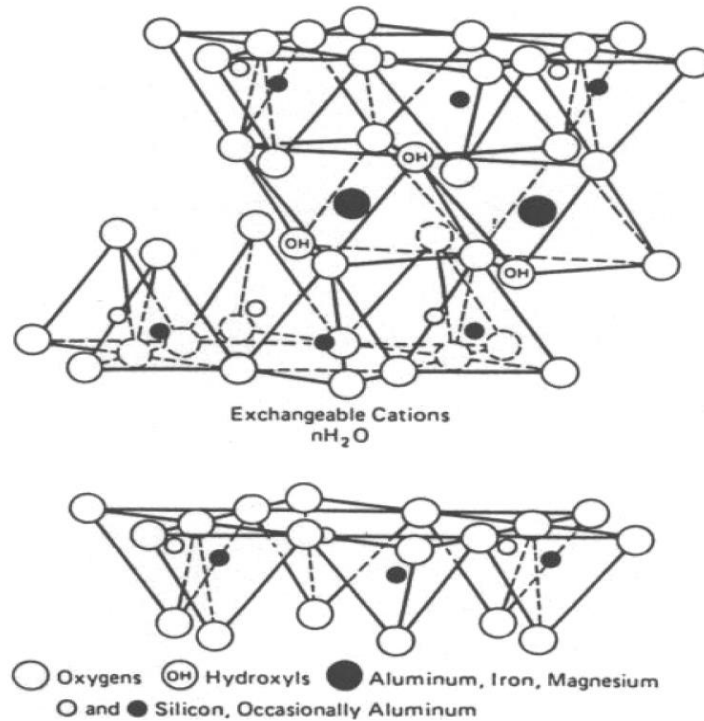


Figure 1.5 Structure of Montmorillonite (Mitchell, 1993)

1.3 LIME

Lime is the burned byproduct of limestone. Lime is the oldest developed material used for construction works, being used by human for more than 2000 years. Also, Lime has been used by Romans for road construction. Stabilization of soils with lime is widely used in various construction works such as foundation base, highways, railways, airports, embankments, slope protection, canal lining etc. Lime is used vastly for stabilizing soil as it is economic, easy construction as well as technology is simple that gives an attraction for the engineers.

Calcium oxide (CaO), usually known as quicklime and it is a widely used chemical compound. At room temperature, calcium oxide is white, caustic and alkaline crystalline solid. As a commercial product, lime typically contains magnesium oxide, silicon oxide and a small amount of aluminum oxide and iron oxide. By calcinations of limestone, Lime is produced in a lime kiln at temperatures higher than 1,000°C. Calcium carbonate (CaCO₃) is converted into calcium oxide (CaO) and carbon dioxide (CO₂). Active calcium oxide is very reactive. In finely ground burnt lime a high level (80 to 90%) of calcium oxide guarantees sensible stabilization reaction

within the soil, favorable water reduction in the soil and an increase in temperature upon slaking.

Lime can be used in the form of quicklime, hydrated lime, or lime slurry to stabilize the soil. Quicklime is manufactured chemically by transforming calcium carbonate (limestone – CaCO_3) into calcium oxide. Hydrated lime is formed when quicklime reacts chemically with water. Hydrated lime reacts with clay Particles and transforms them into a strong cementious matrix permanently. Mostly the lime used for soil treatment is high calcium lime; it contains not more than 5 percent magnesium oxide or hydroxide. However, for some cases dolomitic lime is used, which contains 35 to 46 percent magnesium oxide or hydroxide. Although the magnesium fraction reacts more slowly than the calcium fraction, dolomitic lime perform well in soil stabilization.

Sometimes the term “lime” is used to describe agricultural lime that is generally finely ground limestone, a useful soil modification but not chemically active enough to lead to soil stabilization. “Lime” is additionally sometimes used to describe byproducts of the lime manufacturing process such as lime kiln dust, which although contain some reactive lime; generally have only a fraction of the oxide or hydroxide content of the manufactured product.

The performance of construction work depends on the underlying soils, if it is not stable creates problem to the structures. By adopting suitable construction techniques and design methods, lime treatment of the soils can make the unstable soil to stable soil. Lime can be used to modify the soil of wide range. The reactivity of lime to soil determines the mineralogical properties of soil. Generally, fine grained soils with plasticity index (more than 10) are considered good for lime stabilization. Also, soil containing organic material may need additional lime and special construction techniques adopted. Lime has many effects when mixed with soil:

- a) Soil drying,
- b) Soil modification, and
- c) Soil stabilization.

a) Soil drying

Soil drying is a process in which rapid decrease in soil moisture content occur due to the chemical reaction between water and quicklime. Also, soil drying may happen due to the addition of dry material into a moist soil.

b) Soil modification

Soil modification effects include: reduction in the plasticity of soil, increases optimum moisture content, decreases the maximum dry density, improves compatibility, reduces the soil capacity against swelling and shrinkage, and improved strength and stability after compaction. These effects usually take place within a short period of time after the lime is introduced, generally 1 to 48 hours and are more pronounced in soils with sizable clay content, however may or may not be permanent.

c) Soil stabilization

Lime stabilization occurs in soils containing an appropriate amount of clay and the mineralogy to produce long-term strength, permanent reduction in shrinkage, swelling and reduction in soil plasticity with adequate durability to resist the damaging effects of cyclic freezing and thawing and prolonged soaking.

1.4 LIME STABILIZATION

Stabilization using lime is a longtime practice to modify the characteristics of fine grained soil. The first field applications in the construction of highway and airfield pavements were reported in 1950-60. With the proven success of these attempts, the technique was extended as for large scale soil treatment using lime for stabilization of subgrades also as improvement of bearing capacity of foundations in the form of lime columns.

Lime stabilization occurs over a longer time period of curing. The effects of lime stabilization are usually measured after 28 days or longer. A soil that is lime treated additionally may experience the effects of soil drying and modification. When sufficient amount of lime is added to the soil, stabilization occurs. Stabilization is different than modification as a remarkable strength gain occurs by long-term (pozzolanic) reaction, forms calcium silicate hydrate as lime reacts with silicon and calcium aluminate hydrates produced by aluminates from the clay. This reaction starts quickly but continues for a longer time period, shows strength gain up to a great extent as long as the pH of water remains above 10.

With increased lime content liquid limit decreases due to suppression of diffuse water layer surrounding clay particles as electrolyte concentration increases. These results in increasing the viscosity give resistance against movement of interparticle which in turn increases plastic limit thus plasticity reduces. Due to long-term stabilization cementitious compounds like calcium silicate hydrate, calcium aluminium silicate hydrate is formed and improves strength and stiffness over time. This strength gain occurs up to a certain lime content, after which strength may reduce.

1.4.1 Stabilization mechanism

When lime is mixed with plastic soil in the presence of water, many reactions occur. Two immediate reactions: cation exchange, flocculation-agglomeration modifies plasticity and improves uncured strength and workability. Long-term reaction: pozzolanic reaction is mainly responsible for strength gain.

Lime is widely used to change the geo-engineering properties of plastic soils. Plastic soils are more capable of holding large water, negative charges surrounding the clay particles attract the cations (positive charges) and water dipoles. Thus, there will be an increase in the thickness of diffuse water layer surrounding the clay particles.

Pozzolanic reaction occurs depending upon the soil characteristics forms cementitious compound which improves strength and increases durability. Also, this reaction depends largely on curing period and temperature. The strength improvement is more for a longer time period. The reaction retards below a temperature below 13 to 16°C and accelerates at higher temperature. However, the overall strength of soil depends on the soil type, lime content, compaction condition, type of curing, curing period and curing temperature.

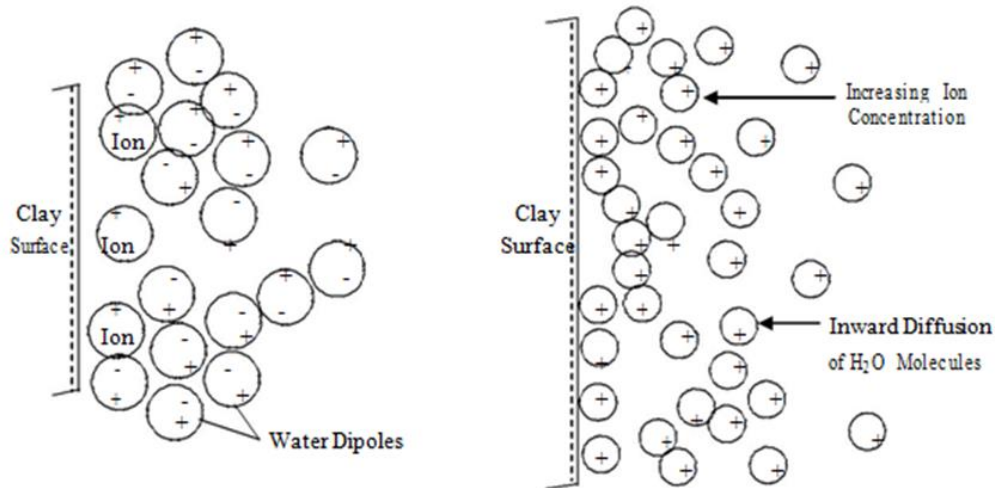


Figure 1.6 Formation of diffused water layer around clay particle (Little, 1987)

When lime reacts with atmospheric carbon dioxide, carbonation may occur. This produces insoluble carbonate which has negative influence on stabilization process and it can be minimized by avoiding the exposure of lime to air and rainfall.

1.4.2 Soil –lime reaction

Generally fine-grained soil shows cation exchange, flocculation-agglomeration when lime is added in presence of water. These reactions occur firstly immediately after mixing the lime to the soil. The following reactions occur when lime is added to the soil:

- a) Cation exchange
- b) Flocculation
- c) Pozzolanic
- d) Carboation

a) Cation Exchange

Cation exchange is an important reaction in soil-lime reactions and mainly causes changes occurring in the plasticity characteristics of soil. The cation replacement takes place in order of their replacing power



The lower valence cations are replaced by higher valence and smaller cations are replaced by higher cations having same valence. As sufficient lime is added to the soil, calcium cation from lime replaces weaker cations from the soil. This cation exchange reduces the thickness of diffuse water layer surrounding the clay particle thus clay particles comes closer to each other and flocculation-agglomeration occur. Cation exchange capacity depends on pH of pore water, clay mineral in the soil. Calcium hydroxide formed due to hydration of quick lime



It increases the electrolytic concentration and pH of the pore water and dissolves the silicates (SiO_2) and aluminates (Al_2O_3) from the clay particles. Na^+ and other cations adsorbed to the clay mineral surfaces are exchanged with Ca^{++} ions.

b) Flocculation

Due to flocculation-agglomeration, the change in texture occurs as clay particles bonded together and form large size particle. Because of cation exchange electrolyte concentration of water increases. Soil modification is the net result of cation exchange and flocculation-agglomeration:

1. Reduction of adsorbed water layer and stabilization occurs.
2. Internal friction increases giving higher strength.
3. More workability achieved as plastic soil texture changes to sand like material.

Untreated clays have a molecular structure similar to some polymers, and shows plastic properties. The structure can trap water between its molecular layers, causing volume and density changes. Figure 1.7 (a) shows the arrangement of clay particles in clay soil surrounded by water permitting them to slide easily and resulting low strength. When clay is treated with lime, calcium and magnesium atoms from lime have replaced sodium and hydrogen atoms and produces soil with very friable characteristics. Figure 1.7 (b) shows reduced water layer and realignment in the clay structure after modification with lime hence increasing strength significantly. Plastic soils generally having dispersed structural arrangement, but by adding lime to the soils flocculated type structure is achieved which makes the soil more workable.

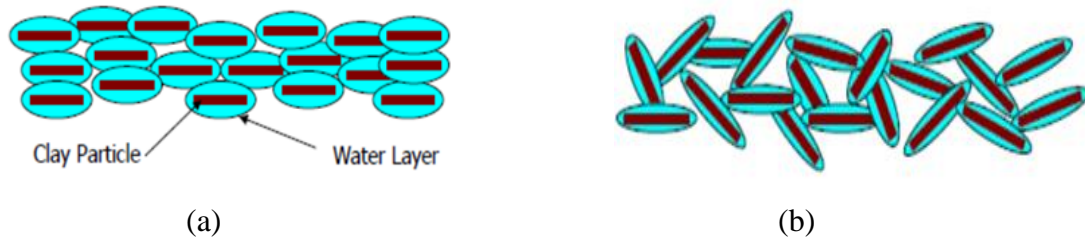


Figure 1.7 Molecular structure in (a) untreated clay (b) treated clay

c) Pozzolanic

The addition of lime to soil changes the properties of soil and this is mainly due to the formation of various compounds such as calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH) and micro fabric changes (Pozzolanic reaction).



In fine-grained soils sources of silica and alumina consists minerals like quartz, micas, feldspars and other silicate or alumino-silicate minerals, either in crystalline or amorphous nature. The clay minerals and amorphous materials in the soil are the important sources for most soils.

When an adequate amount of lime is added to the soil, the pH of soil–lime mixture is reached to 12.4. at these high pH levels, the solubility of silica and alumina are largely increased. Thus, as long as the calcium from lime remains in the soil and the pH remains high enough to maintain the solubility, the pozzolanic reaction continues.

The extent to which the soil–lime pozzolanic reaction proceeds is influenced primarily by natural soil properties. Soils showing a strength increase of greater than 345 kPa after a 28-day curing period at 23°C are considered as reactive, whereas those with strength increases lower than 345kPa are deemed non-reactive.

Various soil properties and characteristics influence the lime reactivity of a soil, which include the following:

1. Soil pH
2. Organic carbon content
3. Natural drainage
4. Excessive quantities of exchangeable sodium
5. Clay mineralogy
6. Degree of weathering
7. Presence of carbonates
8. Extractable iron
9. Silica-sesquioxide ratio
10. Silica-alumina ratio

d) Carbonation

Lime carbonation is an unwanted reaction that may also occur in soil–lime mixtures. In this reaction, lime reacts with carbon dioxide to form calcium carbonate instead of the cementitious CAHs and CSHs.

1.4.3 Applications and Benefits of lime stabilization

The influence of lime stabilization on engineering properties of soils can be classified as immediate and long-term modification. Immediate modification involves cation exchange and flocculation–agglomeration which modifies soil texture and reduces plasticity, swell and shrink properties. Long-term pozzolanic reaction increases the strength and durability.

Lime is used for various constructions including:

1. Highway construction, airfield pavement and roadways
2. Embankment construction
3. Rural roadways
4. Early & late season construction
5. Parking areas
6. Slope and levee repair

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

The term expansive soil indicates to soils, which has the tendency to swell when their moisture content is allowed to increase. The water may come from rain, flooding, leakage of water from sewer lines or from a reduction in surface evapotranspiration when an area is covered by a building or pavement. The term cracking soil is also used for these soils as they have the tendency to shrink and crack when the moisture is allowed to decrease. Soils containing the clay mineral montmorillonite generally show these properties (**Komine and Ogata, 1996; Rao and Triphaty, 2003**).

2.2 EXPANSIVE SOIL

Expansive soils are mostly found in arid and semi-arid regions of the world. The presence of montmorillonite clay in these soils imparts them swell-shrink potentials (Chen, 1988).

Expansive soils cover nearly 20% of the landmass in India and include almost the entire Deccan plateau, Western Madhya Pradesh, parts of Gujarat, Andhra Pradesh, Uttar Pradesh, Karnataka, and Maharashtra (**Gopal Ranjan and Rao, 1991**).

Expansive soils have been reported from several parts of the world, mainly in the arid or semi-arid regions of the tropical and temperate zones like Africa, Australia, India, United States and some regions of Canada. This never means that expansive soils do not exist elsewhere, because they can be found almost everywhere. In humid regions, ground water tables are present at shallow depths and moisture changes, which is responsible for changes in volume in soils under extended drought conditions (**Arnold, 1984; Shuai and Fredlund, 1998 and Wayne et al., 1984**).

In general, expansive soils have high plasticity, and are relatively stiff or dense. The expansive nature of the soil is most obvious near the ground surface where the profile is subjected to

seasonal and environmental changes. The pore water pressure is negative initially and the deposit is generally unsaturated. These soils contain some montmorillonite clay mineral. The expansive soil problem is dependent on the amount of monovalent cations absorbed to the clay mineral like sodium (**Fredlund and Rahardjo, 1993**).

The particles of clay have high negatively charged surfaces that attract free positively charged cations and water dipoles. Thus, a diffused water layer is formed around the clay particles and separates the clay particles which make the clay weak and unstable. This phenomenon depends upon the amount of water present, morphology and mineralogy of the clay. (**Little, 1987**).

2.2.1 Problems associated with expansive soil

Expansive soils have been called the hidden disaster as the damage cost is more than the combined damage from natural disaster such as earthquakes, and floods (**Jones and Holtz, 1973**).

Expansive soils are a worldwide issue that poses many challenges for civil engineers. These are considered a potential natural hazard, which may cause extensive damage to the structures if not properly treated. Such soils swell when giving an access to water and shrink after they dry out (**Al-Rawas et al., 2002**).

The issues with foundations on expansive soils have included heaving, cracking and break-up of pavements, roadways, building foundation, slab-on-grade members, channel and reservoir linings, irrigation systems, water lines, and sewer lines (**Cokca, 2001**).

It is reported that damage to the structures due to expansive soils has been the most costly natural hazard in some countries (in United States more than the cost of damage from flooding, hurricanes, tornados and earthquakes on an average annual basis) for years (**Kehew, 1995; Shuai and Fredlund, 1998**).

2.2.2 Identification of expansive soil

In engineering practice, identification of expansive soil is based on standard classification tests such as liquid limit and plasticity index. However, other laboratory tests such as mineralogical methods, cation exchange capacity, free swell, California Bearing Ratio, coefficient of linear

There are various correlations that are useful in identifying potentially expansive soils. It is also possible to identify them visually. Visual indications include (Wayne et. al., 1984):

1. Deep and wide shrinkage cracks occurring during dry periods
2. Soil is rock-hard when dry, but very sticky and soft when wet
3. Due to expansion of soil damages on the structures.

2.2.2.1 Standard classification tests

Atterberg limits are usually used for identifying swelling soils. The increase in liquid limit and plasticity index are considered as an indicator of increased swelling potential. A classification of swelling soil based on the liquid limit and plasticity index (O'neil and Poormoayed, 1980) as shown in Table 2.1.

Table 2.1 Expansive soil classification based on the liquid limit and plasticity index

W_L (%)	I_P (%)	Swelling potential
< 50	< 25	Low
50 - 60	25 -35	Marginal
<60	>35	High

IS: 2720 (Part III-1980) gives degree of expansion of a soil depending upon its differential free swell is given in table 2.2.

Table 2.2 Expansive soil classification based on differential free swell

FSI (%)	Degree of expansion
< 20	Low
20 – 35	Moderate
35 - 50	High
> 50	Very High

Classification of expansive soil based on linear shrinkage (**Altmeyer, 1955**) is given in Table 2.3.

Table 2.3 Expansive soil classification based on linear shrinkage

LSI (%)	Degree of expansion
< 5	Non-critical
5 - 8	Marginal
> 8	Critical

2.3 LIME STABILIZATION

Lime stabilization is the most widely used means of chemically transforming unstable soils into structurally sound construction foundation. The use of lime in stabilization creates change in engineering behavior in soil, including improved strength, improved resistance to fatigue and permanent deformation, reduced swelling, improved resilient properties and resistance to the damaging effects of moisture. The substantial improvements in these properties are seen in moderately to highly plastic soil, such as heavy clays (**Little et al., 2003**).

All types of clay minerals react with lime. The addition of lime has a significant effect on clay soils containing montmorillonite. In fact, expansive soils tend to react rapidly with lime, losing plasticity immediately (**Bell and Coulthard, 1990**). This is because expansive clay minerals such as montmorillonite exhibit a high cation exchange capacity.

Laboratory testing indicates that lime reacts with medium to fine grained soils to experience decreased plasticity and increased strength and workability (**Little, 1995**).

When lime is added to a clay soil, it must satisfy the affinity of the soil, i.e. ions are absorbed by clay minerals and are not available for pozzolanic reactions until this affinity is satisfied. This is referred to as lime fixation as the lime is fixed in the soil and is not available for the other reactions (**Hilt and Davidson, 1960**).

The lime fixation point corresponds with the point where there will be no change in plastic limit by further addition of lime to the soil. Hence, it is the optimum addition of lime to the soil that is needed for modification of soil. Beyond this point addition of lime will produce a cementitious

compound, which bind the flocculated particles and develops extra strength (**Al-Rawas et al., 2002; Bell, 1996; Mathew and Rao, 1997**).

Extended curing times promote pozzolanic reactions, and thereby produce additional cementing agents (**Bell, 1996**). The development of cementitious products from pozzolanic reactions should be a permanent and non-reversible phenomenon.

Pozzolanic reaction continues as pH remains high till sufficient amount of calcium present in lime to react with silica and alumina (**little, 1995**).

It has been observed that the amount of silica and alumina present in the clay influences strength gain i.e. for montmorillonite lime stabilization is more effective (**Lees, 1982**).

2.3.1 Stabilization Mechanism

When soil is stabilized with lime, chemical reactions occur between the calcium from lime and clay minerals from the particles of plastic soil. These chemical reactions may occur in two phases, with both immediate and long-term benefits.

Cation exchange is the first phase immediate reaction which shows changes in soil texture immediately. An exchange occurs between calcium from lime and cations from clay mineral, thus results a suppression in diffuse water layer surrounding the clay particle. This reduction makes clay particles come closer which in turn causes flocculation and agglomeration and a silt or sand like material formed.

The long-term reaction is the pozzolanic reaction which results in strength gain. The increase in pH of pore water occurs due to addition of lime, silica and alumina from clay released from clay when these become soluble. These react with calcium from lime and form a cementitious material. This strengthens gradually over time (**Eades and Grim, 1960**).

There are two major lime stabilization reactions in clay soils, one in the short-term and another in the long term (**Little, 1995**). The short term process involves ion exchange between calcium ions from lime and cations near the surface of clay particle, which occur only if the calcium ion have a higher charge or if they have a greater concentration than the cations near the clay particle surface. This ion exchange transforms the soil from a weak dispersed structure to a strong

flocculated structure. The long-term pozzolanic reactions begin as an increase in hydroxyl ions from the lime causes an increase in the pH of the soil water, which will begin to dissolve the silicate and aluminates sheets of the clay. As the silica and alumina released, they may combine with calcium from lime to form calcium silicate hydrates and calcium hydrates, which may bind the clay particles together. Lime has an optimal dosage rate for the maximum possible strength gain, which depends mainly on soil type and mineralogy.

Little,1995 describes only long-term pozzolanic reactions in the lime stabilization of kaolinite, while he states that short-term ion exchange must be completed before these same pozzolanic reactions occur in the lime stabilization of montmorillonite. Therefore, higher dosages of lime are required to reach the optimal dosage for clays containing more montmorillonite than kaolinite. Although clays containing montmorillonite may require more lime to reach their optimal dose, they may be able to achieve higher strengths, since montmorillonite may be more receptive to pozzolanic reactions due to its high specific surface area, which allows greater access to silica and/or alumina.

If the lime dosage is so high that the optimal dosage rate is exceeded, the strength gain may begin to slightly decrease due to a decrease in the dry density of the soil-lime mixture (Alexander et al. 1972).

2.3.2 Soil – Lime Reactions

The addition of lime to a soil initiates a two stage reaction. Short-term reactions show their effect right after the addition of lime, while long term reactions are accompanied by a period of time.

The short-term effect of the addition of lime to a clay soil is to cause flocculation and agglomeration of the clay particles, for cation exchange takes place between the metallic ions of the clay particles and the calcium ions of the lime. It is this process which is primarily responsible for the modification of the engineering properties of clay soils when they are treated with lime (Bell, 1993, 1996).

The long-term reactions are the pozzolanic reactions. The addition of lime to soil produces a highly alkaline environment, due to the OH⁻ anions from the hydration of lime, which gives rise

to a slow solution of silica and alumina from clay particles (**Kinuthia et al., 1999; Mathew and Rao, 1997**).

The cementation process develops from the reaction between calcium present in lime and dissolved silica and alumina from soil, forming calcium-silica-hydrates (CSH), calcium-alumino-hydrates (CAH), and calcium-alumino-silica-hydrates (CASH) (**Nalbantoglu and Tuncer, 2001**).

$\text{Ca}_2+ + 2(\text{OH})^- + \text{SiO}_2$ (Clay Silica) CSH

$\text{Ca}_2+ + 2(\text{OH})^- + \text{Al}_2\text{O}_3$ (Clay Alumina) CAH

$\text{Ca}_2+ + 2(\text{OH})^- + \text{SiO}_2$ (Clay Silica) + Al_2O_3 (Clay Alumina) CASH

These cementitious reaction products contribute to flocculation by bonding adjacent soil particles together and as curing occurs they strengthen the soil. Such pozzolanic reactions are time and temperature dependent, with strength developing gradually over a long period of time (in some instances this may take several years).

Apart from pozzolanic reactions, carbonation also results long-term strength increases for soils stabilized with lime. Carbonation occurs when lime reacts with atmospheric carbon dioxide produces a relatively insoluble calcium carbonate. This is advantageous after mixing; the slow process of carbonation and formation of cementitious products can lead to long-term strength increases (**Arman and Munfakh, 1970**).

However, prior to mixing, exposure of lime to air should be avoided through proper handling methods and expedited construction procedures in order to avoid premature carbonation of the lime (**Chou, 1987**).

Carbonation is the reaction of lime with carbon dioxide from the air to form weak cementing agents i.e. calcium carbonate. This is undesirable (**Bell, 1993**), becoming unstable, calcium-silica, calcium-alumino alumina, and calcium carbonate if the pH of the lime-stabilized soil drops sufficiently low. Carbonation reactions are harmful to the long-term strength and durability of the lime-stabilized soil. Using sufficient amount of lime (to provide enough alkalinity), compaction of the soil to high density and prompt placement after mixing lime with the soil (to minimize carbon dioxide penetration) can minimize potential carbonation problems.

2.3.3 Effects of lime stabilization on geotechnical properties of soil

A change in soil texture takes place when lime is mixed with clays. With the increase in lime content, there is an apparent reduction in clay content and a corresponding increase in percentage of coarse particles (**Chen, 1975**).

When lime is used to the soil, it improves the engineering properties, and these are due to two soil-lime reactions i.e. colloidal and pozzolanic reaction.

Colloidal reaction occurs immediately after addition of lime to the soil, which lowers the plasticity and swelling properties thus, increases workability (Thompson, 1968).

In pozzolanic reaction, pH value increases up to 12. Dissolution of minerals like silica and alumina from clay occurs and the dissolved substances react with the calcium from lime thus forms cementitious compounds which help in improving the strength (**Ingles, 1964; Yoder and Witczak, 1975**).

The strength improvement in the soil is due to the formation of cementitious compounds like calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH) when the soil is stabilized with lime (**Narasimha Rao and Rajesekaran, 1996**).

Treatment of fine-grained soil with lime generally results in an increased plasticity index of low-plasticity soils and the reduction of the plasticity of cohesive soils. The soils become friable and easy to work (**Mateous, 1964; Marks and Haliburton, 1970**).

The swelling characteristic of the soil is reduced and the shrinkage limit is increased when lime is added (**Mateous, 1964**).

Consistency limit decreases initially with increased lime content, but beyond 5% lime content there will be a negligible change. As major cementitious compounds formed are gyrolite, calcium silicate hydrate, and calcium aluminum silicate hydroxide hydrate, lime improves strength of the soil, but beyond certain limit, further addition of lime strength reduces due to excess formation of silica gel, a highly porous material (**Dash and Hussain, 2012**).

Liquid limits and plasticity of montmorillonite reduced with the addition of lime. Lime treatment increases the strength of clay materials. Montmorillonite clays respond much more rapidly to lime stabilization, and so exhibit earlier gains in strength (**Bell, 1996**).

The swelling phenomenon in clays is attributed predominantly to the presence of montmorillonite clay mineral in them. The addition of lime, quicklime (CaO) or hydrated lime [Ca (OH)₂] to expansive soil reduces swelling & swelling pressure and improves strength(**Kate, 2009**).

The change of expansive soil texture takes place due to the function of cation exchange when lime is added. Plastic limit increases, liquid limit reduces and the plasticity index decreases when lime is added (**Zhang &Xing, 2002**).

The addition of lime to highly plastic clay shows a reduction in shrinkage with increasing additive percentages. The linear shrinkage decreased from approximately 4 to 7 % with the addition of 5% lime (**Russell, Buhler, Cerato, 2007**).

Swelling phenomenon in clays is due to the presence of montmorillonite clay mineral. Bentonite clay contains sodium montmorillonite as its predominant mineral constituent(**Kate, Sunil Kumar, Bhorkar, 2012**).

With the increase in lime content, the maximum dry density of the soil-lime mixes decreases and optimum moisture content increases. The fall in density is more significant at lower percentages of lime. Time of curing does not produce much increase in strength upto 4% of lime content (**Arvind Kumar, BaljitsinghWalia, AsheetBjaj, 2007**).

Lime reduces the maximum dry density and increases the optimum moisture content of clayey soils at a given compactive effort (**Ladd et al. 1960, Marks and Haliburton 1970**).

Strength gain largely depends on the amount of silica and alumina available from the clay itself; thus, it has been found that lime stabilization is more effective for montmorillonite soils than for kaolinite soils (**Lees et. al, 1982**). In a soil-lime mixture the reaction occurring is between lime and clay minerals. Montmorillonite minerals were shown to require lime in excess of 4-6% of lime for the development of maximum strength (**Eades and Grim, 1960**).

The plasticity of soil increases with lime as there is an increase in pH. The high alkalinity of soil-lime mixtures (pH=12) can be considered as being responsible for the modification of soil behavior (**Clare and Cruchley, 1957**).

The high alkalinity changes the physico-chemical condition of the mineral surfaces (quick action), making the development of new cementitious compounds (**Rossi et al., 1983**), belong mainly to chemicals such as hydrated tricalcium aluminate, hydrated calcium silicate, hydrated carbon tetracalcic aluminate, calcium carbonate (calcite), and some unidentified aluminium silicates.

Some of the cementitious compounds were identified due to treatment of clay with lime using X-ray diffraction (XRD) studies to be CSHs, CAHs, hillebrandite, etc(**Heller and Taylor, 1956; Midgley, 1957**).

In the case of lateritic soils, the more active substances for lime are amorphous silica and alumina, are the principal components responsible for the reactivity of lateritic soil with lime (**Cabrera and Nwakanma, 1979; Queiroz de Carvalho, 1981, 1983**).

2.4 SCOPE OF PRESENT STUDY

Thus, through appraisal of the literature review, it is observed that several attempts have already been made by researchers to study the effect of additive on stabilization of expansive soil. However the researches on the plasticity swell and shrink behavior, strength aspect of soils covering wide ranges of plasticity upon lime stabilization is comparatively less. The experimental program undertaken investigates:

1. Plasticity characteristics of raw soil and lime treated soil
2. Swell and shrinkage properties of virgin soil and lime stabilized soil
3. Effect of lime on the compaction characteristics of soil
4. Effect of lime content on the strength aspect of lime treated soil.
5. The effect of curing period on unconfined compressive strength values of lime treated soil
6. The effect of variation in moisture content on unconfined compressive strength of soil

CHAPTER 3

EXPERIMENTAL WORK AND METHODOLOGY

3.1 INTRODUCTION

Lime stabilization of plastic soils reduces the problems associated with constructions faced by the expansiveness of these soils, mostly because of its swell and shrinkage property. So assessment of the behavior of plastic soil at different condition is required before commencing the construction activity. Even though adequate substitute for full scale field tests are not available, tests at laboratory scale provide a measure to control many of the variable encountered in practice. The trends and behavior pattern observed in the laboratory tests can be used in understanding the performance of the structures in the field and may be used in formulating mathematical relationship to predict the behavior of field structures. Details of material used, sample preparation and testing procedure adopted have been outlined in this chapter.

3.2 MATERIAL USED

An experimental program has been done taking highly expansive montmorillonite soil and a residual silty soil. Four types of plastic soils have been prepared from the raw soils. These artificial soils are added with different proportions of lime and a series of tests were conducted.

3.2.1 Soils

Primarily, an expansive soil and a residual soil, which represent the extreme soil type, are used in this study. The expansive soil is a commercially available bentonite. Its liquid limit and plastic limit are found to be 340% and 68%. The liquid limit and plastic limit of the residual soil are found to be 34% and 18% respectively. As per Indian standard soil classification system for fine grained soil as shown in Figure 3.1, the expansive soil is classified as clay with high plasticity and the used residual soil is classified as clay with low compressibility following Table 3.1.

Table 3.1 IS classification of soil

Low plasticity	$W_L < 35\%$
Intermediate plasticity	$W_L < 35\% < 50\%$
High plasticity	$W_L > 50\%$

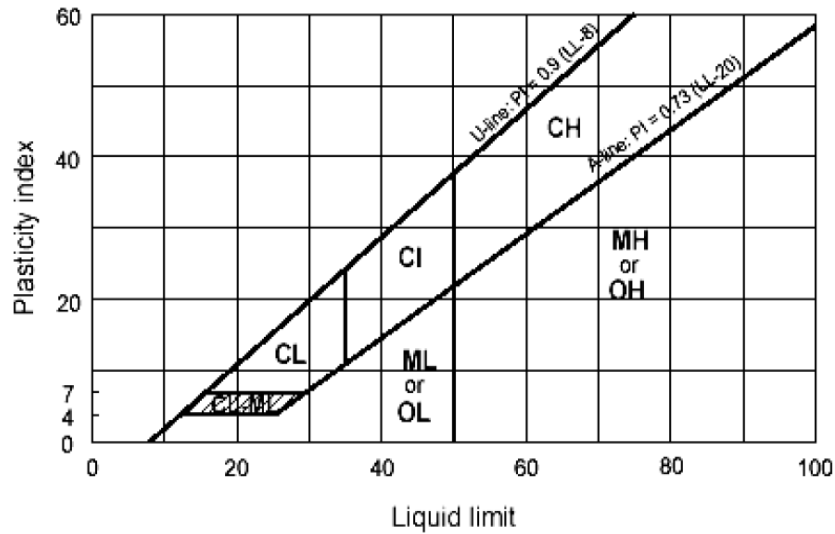


Figure 3.1 Casagrande's plasticity chart for laboratory classification of fine-grained soil

3.2.2 Lime

A laboratory hydrated lime [$\text{Ca}(\text{OH})_2$] was used as the stabilizing agent. The specific gravity of the lime used is found to be 2.4.

3.3 SAMPLE PREPARATION AND EXPERIMENTAL PROGRAM

To cover a wide range of plasticity, four different soil samples were constituted by mixing the expansive soil and residual soil in different proportions (i.e. 100% expansive soil, 50% expansive soil with 50% residual soil, 10% expansive soil with 90% residual soil, 100% residual soil). All of these soils amended with different percentage of lime (i.e. 0, 2, 5, 9, 13 percentage by weight of dry soil) and various engineering properties were studied by conducting the experiments.

Liquid limit tests are done using a mechanical liquid limit device, whereas the plastic limit is obtained by the thread rolling method as per IS : 2720 (Part 5)-1985. The free swell index is determined as per IS: 2720(part XL) -1977. The linear shrinkage has been determined as per IS: 2720 (Part 20) - 1992. The Optimum moisture content and dry density of soils were determined by performing the "standard proctor test" as per IS: 2720 (part VII) - 1965. The strength improvement of soils will be evaluated using the "unconfined compressive strength (UCS) tests"

as per IS: 2720 (part-10)-1991. Also, the change in plasticity, swell and shrink characteristics, strength by adding lime as additive has been studied by conducting geotechnical experiments on stabilized soils referring IS: 4332.

The overall testing program is conducted in two phases. In the first phase the index properties and geotechnical characteristics of the soils were studied by conducting laboratory experiments. In the second phase, soils are mixed with 2%, 5%, 9% and 13% of lime by dry weight of soil. The geotechnical properties of these lime stabilized soil samples were evaluated and compared with that of soil before stabilization. The overall testing program is outlined in Table 3.2.

Table 3.2 Test program for soils

Sl. No.	Test Method	Complying standards	Samples Variable		Parameters
			Lime content (%)	Curing period (Days)	
1.	Liquid and plastic limit tests	IS:2720(Part 5)-1985	0,2,5,9,13	Immediate	LL, PL, PI
2	Free swell test	IS:2720(Part XL)-1977	0,2,5,9,13	Immediate	FSI
3.	Linear shrinkage test	IS:2720(Part 20)-1992	0,2,5,9,13	Immediate	LSI
4.	Light compaction test	IS:2720(Part VII)-1980 and IS:4332(Part III)-1967	0,2,5,9,13	Immediate	OMC, MDD
5.	UCS test	IS:2720(Part-10)-1991 And IS:4332(Part V)-1970	0,2,5,9,13	0,7,14,28,60	UCS

The four soils were classified according to IS classification of soil following Casagrande's plasticity chart by knowing the liquid limit and plasticity index for the soils. The engineering properties of these plastic soils are tabulated in Table 3.3.

Table 3.3 Engineering properties of soils showing IS classification

Engineering properties	Values			
	100% ES	50%ES+50%RS	10%ES+90%RS	100%RS
Liquid limit, W_L (%)	340	144	44	34
Plastic limit, W_P (%)	68	36	23	18
Plasticity index, I_P (%)	272	108	21	16
IS classification	CH	CH	CI	CL
Free swell index, FSI (%)	712.5	475	81	10
Linear shrinkage index, LSI (%)	49	22	12	8
Optimum moisture content (%)	26.7	22.8	17.4	15.1
Maximum dry density, γ_d (g/cc)	1.39	1.58	1.75	1.79
Unconfined compressive strength, σ (kPa)	295	312	372	438

3.4 PHYSICAL PROPERTIES OF SOILS

The physical properties of both the soils were determined and are presented in the Table 3.4.

Table 3.4 Physical properties of soils

Physical Parameters	Value		Physical parameters	Value	
	Bentonite	Residual soil		Bentonite	Residual soil
Color	Grey	Brown	Shape	Platy	Sub-rounded
Silt and Clay (%)	100	86.94	Coefficient of uniformity, C_u	-	7.3
Fine Sand (%)	-	13.06	Coefficient of Curvature, C_c	-	2.2
Medium Sand (%)	-	-	Specific Gravity, G	2.79	2.6
Coarse Sand (%)	-	-	Plasticity index, I_P (%)	272	16

3.4.1 Morphological study of soils

The morphology of soils was studied by Scanning Electron Microscope which produces images of a sample by scanning it with a focused beam of electrons. The electrons interact with atoms in the sample, producing various signals that can be detected and that contain information about the sample's surface topography and composition. The SEM images of bentonite and the residual soil are as shown in figure 3.2. From this analysis, bentonite shows dispersed type fabric arrangement and the residual soil mainly contains individual sub-rounded particles.

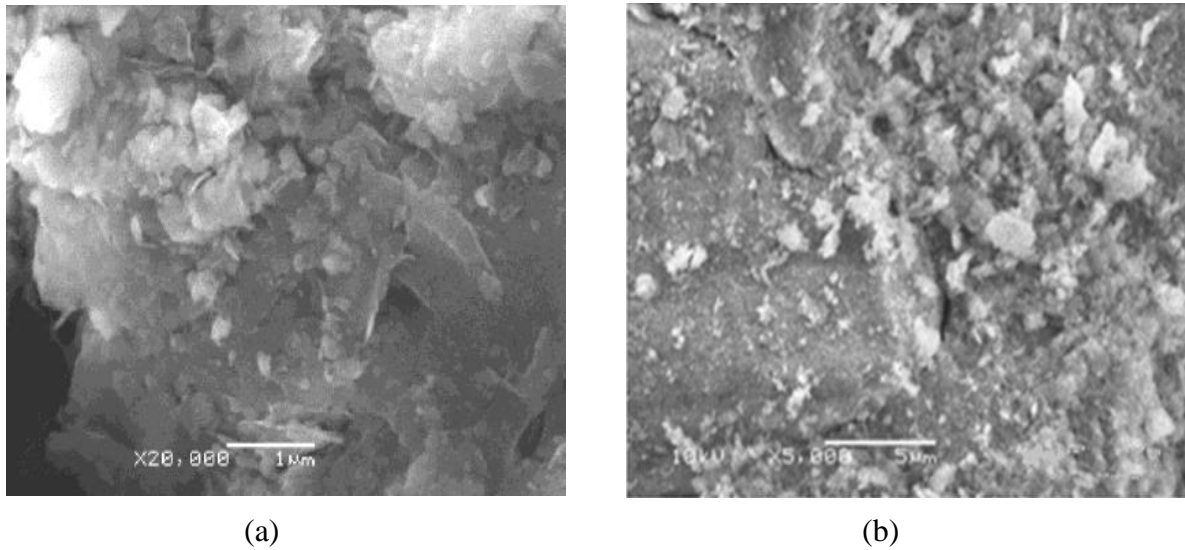


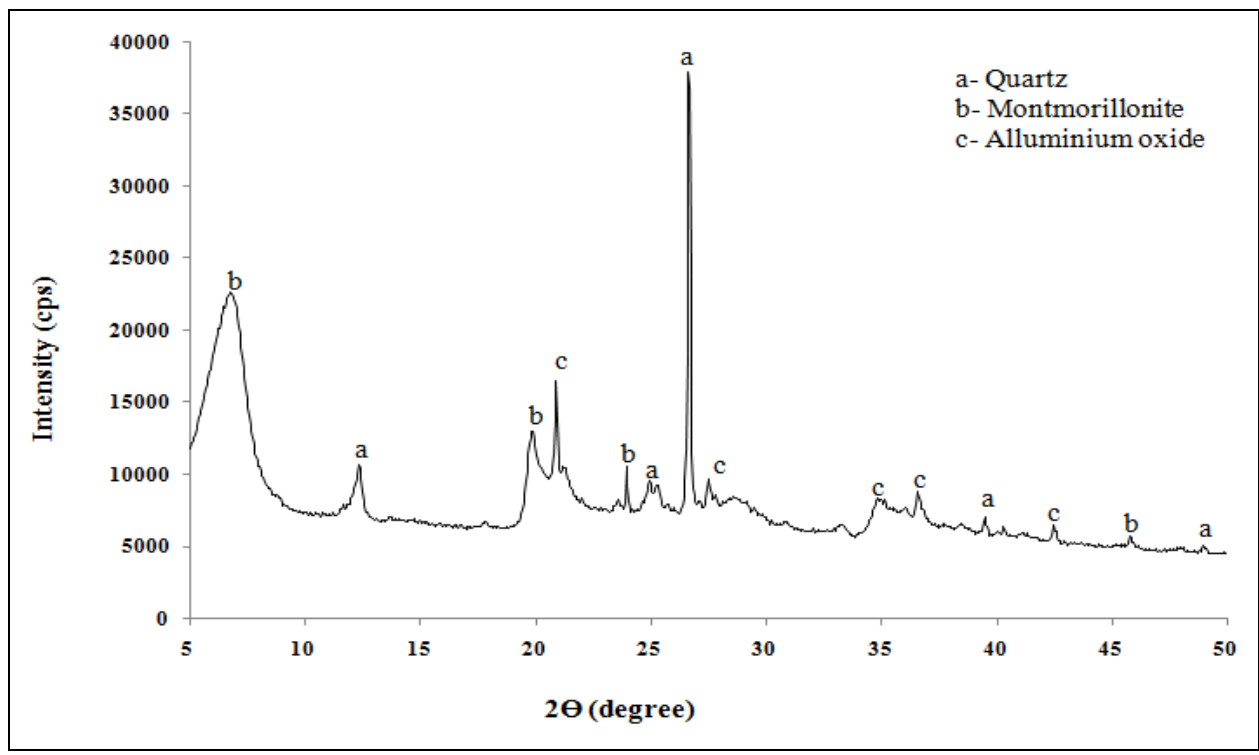
Figure 3.2 SEM of (a) Bentonite and (b) Residual soil

3.4.2 Mineralogical analysis of soils

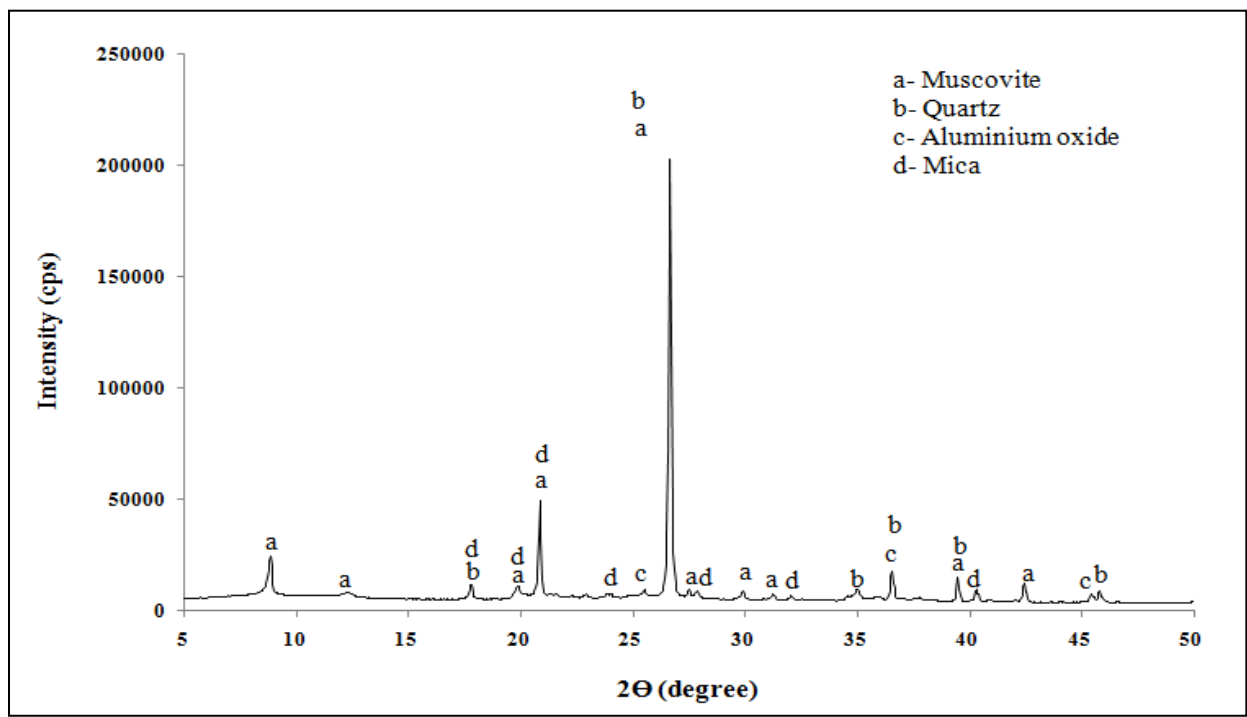
The mineralogical analysis of the materials has been done by XRD analysis using X-ray diffractometer which is based on the principle that beams of X-rays diffracted from crystals are characteristics for each clay mineral group. The XRD analysis results for both the soils and lime are shown in figure 3.3 and the mineralogical elements present in the materials are tabulated in the Table 3.5.

Table 3.5 Qualitative mineralogical composition of materials

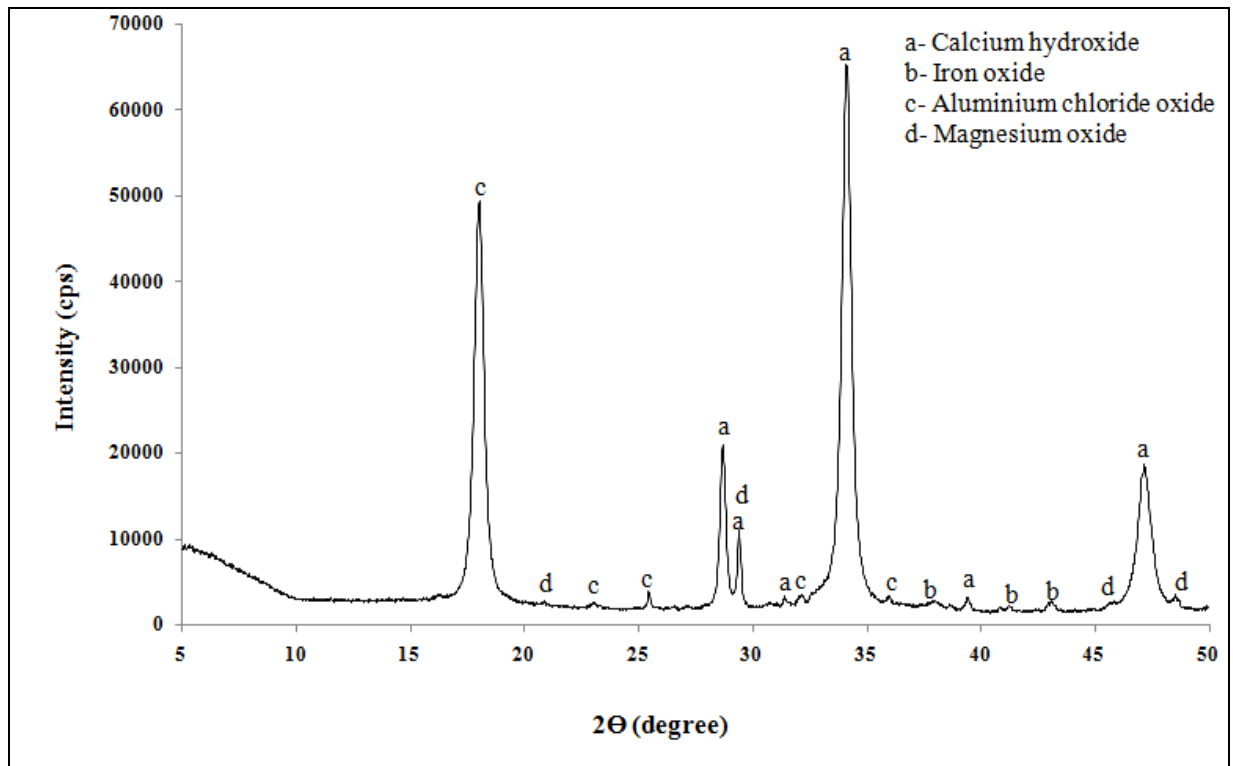
Bentonite	Residual soil	Lime
Quartz	Muscovite	Calcium hydroxide
Montmorillonite	Quartz	Iron oxide
Alluminium oxide	Alluminium oxide	Alluminium chloride oxide
-	Mica	Magnesium oxide



(a)



(b)



(c)

Figure 3.3 XRD analysis for (a) Bentonite, (b) Residual soil and (c) Lime

3.5 DETERMINATION OF INDEX PROPERTIES

3.5.1 Determination of Specific Gravity

The specific gravity of soils was determined according to IS: 2720 (Part III/sec 1)-1980. However, the Specific gravity of bentonite and lime were determined by using Le-Chatelier flask with Kerosene as the solvent. The specific gravity of soils and lime are listed in the Table 3.6.

Table 3.6 Specific Gravity of the materials

Materials	G
Bentonite	2.79
Residual soil	2.6
Lime	2.4

3.5.2 Determination of Grain Size Distribution

For determination of grain size distribution, the soils were passed through IS sieve size 75 μ . Sieve analysis for coarser particles (i.e. portion retained on 75 μ sieve) and sedimentation analysis for finer

particles (i.e. portion passing 75 μ sieve) were conducted.

The percentage of soil passing through 75 μ IS sieve for bentonite was found to be 100% and in case of residual soil was 86.94%. Hence, the grain size analysis is done by sedimentation analysis, using a density hydrometer as following IS: 2720 (part 4)-1985. The grain size distribution curve for bentonite and the residual soil are shown in Figure 3.4(a) and Figure 3.4(b) respectively.

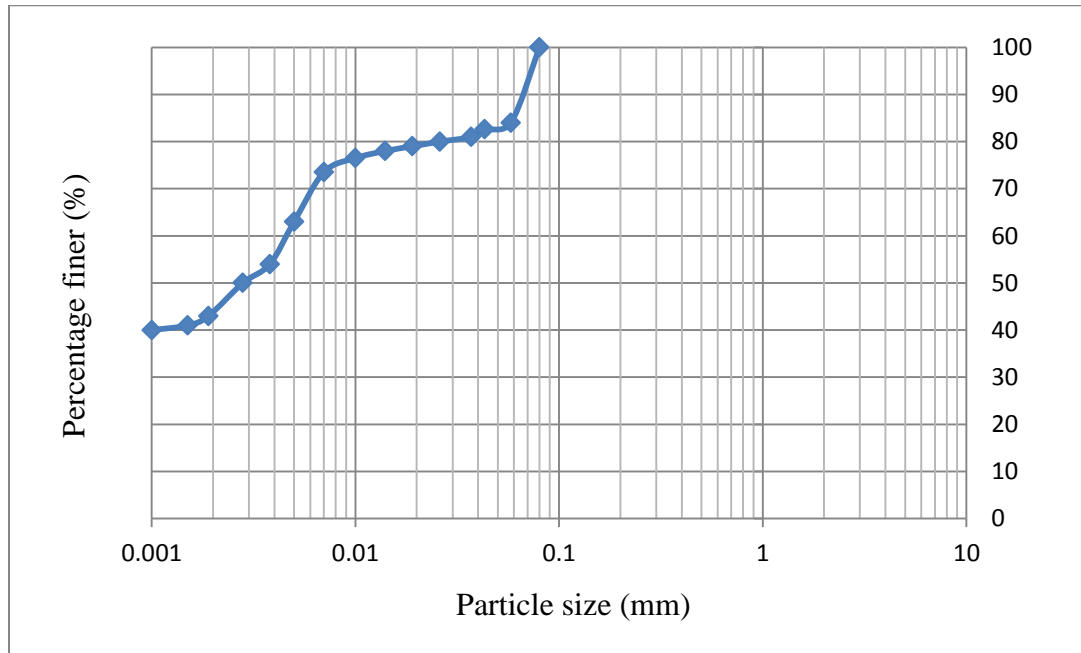


Figure 3.4(a) Grain size distribution curve for bentonite

The uniformity of the soil can be expressed by uniformity coefficient given by:

$$C_u = D_{60}/D_{10}$$

The general shape of particle size distribution curve is described by coefficient of curvature or coefficient of gradation given by:

$$C_c = (D_{30})^2 / (D_{60} \times D_{10})$$

Where,

D_{60} = Particle size such that 60% of the soil is finer than this size

D_{30} = Particle size such that 30% of the soil is finer than this size

D_{10} = Particle size corresponding to 10% finer

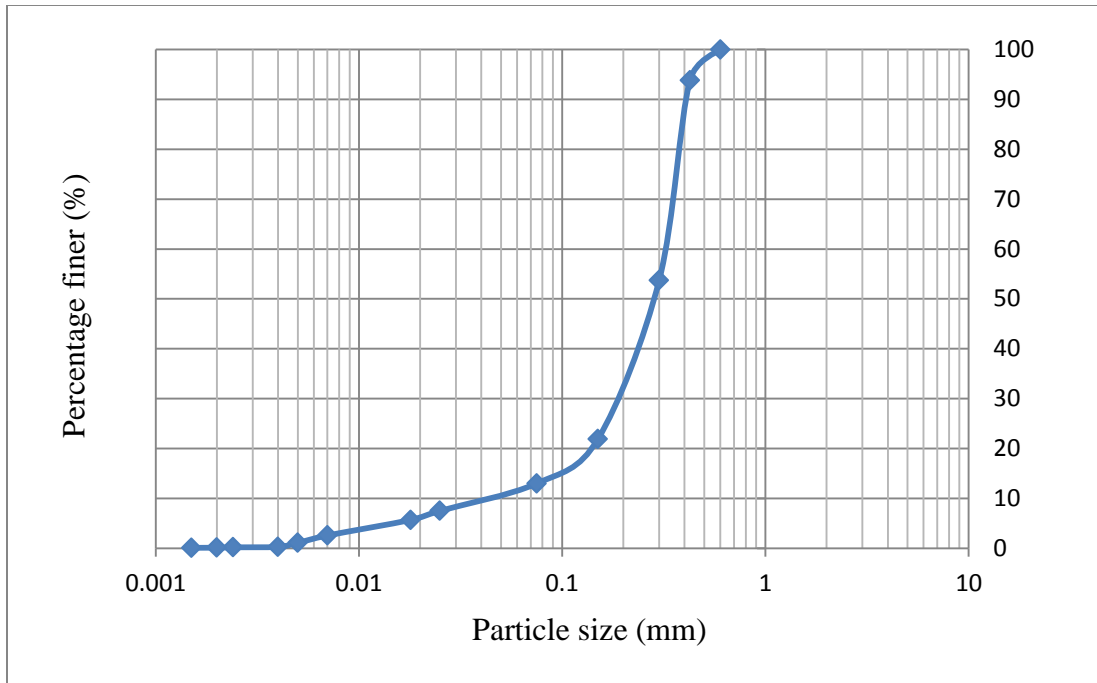


Figure 3.4 (b) Grain size distribution curve of residual soil

3.6 DETERMINATION OF ENGINEERING PROPERTIES

3.6.1 Determination of Atterberg Limits

The liquid limit and plastic limits of different soils are determined and the test procedure is shown in Figure 3.5

a) Determination of Liquid Limit

The liquid limit was determined in the laboratory by the help of Casagrande type liquid limit apparatus. About 120g of the specimen passes through 425 μ IS sieve was taken and mixed with thoroughly with water. This paste was left for sufficient time for maturing, for bentonite maturing time was 24. Hours. A portion of paste was placed at centre of cup and a groove was made by grooving tool which is IS: 9259-1979 and that for designates. The handle was turned at the rate of 2 revolutions per second, brass cup was raised and allowed to fall on a rubber base until the two parts of soil come in contact. The water content corresponding to 25 blows was taken as liquid limit. The liquid limit tests were conducted for the soils with different lime content. The liquid limit values for all lime-soil mixes are tabulated in Table 3.7.

Table 3.7 Liquid limit of soils for different lime content

Lime content (%)	Liquid limit (%)			
	100% ES	50%ES+50%RS	10%ES+90%RS	100%RS
0	340	144	44	34
2	149.2	96.6	46.38	36.3
5	121.8	90.85	47.59	37.8
9	116.1	88.4	47.1	40.1
13	122.7	90.4	50	44.5

b) Determination of Plastic Limit

Plastic limit test is done to determine the plastic limit of soil. The plastic limit of fine-grained soil is the water content of the soil below which it ceases to be plastic. It gets crumble when rolled into threads of 3mm diameter.

Table 3.8 Plastic limit of soils for different lime content

Lime content (%)	Plastic limit (%)			
	100% ES	50%ES+50%RS	10%ES+90%RS	100%RS
0	68	36	23	18
2	98	59	32	28
5	95	66	34	28
9	92	65	34	27
13	89	67	36	15



Figure 3.5 Liquid limit and plastic limit test for soil

c) Determination of Plasticity Index

Plasticity index is calculated from the relation

$$I_p = W_L - W_p$$

Where,

I_p = plasticity index in %,

W_L = liquid limit in %

W_p = plastic limit in %

The values of plasticity index for the soils are tabulated in Table 3.9.

Table 3.9 Plasticity index of soils for different lime content

Lime content (%)	Plasticity index (%)			
	100% ES	50%ES+50%RS	10%ES+90%RS	100%RS
0	272	108	21	16
2	51	38	14	9
5	27	25	14	10
9	24	24	14	13
13	34	23	15	15

3.6.2 Determination of Free Swell Index

This procedure involved is to take two oven dried soil samples (passing through 425 μ IS sieve), 20g each sample were placed separately in two 100ml graduated soil sample. In one cylinder distilled water was filled and in other kerosene (non-polar liquid) is filled up to 100ml mark, in case of bentonite 10 gm of sample were taken for test. Figure 3.6 shows the free swell tests for the soils. The final reading of volume of soil was taken after 24hours to calculate free swell index.

The percent free swell index was calculated as

$$FSI (\%) = [(V_d - V_k) / V_k] * 100$$

Where,

V_d = The volume of sample noted from the graduated cylinder containing distilled water

V_k = The volume of sample noted from graduated cylinder containing distilled kerosene.



Figure 3.6 Free swell test for soil

Free Swell Index of different lime-soil mixes were determined from the test and tabulated in Table 3.10.

Table 3.10 Free swell index of soils for different lime content

Lime content (%)	Free swell index (%)			
	100% ES	50%ES+50%RS	10%ES+90%RS	100%RS
0	712.5	475	80.95	10
2	400	52	8.33	-4.54
5	115.38	38.46	16	-8.33
9	120	25.92	3.57	-8
13	75	19	3.33	0

3.6.3 Determination of Linear shrinkage

About 150 g of the soil sample passing 425 micron IS Sieve shall be mixed with water; approximately 2 percent above the liquid limit and the soil paste were left to stand for a sufficient time (24 hour) to allow the moisture to permeate throughout the soil mass. The thoroughly mixed soil-water paste was placed in the shrinkage mould as shown in Figure 3.7, and then gently jarred to remove any air pockets in the paste. The soil was leveled off along the top of the mould with the palette knife. The mould was placed so that the soil-water mixture (paste) can air dry slowly, until the soil has shrunk away from the walls of the mould. Drying mould first started at a temperature of 60 to 65°C until shrinkage has largely ceased and then at 105 to 110°C to complete the drying. After cooling of mould containing dried soil, the mean length of soil bar in the mould was measured by vernier calliper, if the specimen has become

curved during drying, the measurement should be made along the mean arc. The linear shrinkage of the soil was calculated from the following formula:

$$\text{LSI (\%)} = [1 - (L_{\text{avg}}/L_0)] \times 100$$

Where,

L_{avg} = Average length of soil (mm)

L_0 = Original length of brass mould (mm)

The linear shrinkage values for soils stabilized with different lime contents are given in Table 3.11.

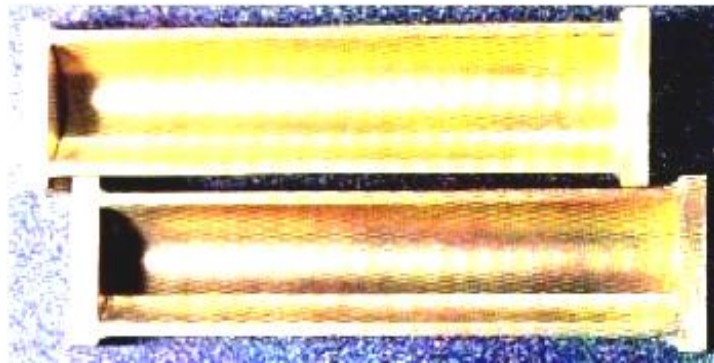


Figure 3.7 Linear shrinkage moulds

Table 3.11 Linear shrinkage of soils for different lime content

Lime content (%)	Linear shrinkage index (%)			
	100% ES	50%ES+50%RS	10%ES+90%RS	100%RS
0	49	22	12	8
2	36	20	10	6
5	26	16	8	5
9	22	14	7	4
13	18	14	6	4

3.6.4 Determination of compaction characteristics

The compaction characteristics of soils were found by conducting standard proctor compaction test. For this test, samples were mixed with required amount of water and the wet sample was compacted in proctor mould as shown in Figure 3.8 in three equal layers, each layer by 25 blows using standard proctor rammer of 2.6 kg dropped from a height of 31cm. The moisture content of the compacted mixture was determined. From the dry density and moisture content relationship, the optimum moisture content and corresponding maximum dry density were

determined. Soils were then stabilized with varying percentage of lime. The lime contents were 0%, 2%, 5%, 9%, and 13% of the dry weight of soil. Similar compaction tests were conducted with varying lime percentages and the optimum moisture contents and maximum dry densities were determined. This was done to study the effect of lime on compaction characteristic of soils.



Figure 3.8 Compaction test for soil

Table 3.12 Compaction characteristics of soils for different lime content

Lime content (%)	100% ES		50%ES+50%RS		10%ES+90%RS		100%RS	
	OMC (%)	MDD (g/cc)	OMC (%)	MDD (g/cc)	OMC (%)	MDD (g/cc)	OMC (%)	MDD (g/cc)
0	26.7	1.39	22.8	1.58	17.4	1.75	15.1	1.79
2	33.13	1.28	27.5	1.43	19.4	1.67	18.25	1.71
5	35	1.28	28.9	1.36	21.2	1.63	18.7	1.67
9	34	1.29	28.6	1.35	20.5	1.6	18.4	1.67
13	34	1.29	29.4	1.38	18.4	1.59	17.8	1.66

3.6.5 Determination of Unconfined Compressive Strength

The Unconfined compressive strength test is one of the common tests used to study the strength characteristics of soil and stabilized soil. To get Immediate UCS strength, UCS tests on soils and lime stabilized soil specimens are compacted to their maximum dry densities with optimum

moisture contents obtained from standard proctor compaction tests. The UCS tests were performed for the plastic soils as well as for stabilized soils. For this test, cylindrical specimens were prepared by compacting to their maximum dry density at optimum moisture content in the metallic split mould with dimension 50mm (diameter) × 100mm (height). These specimens were tested in a compression testing machine with strain rate of 1.25% per minute till failure of the sample.

To determine the effect of curing period on strength property all samples were coated with wax as shown in Figure 3.9. Wax coat has been made to the test samples to maintain at its specified moisture content and cured at a temperature of 27° C in a humidity chamber for a period of 0, 7, 14, 28 and 60 days. After curing period and before testing, the wax was removed from the end of the specimen. The unconfined compressive strengths of specimens were determined from stress versus strain curves plots and hence the unconfined strength and corresponding failure strain for different raw soils and lime stabilized soils for different days of curing are obtained. The compressive strengths for soils stabilized with lime for different curing periods are tabulated in Table 3.13, Table 3.14, Table 3.15 and Table 3.16.



Figure 3.9 UCS test samples

Table 3.13 Compressive strength of lime stabilized soil (100%ES) for different curing periods

Lime content (%)	UCS (kPa)				
	0 Days	7 Days	14 Days	28 Days	60 Days
0	295	295	295	295	295
2	492	505	540	559	614
5	728	890	960	1490	1908
9	917	1050	1480	2590	3069
13	805	895	1180	2526	2996

Table 3.14 Compressive strength of lime stabilized soil mix (50%ES+50%RS) stabilized for different curing periods

Lime content (%)	UCS (kPa)				
	0 Days	7 Days	14 Days	28 Days	60 Days
0	312	312	312	312	312
2	401	419	500	532	618
5	744	859	990	1160	1799
9	846	912	1460	2340	2859
13	745	815	1050	2165	2689

Table 3.15 Compressive strength of lime stabilized soil mix (10%ES+90%RS) for different curing periods

Lime content (%)	UCS (kPa)				
	0 Days	7 Days	14 Days	28 Days	60 Days
0	372	372	372	372	372
2	375	379	400	432	467
5	712	839	985	1275	1622
9	462	535	640	2033	2361
13	390	470	605	1028	1530

Table 3.16 Compressive strength of lime stabilized soil (100%RS) for different curing periods

Lime content (%)	UCS (kPa)				
	0 Days	7 Days	14 Days	28 Days	60 Days
0	438	438	438	438	438
2	400	430	450	472	469
5	620	750	880	1611	1951
9	531	630	690	740	948
13	358	445	450	473	524

Also, the change in compressive strengths for soils without lime and lime stabilized soils compacted with varying moisture contents are studied by conducting the UCS test following the above procedure. The UCS tests were conducted for all the four types of soils with varying moisture contents i.e. increasing the moisture contents by 10% and 20% and reducing the moisture contents by 10% and 30% of optimum moisture content. The specimens compacted with these varying moisture contents were coated with wax and cured for a period of 28 days. The wax was removed from the specimens after 28 days and before testing the compressive strength. The compressive strength test results for lime-soils with varying moisture contents are presented in Table 3.17, Table 3.18, Table 3.19 and Table 3.20.

Table3.17 Compressive strength of lime stabilized soil (100% ES) for varying moisture contents

Lime content (%)	UCS (kPa)				
	0.7 OMC	0.9 OMC	OMC	1.1 OMC	1.2 OMC
0	600	438	295	290	206
2	1050	940	459	416	363
5	1130	1340	1490	910	838
9	1100	1410	2590	2622	2084
13	1050	1400	2526	2600	2678

Table 3.18 Compressive strength of lime stabilized soil mix (50%ES+50%RS) for varying moisture contents

Lime content (%)	UCS (kPa)				
	0.7 OMC	0.9 OMC	OMC	1.1 OMC	1.2 OMC
0	664	450	312	305	260
2	1044	697	632	544	355
5	1240	1187	1160	806	745
9	1990	2330	2340	2373	1780
13	1211	1134	2165	2300	2428

Table 3.19 Compressive strength of lime stabilized soil mix (10%ES+90%RS) for varying moisture contents

Lime content (%)	UCS (kPa)				
	0.7 OMC	0.9 OMC	OMC	1.1 OMC	1.2 OMC
0	750	584	372	310	275
2	965	785	732	472	300
5	1128	1110	1275	943	605
9	1094	1090	2033	1915	1232
13	820	900	928	1080	740

Table 3.20 Compressive strength of lime stabilized soil (100%RS) for varying moisture contents

Lime content (%)	UCS (kPa)				
	0.7 OMC	0.9 OMC	OMC	1.1 OMC	1.2 OMC
0	765	600	438	329	280
2	1393	1510	472	401	240
5	1110	1310	1611	1526	1200
9	917	934	740	600	413
13	818	839	473	302	265

CHAPTER 4

RESULT AND DISCUSSION

4.1 INDEX PROPERTIES

4.1.1 Specific gravity

Specific gravity is one of the important physical properties needed for the use of lime for geotechnical and other applications. In general, the specific gravity of hydrated lime lies between 2.3 to 2.9 but can be for quick lime i.e. 3.2 to 3.4. The average specific gravity of lime used for the present research found to be 2.4. The specific gravity value for inorganic soil ranges between 2.6 to 2.8 and the value of specific gravity for bentonite and residual soil used are 2.79 and 2.6 respectively.

4.1.2 Grain size distribution

In Figure 3.4 (a), particle size distribution curve for bentonite is situated higher up and to the left indicating fine grained soil and the bentonite contains silt and clay size particles only. The curve for residual soil shown in Figure 3.4 (b) represents well graded soil contains particles of different sizes in good proportion as the coefficient of uniformity and coefficient of curvature for the residual soil are found to be 7.3 & 2.2 respectively. The particle size of residual soil ranges from fine sand to clay size.

4.2 ENGINEERING PROPERTIES

4.2.1 Atterberg limits

4.2.1.1 Liquid limit

Influence of lime on liquid limit of soils is shown in Figure 4.1. There is a significant reduction in liquid limit at 2% of lime content for soils 100%ES and 50%ES+50%RS. Beyond, this lime content the change in liquid limit is comparatively less. It is also observed that there is not much more variation in liquid limits for the soils 10%ES+90%RS and 100%RS with increased lime content. This reduction occurs as the lime is added to the soil, Ca^{+} ions are released into the pore fluid and results in increase in electrolyte concentration which reduces the thickness of diffuse double layer held around soil particle leading to a lower liquid limit.

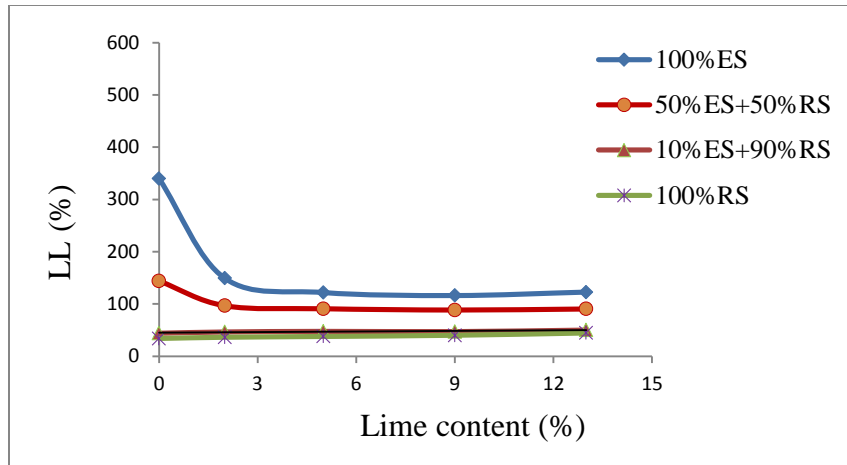


Figure 4.1 Variations in liquid limit with lime content

4.2.1.2 Plastic limit

The plastic limit is a measure of cohesion of soil particles against cracking and also it is the water content of soil when it approaches a certain shear resistance. With the addition of lime, the thickness of diffuse double layer decreases which increase charge concentration and thereby viscosity of pore fluid, it results interparticle shear resistance, leading to an increased plastic limit. Figure 4.2 shown variations in plastic limit with lime content for soils. It is observed that, plastic limit increases with lime content. The influence of lime on plastic limit is more at 2% lime content. Also, it has been seen that beyond 5% lime content liquid limit and plastic limit does not change more.

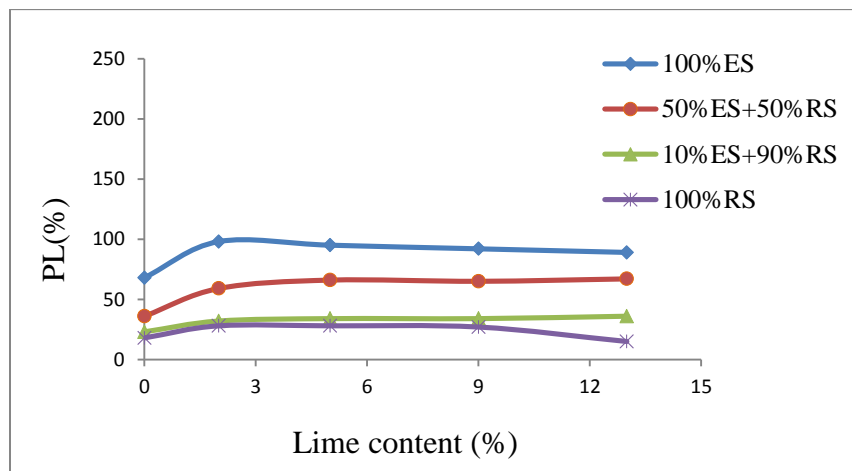


Figure 4.2 Variations in plastic limit with lime content

4.2.1.3 Plasticity index

A decrease in plasticity index has been observed as the lime content increases. The variations in plasticity index with lime content for soils are shown in Figure 4.3. This variation is more in case of expansive soil (i.e. 100%ES) at 2% of lime content. Also, it has been seen that, there is comparatively more reduction in plasticity index in case of soil mix 50%ES+50%RS in compare to other two soils (i.e. 10%ES+90%RS and 100%RS).

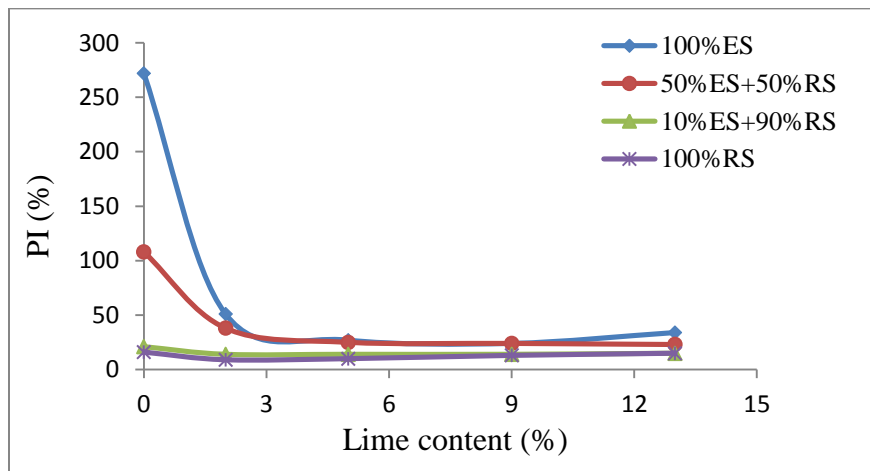


Figure 4.3 Variations in plasticity index with lime content

4.2.2 Free swell index

The variation of free swell index with percentage of lime is illustrated in Figure 4.4. For the soil 100%ES, free swell index reduces largely upto 5% lime content, thereafter the decrease is not substantial. However, for the soil mix 50%ES+50%RS and 10%ES+90%RS a reduction in free swell index value is observed upto 2% lime content beyond which the reduction is not there is not substantial. This indicates that the colloidal reaction in soil of 100%ES continues upto 5% lime content whereas this stops at 2% lime content for soil mix 50%ES+50%RS and 10%ES+90%RS. It has been observed that for the residual soil or residual soil added with lime, the free swell index value is either very close to zero or negative. This observation is in line with the previous researchers who have reported a negative free swell index values for finely powered non cohesive earth material like rock powder.

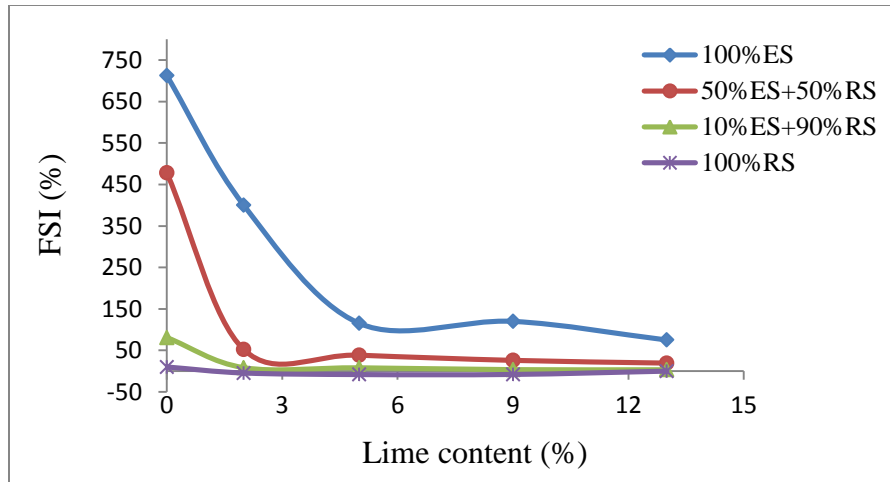


Figure 4.4 Variation in free swell index with lime content

4.2.3 Linear shrinkage index

Figure 4.5 represents the variations in linear shrinkage value for lime modified soils. The addition of lime to highly plastic clay showed a reduction in shrinkage with increasing lime percentage. The addition of lime to plastic soils causes a colloidal reaction which includes a replacement of naturally carried cations on clay surface by Ca^{++} cations, an increase in pH value and a reduction in double layer water. This helps in flocculation and aggregation of colloidal clay particles making them less plastic. This shrinkage arrest is caused by the addition of lime which is well correlated with raw soil's specific surface area and double layer water thickness.

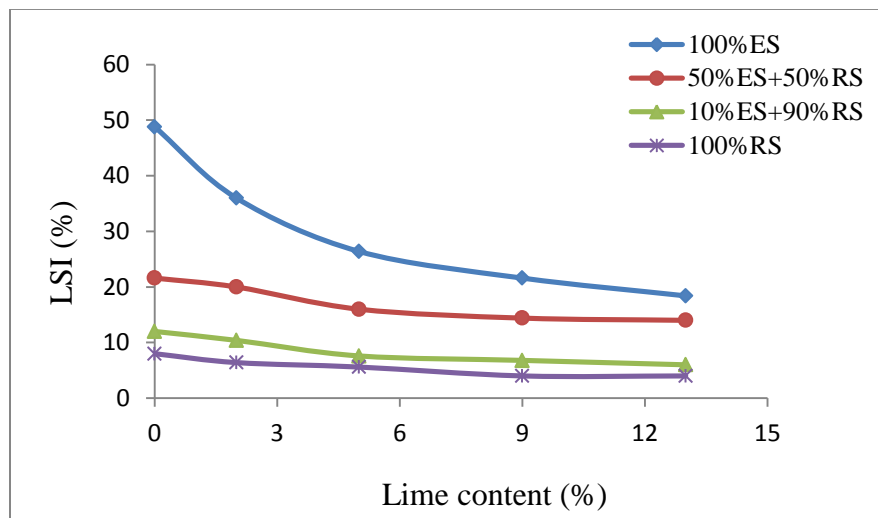


Figure 4.5 Variations in linear shrinkage with lime content

4.2.4 Compaction characteristics

The compaction characteristics of the soils have been studied by varying the lime content. The compaction curves showing optimum moisture contents and maximum dry density for different soils have been shown in Figure 4.6. It is observed that the optimum moisture content is less and the maximum dry density is more for the soil 100%RS compared to other three soils. However, the expansive soil (i.e. 100%ES) has minimum dry density and maximum moisture content. Hence, the more the percentage of bentonite in the residual soil, more the moisture content and lesser the dry density. This is obvious as the affinity of expansive soil to water is much higher than the residual soil. The water molecules adsorbed to the surface of the clay minerals and higher amount of water is needed to lubricate the clay surface.

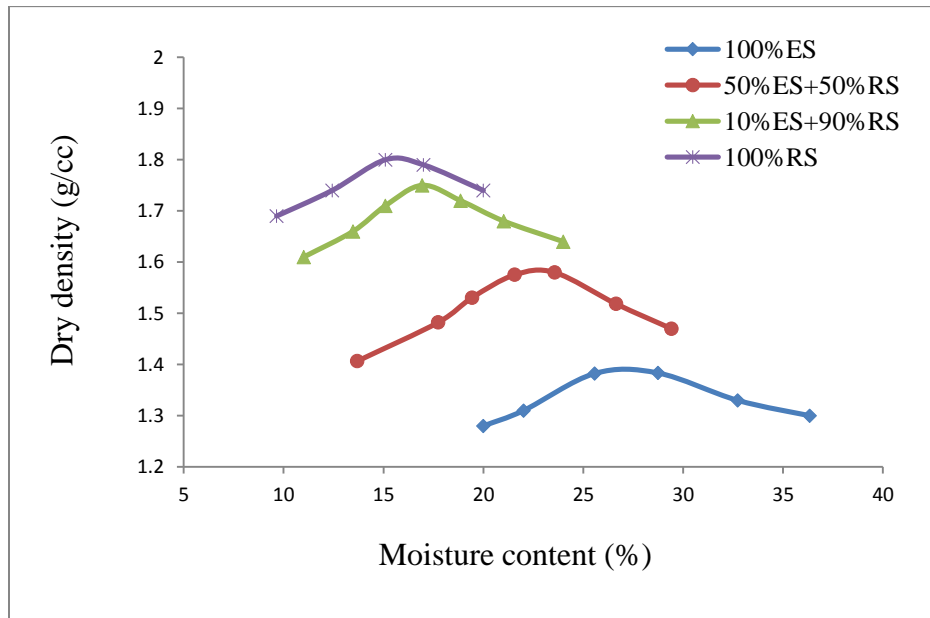


Figure 4.6 Compaction curves for different soils

Further, compaction characteristics have been studied by adding lime in different percentage to the soils. It has been observed that, optimum moisture content increases and maximum dry density decreases with lime content. The variation in compaction characteristics with lime content have been shown in Figure 4.7 and Figure 4.8. Also, it is seen that the fall in maximum dry density is significant at lower percentage of lime. At lower percentage of lime reacts very quickly with soil, base exchange, aggregation and flocculation occurs resulting increased void ratio leading to a decrease in the density of the mix. Beyond this lime content, addition of lime is

utilized for pozzolanic reactions causes an increase in optimum moisture content and also, more water is retained in the void space between the particles which water content may increase.

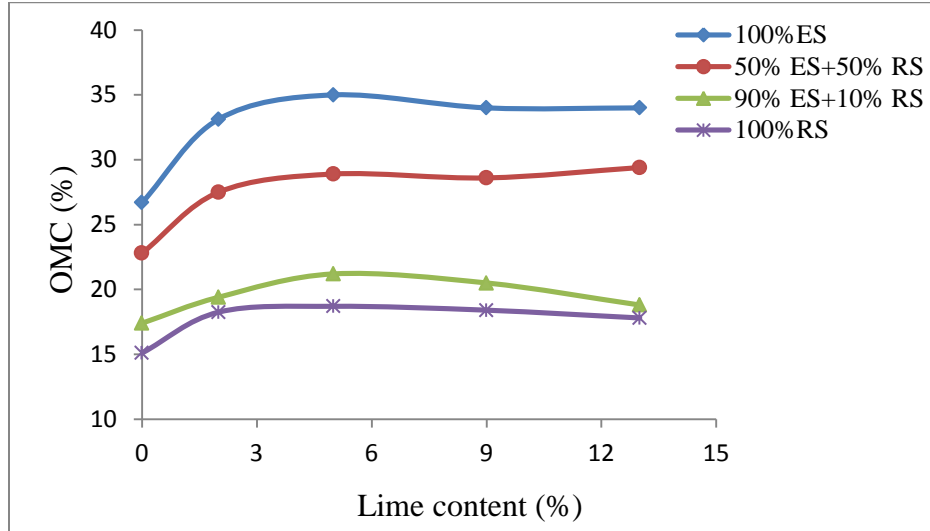


Figure 4.7 Variations in optimum moisture content with lime content

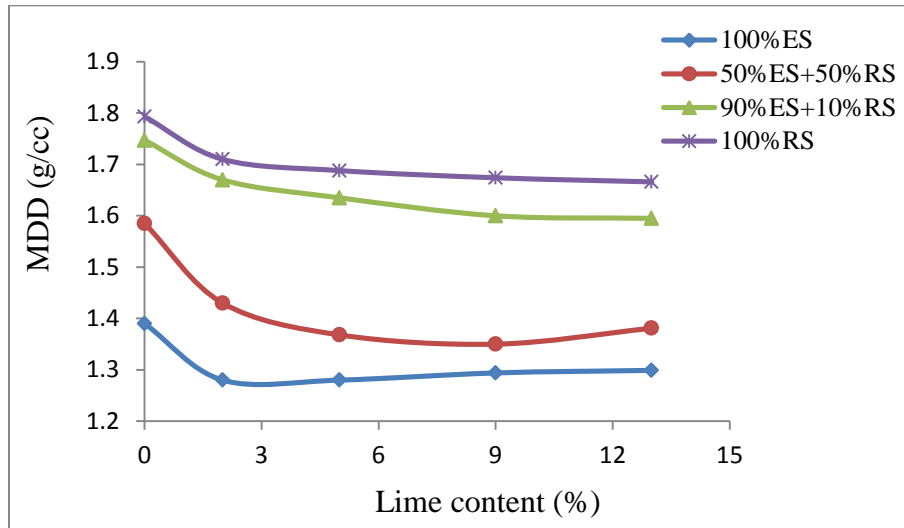


Figure 4.8 Variation in maximum dry density with lime content

There is a very less variation in optimum moisture content beyond 5% lime content for soils 100%ES, 50%ES+50%RS and 10%ES+90%RS but for 100%RS it is 2%. The reduction in density of soils is remarkable at 2% lime content.

4.2.5 Unconfined compressive strength

Unconfined compressive strength tests were carried out on untreated soils compacted to their maximum dry density at optimum moisture content. The unconfined compressive strengths are obtained from stress-strain curves for different untreated soils the soils as shown in Figure 4.9. It is observed that for untreated expansive soil (i.e. 100%ES), the compressive strength is less than the Residual soil (i.e. 100%RS). The expansive soil possesses more 11.6% moisture content than residual soil and also the dry density is less. As the expansive soil has high optimum moisture content and lesser maximum dry density, is obvious to show less compressive strength than the residual soil.

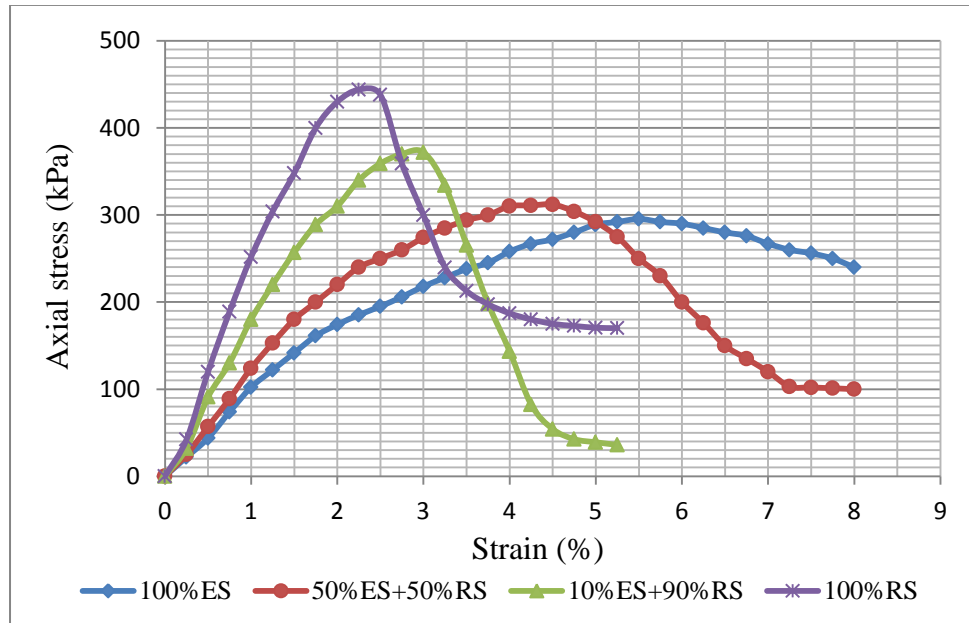


Figure 4.9 Stress-strain curves for untreated soils

4.2.5.1 Effect of curing on strength of soil

Typical strength (i.e. unconfined compressive strength at failure) variation of lime treated soils after different curing period are depicted in Figure 4.10, Figure 4.11, Figure 4.12 and Figure 4.13. At 2% lime content the strength gain is not remarkable but by increasing the lime content the strength gain is more significant for all the soils.

While considering the curing period, strength gain is more rapid after 28 days and after 60 days curing, soils gain maximum strength after these curing time. For all types of soils the compressive strength is maximum after 60 days curing. Pozzolanic reaction occur as long as the clay mineral react with lime. For bentonite, to continue the pozzolanic reaction high amount of lime is required, where as for residual soil it is comparatively less as less amount of clay mirals present in the residual soil to react with lime.

For the soil 100%ES, at 9% lime content the strength is maximum and beyond this, decrement in strength is comparatively less. So, for the economical point of view 9% may be considered as optimum lime content. Whereas, beyond 5% lime content the strength is decreased for the soil 100%RS. Thus for the residual soil the optimum lime content is 5%. As the expansive soil requires higher percentage of lime to alter its physicochemical characteristics, the optimum lime content is comparatively high and beyond this point there is not remarkable decrement in strength for 28 days and 60 days curing. The fall in strength is sudden beyond 5% lime content for residual for 28 days and 60 days curing,

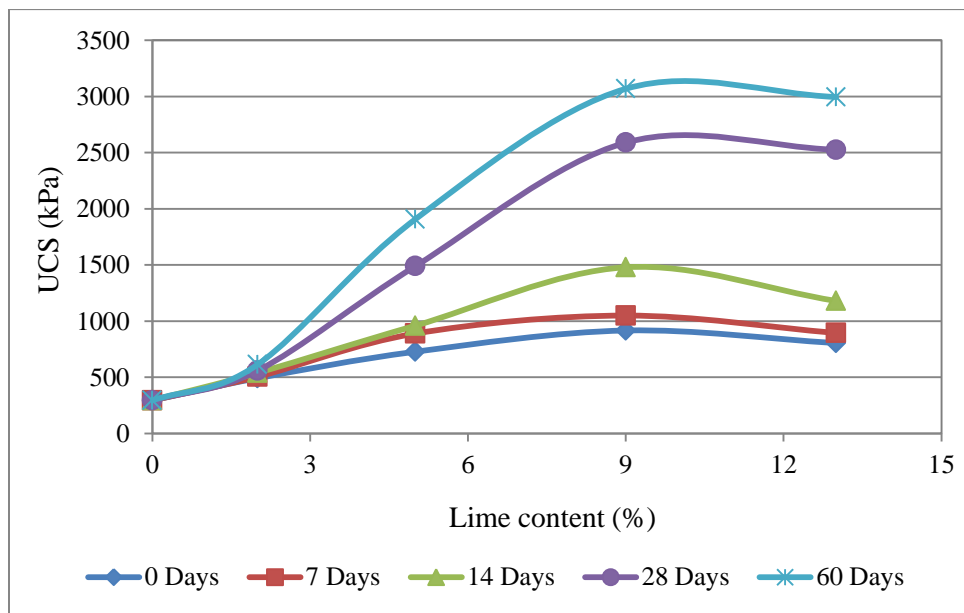


Figure 4.10 Variation in UCS for lime stabilized soil (100%ES) at different curing period

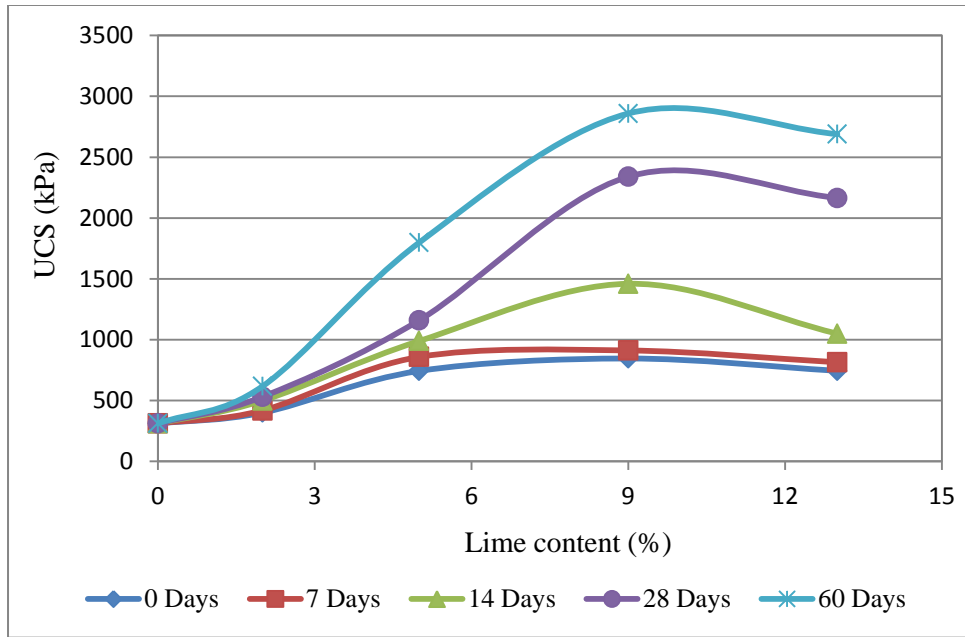


Figure 4.11 Variation in UCS for lime stabilized soil mix (50%ES+50%RS) at different curing period

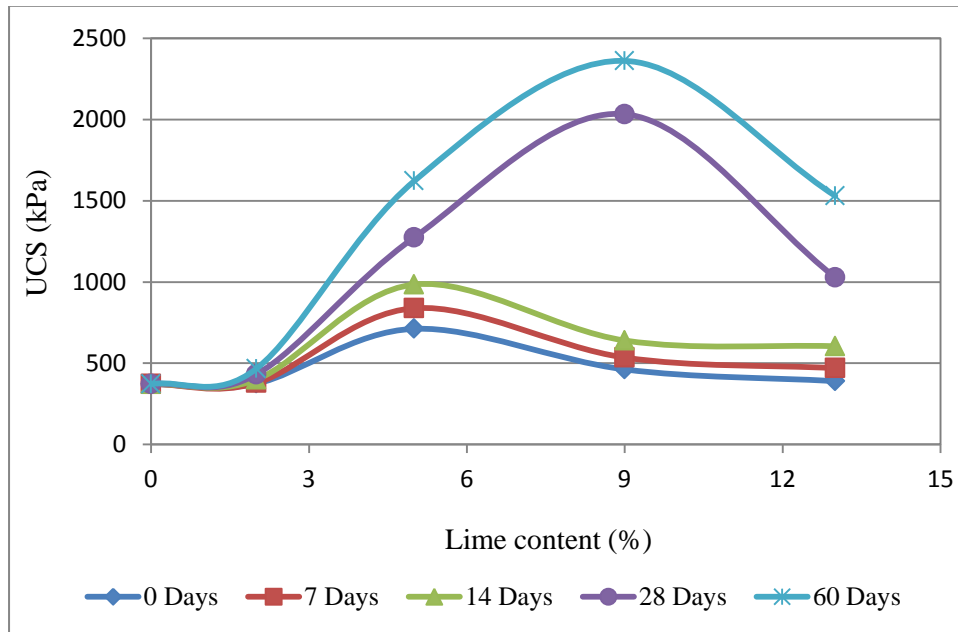


Figure 4.12 Variation in UCS for lime stabilized soil mix (10%ES+90%RS) at different curing period

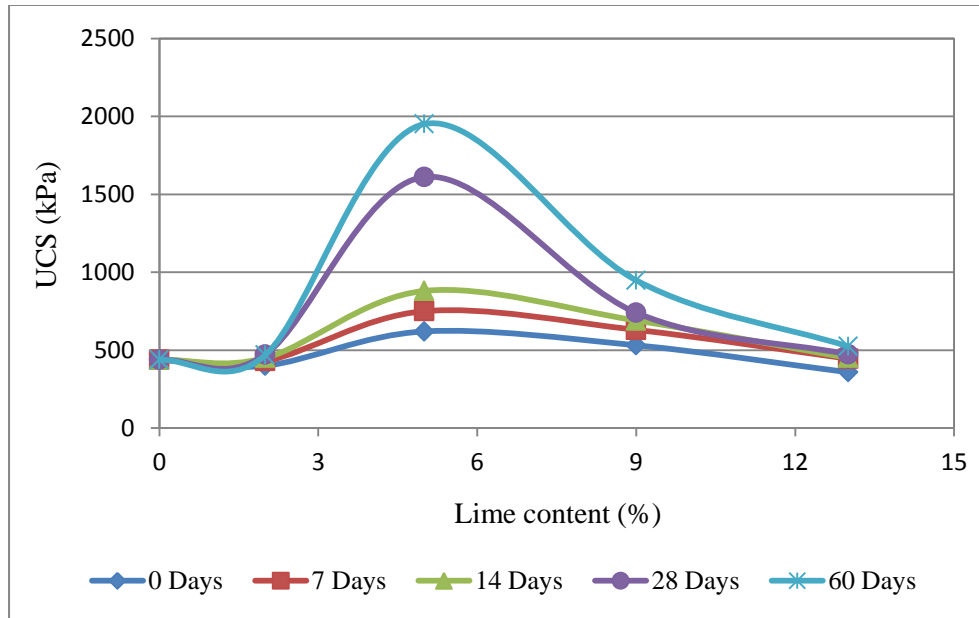


Figure 4.13 Variation in UCS for lime stabilized soil (100%RS) at different curing period

Practically, at a lower lime content strength improvement is negligible as at this stage, lime is mainly used to meet the initial requirement of soil like reducing thickness of diffuse double layer due to which plasticity of soil changes. Also, the cementation is too weak and gives a marginal strength gain. By increasing lime content the improvement in strength gain is remarkable as the lime is adequate for formation of cementitious compounds once pozzolanic reaction starts. The compressive strength of soil increases upto certain lime content after this lime content, the strength starts decreasing. This is because the excess lime behaves as lubricant to the soil particles as lime has not appreciable friction and cohesion.

However, the strength improvement is also depends upon the curing period. Compressive strength is high for longer curing period. For low lime content the strength increment is very less even after a long period of curing. It has been observed that after prolonged curing period and for appreciable lime content, the compressive strength is maximum.

Optimum lime content varied with soil type and curing period. The summary of optimum lime content giving maximum compressive strength for the soils after different curing periods are produced in Table 4.1.

Table 4.1 Summary of optimum lime content giving maximum strength for soils cured at different curing periods

Curing period (Days)	Optimum lime content (%)			
	100%ES	50%ES+50%RS	10%ES+90%RS	100%RS
0	9	9	5	5
7	9	9	5	5
14	9	9	5	5
28	9	9	9	5
60	9	9	9	5

4.2.5.2 Effect of moisture content on strength of soil

Figure 4.14 shows the effect of moisture content on strength of untreated soils. The strength of residual soil is more than the expansive soil when compacted at their corresponding optimum moisture contents.

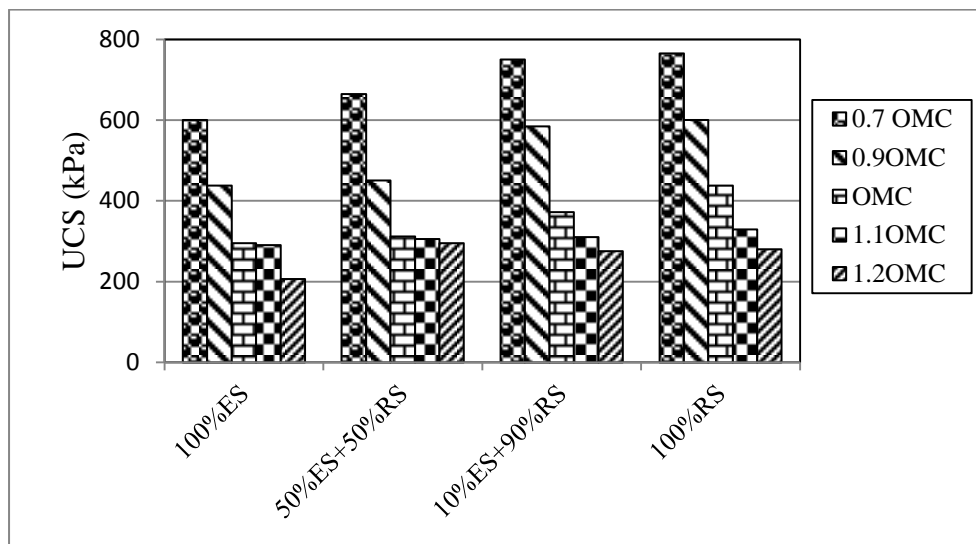


Figure 4.14 Variation in UCS with moisture content for untreated soils

Untreated soil compacted at a dry side of optimum moisture content shows higher unconfined compressive strength. If the soil sample is compacted at dry of optimum moisture content, the structure forms to a flocculated one at same compacted dry density. Flocculated structures have much higher strength than dispersed one. As the water content increases, water acts as a

lubricating agent and a given compactive energy, soil-structure changes towards a more dispersed one which is responsible for reduced strength.

Variation in compressive strength with moisture content for lime stabilized soils are shown in Figure 4.15, Figure 4.16, Figure 4.17 and Figure 4.18.

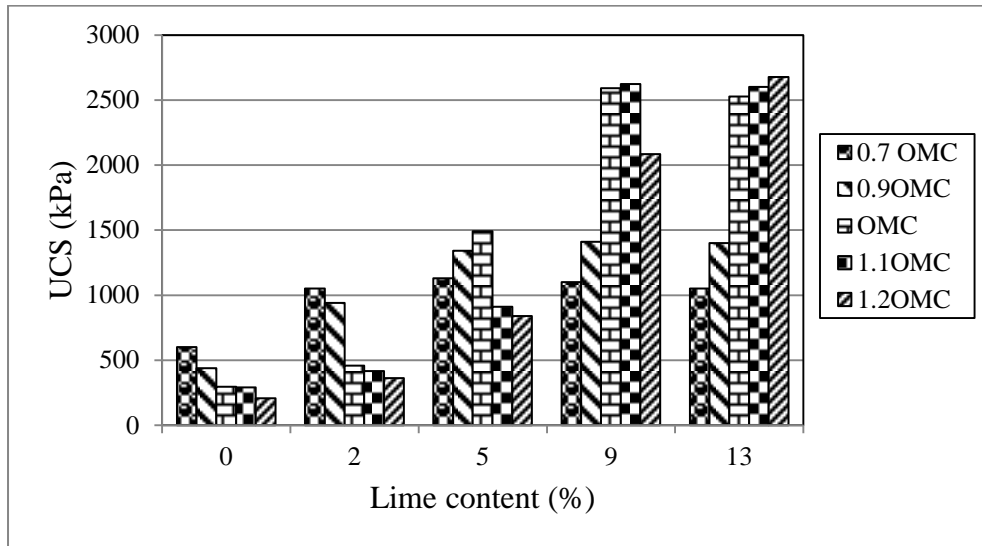


Figure 4.15 Variation in UCS with moisture content for lime stabilized soil (100%ES)

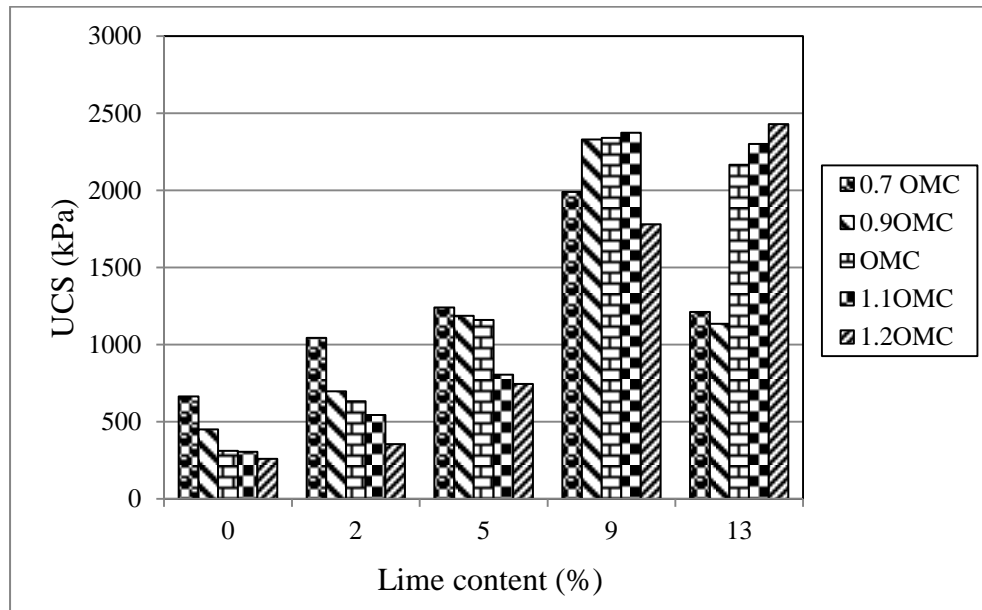


Figure 4.16 Variation in UCS with moisture content for lime stabilized soil mix (50%ES+50%RS)

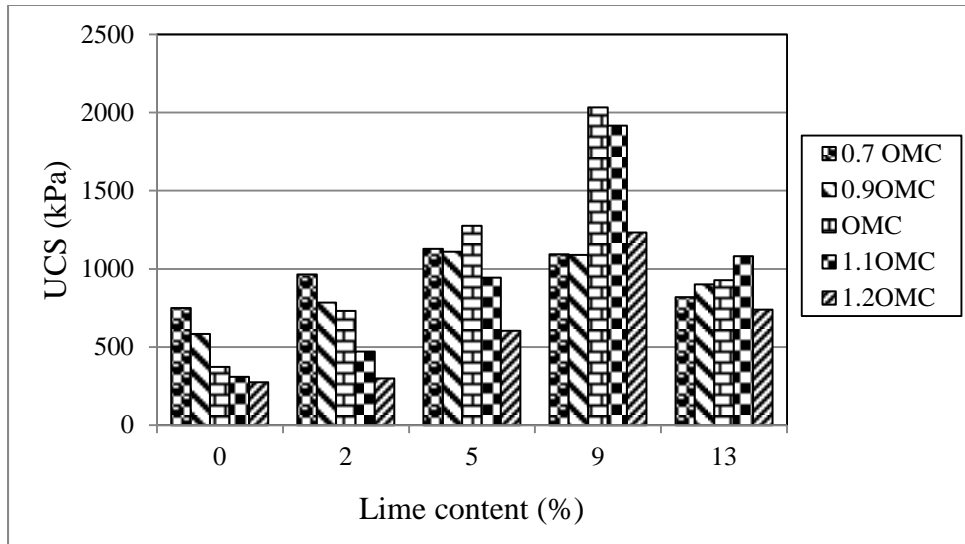


Figure 4.17 Variation in UCS with moisture content for lime stabilized soil mix (10%ES+90%RS)

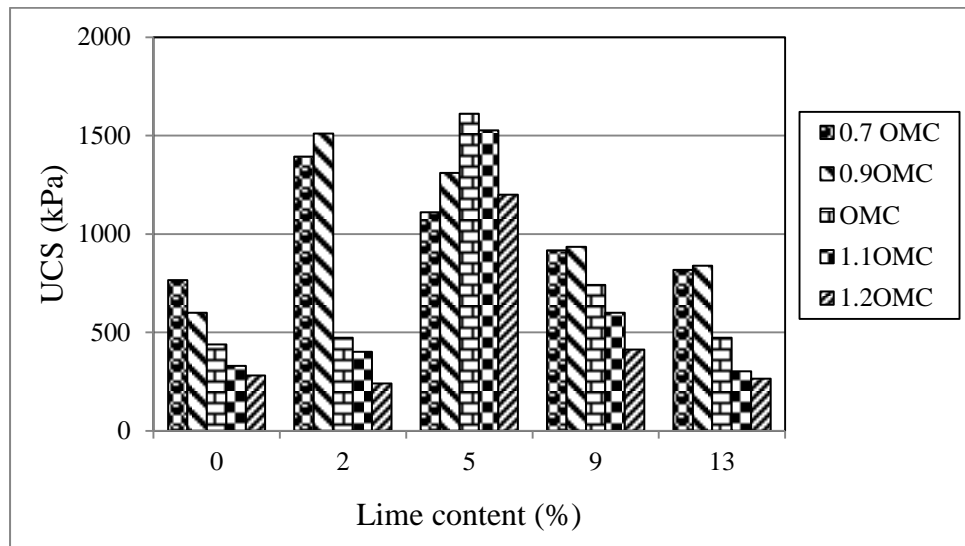


Figure 4.18 Variation in UCS with moisture content for lime stabilized soil (100%RS)

When lime is added to the expansive soil and cured for 28 days, pozzolanic reaction occur leads to strength gain. When 13% lime is added to the expansive soil in presence of sufficient water, pozzolanic reaction occur as long as there is enough residual calcium from lime presents in the system. But, at higher lime content and at dry of optimum moisture content, strength of expansive soil reduces as the excess calcium from lime remains non-reactive.

The mica and muscovite in residual soil are non-reactive to lime and strength improvement is not achieved significantly. The strength gain is due to silica and alumina presents in the soil. Maximum strength gain occurs for 5% lime content compacted at optimum moisture content. Low strength occurred due to an excess or a less lime content and for an excess or deficiency of moisture content. Insufficient water obstructs pozzolanic reaction and at higher lime content and moisture content, excess formation of cementitious gel (highly porous material) causes reduction in strength.

For soils 100%ES and 50%ES+50%RS, at 13% lime content, for soil mix 10%ES+90%RS at 9% lime content and for 100%RS at 5% lime content compacted at optimum moisture content and wet of optimum moisture content maximum strength is achieved. The optimum lime content giving maximum compressive strength for soils compacted at different moisture contents is tabulated in Table 4.2.

Table 4.2 Summary of optimum lime content giving maximum strength for soils compacted at different moisture contents

Moisture contents (%)	Optimum lime content (%)			
	100%ES	50%ES+50%RS	10%ES+90%RS	100%RS
0.7 OMC	5	5	5	2
0.9 OMC	9	9	5	2
OMC	9	9	9	5
1.1 OMC	9	9	9	5
1.2 OMC	13	13	9	5

CHAPTER 5

CONCLUSION

5.1 SUMMARY

The main objectives of this research were identified as follows:

1. To study the plasticity, swell and shrinkage characteristic of soils
2. To study the compaction characteristics of lime treated soils
3. To study the strength development in lime treated soils and hence to find out the optimum percentage of lime required to stabilize the soils

To achieve the above objectives, an overview on lime stabilization of clay soil has been done and also, a review of various studies on geotechnical behaviour (plasticity, swelling, shrinkage and strength) of lime stabilized expansive soils was carried out and the literature review has been summarized in chapter 2. Indian standard (Methods of test for soils) codes are followed to conduct various soil tests.

An extensive experimental program has been undertaken. Four types of soils have been synthesized taking highly expansive montmorillonite clay and a residual silty soil. The artificially prepared soils have a wide range of plasticity. These artificial soils are added with different proportions of lime and a series of tests were conducted to find out their plasticity, swell and shrinkage characteristics, compaction characteristics and the strength properties.

5.2 CONCLUSIONS

Many of the important engineering properties of soils can be enhanced by the addition of lime. The properties of such soil-lime mixtures vary and depend upon the type of soil. To develop an understanding of the possible mechanisms involved, a series of experiments through variation of parameters were carried out, based on which the following conclusions are drawn:

The liquid limit of soil decreases with an increase in lime content. This result is obtained due to reduction in thickness of double layer as the electrolyte concentration increases in the pore fluid.

1. The plastic limit of soil increases with lime contents as the charge concentration of pore water increases, the viscosity increases and offers high resistance against interparticle movement.
2. The plasticity of soil reduces with increased lime content.
3. There is a significant reduction in free swell index of high plastic clay at low lime content in comparison to other low and medium plastic clays.
4. The amount of shrinkage arrest caused by addition of lime correlated with raw soils specific surface area. More shrinkage occurred for high plastic clays.
5. The compaction characteristics of soils vary significantly at low lime content. The optimum moisture content increases and maximum dry density decreases with increased lime content.
6. The strength of lime treated soil depended on type of soil, lime content, curing period and moisture content. For high plastic clays maximum strength was achieved at 60 days cured sample with 9% lime content whereas, for low plastic clay occurred with 5% lime.
7. More strength is achieved at dry side of optimum than wet side of optimum moisture content for virgin soils. When the soils are cured for 28 days with varying moisture content, there is a significant increase in strength for high plastic clays when treated with high lime content with high moisture content, and for low and intermediate plastic clay, at comparatively low lime content.

5.3 SCOPE OF FUTURE WORK

1. Durability aspects of lime treated soil like drying and wetting, freezing and thawing actions and response to various chemicals like alkalis, chlorides, sulphides are not investigated.
2. Research can be extended to find out the hydration products, morphology and microstructure of lime treated soil and co-relations with developed strength.
3. Other geotechnical parameters such as hydraulic conductivity, consolidation parameters can also be performed.
4. The response of lime stabilized clays under repeated loading can be evaluated.

REFERENCES

1. Arvind Kumar, Baljit Singh Walia and Asheet Bajaj (2007), "Influence of fly ash, lime, and polyster fibres on compaction and strength properties of expansive soil", ASCE.
2. Arman A. and Munfakh G.A. (1970), "Stabilization of Organic soils with Lime", engineering Research Bulletin No. 103, Division of Engineering Research, Louisiana State University, Baton Rouge.
3. Arnold M. (1984), "The genesis, Mineralogy and Identification of Expansive Soils", Fifth International Conference on Expansive Soils, Adelaide, South Australia, pp. 32-36.
4. Al- Rawas A.A., Taha R., Nelson J.B., Belt Al-Shab T. and AL-Siyabi H. (2002), "A Comparative Evaluation of Various Additives Used in the Stabilization of Expansive Soils", Geotechnical Testing Journal, GTJODJ, ASTM, 25(2), pp. 199-209.
5. Bell F.G. (1996), "*Lime Stabilization of Clay minerals and Soils*", Engineering geology, Vol. 42, pp. 223-337.
6. Bell, F.G. (1993), "*Engineering treatment of soils*", Chapman and Hall, London.
7. Chen F.H. (1988), "*Foundations on Expansive Soils*", Elsevier, New York.
8. Cokca E. (2001), "*Use of Class-C Fly-ashes for the stabilization of an Expansive soil*", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 127 (7), pp. 568-573.
9. Chou L. (1987), "*Lime Stabilization: Reaction, Properties, Design and Construction*", State of the Art Report 5, transportation Research Board, Washington, DC.
10. Clare K.E and Cruchley A.E. (1957), "*Laboratory experiments on the stabilization of clays with hydrated lime*", Geotechnique, Vol. 7, pp. 97-100.
11. Dash S.K. and Hussain M. (2012), "*Lime Stabilization of Soils: Reappraisal*", ASCE.
12. Eades, J.L. and Grim R.E. (1960), "*Reaction of Hydrated lime with Pure Clay minerals in Soil Stabilization*", Bulletin 262, Highway Research Board, Washington, DC.
13. Fredlund D.G. and Rahardjo h. (1993), "*Soil Mechanics for Unsaturated Soils*", John Wiley and Sons, Inc.: New York.
14. Gopal Ranjan and Rao A.S.R. (1991), "*Basic and Applied Soil Mechanics*", Wiley Eastern, New Delhi.

15. Hilt G.H. and Davidson D.T. (1960), "*Lime Fixation on clayey Soils*", Highway Research Board, No. 262, pp 20 - 32.
16. Heller L. and Taylor H.G.W.(1956), "*Crystallographic data for calcium silicates*", Dept. of Sci. and Ind. Res., Build. Res.Station, H.M. Stationery Office, London.
17. Ingles O.G. (1964), "*The nature and strength of the interparticle bonds in natural and stabilized soils*", Mechanism of soil stabilization, D-9, Proc. Of a colloquium held at Syndal Victoria, Australia.
18. Jones D.E. and Holtz W.G. (1973), "*Expansive soils- The hidden disaster*", Civil Engg., ASCE, New York, NY, pp. 87-89.
19. Komine H. and Ogata N. (1996), "*Prediction for swelling Characteristics of Compacted Bentonite*", Canadian Geotechnical Journal, Vol. 33, pp. 11-22.
20. Kehew A.E. (1995), "*Geology for Engineers and Environmental Scientists*", Prentice Hall, 574 pages.
21. Kinithua J.M., Wild S. and Jones G.I. (1999), "*Effects of Monovalent and Divalent Metal Sulphates on Consistency and Compaction of Lime-Stabilized Kaolinite*", Applied Clay Science, Vol. 14, pp. 27-45.
22. Kate J.M. (2009), "*Behaviour of expansive clays treated with lime –flyash admixtures*", IGC, Guntur, India.
23. Kate J.M., Kumar Sunil, Bhorkar M.P. (1012), "*Influence of bentonite proportions on swelling characteristics of clay mixes*", IGC, Delhi.
24. Little D.L. (1987), "*Fundamentals of the Stabilization of Soil with Lime*", Bulletin 332, NLA, Arlington, V.A.
25. Lees G., Abdelkader M.O. and Hamdani S.K. (1982), "*Sodium Chloride as an Additive in Lime-Soil Stabilization*", Highway Engineer, Vol. 29, No. 12, pp. 2-8.
26. Little D.N. (1995), "*Handbook for Stabilization of Pavement Subgrades and Base Courses with Lime*", Kendall/Hunt, Iowa.
27. Mathew P.K. and Rao S.N. (1997), "*Effect of Lime on cation exchange capacity of Marine Clay*", *Journal of Geotechnical and Geoenvironmental Engineering*", Vol. 123, No. 2, pp. 183-185.
28. Mateous M. (1964), "Soil lime research at Iow a State University", J. Soil Mech. and Found. Engg. Div., ASCE, 90(2), 127-153.

29. Marks B.D., and Halibunon T.A. (1970), "*Effects of sodium chloride and sodium chloride-lime admixtures on cohesive Oklahoma soils*", 49th Annual Meeting of the Highway Research Board.
30. Narasimha Rao S. And Rajesekaran G. (1996), "*Reaction products for medin lime-stabilized marine clays*", J. Geotech. Engrg., Geotech.Div., ASCE, 122(5), 329-336.
31. Nalbantoglu Z. and Tuncer E.R. (2001), "*Compressibility and Hydraulic Conductivity of a Chemically Treated Expansive Clay*", Canadian Geotechnical Journal, Vol. 38, pp. 154-160.
32. Rao K.S.S. and Triphaty S. (2003), "*Effect of Aging on swelling and Swell-Shrink Behavior of a Compacted Expansive Soil*", Geotechnical Testing Journal, Vol. 26, No. 1, pp. 1-11.
33. Russell L., Buhler and Cerato B. (2007), "*Stabilization of Oklahoma expansive soils using lime and class C fly ash*", Geo-Denvar: New Peaks in Geotechnics, Denver.
34. Shuai F. and Fredlund D.G. (1998), "*Model for the Simulation of swelling pressure Measurements on Expansive Soils*", Canadian Geotechnical Journal, Vol. 35, pp. 96-114.
35. Thompson M.R. (1968). "*Lime-treated soils for pavement construction*", Highway Div., ASCE, 94(2), 1991-217.
36. Wayne A.C., Mohamed A.O. and El-Fatih M.A. (1984), "*Construction on Expansive Soils in Sudan*", Journal of Construction Engineering and Management, Vol. 110, No. 3, pp. 359-374.
37. Zhang Ji-ru and Cao Xing (2002), "*Stabilization of expansive soils by lime and fly ash*", Journal of Wuhan University of Technology.