STABILIZATION OF LIGHT GREY AND RED CLAY SUBGRADE SOIL USING SA-44/LS-40 CHEMICAL AND LIME

A Thesis submitted to School of Graduate Studies of Addis Ababa University In Partial Fulfilment of the Requirements for the Masters Degree of Civil Engineering in Geotechnical Engineering

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January, 2008

ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

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ACKNOWLEDGEMENT

My best gratitude goes to my mother W/o Semegn Metekia and all my family members for their material support and moral encouragement through the study period. My indebted gratitude also goes to my wife W/o Meseret Addisu for being along side from the inception to completion of this study.

My gratitude goes to my advisor Dr. Ing. Asrat Worku for his diligent advice and patience which makes the study fruitful. My grateful appreciation also goes to Ato Yared Bayou, Senior Geotechnical Engineer who provides technical support while I was conducting the study.

Lastly, I like to appreciate Dallas Roadway Product Inc. for the generous logistic and financial support of the study.

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SYMBOLS AND ABBREVIATIONS

AACRA	Addis Ababa City Roads Authority
AASHTO	American Association State Highway Transport Officials
AAU	Addis Ababa University
$Ca(OH)_2$	Calcium Hydroxide
Ca^{++}	Calcium Ion
CaO	Calcium Oxide
CBR	California Bearing Ratio
CH	Inorganic clays of high plasticity, fat clays
DRP	Dallas Road Way Product
ERA	Ethiopian Roads Authority
H^+	Hydrogen Ion
LL	Liquid Limit
MgO	Magnesium Oxide
MC	Mix Code
OH^-	Hydroxyl
PI	Plasticity Index
PL	Plastic Limit
SGS	School of Graduate Studies
USCS	Unified Soil Classification System

ABSTRACT

This study is undertaken on a light grey (expansive) and a red clay soil samples taken from within Addis Ababa. The change in engineering properties of these soils treated with SA-44/LS-40 chemical alone, lime alone and combinations of both at different application rates are investigated.

The investigation of the modifications in engineering properties of the stabilized soils are made using laboratory tests that include soaked CBR and CBR swell tests, swelling pressure test, Atterberg limit tests, percent swell test and free swell tests. The modifications in engineering properties were evaluated after curing the treated sample for fourteen days.

From the study, it is observed that the application of lime alone and SA-44/LS-40 chemical with lime shows improvements of varying degree on the engineering properties of both the light grey and the red clay soils. However, no specific mix ratio showed the maximum improvement in all the engineering properties of each soil. This suggests that before deciding for large-scale applications, the desired improvement level of the engineering properties of a particular soil should be specified first, and tests be conducted for different mix ratios to arrive at the optimum one.

1.0 INTRODUCTION

1.1 Background

Large areas of our country are covered with expansive soils such as dark and light grey clay soils. These clays have caused persistent difficulties in road construction and are common occurrences in Ethiopia [10].

Particular problems associated with road construction over expansive soils are commonly the seasonal volumetric change in these soils rather than their low bearing strength, since expansive soils are often relatively strong at equilibrium moisture content. Expansive soils shrink and crack when they dry out and swell when they get wet. The cracks allow water to penetrate deep into the soil, hence causing considerable expansion. This results in deformation of road surfaces, since the expansion and the subsequent heave are never uniform. Furthermore, these volume changes may produce lateral displacements (“creep”) of the expansive clay, if the side slopes of roads are not gentle enough. Seasonal wetting causes the road edges to wet and dry at a different rate than those under the surfacing. This mechanism in turn causes differential movements over the cross section of the road and associated crack developments, first occurring in the shoulder area, and subsequently propagating towards the carriageway [9].

To mitigate expansive soil problems several alternative solutions can be applied, of which stabilization is one of these alternatives. Stabilization techniques can also be applied to less expansive to non-expansive soils like the red clay non-expansive soil included in this study to improve important engineering properties. Various organic and inorganic fractions of different soil types undergo modifications when "catalyst" agents are introduced into the soil [7]. In turn the "chemical reaction" can convert inferior and formerly unstable materials to highly satisfactory road bed materials [7].

This study has been carried out on samples of the light grey expansive and the red clay soil predominant in the capital Addis Ababa. It is intended to stabilize these samples by the application of SA-44/LS-40 chemical alone, lime alone and different combinations of both. Various application rates have been used for the treatment of both soil samples. Following the application of SA-44/LS-40 chemical and/or lime, the samples are cured for fourteen days. Subsequently, the influence of each application rate has been evaluated via various laboratory tests.

1.2 Objectives of the Study

The general objective of this thesis is to study the potential use of chemical stabilization to render a typical expansive light grey clay soil suitable as road subgrade, by increasing its bearing capacity and decreasing its swelling pressure. In addition to this, it also aims at the potential to reduce pavement thickness provisions for road projects over less expansive soils like the red clay soils of Addis Ababa by increasing their bearing capacity.

The study has the specific objective of investigating the engineering properties of these soils through the application of SA-44/LS-40 chemical and lime in different proportion. The influence of applying these stabilizers have been examined via various laboratory tests mentioned in subsection 1.4.

1.3 Scope of Investigation

To meet the above-mentioned objectives soil samples of light grey and red clay soil of Bole and Addisu Gebeya areas of Addis Ababa respectively, were collected for the investigation. The samples of both soils were taken from 2m depth below the ground surface, from one test pit for each site. The construction of a road didn't affect the subgrade soil up to such depth. However, the samples are collected from areas where intermingling of the surface soil are observed up to considerable depth. Thus, it becomes necessary to go up to 2m depth to begin sample collection. The collected samples were treated with the chemical and lime of various mix ratios shown in the next subsection. The chemical and lime are applied to the natural soil by loose volume and dry mass of the soil respectively. Subsequently, the treated soil samples were cured in two different ways, for fourteen days. That are samples for CBR and CBR swell tests were cured in compacted state in the CBR molds, whereas for the rest of the tests, the curing was made in a loose state of the soil packed in plastic bags. The curing period is taken to be fourteen days based on the advice of Dallas Roadway Product Inc. Curing is undertaken in the laboratory room without controlling both temperature and humidity of the room. The various tests used to evaluate the change in engineering properties of the soils are described next.

1.4 Methodology

The following application rates of SA-44/LS-40 chemical and lime are employed in the study. These applications are taken based on the advice of Dallas Roadway Product Inc. and from the results obtained.

- 0.08 liters of SA-44/LS-40 per 1m^3 of soil
- 0.15 liters of SA-44/LS-40 per 1m^3 of soil

- 0.30 liters of SA-44/LS-40 per 1m^3 of soil
- 0.08 liters of SA-44/LS-40 per 1m^3 of soil plus 2 percent lime by the dry weight of soil
- 0.15 liters of SA-44/LS-40 per 1m^3 of soil plus 2 percent lime by the dry weight of soil
- 0.30 liters of SA-44/LS-40 per 1m^3 of soil plus 2 percent lime by the dry weight of soil
- 0.08 liters of SA-44/LS-40 per 1m^3 of soil plus 4 percent lime by the dry weight of soil
- 0.15 liters of SA-44/LS-40 per 1m^3 of soil plus 4 percent lime by the dry weight of soil
- 0.30 liters of SA-44/LS-40 per 1m^3 of soil plus 4 percent lime by the dry weight of soil
- 4 percent lime by the dry weight of soil
- 8 percent lime by the dry weight of soil

The changes in the engineering properties of the stabilized soil have been evaluated after curing the treated sample for fourteen days. Curing for the CBR test is accomplished in the compacted state placed in the CBR molds whereas for the rest of the tests, this is done in the loose state placed in plastic bags. Following are laboratory tests conducted to evaluate the change in important engineering properties of the soils;

- Soaked CBR and CBR swell tests,
- Swelling pressure test,
- Atterberg limit tests and
- Percent swell and free swell tests

1.5 Organization of the Thesis

The thesis is comprised of five chapters. Chapter one gives a brief description of the thesis background, objectives of the thesis, scope of the investigation, and the methodology. The second chapter discusses basic soil characteristics, formation, structural arrangement, and physical properties of clay soil and treatment of expansive soils. The third chapter gives a brief description of the tests conducted and their results. In the fourth chapter, analysis of the laboratory tests conducted is given. The last chapter, chapter five, presents the conclusions drawn and the recommendations made regarding the influence of the chemical under study.

2.0 PROPERTIES OF FINE GRAINED SOILS

2.1 Soil Characteristics

Soil characteristics may be considered either as microscale or macroscale factors. Microscale factors include the mineralogical and chemical properties of the soil. Macroscale factors include the engineering properties of the soil, which in turn are dictated by the microscale factors [2].

2.1.1 Microscale Factors

Microstructure is more important from a fundamental than from an engineering view point, but it is useful as an aid in the general understanding of soil behavior. The microstructure of clay is the complete geological history of that deposit, including both the stress changes and environmental conditions during deposition. These geological imprints tend to affect the engineering response of the clay very considerably. Recent research on clay microstructure suggests that the greatest single factor influencing the final structure of clay was the electro chemical environment existing at the time of deposition. Flocculated structure or aggregations, of varying degrees of packing and interconnections result during sedimentation [12].

Clay Mineralogy and Soil Water Chemistry: Clay minerals of different types typically exhibit different swelling potentials because of variations in the electrical field associated with each mineral. The swelling capacity of an entire soil mass depends on the amount and type of clay minerals in the soil, the arrangement and specific surface area of the clay particles, and the chemistry of the soil water surrounding those particles [2].

2.1.2 Macroscale Factors

Macrostructure, including the stratigraphy, of fine-grained soil deposits has an important influence on soil behavior in engineering practice. Joints, fissures, root holes, varves, silt and sand seams and lenses, and other discontinuities often control the behavior of the entire soil mass [2].

Plasticity and Density: Macroscale soil properties reflect the microscale nature of the soil. Because they are more conveniently measured in engineering work than microscale factors, macroscale characteristics are primary indicators of swelling behavior. Commonly determined properties such as soil plasticity and density can provide a great deal of insight regarding the expansive potential of soils [2].

Soil consistency, as quantified by the Atterberg limits, is the most widely used indicator of expansive potential. Most expansive soils can exist in a plastic condition over a wide range of moisture contents. This behavior results from the capacity of expansive clay minerals to contain large amounts of water between particles and yet retain a coherent structure through the inter-particle electrical forces. Soil plasticity is influenced by the same microscale factors that control swell potential and provides therefore, a useful indicator of swell potential [2].

The electrical force fields between particles are related to the inter-particle spacing. Hence the dry density and physical arrangement of particles (that is. fabric) will affect the swell potential. Increased soil particles (that is. fabric) will affect the swell potential. Increased soil density through compaction or natural depositional history leads to greater amounts of swell and higher swell pressure [2].

2.2 Clay Formation

Clay minerals are formed through a complicated process from an assortment of parent materials. The parent materials include feldspars, micas, and limestone. The constituents of the parent material during the early and intermediate stages of the weathering process determine the type of clay formed. The nature of the parent material is much more important during these stages than after intense weathering for long period of time. The weathering processes by which clays are formed includes the following [1];

- Physical weathering: it changes the particle size and bulk volume of the parent material with no significant change in composition. The processes include expansion due to unloading, crystal growth, thermal expansion and contraction, organic activity and colloidal plucking.
- Chemical weathering: it causes a complete change in physical and chemical properties, accompanied by an increase in bulk volume caused both by the lesser density of the new compounds and by additional porosity of the weathered aggregate. It includes hydration, hydrolysis, oxidation, carbonation and solution. All these processes depend on the presence of water as the means by which they occur.
- Biological weathering: it is similar to chemical weathering in that it changes both the state of the aggregate and chemical composition.

The principal source of clay minerals is the chemical weathering of rocks which contain orthoclase feldspar, plagioclase feldspar and mica (muscovite) [12].

The three most important groups of clay minerals are montmorillonite, illite, and kaolinite which are crystalline hydrous aluminosilcates. Montmorillonite is the clay mineral that presents most of the expansive soil problems. The formation of clay or expansive clays or the montmorillonite clay is favored in alkaline environment and the absence of leaching the presence of ferromagensium minerals in the parent materials and the presence of bases. Prolonged leaching under high temperatures or tropical conditions with ferric iron parent rocks favors the formation of minerals of kaolinite group which are non expansive. The presence of potash in the parent mineral under these conditions results in formation of illite [1].

2.3 Clay Structure

Clay minerals are predominantly silicates of aluminum and/or iron and magnesium. Some of them also contain alkalies and/or alkaline earths as essential components. These minerals are predominantly crystalline in that the atoms composing them are arranged in definite geometric patterns. Most of the clay minerals have sheet or layered structures. A few have elongated tubular or fibrous structures [12].

The result of studies using the electron microscope and X-ray diffraction techniques show that the clay minerals have a lattice structure, in which the atoms are arranged in several sheets similar to the pages of a book. The arrangement and the chemical composition of these sheets determine the type of clay mineral. The basic building blocks of the clay minerals are the silica tetrahedron and the alumina octahedron. The blocks combine into tetrahedral and octahedral sheets to produce various types of clays [1].

The silica tetrahedron consists of a silicon atom surrounded tetrahedrally by oxygen ions as shown in figure 2.1 (a). The alumina octahedron consist of an aluminum atoms surrounded octahedrally by six oxygen ions as shown in figure 2.1 (b). When each oxygen atom is shared by two tetrahedral, a plate-shaped layer is formed [1]. All the possible combinations of these basic units to form clay minerals produce a net negative charge on the exterior of the clusters. A soil water suspension will thus have an alkaline reaction (PH > 7) unless the soil is contaminated with an acidic substance [12].

Some of the most common clay minerals are the following;

- Kaolinite

The kaolinite structural unit consists of alternating layers of silica tetrahedral with the tips embedded in an aluminum (gibbsite) octahedral unit. The alternating of silica and gibbsite layers produces what is sometimes called a 1: 1 basic unit. The kaolinite cluster is a stacking of 70 to 100 or more sheets as a book with hydrogen bonds and van der Waals forces at the interface. The resulting formula is approximately as follows [12];

$$(OH)_8Al_4Si_4O_{10}$$

The bonding combination of hydrogen and van der Waals forces results in considerable strength and stability with little tendency for inter-layers to take on water and swell. Kaolinite is the least active of the clay minerals. Kaolinite can be produced by weathering of certain of the more active clay minerals as well as being directly formed as a byproduct of rock weathering. Kaolinite tends to be found in regions of heavier rainfall [12].

- Illite

The illite clay mineral consists of an octahedral layer of gibbsite sandwiched between two layers of silica tetrahedral. This produces a 1 : 2 mineral with additional difference that some of the silica positions are filled with aluminum atoms and potassium ions are attached between layers to make up the charge deficiency. The bonding results in a less stable condition than for kaolinite, and thus activity of illite is greater than kaolinite. Its general equation is [12];

$$(OH)_4K_y(Si_{8-y}.Al_y)(Al_4.Mg_6.Fe_4.Fe_6)O_{20} \ ,$$

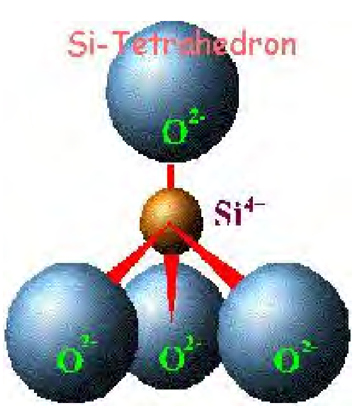
Where y is between 1 and 1.5

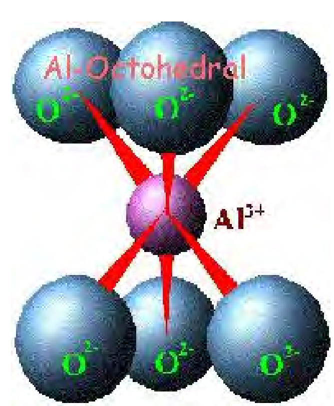
Illite basically is derived principally from muscovite (mica) and biottites and is sometimes called mica clay. Illite clays tend to be found in areas of moderate rainfall [12].

- Montmorillonite

The montmorillonite clay mineral is made of sheet like units ordered, also as a 1 : 2 unit. The inter-sheet bonding is due to mainly to van der Waals force and is, thus, very week relative to hydrogen or other ion bonding. Various substitutions take place, including Al for Si in the tetrahedral and Mg, Fe, Li, or Zn, for Al in the octahedral layer. These exchanges result in a relatively large net unbalanced negative charge on the mineral, with resulting large cation exchange capacity and affinity for water with H^+ ion in the absence of metallic ions. Weathering of montmorillonite clay minerals often produces kaolinite clay. It is found in the more arid regions of the world. Its general formula is [12]

$$(OH)_4Si_8Al_4O_{20} \cdot nH_2O$$


Si-Tetrahedron
O^{2-}
Si^{4-}
O^{2-}
O^{2-}
O^{2-}


Al-Octohedral
O^{2-}
O^{2-}
O^{2-}
Al^{3+}
O^{2-}
O^{2-}
O^{2-}

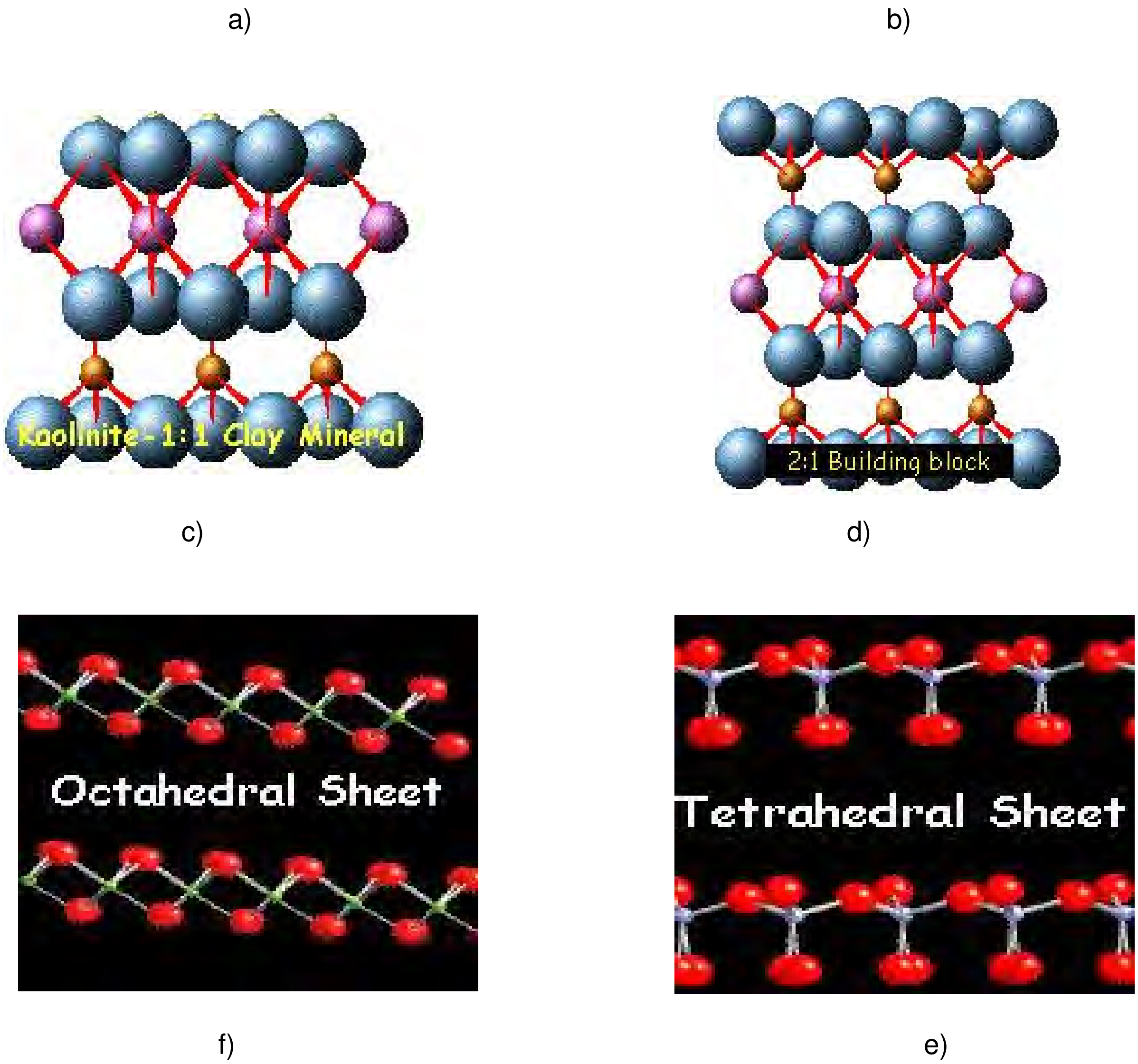


Figure 2.1 Silica tetrahedron, alumina octahedral, clay mineral building blocks, octahedral and tetrahedral sheets [14]

2.4 Physical Properties of Clay Soils

It is well known to soil engineers that montmorillonite clays swell when the moisture content is increased, while swelling is absent or limited in illite and kaolinite. The types of soils, and the conditions under which the most critical situations exists, can be outlined as follows [1]:

- Moisture Content

Irrespective of high swelling potential, if the moisture content of the clay remains unchanged, there will be no volume change; structures founded on clays with constant moisture content will not be subjected to movement caused by heaving. When moisture content of the clay is changed, volume expansion, both in the vertical and horizontal direction, will takes place. The initial moisture content of the expansive soils controls the amount of swelling. This is true both for soils in undisturbed and in remolded states.

- Dry Density

Directly related to initial moisture content, the dry density of the clay is another index of expansion. The dry density of the clay is also reflected by the standard penetration resistance test results. Clays with penetration resistance in excess of 15 usually possess some swelling potential [1].

2.5 Stabilization of Expansive Soils

Treatment procedures that are available for stabilizing expansive soils before and after construction of structures and highways include [2];

- Removal and Replacement
- Remolding and Compaction
- Surcharge Loading
- Prewetting
- Moisture Control
- Chemical Admixtures

2.5.1 Removal and Replacement

Removal of expansive soils and replacement with non expansive soils is one method to provide stable foundation material. In some cases the expansive strata may be entirely removed. Generally, the expansive layer extends to a depth too great to economically allow complete removal and replacement. One mechanism by which the removal and replacement method mitigates expansive potential is by the control of the moisture content in the underlying clay layer [2].

The pertinent requirements concerning soil replacement are the type of replacement material, the depth of replacement, and the extent of replacement [1].

Some advantages of treatment by removal and replacement are [2];

- Non expansive soils can be compacted at higher densities, yielding higher bearing capacities than can be produced by prewetting the expansive clay or compacting it at low densities.
- Removal and replacement require less delay to construction than some other procedures such as prewetting.

Some disadvantages of removing and replacing the expansive soil are [2];

- Non expansive, preferably impervious, fill must be obtained. This can have a significant cost factor if the fill must be imported.
- The required thickness of the non expansive fill material may be too great to be practical.
- Granular fill may serve as a reservoir and provide long term sources of water to foundation or subgrade soils.

2.5.2 Remolding and Compaction

The swell potential of expansive soils can be reduced by decreasing the dry density. Compaction at low densities and at water contents above the optimum water content, as determined by standard proctor test, produces less expansion potential than compaction at high densities and low water contents [2].

Some advantages of remolding and re-compacting include the following [2];

- Cost of importing selected material is eliminated.
- If compaction is performed properly, the impermeable fill will minimize migration of water into underlying soils.
- It is economically feasible to scarify, pulverize, and recompact expansive soils.

Some disadvantages of remolding and re-compaction control include [2];

- The lower bearing capacity of the low density compaction may not be adequate. However, for light structures, the bearing capacity of clays compacted at low densities is generally adequate.
- Some soils have such high potential for volume change that compaction control doesn’t reduce swell potential significantly and replacement may be necessary.
- Compacting at specified densities and water contents may necessitate frequent testing to maintain quality control, which may increase the cost of the project.

2.5.3 Surcharge Loading

Swell can be prevented if expansive clays can be loaded with a surcharge large enough to counteract the expected swell pressures. This method is most effective when swell pressures are low and some heaving can be tolerated in the construction project, such as in secondary highway system. However, many soils exhibit swell pressure too high to be controlled by normal surcharge loads alone [2].

2.5.4 Prewetting

Prewetting or ponding is based on the theory that increasing the moisture content in the expansive foundation soils will cause heave to occur prior to construction and there by eliminate problems afterward. It is assumed that if the high moisture content is maintained, there will be no appreciable increase in soil volume to damage the structure. This procedure may have serious drawbacks that limit its application. Expansive soils typically exhibit low hydraulic conductivity and the time required for adequate wetting can be up to several years. Furthermore, after the water has been applied for long periods of time serious loss of soil strength can result causing reductions in bearing capacity and slope stability. Another major drawback to the use of this procedure is that after a prolonged period of surface ponding, the wetting front of the infiltrating water will have advanced to only to a depth much less than that of the active zone. Redistribution of water throughout the active zone can continue after construction due to the high water content in the zone above the wetting front. The continued migration of water into lower layers can result in continued heave after construction [2].

2.5.5 Moisture Control

Soil expansion problems are primarily the result of fluctuations in water content. Non uniform heave will result from either non uniform water content changes, non-uniform soil conditions, or both. If fluctuations in water content over time can be minimized and if the water content in the subsoil can be made uniform, a major part of the expansion problem can be mitigated [2].

The placement of a structure or pavement on the ground surface will change the evapo-transpiration from the surface. Changes in land use, such as irrigation of landscaping, will change the potential for infiltration. These changes will, in turn, change the water content and its distribution in the subsoils.

If the change in water content can be made to occur slowly and if the water content distribution can be made uniform, differential heave can be minimized [2].

Moisture barrier offer a viable solution in this regard. The basic principle on which moisture barriers act is to move edge effects away from the foundation or pavement and minimize seasonal fluctuation of water content directly below the structure. Also the time during which moisture changes occur is longer because the barrier increases the path length for water migration under the structure. This allows for water content to be more uniformly distributed due to capillary action in the subsoil. Thus the heave will occur more slowly and in more uniform fashion [2].

The various treatment alternatives presented above can be employed either singly or in combination to obtain the anticipated end result. The choice of the techniques to be used is assessed basically with respect to economic factors [2].

2.5.6 Chemical Admixtures

Soil stabilization has been long recognized as the “art” of improving the behavior of foundation materials through careful selection of moisture control and compaction. Various organic and inorganic fractions of different soil types undergo modification when a “catalyst” agent is introduced into the soil. In turn, the “chemical reaction” converts inferior and formerly unsuitable materials to highly satisfactory roadbed materials [7].

Description of the stabilization mechanisms and result of research conducted on each type will be given below. The stabilizers selected for the discussion are lime, DRP, con-aid, cement and RBI Grade 81.

a) Lime stabilization

Stabilization as applied to highway construction can be defined as a means of permanently consolidating soils and base materials by markedly increasing their strength and bearing capacity and decreasing their water sensitivity and volume change during wet/dry cycles. To achieve stability an additive can be incorporated with the soil. This additive is particularly effective with clay-bearing soils and aggregates, with which it reacts both chemically and physically to yield quality road building materials [5].

According to Webster and precise definition, lime can only refer to quicklime (calcium oxide) or hydrated lime (calcium hydroxide), which are burned forms of lime stone (calcium carbonate). Lime stabilization, embraces only the burned lime products-not pulverized limestone [5].

The most common form of commercial lime used in lime stabilization is hydrated high calcium lime, Ca $(OH)_2$. But, monohydrated dolomitic lime, $Ca(OH)_2$, MgO, calcitic quick lime, CaO, and dolomitic quicklime, CaOMgO are also used [10]. Quick limes or hydrated limes can improve the engineering properties of heavy clay soils or granular soils with silt-clay fractions. Clayey gravels materials have been successfully stabilized for use as pavement basis [2].

Quicklime is an excellent stabilizer if the material is very wet. When it comes in contact with the wet soil the quicklime absorbs a large amount of water as it hydrates. This process is exothermic and the heat produced acts as a further drying agent for the soil. The removal of water and the increase in plastic limit cause a substantial and rapid increase in the strength and trafficability of the wet material [10].

When lime is added to a cohesive soil, calcium ions replace sodium ions in the clay fraction until the soil becomes saturated with calcium and the PH rises to a value in excess of 12 (i.e. highly alkaline). The solubility of silica and alumina in the soil increase dramatically when the PH is greater than 12, and their reaction with lime can then proceed producing cementitious calcium silicates and aluminates. Amorphous silica reacts particularly well with lime. The cementitious compounds form a skeleton that holds the soil particles and aggregates together [5].

i) Application of lime

By lime-stabilization, both the ion exchange reaction and the production of cementitious materials increase the stability and reduce the volume change within the clay fraction. The chemical reaction that occurs between lime and soil is quite complex [10]. Application of lime to pavement subgrade can provide significantly improved engineering properties, these are essentially two forms of improvements [3]:

a) Short term changes to soil, called soil modification
b) Long-term improvements to the soil characteristics, called soil stabilization.

Whether the soil is modified or stabilized depends primarily on the soil properties, quantity and quality of stabilizing agent and how well the soil is compacted [3].

Modification of Clay by Lime: At the microscopic level, clay particles tend to form into sheets or platelets with a negative ionic charge along the length of the particle. Positively charged cations are attracted to the surfaces. These ions are not orderly arranged at the clay surface but form a diffused layer. Water is attracted to the particles. The clay particles, surrounded by water, align in a parallel pattern that offers little resistance to shear forces (see figure 2.2). The clay can expand as more water is absorbed [3].

When lime is introduced, the Ca^{++} replaces the cations that are normally adsorbed at the clay surface. This cation exchange occurs because divalent calcium cations normally replace cations of single valance and ions in high concentration will replace those in a lower concentration [3].

This cation exchange dramatically reduces the size of the water layer surrounding the clay particles. The clay particles can now approach each other more closely. An edge-to-face attraction now occurs as shown in figure 2.3. The particles flocculate and agglomerate with edge-to-face particle contact. This arrangement has much higher internal friction and shear strength than the face-to-face particle arrangement in the untreated clay. The texture of the soil distinctly changes from plastic and sticky to crumbly and sand-like (friable) [3].

Soil-Stabilization for long term improvement: In addition to the immediate benefit of lime modification, lime can improve the long term properties of pavement subgrade, including [3]:

a) Substantial increases in subgrade strength
b) Continued strength gain with time
c) Resistance to moisture infiltration
d) Long term durability over decades of service even under severe environmental conditions (freezing-thawing)

These long term benefits come from stabilizing the soil with an adequate amount of lime, lime and fly ash, or lime kiln dust through pozzolanic reactions [3].

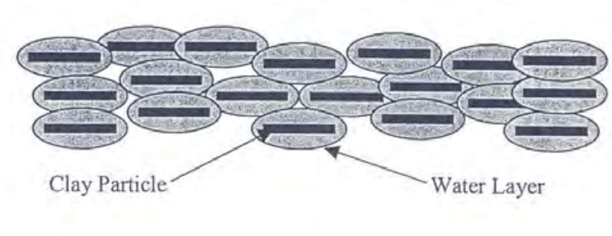


Figure 2.2 Parallel arrangements of natural clay particles [3]

Figure 2.3 Edge-to-face contacts of lime-treated clay particles [3]

Pozzolanic reaction (the key to long term stability): Many, but not all, soils containing clay have significant amount of silica and alumina to react with lime to produce pozzolanic reactions. A pozzolan is defined as a finely divided siliceous or aluminous material that in the presence of water and calcium will form a cemented product. The reaction is illustrated by the following equations [3]:

Ca^{++} + OH + Soluble clay silica $\longrightarrow$ Calcium Silica Hydrate (CSH)
Ca^{++} + OH + Soluble clay alumina $\longrightarrow$ Calcium Aluminate Hydrate (CAH)

With an adequate amount of lime and water, lime-water-clay soil system has a PH high enough to solubilize silica and alumina for pozzolanic reaction. As long as clay-alumina and the PH remain high enough to maintain solubility, this pozzolanic reaction will continue [3].

What is unique about this pozzolanic reaction is the cooperative reaction between the lime and the clay. The lime induces high PH environment that solubilizes the silica and alumina. The lime also provides the free residual calcium that combines with silica and alumina from the clay to produce the pozzolanic reaction [3].

The extent to which this pozzolanic reaction proceeds is influenced primarily by the natural soil properties. With some soils, even some containing a substantial amount of clay, the pozzolanic reaction is inhibited and cementing agents are not extensively formed. Because natural soil properties vary so greatly, significant variations occur from different soils stabilized with lime [3].

ii) Construction practices of lime stabilization

The following discussions elaborate further on the best current lime stabilization practices in road constructions [5];

Scarification and pulverization

The grader-scarifier and disc harrow are commonly used for initial scarifying, and the disc harrow and rotary mixer for pulverizing. When the soil is unusually dry, water is added to aid pulverization; if extremely wet, the rotary mixer or disc harrow can be used for aerating and drying out the soil, particularly the heavy clays [5].

Although the common practice is to scarify before spreading lime the reverse procedure can be possible. The main disadvantage of this procedure however pertains to weather conditions; when lime is placed on a smooth surface, there is greater chance for loss due to wind and rain particularly if mixing is not started immediately [5].

Lime spreading

Both dry (including bag and bulk) and slurry methods have been used successfully for applying lime. In the former method it is advisable to sprinkle the roadway lightly prior to lime spreading to reduce dusting [5].

1. Dry lime-bags

The use of bagged lime is generally the simplest but also the most costly method. Bags are delivered in dump or flatbed trucks and spotted by hand to give the required distribution. After the bags are spotted properly, they are slit with a knife or shovel and the lime dumped into piles or transverse windrows across the roadway. The lime is then leveled by either hand raking or by means of a spike-tooth harrow or drug pulled by a tractor or truck. Generally, two passes are required to level the lime. Immediately thereafter, the lime is sprinkled to reduce dusting. Disadvantage of the bag method over the dry bulk and slurry method are higher cost of lime (due to cost of bagging), greater labor cost due to considerable extra physical handling and slower operation [5].

2. Dry Lime-bulk

For large stabilization projects, particularly where dusting is no problem, the use of bulk lime has become common practice. Lime is delivered to the job either in self-unloading transport trucks or dump trucks. With the auger trucks, spreading is handled by means of a mechanical-type spreader attached to the rear or through metal downspouts or flexible rubber boots. The mechanical spreaders incorporate belt, screw, rotary vane, or drug chain conveyors to distribute the lime uniformly across the spreader width. The rate of lime application can be regulated by varying the spreader opening, spreader drive speed, so that the required amount of lime can be applied in one or two passes. Obviously, the self-unloading tank truck is the least costly method of spreading lime, since there is no rehandling of material and large payloads can be carried and spread quickly [5].

3. Slurry method

In this method hydrated lime and water are mixed in to slurry, either in a central mixing tank, jet mixer, or in a tank truck; in either case the slurry is spread over the scarified roadbed through the tank truck spray bars. The slurry is distributed by one or more passes over a measured area until a specified percentage (based on lime solids content) is obtained. To prevent run-off and consequent non uniformity of lime distribution, slurry is mixed immediately after each spreading pass. The slurry method is also necessarily disadvantageous on wet soils, and during wet weather, when the soils are already near or over optimum moisture content. Further, its use is generally limited to jobs requiring smaller amounts of lime (4% or under) , since at higher percents so much water is required to carry the lime in suspension that the soil would be over optimum most of the time [5].

Mixing and watering

Prior to compaction, lime stabilized mixtures should be pulverized so that 100% of the binder content passes a 1-inch screen and 60% passes a #4 sieve, exclusive of non-slaking fractions. To meet this specification, two-stage mixing is generally required for the heavy clay soils used in subgrade work, one-stage mixing for granular bases. This difference is due to the fact that the heavy clays are much more difficult to break down than the granular soils containing less clay [5].

Compaction

Clay subgrades can also be compacted soon after final mixing, although delays of up to four days are not injurious. When longer days (two weeks or more) can not be avoided, it may be necessary to incorporate a small amount of lime in to the soil (example, ½%) to compensate for losses due to carbonation and erosion. Various rollers and layer thickness have been used in lime stabilization. The most common practice is to compact in one lift, using the sheepsfoot roller until it “walks out”, followed by a multiple-wheel pneumatic roller; the flat wheel roller is then used in finishing. During compaction light sprinkling may be required, particularly during hot, dry weather, to compensate for evaporation losses [5].

b) DRP or SA-44/LS-40 chemical stabilization

Dallas roadway stabilizer system is a proprietary liquid acid chemical road base stabilizer that when introduced into the road base during road construction, results in a chemical and physical reaction. A result of this reaction is that a portion of the naturally occurring binding elements in the road base, such as calcium lime, and other minerals and rock fines are liquefied. These liquefied mineral particles are then forced into the voids between the aggregate during compaction of the road base. Upon curing they re-solidify to form a strong bond between the aggregate resulting in a stabilized road base with more strength and bearing capacity than the same untreated road base. DRP or SA-44/LS-40 stabilizer system acts as an anti-swelling material, successfully solving heaving problem in roadways and foundations [7].

i) Physical description of SA-44/LS-40

The SA-44/LS-40 or DRP chemical stabilizer is comprised of a liquid sulfuric acid chemical stabilizer (primary: SA-44) and lignosulfonate stabilizer (secondary: LS-40). The primary acidic stabilizer induces the chemical reactions necessary for the strength increase by pozzalonic reactions. The reactions include forming liquefied zones between soil particles, which re-solidifies into stronger bonds between some particles during the curing process. The secondary lignosulfonate stabilizer is used for minimizing brittleness, controlling dust and increasing the shear strength of the re-solidified soil [8].

SA-44/LS-40 solutions consist of solvents and surfactant, one or more acid oxidizers and a common dispersing agent. The solvents and surfactant “wash” the soil particles thereby enabling the oxidizers to the catalyst to speed up the chemical action. The dispersing increases the reactivity of the chemical soil road base stabilizer component by reducing surface tension and facilitating the flow of the chemical stabilizer through the soil to ensure uniform treatment of the stabilizer layer [7].

ii) Typical physical and chemical characteristics

The components that comprise SA-44/LS-40 chemical soil/road base stabilizer include:

1. Proprietary liquid acid chemical soil/road base-stabilizer that when introduced into the soil/road during road construction produces a chemical and physical reaction within existing soil/road base mass. This reaction includes the naturally accruing chemical elements such as calcium, lime and other minerals and rock fines which are “liquefied”. These liquefied mineral particles are then forced into the voids of the soil and aggregate during compaction, thus eliminating “channels” for capillary action and the migration of moisture through the compacted soil road base mass. Upon

curing, the re-solidification of material creates a strong bond between the soil and aggregate in the stabilized road base with measurably added strength and bearing capacity beyond that of the same untreated road base [7].

2. LS-40 soil treatment must be used with the chemical stabilizer to minimize brittleness and enhance the shear strength of the re-solidified and compacted soil/road base following treatment and curing. The LS-40 has secondary effect of further reducing capillary action through the compacted mass. A tertiary benefit of the LS-40 is dust control [7].

iii) Application practices of SA-44/LS-40

Mixing ratio and application

The normal ratio is one part SA-44 to one part LS-40 for normal stabilization [7]. Once the required amount of SA-44/LS-40 stabilizer has been determined it is poured into a water distributor with spray-bar, and diluted to a ratio of two litters of SA-44/LS-40 concentrated stabilizer to 200 litters of water. Even though the concentrated chemical stabilizer is readily soluble in water, the operator should ensure that the solution is adequately mixed. Application, by calibrated water distributor should be made in at least two applications. Two applications ensure that spots missed during the first application are caught during the second application. More than two applications are acceptable so long as the proper amount of SA-44/LS-40 solution is used [7].

The next step is to windrow the material across the width of the roadway being stabilized. Repeated passes of the grader or other equipment will ensure that the solution is thoroughly mixed into the base material. As noted above, extra water may need to be added to obtain optimum moisture levels in accordance with achieving specified maximum in place soil densities. The ultimate goal is to uniformly mix the solution into the base material leaving no “unwetted” areas [7].

When ‘’pulverize-mixer’’ equipment is used, the application process involves [7];

1. Scarification
2. a single application of diluted chemical soil road base stabilizer onto the scarified material
3. pulverizing mixing with a pulverize mixer
4. leveling, and
5. compaction

‘’Pulverizing mixing’’ the preferred method routinely produces a more uniform mixture of base material and stabilizer, and more complete wetting of all the soil/soil aggregate particles, thereby resulting in a more uniformly treated stabilized layer [7].

Compaction

Once the soil/road base material is totally wetted and optimum moisture (plus 1-2% moisture content) is obtained, compaction should begin. As the moisture content is increased above optimum, the soil becomes increasingly more workable [7].

Quality control tests of in place density using Sand-Cone, Nuclear Density Field Test Equipment, and other approved specified methods must be applied to ensure that specified maximum density levels of compaction are obtained [7].

c) Con-Aid Chemical Stabilization

Con-aid is a surfactant (surface active agent) that changes the hydrophilic (water adsorption) nature of clay minerals to hydrophobic (non affinity for water). The reaction of con-aid on clay minerals is particularly effective because of the ion-exchange capacity of clay minerals the property that the clay minerals have of adsorbing certain ions thereby changing its physical properties [21].

Con-aid is a cation reactive synthetic compound which forms protective, oily clay layers on the surface of soil and clay particles. It reduces ion mobility and ion exchange and simultaneously makes the material hydrophobic by eliminating the absorption of water. The result is a soil material that is much sensitive to moisture, more workable and it can be compacted to a better particle interlock system by equipment and traffic forces [21].

The effect of con-aid on Atterberg limits, moisture-density relationship and CBR value of a soil were investigated in Addis Ababa University, by Daniel Nebro in 2002. To investigate the effect the soil was treated with the con-aid chemical using volumetric application rate while the manufacturer recommended an application rate in aerial coverage. Based on the results of the laboratory tests of this investigation, the following conclusions are made [20].

1. Con-aid treatment didn’t cause a significant change in the Atterberg limits of the expansive soil investigated here.
2. Neither the optimum moisture content nor the maximum dry density as determined by AASHTO T-99 was appreciably affected.
3. Con-aid had no effect on the soaked CBR and CBR swell of the treated soil.

d) Cement Stabilization

Strength gain in soils using cement stabilization occurs through the same type of pozzolanic reactions found using lime stabilization. Both lime and cement contain the calcium required for the pozzolanic reactions to occur; however, the origin of the silica required for the pozzolanic reactions to occur differs. With lime stabilization, the silica is provided when the clay particle is broken down. With cement stabilization, the cement already contains the silica without needing to break down the clay mineral. Thus, unlike lime stabilization, cement stabilization is fairly independent of the soil properties; the only requirement is that the soil contains some water for the hydration process to begin. Similar to lime stabilization, carbonation can also occur when using cement stabilization [19].

When cement is exposed to air, the cement will react with carbon dioxide from the atmosphere to produce a relatively insoluble calcium carbonate. Thus, similar to lime, proper handling methods and expedited construction procedures should be employed to avoid premature carbonation of cement through exposure to air [19].

Unlike lime stabilization, the goal of mixture design using cement stabilization is to find the lowest cement content that will produce a desired strength. The strength gain of soil-cement mixtures increases linearly with cement content. Accordingly, many mixture design procedures involve molding and curing specimens at varying cement contents until the lowest cement content that provides the required strength is achieved. The primary factor governing the behavior of cement-stabilized soil is the water-cement ratio. The water-cement ratio is defined as the ratio of moisture content of the soil to the cement content, with both the moisture content and cement content expressed in terms of dry weight of soil. Test results indicated that increasing water-cement ratio produced decreasing strength of the cement-stabilized soil [19].

As per the conclusion made from an investigation undertaken on cement stabilization in Addis Ababa University, by Aksum Tesfaye in 2001, high percentages of cement applications (i.e. 15% cement) result in reduction in expansion potential from highly expansive to moderately expansive soil.

e) RBI Grade 81 Chemical Stabilization

This stabilizer, which is known as RBI Grade 81, is produced in powdered form by Anyway Solid Environmental Solutions, LTD. Due to the proprietary nature of the stabilizer, the constituents of the product are not revealed by the manufacturer [19].

According to the manufacturers manual, RBI Grade 81 Natural Soil Stabilizer is composed of a suite of natural components that act together to produce an extremely effective, inexpensive and environmentally friendly product for social and economic development through infrastructure creation. Soil stabilization with RBI Grade 81 is effected by means of a hydration process. Mixing of soil with RBI Grade 81 stabilizer creates a heat emitting chemical reaction (exothermic) similar to the reaction in concrete. During the reaction, the natural soil and the stabilizer exchange ions creating ionic connections between the various material particles. The pores in the soil are filled with RBI Grade 81 crystalline reaction products allowing mechanical ties to be created between the particles. At the end of the process, a soil matrix has been created throughout the stabilized system due to the formation of complex hydration products. Changes that occur in the stabilized soil due to exposure to RBI Grade 81 cause modification of its inductive and engineering properties and performance. Due to the fact that the chemical process continues over a period of time, the changes in the soil properties also improve over time [19].

Stabilization using RBI Grade 81 is subject to the quality of materials available and the impact of the environment (traffic, climate, etc.) on the structural design. The producer claims that stabilization with RBI Grade 81 endeavors to increase the quality of the project and reduce construction costs by improving the properties of substandard, readily available material to comply with the relevant specifications [19].

An M. Sc. research is currently underway to investigate the effect of RBI grade 81 stabilizer on both the light grey and the red clay soils of Addis Ababa investigated in this work.

3.0 LABORATORY TEST AND RESULTS

3.1 Tests on Natural Soils

The study was carried out on a light grey and a red clay soil of Addis Ababa city. The light grey clay soil samples were collected from Bole High School of GPS point (N 0476579, E 0994551 and Z 2347m). Also the Red clay soil samples were collected from Addisu Gebeya area of GPS point (N 0470845, E 1000830 and Z 2572m). Both soil samples were taken from 2m depth below the ground surface. The following laboratory tests were undertaken on these soil samples;

- Soaked CBR and CBR swell tests
- Swelling pressure test
- Atterberg limit tests and
- Hydrometer test
- Percent swell and free swell tests

3.2 Test Results of Natural Soils

The summary of the test results on the natural soils is indicated in Table 3.1, whereas the standard testing procedures followed while undertaking each test are indicated in subsection 3.4. From the summary of the test result shown in Table 3.1, the light grey clay soil can be characterized as highly expansive and of high plasticity. The red clay soil is non-expansive and of relatively low plasticity.

Table 3.1 Test Results of Natural Light Grey and Red Clay Soil Samples

Light Grey Clay Soil										
CBR Value %	Swelling Pressure Kpa	LL	PL	PI	Soil Classification		Percent Swell, %	Free Swell %	%,Passing No. 200 Sieve	MDD,g /cc OMC, %
					AASHTO	USCS				
2.89	299.0	109	42	67	A-7-5	CH	14.20	158	99.19	1.23 37.09
Red Clay Soil										
CBR Value %	Swelling Pressure Kpa	LL	PL	PI	Soil Classification		Percent Swell, %	Free Swell %	%, Passing No. 200 Sieve	MDD,g /cc OMC, %
					AASHTO	USCS				
6.99	--	62	30	32	A-7-5	CH	--	--	95.76	1.33 34.87

3.3 Preparation of Treated Soil Samples

The collected natural soil samples of the light grey and the red clay soils were treated with the SA-44/LS-40 chemical and lime as described in subsection 1.4. Subsequently, the treated soil samples were cured. Then the following laboratory tests were carried out on the cured soil samples;

- Soaked CBR and CBR swell tests
- Swelling pressure test
- Atterberg limit tests and
- Percent swell and free swell tests

The natural moisture content of the natural soil is determined after the samples are air dried and pulverized. Subsequently, compaction tests were undertaken to determine the optimum moisture content of the natural soil samples. Then the volume of SA-44/LS-40 chemical required for each application rate and the volume of water required to dilute the chemical are determined. Simultaneously, the mass of lime required for each application rate is also established. The volume of water required to dilute the chemical is 200 times the volume of the chemical. Subsequently, the soil samples are mixed with the diluted SA-44/LS-40 chemical and/or the mass of lime required for each mix. Following this, the extra amount of water required to bring the treated soil to optimum moisture content is added and mixed well. Finally, the treated soil samples were cured for fourteen days in two different ways. Samples for CBR and CBR swell tests were cured in compacted state in the CBR molds in a cold room, whereas for the rest of the tests, the curing was made in a loose state of the soil packed in plastic bags. The curing is taken place in the laboratory without controlling both temperature and humidity of the curing room. The details of the mix design are given in Annex - 1.

3.4 Test Results of Treated Soil Samples

3.4.1 CBR and CBR Swell Tests and Results

The CBR test indirectly measures the shearing resistance of a soil under controlled moisture and density conditions. The CBR is obtained as the ratio of load required to effect a certain depth of penetration of a standard penetration piston into a compacted specimen of the soil at some water content and density to the standard load required to obtain the same depth of penetration on a standard sample of crushed stone.

In equation form, this is

CBR= <u>Test Load on the Sample   </u> * 100%

Standard Load on Crushed Stone

CBR tests were conducted on the compacted specimens at the optimum moisture content using standard compaction test.

The compacted soil samples of the CBR mold are soaked for 96 hours in a water bath to get the soaked CBR value and the CBR swell of the soil. The CBR swell of the soil is measured by placing the tripod with the dial indicator on the top of the soaked CBR mold. The initial dial reading of the dial indicator on the soaked CBR mold is taken just after soaking the sample. At the end of 96 hours the final dial reading of the dial indicator is taken hence the swell percentage of the initial sample length is given by [4],

$$\text{Percent Swell (CBR Swell)} = \frac{\text{Change in Length in mm during Soaking}}{116.43 \text{ mm}} * 100\ \%$$

The CBR and CBR swell tests were conducted on the cured soil samples of both the light grey and the red clay soils. Testing procedures of AASHTO T 193-93 was followed for the evaluation of CBR value and CBR swell. The details of the tests are provided in Annex -3.

The summary of CBR and CBR swell test results and normalized CBR of the treated light grey and red clay soils are given in Tables 3.2 and 3.3, respectively. The CBR values of various mixes are normalized with respect to the CBR value of the natural soil. The results for the light grey and red clay soils are presented in bar chart form in Figure 3.1 and 3.2, respectively.

The results of CBR and CBR swell tests of the treated light grey clay soil show up to four fold increments in the CBR value and up to ninety three percent reductions in the CBR swell from the natural light grey clay soil. Additionally, the results of CBR tests of the treated red clay soil show up to three fold increments in the CBR value.

Table 3.2 CBR and CBR Swell Test Results of Light Grey Clay Soil

Mix Code	Mix Type	CBR Value (%)	Normalized CBR Value	CBR Swell (%)
1	Natural Soil	2.89	1.00	5.95
2	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 alone	0.87	0.30	0.84
3	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 alone	0.58	0.20	0.98
4	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 alone	1.93	0.67	0.58
5	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 and 2% Lime	2.47	0.85	2.26
6	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 and 2% Lime	2.23	0.77	3.64
7	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 and 2% Lime	6.50	2.25	1.29
8	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 and 4% Lime	7.61	2.63	0.50
9	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 and 4% Lime	11.46	3.97	0.67
10	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 and 4% Lime	6.26	2.17	1.18
11	Natural Soil Plus 4% Lime alone	3.98	1.38	1.47
12	Natural Soil Plus 8% Lime alone	5.51	1.91	0.40

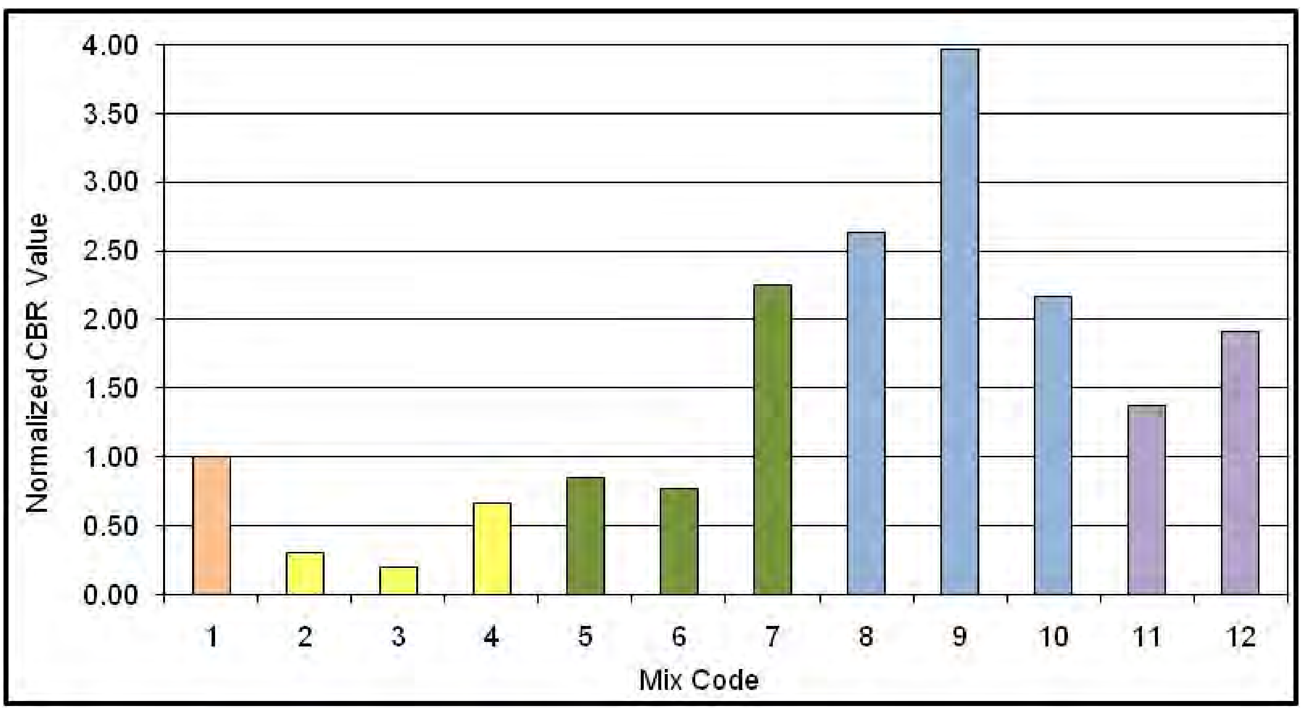


Figure 3.1 Normalized CBR Value vs. Mix Code of Light Grey Clay Soil

Table 3.3 CBR and CBR Swell Test Results of Red Clay Soil

Mix Code	Mix Type	CBR Value (%)	Normalized CBR Value	CBR Swell (%)
1	Natural Soil	6.99	1.00	0.40
2	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 alone	7.49	1.07	0.39
3	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 alone	3.11	0.44	0.42
4	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 alone	1.56	0.22	0.12
5	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 and 2% Lime	5.10	0.73	0.32
6	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 and 2% Lime	9.05	1.29	0.34
7	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 and 2% Lime	7.25	1.04	0.44
8	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 and 4% Lime	20.28	2.90	0.32
9	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 and 4% Lime	15.94	2.28	0.36
10	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 and 4% Lime	13.21	1.89	0.53
11	Natural Soil Plus 4% Lime alone	12.13	1.74	0.36
12	Natural Soil Plus 8% Lime alone	4.44	0.64	0.14

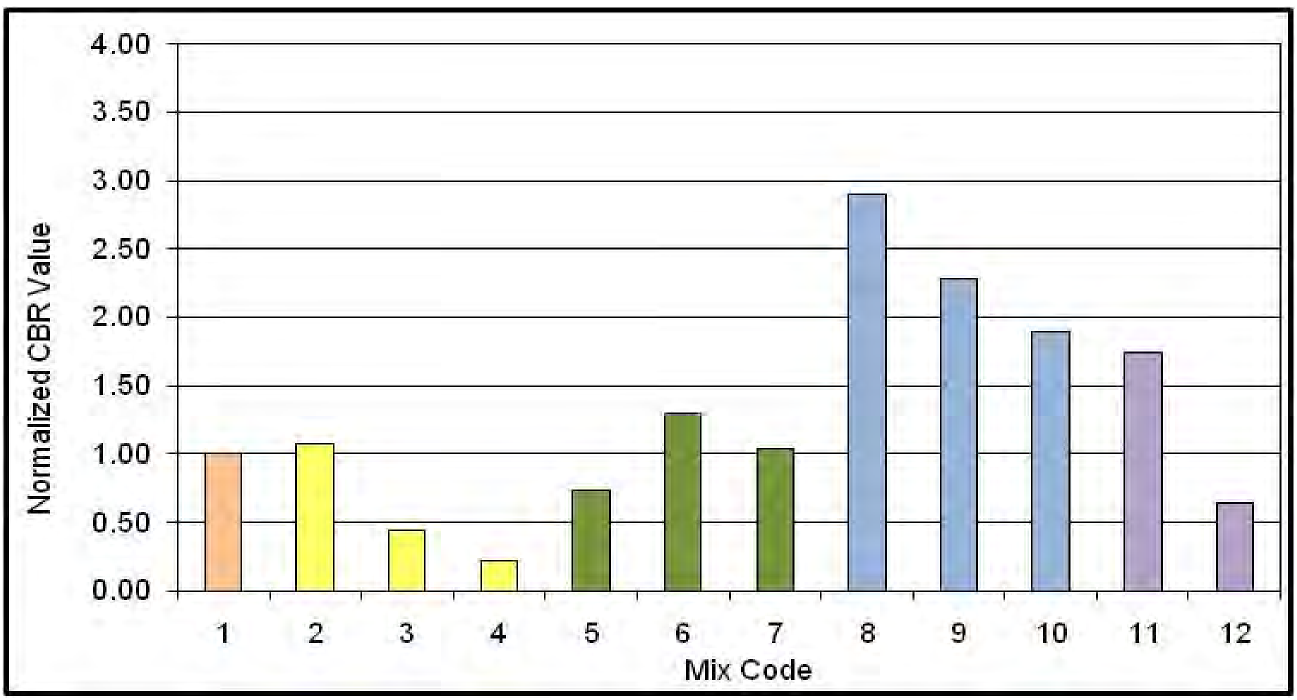


Figure 3.2 Normalized CBR Value vs. Mix Code of Red Clay Soil

3.4.2 Swelling Properties

i) Swelling Pressure Tests and Results

Swelling pressure is the pressure, which prevents the specimen from swelling or that pressure which is necessary to retain the specimen back to its original state (void ratio, height) after swelling [1].

The samples are placed in the consolidation ring of height of 20mm then they are subjected to a vertical pressure of 6.9 Kpa. Up on completion of the consolidation, water is added to the sample. When swelling of the sample has ceased the vertical stress is increased in increments until it has compressed to its original height. The stress required to compress the sample to its original height is commonly termed the zero-volume-change swelling pressure.

The above testing procedure was followed for the evaluation of swelling pressure. The swelling pressure tests were conducted on the treated and natural samples of the light grey clay soil. The details of the tests are given in Annex -2. In addition, swelling pressure test was also conducted on the natural red clay soil sample. However, the test results reveal that this soil is non expansive. Hence swelling pressure tests were not conducted on the treated red clay soil samples.

The summary of swelling pressure test results and percent reduction in the swelling pressure of the treated light grey clay soil are given in Table 3.4. The percent reduction of the swelling pressure is made with respect to the swelling pressure of the natural light grey clay soil. The results are plotted as shown in Figure 3.3. The results of swelling pressure tests of the treated light grey clay soil show up to eighty percent reduction from the swelling pressure of the natural light grey clay soil.

Table 3.4 Swelling Pressure Test Result of Light Grey Clay Soil

Mix Code	Mix Type	Swelling Pressure in Kpa	Percent Decrease in Swelling Pressure
1	Natural Soil	299.00	--
2	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 alone	140.00	53.18
3	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 alone	187.50	37.29
4	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 alone	194.50	34.95
5	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 and 2% Lime	64.50	78.43
6	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 and 2% Lime	60.00	79.93
7	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 and 2% Lime	73.80	75.32
8	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 and 4% Lime	175.00	41.47
9	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 and 4% Lime	170.00	43.14
10	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 and 4% Lime	152.50	49.00
11	Natural Soil Plus 4% Lime alone	184.00	38.46
12	Natural Soil Plus 8% Lime alone	85.00	71.57

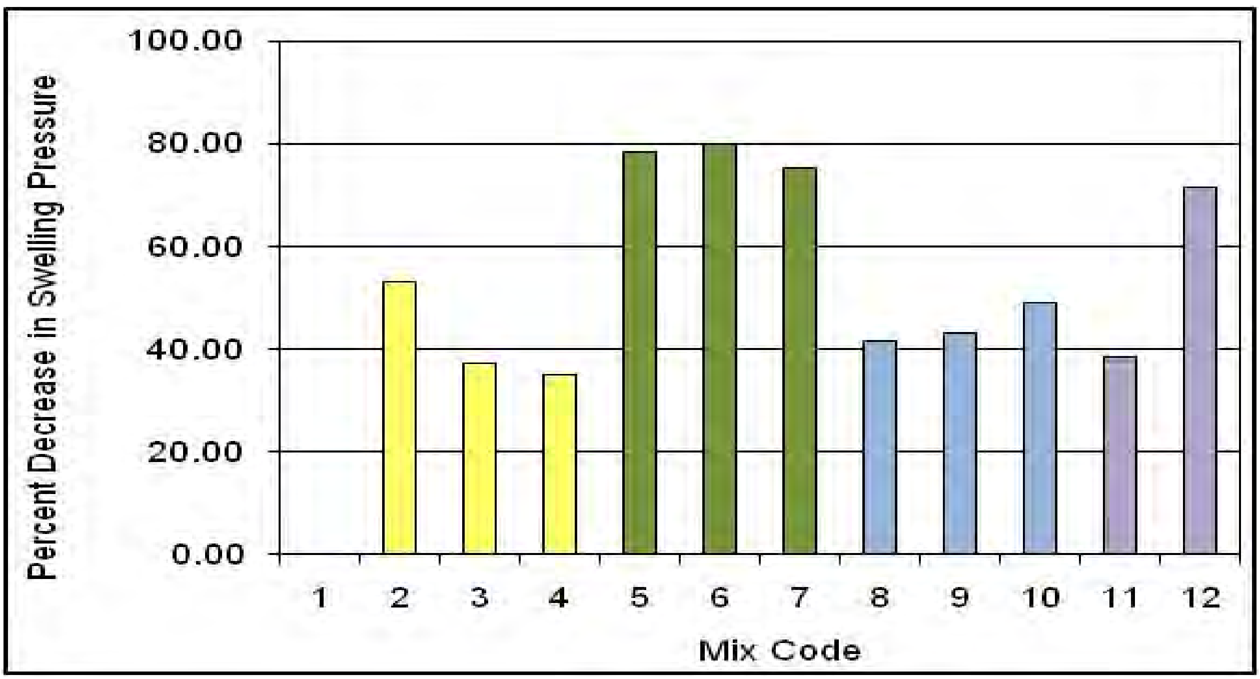


Figure 3.3 Percent Decrease in Swelling Pressure vs. Mix Code of Light Grey Clay Soil

ii) Percent Swell and Free Swell Tests and Results

The parameters determined from expansive soil identification tests have been combined in a number of different classification schemes. Unfortunately, there has not yet evolved a standard classification procedures, and a different scheme is used in practically every different location [2].

The most confusing aspect of classification is the lack of a standard definition of swell potential. Not only do the sample conditions vary in the different swell tests used to evaluate swell potential (i.e. remolded or undisturbed), but surcharge loading and other testing factors vary over a wide range of values. For the purpose of this research, swell potential is defined according to Seed et al. (1962b): swell potential is defined as the volume change of a remolded sample at optimum moisture and maximum density (Standard AASHTO) under 6.9 Kpa load [2].

Percent swell tests were conducted on the light grey clay soil samples only to study the swelling potential of this soil. Summary of the test results and percent reduction in the percent swell of the treated light grey clay soil are given in Table 3.5. The percent reduction of the percent swell is made with respect to the percent swell of the natural light grey clay soil. The plots of the percent reduction in the percent swell of the different mixes are shown in Figure 3.4. The results show up to eighty percent reductions from the percent swell of the natural light grey clay soil.

The free swell test consists of placing a known volume of dry soil passing the No. 40 sieve into a graduated cylinder filled with water and measuring the swelled volume after it has completely settled. The free swell of the soil is determined as the ratio of the change in volume to the initial volume, expressed as a percentage [2].

Free swell tests were conducted on the light grey clay soil samples only. The graphical presentation of the percent reductions in the free swell are shown in Figure 3.5. The results show up to eighty two percent reduction from the free swell of the natural light grey clay soil.

Table 3.5 Percent Swell and Free Swell Test Result of Light Grey Clay Soil

Mix Code	Mix Type	Percent Swell (%)	Percent Decrease in Percent Swell	Free Swell (%)	Percent Decrease in Free Swell
1	Natural Soil	14.20	--	158	--
2	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 alone	4.75	66.55	160	0.00
3	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 alone	6.75	52.46	165	0.00
4	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 alone	3.13	77.96	65	58.86
5	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 and 2% Lime	2.54	82.11	95	39.87
6	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 and 2% Lime	3.45	75.70	115	27.22
7	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 and 2% Lime	3.73	73.73	50	68.35
8	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 and 4% Lime	4.00	71.83	85	46.20
9	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 and 4% Lime	4.20	70.42	100	36.71
10	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 and 4% Lime	3.13	77.96	28	82.28
11	Natural Soil Plus 4% Lime alone	2.98	79.01	50	68.35
12	Natural Soil Plus 8% Lime alone	0.90	93.66	75	52.53

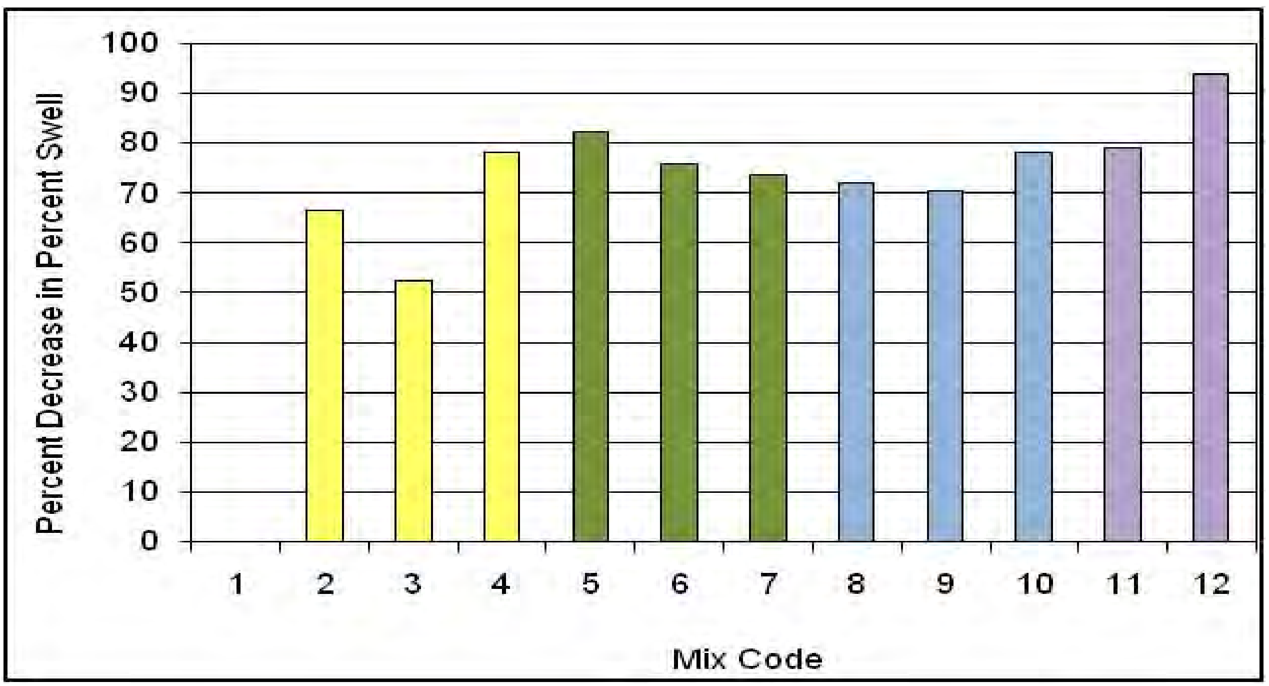


Figure 3.4 Percent Decrease in Percent Swell vs. Mix Code of Light   Grey Clay Soil

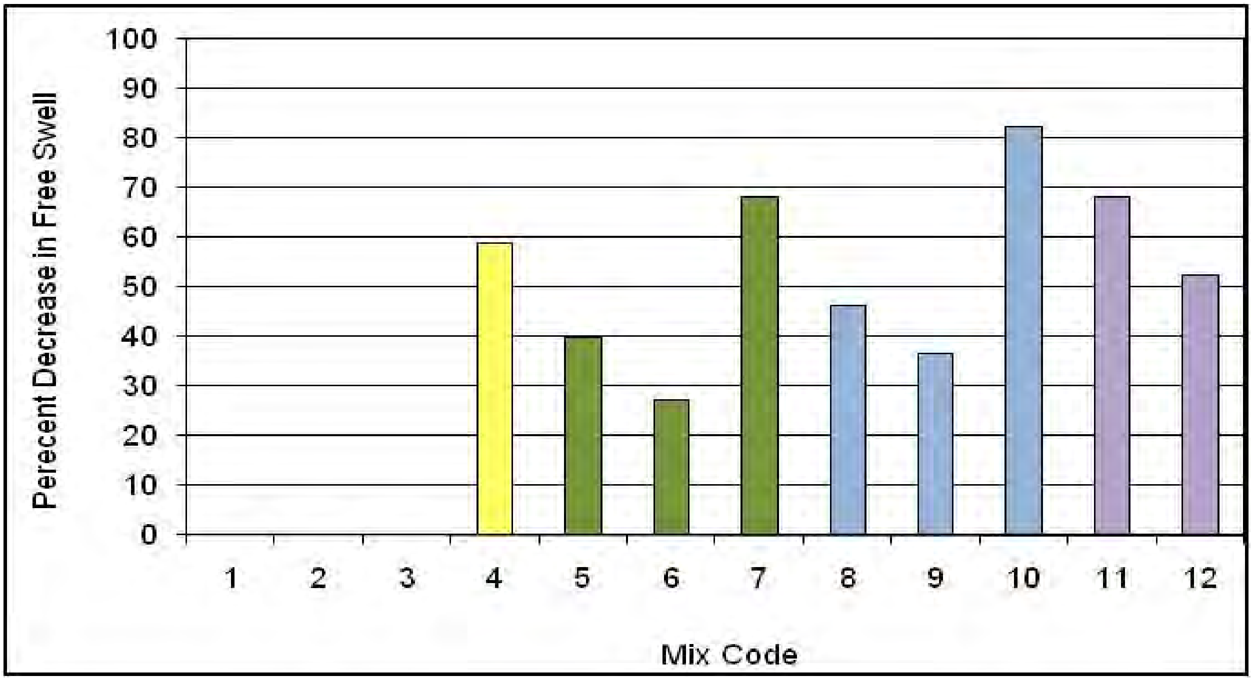


Figure 3.5 Percent Decrease in Free Swell vs. Mix Code of Light Grey Clay Soil

3.4.3 Atterberg Limit Tests and Results

Fine grained soils in particular can exhibit several states depending on the amount of water in the soil. When water is added to dry soil, the individual particles are covered with adsorbed water forming a thin film around each grain. If the addition of water is continued, the thickness of the water film will continue to increase, thereby facilitating the sliding effect between adjoining particles. Thus, it is a fact that the behavior of the soil is related to the amount of water in the system [6].

The liquid limit is the water content at which the soil has negligible shear strength such that it flows to close a groove of standard width when agitated with a standard liquid limit apparatus [6].

The plastic limit is the water content at which the soil begins to break apart when rolled in to a thread of 3 mm in diameter. The plasticity index is the range of water content over which the soil exhibit plasticity that is the difference between the liquid limit and plastic limit [13].

The liquid limit and plastic limit tests were conducted on the natural and treated soil samples of both the light grey and the red clay soils. Testing procedures of AASHTO T89-96 and T90-96 were followed for the evaluation of liquid limit and plastic limit tests, respectively.

The percent reductions in plasticity indexes of the treated light grey and red clay soils with respect to the plasticity index of the natural soils are given in Tables 3.6 and 3.7, respectively. The percent reduction in the plasticity indexes of the various mixes are calculated with respect to the plasticity index of the respective natural soil. The graphical presentation of the percent reductions are shown in Figure 3.6 and 3.7, respectively.

The results for the light grey clay soil show up to sixty four percent reductions in plasticity index, whereas the results for the red clay soil show up to fifty three percent reduction in the plasticity index.

Table 3.6 Liquid Limit and Plastic Limit Test Results of Light Grey Clay Soil

Mix Code	Mix Type	LL	PL	PI	Percent Decrease in PI Value
1	Natural Soil	109	42	67	--
2	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 alone	100	43	57	14.93
3	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 alone	100	37	63	5.97
4	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 alone	90	38	52	22.39
5	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 and 2% Lime	96	49	47	29.85
6	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 and 2% Lime	96	48	48	28.36
7	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 and 2% Lime	96	48	48	28.36
8	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 and 4% Lime	82	51	31	53.73
9	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 and 4% Lime	86	49	37	44.78
10	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 and 4% Lime	74	43	31	53.73
11	Natural Soil Plus 4% Lime alone	85	46	39	41.79
12	Natural Soil Plus 8% Lime alone	72	48	24	64.18

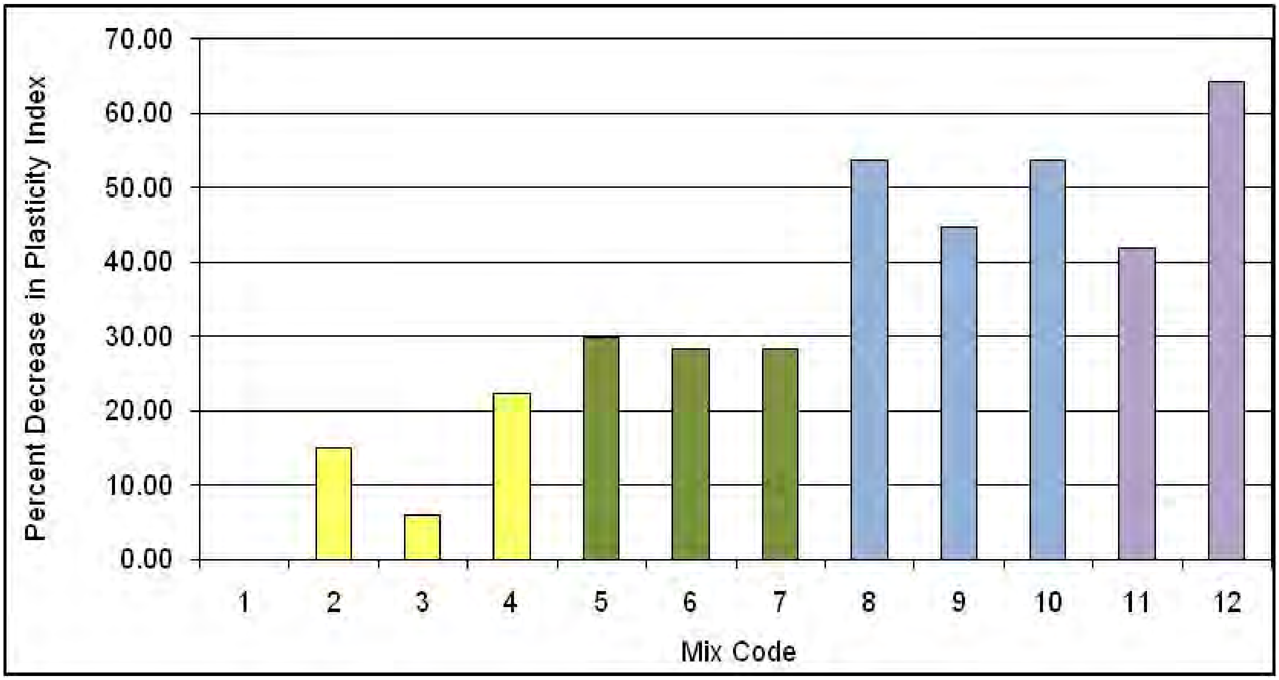


Figure 3.6 Percent Decrease in Plasticity Index vs. Mix Code of Light Grey Clay Soil

Table 3.7 Liquid and Plastic Limit Test Results of Red Clay Soil

Mix Code	Mix Type	LL	PL	PI	Percent Decrease in PI Value
1	Natural Soil	62	30	32	--
2	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 alone	63	32	31	3.13
3	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 alone	60	34	26	18.75
4	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 alone	57	34	23	28.13
5	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 and 2% Lime	59	31	28	12.50
6	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 and 2% Lime	58	33	25	21.88
7	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 and 2% Lime	55	32	23	28.13
8	Natural Soil Plus 0.08lit/m^3 SA-44/LS-40 and 4% Lime	50	35	15	53.13
9	Natural Soil Plus 0.15lit/m^3 SA-44/LS-40 and 4% Lime	49	31	18	43.75
10	Natural Soil Plus 0.30lit/m^3 SA-44/LS-40 and 4% Lime	53	33	20	37.50
11	Natural Soil Plus 4% Lime alone	50	33	17	46.88
12	Natural Soil Plus 8% Lime alone	51	35	16	50.00

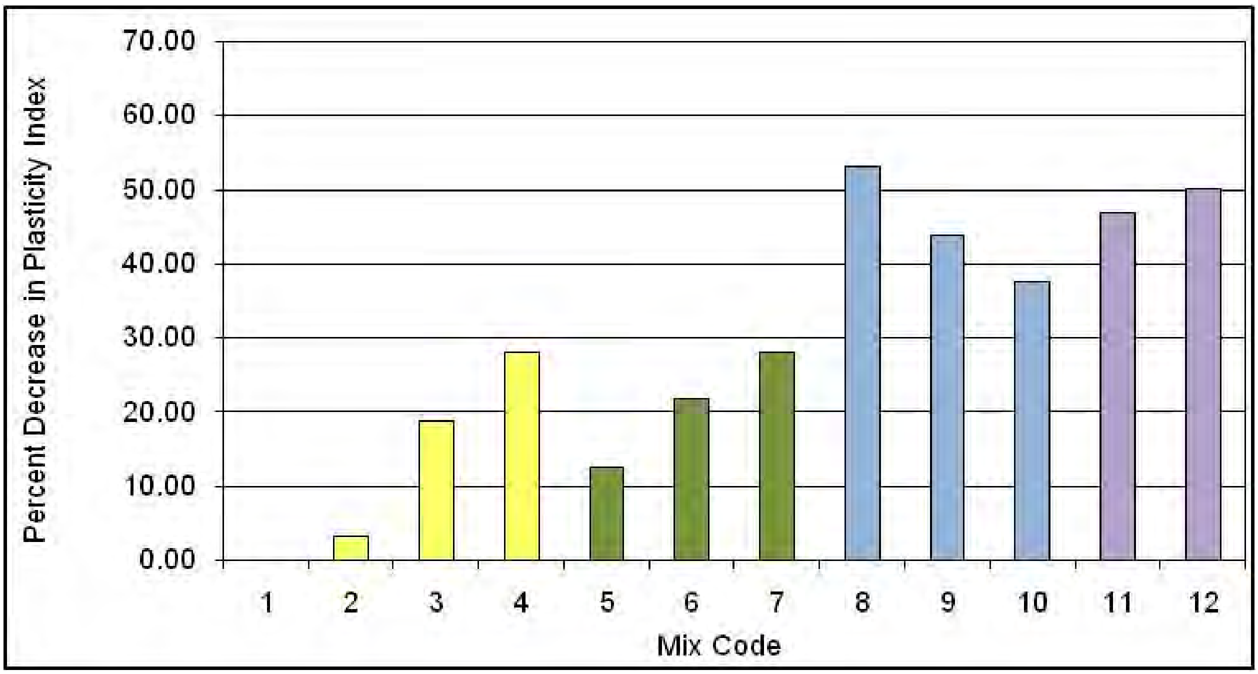


Figure 3.7 Percent Decrease in Plasticity Index vs. Mix Code of Red Clay Soil

4.0 DISCUSSION OF TEST RESULTS

4.1 CBR and CBR Swell Test Results

The CBR test results of the light grey clay soil are given in Table 3.2 and Figure 3.1. Thus, the subsequent discussions are made by scrutinizing these results. The applications of SA-44/LS-40 chemical alone are ineffective in improving the CBR value of this soil. However, the applications of lime alone results in modest improvement in the CBR value of this soil. In addition, the applications of the chemical with lime results in remarkable improvement in the CBR value of this soil.

Furthermore, the applications of the chemical with 4% lime on the light grey clay soil improve the CBR value for all chemical application rates though the improvement reduces with the increase in chemical application rates. However, the applications of the chemical with 2% lime on this soil improve the CBR value only for the highest chemical application rate. Additionally, the applications of lime alone on this soil improve the CBR value more with the increase in lime percentage.

The CBR test results of the red clay soil are given in Table 3.3 and Figure 3.2. Thus, the subsequent discussions are made by scrutinizing these results. The applications of SA-44/LS-40 chemical alone and the chemical with 2% lime are ineffective in improving the CBR value of this soil. However, the applications of lime alone results in modest improvement in the CBR value of this soil. In addition, the applications of the chemical with 4% lime results in remarkable improvement in the CBR value of this soil.

Furthermore, the applications of the chemical with 4% lime on the red clay soil improve the CBR value for all chemical application rates though the improvement reduces with the increase in chemical application rates. However, the applications of lime alone on this soil improve the CBR value only for the smaller application percentage.

4.2 Swelling Properties

The swelling pressure tests were conducted on the light grey clay soil samples only since the volume change of the red clay soil is much lower.

4.2.1 Swelling Pressure Test Results

The swelling pressure test results of the light grey clay soil are given in Table 3.4 and Figure 3.3. Thus, the subsequent discussions are made by scrutinizing these results. The applications of SA-44/LS-40 chemical alone, the chemical with 4% lime and lime alone results in modest reduction in the swelling pressure of this soil. However, the applications of the chemical with 2% lime results in remarkable reduction in the swelling pressure of this soil

In addition, the applications of the chemical with lime have less influence on the swelling pressure of the light grey clay soil with the change in the chemical application rate. However, the applications of the chemical alone results in more reduction in the swelling pressure with the decrease in chemical application rates. In contrast the application of lime alone reduces the swelling pressure more with the increase in lime percentage.

4.2.2 Percent Swell Test Results

The percent swell test results of the light grey clay soil are given in Table 3.5 and Figure 3.4. Thus, the subsequent discussions are made by scrutinizing these results. The applications of SA-44/LS-40 chemical alone, the chemical with lime and lime alone results in remarkable reduction in the percent swell of this soil.

In addition the applications of the chemical alone results the highest reduction in the percent swell of the light grey clay soil for the highest chemical application rate. Furthermore, the applications of the chemical with 2% lime results in reduction the percent swell of this soil more with the decrease in the chemical application rates. However, the applications of the chemical with 4% lime results in reduction the percent swell of this soil more with the increase in the chemical application rates. Additionally the application of lime alone reduces the percent swell of this soil more with the increase in lime percentage.

4.2.3 Free Swell Test Results

The free swell test results of the light grey clay soil are given in Table 3.5 and Figure 3.5. Thus, the subsequent discussions are made by scrutinizing these results. The applications of SA-44/LS-40 chemical alone, the chemical with lime and lime alone results in modest reduction in the free swell of this soil.

In addition, the applications of the chemical alone on the light grey clay soil reduce the free swell only at the highest chemical application rate. Furthermore, the applications of the chemical with lime results in the highest reduction in the free swell of this soil at the highest chemical application rates. However, for lime alone application the reductions in the free swell of this soil decreases with the increase in lime percentage.

4.3 Atterberg Limit Test Results

The Atterberg limit test results of the light grey clay soil are given in Table 3.6 and Figure 3.6. Thus, the subsequent discussions are made by scrutinizing these results. The applications of SA-44/LS-40 chemical alone and the chemical with 2% lime results in modest reduction in the plasticity index of this soil. In addition, the applications of the chemical with 4% lime and lime alone results in remarkable reduction in the plasticity index of this soil.

Furthermore, the applications of the chemical alone on the light grey clay soil results in the highest reduction in the plasticity index of this soil only at the highest chemical application rate. However, for chemical with lime applications, the change in chemical applications rate have less influence on the plasticity index of this soil.

The Atterberg limit test results of the red clay soil are given in Table 3.7 and Figure 3.7. Thus, the subsequent discussions are made by scrutinizing these results. The applications of SA-44/LS-40 chemical alone and the chemical with 2% lime results in modest reduction in the plasticity index of this soil. However, the applications of the chemical with 4% lime and lime alone results in remarkable reduction in the plasticity index of this soil.

In addition, the applications of the chemical alone and the chemical with 2% lime results more reduction in the plasticity index of the red clay soil with increasing the chemical application rate. However, the applications of the chemical with 4% lime results more reduction in the plasticity index of this soil with decreasing the chemical application rate. However, for lime alone applications, the change in lime percentage has less influence on the plasticity index of this soil.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The subsequent conclusions and recommendations are made on the stabilization of light grey and red clay sub grade soils using SA-44/LS-40 chemical and lime. The chemical and lime utilized during this study are produced by Dallas Road Way Product Inc. and Senkele Lime Factory, respectively.

5.1 Conclusions

The applications of SA-44/LS-40 chemical alone results in promising reduction in the swelling pressure of the light grey clay soil however, it is ineffective in improving the soaked CBR value of this soil. In addition the applications of lime alone results in an increase in the soaked CBR value and reduction in the swelling pressure of the light grey clay soil. Furthermore the applications of the chemical with lime results in even more increase in the soaked CBR value and reduction in the swelling pressure of the light grey clay soil.

Thus, the application of the chemical with lime is more suitable for the light grey clay soil. In particular the application of 0.30lit/m^3 of SA-44/LS-40 chemical and 2% lime is an optimum proportion in increasing the soaked CBR value and reducing the swelling pressure of the light grey clay soil.

The applications of SA-44/LS-40 chemical alone are ineffective in improving the soaked CBR value of the red clay soil. In addition the applications of lime alone results in an increase in the soaked CBR value of the red clay soil. Furthermore similar to the case of the light grey clay soil, the applications of the chemical with lime results in further increase in the soaked CBR value of the red clay soil.

Thus, the application of the chemical with lime is more suitable for the red clay soil. In particular the application of 0.08lit/m^3 of SA-44/LS-40 chemical and 4% lime is an optimum proportion in increasing the soaked CBR value of the red clay soil. This application changes the sub grade strength class of the red clay soil from S_3 to S_5 as per the pavement design manual of Ethiopian Roads Authority. This change in subgrade strength class causes reduction in pavement thickness on the red clay subgrade soil.

5.2 Recommendations

From the results of the study, it is observed that the application of lime alone and SA-44/LS-40 chemical in combination with lime shows improvements in the engineering properties of both the light grey and the red clay soils. Even though, no single mix type results in the maximum improvement in all the required properties of the soils. In addition, the following recommendations are made;

1. From the results of the study:
   - The application of 0.30lit/m^3 of SA-44/LS-40 chemical and 2% lime is an optimum mix for the light grey clay soil.
   - The application of 0.08lit/m3 of SA-44/LS-40 chemical and 4% lime is an optimum mix for the red clay soil.
2. The study results of both soils show that the improvements in engineering properties of the soils after applying the stabilizers are erratic. Thus, before large-scale applications, the optimum mix of a given soil should be established from test results of various trials.
3. The effect of weather condition on the stabilized soils is not taken in to account while undertaking the investigation. Hence before large-scale application of the chemical, it may be helpful to study the effect of different weather conditions on the response of the soils.
4. The environmental impacts of the chemical application are not taken into account in the study. Further studies of this aspect may be important.
5. The results of the investigations were obtained based on immediate compaction after treating the soil. The effect of delay between mixing and compaction on strength and swelling properties of the soils should be investigated, since immediate compaction after mixing is not practical for large-scale applications.
6. The effectiveness of the stabilized soils is encouraging although the depth of treatment and result of treatment on long term basis needs further evaluation. Hence performance–based testing might be necessary using large-scale field trial.
7. The conclusions drawn are based on technical feasibility alone. Hence, before large-scale application, financial viability of the applications should be thoroughly checked.

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