EFFECT OF WET AND DRY CYCLES ON SHEAR STRENGTH OF CLAYS UNDER LOW OVERBURDEN STRESS

By

Muhammad Hussain Ahsan

(NUST-2016-MS GEOTECH-00000172031)

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

Master of Science

In

Geotechnical Engineering

NUST Institute of Civil Engineering (NICE)

School of Civil and Environmental Engineering (SCEE)

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<u>THESIS ACCEPTANCE CERTIFICATE</u>

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EFFECT OF WET AND DRY CYCLES ON SHEAR STRENGTH OF CLAYS UNDER LOW OVERBURDEN STRESS

Submitted by

Muhammad Hussain Ahsan

has been accepted towards the partial fulfillment

of

the requirements

for

Master of Science in Geotechnical Engineering

____

Dr. Liaqat Ali

Associate Dean

NUST Institute of Civil Engineering (NICE)

DEDICATED

TO

MY BELOVED MOTHER AND MY DEAREST FATHER (Late)

Who were always been supportive and encourging

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ABSTRACT

The seasonal variations in soil water content are one of the major important environmental factors that causes the reduction of shear strength of expansive clays. Highway embankments which are constructed on expansive soils all over the world pose a severe maintenance problem due to shallow slope failures. Proper understanding of strength loss mechanism of the expansive clays will give some useful guidelines to design embankment with adequate factor of safety for long term drained conditions. Effect of wet and dry cycles on the high plastic clay strength has been studied by several researchers but the effect of wet dry cycle at low overburden stress and self-healing phenomenon was not clear, which might be very important consideration in the geotechnical design.

This study focuses on the shear strength parameters of the soil samples collected from ten different locations in the vicinity of Islamabad region. Shear strength parameters were determined after 2^{nd} ,4^{th}, 6^{th} wet and dry cycles and 7 days strength recovery period after 6^{th} wet and dry cycle by Direct Shear Test (DST). X-Ray Diffraction (XRD) analysis of the soil samples was conducted. Other parameters like void ratio, density and moisture contents were also determined for the same number of cycles. Several anomalies in the trends of shear strength parameters were observed. It is concluded that cohesion is a non-reliable parameter and it is safe to use zero cohesion for long term design of slopes. Angle of internal friction is reduced both for low plastic and high plastic. This reduction is less in low plastic soils. The self-healing in low plastic clays is much more than in high plastic clays.

Chapter 1

INTRODUCTION

1.1 GENERAL

Wet and dry cycles are considered as the most destructive environmental factor that induce damage to the civil engineering infrastructures like highways and pavements (Allam and Sridharan, 1981). Several Investigations have been done to understand the catastrophic effect of cyclic wetting and drying on soil physical as well as mechanical properties. It directly effects the particle cementation, void ratio and moisture content of soil which not only leads to the formation of cracks fissures in expansive soils but also increase the compressibility of soils. Therefore, the variation in shear strength of expansive soils subjected to wet and dry cycles controls the slope stability analysis (Md *et al.*, 2016).

Once a soil reaches its peak strength, the resistance fell to a lower value when subjected to further shear deformation and this reduction in the shear strength becomes zero after some time. This lower value of strength is defined as residual shear strength and it is applicable to slope stability analysis of natural slopes, excavations and previously failed soil slopes (Skempton, 1964). With more advancement in shear strength, it is realized that there is a fully softened shear strength which lies in between the peak shear strength and residual shear strength and is numerically equal to the peak shear strength of clays in normally consolidated state (Skempton, 1964).

One of the most common and problematic failure of the soils is shallow slope failures. It is a term which is used to explain the surficial slope instabilities especially in the embankments of pavements. These instabilities in shallow slopes mostly occurs in fine grained soils especially after long rainfall events. A detailed study was made on the repair cost of almost 20 percent of all U.S. highways and roads and it was concluded that the total cost for maintenance and repair of landslides exceeded 100 million U.S. dollars annually. Therefore, many private and government agencies including the U.S. Army Corps of Engineers (USACE), U.S. Forest services and railroad industries significantly increased the total cost for landslides, cut slopes and embankment repairs (Loehr and Bowders, 2007).

Previous researchers performed several direct shear testing on high plastic clays and they concluded that the cohesion of the soil reduces to zero upon wetting and drying cycles with a minor change in angle of internal friction (Stark and Hussain, 2012). But when the back analysis of several failed slopes was done, the angle of internal friction in the first time shear strength failure was very less than the peak shear strength when subjected to cyclic wetting and drying (Wright *et al.*, 2007). Therefore, it was recommended to made a thorough study on the reduction of shear strength parameters when subjected to wetting and drying cycles.

1.2 NEED FOR RESEARCH

It is reported by several researchers that the shear strength parameters reduces when subjected to several wet and dry cycles (Lade, 2010; Stark and Hussain, 2012; Wright *et al.*, 2007). But no clear idea was given to estimate this reduction from soil index properties. Several ring shear and triaxial tests were performed on expansive clays after being subjected to wet and dry cycles. Major loss in cohesion of the clays along with a minor change in angle of internal friction were observed (Wright *et al.*, 2007). It was recommended for further laboratory testing to observe this strength loss behavior of clays upon cyclic wetting and drying. Correlations for estimating the fully softened shear strength are available in the literature under 50, 100 and 400 kPa normal stresses (Stark and Hussain, 2012). But these correlations are not suitable for determination of fully softened shear strength for shallow slopes where the overburden stresses are expected to be smaller than 50 kPa (Lade, 2010).

A self-healing in the expansive soils was observed in the pre-existing shear surfaces, cracks and fissures that resulted in increased shear resistance. The magnitude of this self-healing was appeared to increase with increasing the plasticity of the soil. This increase could have implications for the size, cost of landslide remediation (Stark *et al.*, 2005). This estimation of the self-healing phenomenon in the soils after wet and dry cycles had never not been clarified previously.

1.3 RESEARCH OBJECTIVES

Main objective of this research is the characterization of ten different soil samples and determination of variation in shear strength of these samples in shallow slope stability analysis. This research will be focusing specifically on:

- Characterization of existing soil properties of different clay samples taken at shallow depths from natural slopes, excavations and road embankments
- Effect of wet and dry cycles on shear strength of clays under low overburden stress (< 50 kPa)
- Correlation between reduced angle of internal friction and liquid limit
- Estimation of gain in strength after cyclic wetting and drying

1.4 SCOPE AND METHODOLOGY

This research includes the effect of wet and dry cycles on shear strength of ten different soil samples. The soil samples are characterized on the bases of their index properties. DST are performed on all the soils at Maximum dry density, 2nd,

4^{th} and 6^{th} wet and dry cycles. Also, the gain in strength 7 days after 6^{th} wet and dry cycles is also determined to understand the self-healing phenomenon in the soils. XRD analysis of the soils is done to determine the mineralogical configuration of the samples and presents of cementitious chemicals and salts. This research is divided in three phases. The detailed methodology is discussed in chapter 3 of this thesis but the general overview of the methodology is shown as under.

1.4.1 Phase I: Characteristics of Clay Samples

- Grain size distribution
    - Sieve Analysis
    - Hydrometer Analysis
- Atterberg limits
    - Liquid Limit
    - Plastic Limit
- Maximum dry density and optimum moisture content
- Swell index
- XRD Analysis

1.4.2 Phase II: Effect of Wet and Dry Cycles

- Low plastic clays
    - 0^{th} wet and dry cycles
    - 2^{nd} wet and dry cycle
    - 4^{th} wet and dry cycle
    - 6^{th} wet and dry cycle
- High plastic clays
    - 0^{th} wet and dry cycles

    - 2nd wet and dry cycle
    - 4th wet and dry cycle
    - 6th wet and dry cycle

1.4.3 Phase III: Self Healing in Clays

- Group-I Clays
    - Gain in strength 7 days after 6th wet and dry cycle
- Group-II Clays
    - Gain in strength 7 days after 6th wet and dry cycle
- Group-III Clays
    - Gain in strength 7 days after 6th wet and dry cycle

1.5 THESIS CONTENTS

This thesis consists of five chapters. Chapter 1 is limited to general introduction of the thesis proving the cyclic wetting and drying as catastophic environmental factor and need of present research. It also includes the main objectives of the research, its scope and brief methodology. methodology.

In Chapter 2, light is thrown upon the background of shallow slope stability analysis. The order is started with introduction to the shear strength of clays. Description on different environmental factors causing the reduction in the shear strength is done. Also, the evolution of fully soften shear strength is highlighted. Finally, the correlations for fully soften shear strength and reasons for self-healing in soils are presented.

Chapter 3 is consisted of detailed methodology, which involves the discussion on the measurement of soil properties from laboratory testing. The detailed procedures of the laboratory tests are presented. Sample preparation techniques and methodology of cyclic wetting and drying is also discussed in detail.

Chapter 4 contains all the data obtained from testing. Graphs, trends and numerical data is presented in this section. In discussion, the interpretation of results is done, and critical reasoning is presented against changes and trends observed in results, i.e., Index properties of soil, Modified proctor test, Direct shear test, and Swell potential, etc.

Chapter 5 circumscribes the conclusions made by this research and general recommendations for further research in this area.

Chapter 2

REVIEW OF LITERATURE

2.1 GENERAL

Shallow slope failure is always been a major issue with the highway embankments constructed on expansive clays. This failure is due to the loss of shear strength with time due to weathering processes either human induced or naturally occurring events. Usually, shallow slope failures occur after prolonged rainfall events which results in the reduction of shear strength (Titi and Helwany, 2007). In most of the cases the depth of the failure varied from 3 ft to 6ft in shallow slope failure (Loehr and Bowders, 2007). Figure 2.1 shows some shallow slope failures due to excessive rainfall (Gamez and Stark, 2014).

Figure 2.1: Shallow slope failures

During heavy rains, water seeps into the ground, saturating the upper layers of soil. Porewater pressures develop and reduce the shearing resistance of the soil. When the surficial soils are underlain by a more impermeable material, seepage flow parallel to the slope begins to take place (Skempton, 1970). This condition reduces the factor of safety of the slope and may cause surficial failures.

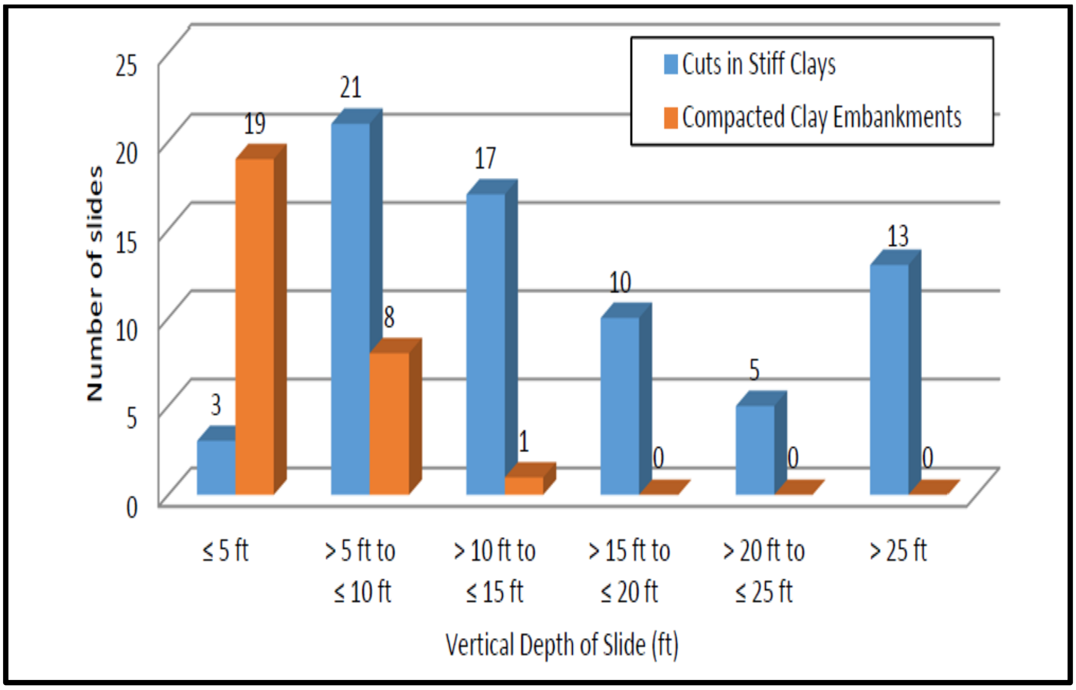

Figure 2.2: Number of slope failures with depth

Figure 2.13 is showing that most probable depth of sliding for the slope constructed with clay was less than or equal to 5 ft and none of them was recorded beyond 15 ft depth (Castellanos, 2014).

Shallow slope failure may happen anywhere, they tend to attract more attention in semi-arid areas of the world in which the upper layer of the soil dries out for some years followed by a year with heavy rainfalls which saturate the upper layers and cause a large number of surficial failures (Lade, 2010). Change in climate might have an adverse effect on the slope. During the summer, top layer of soil dries out and longer periods of several years with little rainfall the depth of the dry soil zone increases slowly. Large surficial cracks might occur during this time which will provide a flow path for the rain water. Water content varies only in the soil close to the surface and it remains relatively constant below the zone of annual fluctuation.

An active zone is shown in Figure 2.14 which is the evidence of a zone of relatively constant water content. The zone followed by the active zone may be considered as an impermeable layer and soil is expected to lose its cohesion within the zone of moisture variation. Just before the first time of sliding, wetting front reaches up to the maximum depth of active zone after a period of heavy rainfall and reduces the soil internal shear strength to mobilized shear. It was concluded that, the water in the partly saturated soil below the dry soil is under tension and this provides an effective confining pressure in the partly saturated soil (Lade, 2010). Sliding failure will occur at the level of the lowest factor of safety and this is just above the depth to which the upper layer has previously dried out. There are several iterative models developed for calculation of this depth of active zone, but these are not in the scope of this research. From the depth of active zone safe factor of safety for shallow slopes may be determined (Khan, 2016a).

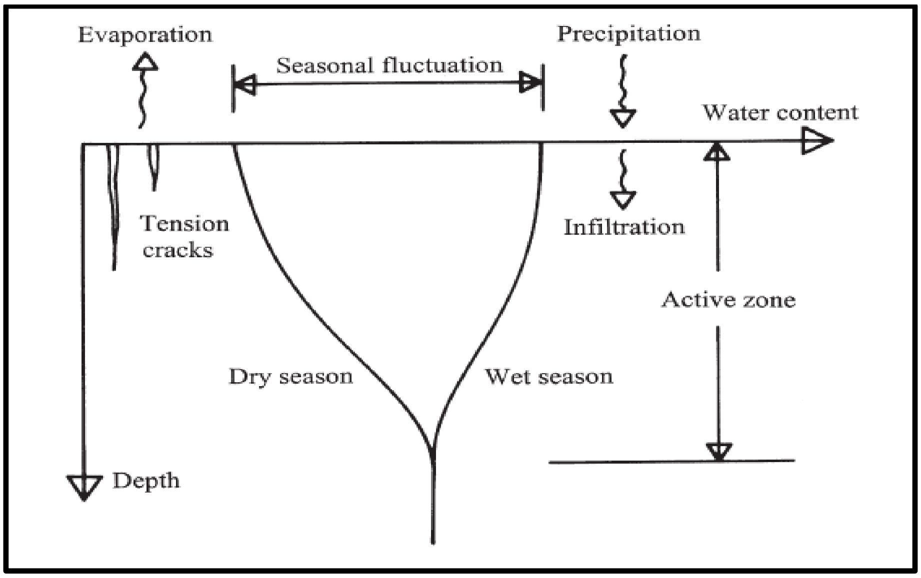

Figure 2.3: Depth of active zone

Although slope stability analysis is very important for counter measure planning and evaluation of landslide mitigation works, no single method is globally accepted in practice. Many countries still ignore the need of soil testing for determination of shear strength parameters of soil during stability analysis of landslides and assume it by empirical formulas. Practitioners in some countries assume mobilized cohesion “c” according to the average depth of sliding surface whereas some others assume zero (for highly plastic clays). Internal angle of internal friction “φ” is calculated by back analysis, using those values of cohesion for limiting equilibrium condition. As the difficulty to locate exact sliding surface, ground water table and position of different formation layers in preexisting landslides bring serious errors in stability analysis. Therefore, it is required to study the appropriate shear strength that must be assigned for the analysis of preexisting landslides (Tiwari and Marui, 2002).

2.2 CLAY MINEROLOGY

The understanding of the origin of clay minerals is a very interesting aspect of clay mineralogy. Clay minerals occurs under very specific geologic environments. The favorable environment of clay formation includes marine and continental sediments, soil horizons, weathering rock formations and volcanic deposits. Clays minerals are generally formed from the weathering of the rocks by air, water or steam (Al-Ani and Sarapaa, 2008). The nature of the clay mineral formation depends mainly on three factors. Firstly, the mineralogical configuration and textural composition of the parent rock. Secondly, the composition of weathering agents (water air or steam) and thirdly the nature of water flow. Therefore, the interaction of rocks with water produce clay minerals either at or near the surface of the earth. Introduction of water to a specific clay may also change its mineral type under certain conditions (Righi and Meunier, 1995).

$$Rock + Water \rightarrow Clay$$

This may be clear by a simplest example. Carbon dioxide gas (CO_2) may dissolve in water producing carbonic acid. Water will be converted into H^+ ion and HCO_3^- ions are produced making water slightly acidic. This mechanism can be shown with the help of equation below.

$$CO_2 + H_2O \rightarrow H_2CO_3 \rightarrow H^+ + HCO_3^-$$

This acidic water will react with the rock surfaces. This reaction will dissolve the silica and K ions from feldspar which is very commonly occurring rock. In this way, the feldspar is transformed into a clay mineral kaolinite. The weathering effect of rock is not only a major natural source of clay minerals but also main reason of presence of metal concentrations in clays. Accordingly, the flow rate of the water also effects this process. If the rate of flow is faster, the contact time of solution will be lesser, and reaction will be very slow. Figure 2.4 shows the change of clay mineral from parent rock basalt. Smectites are formed at low rainfall, whereas kaolinite and illite are formed at moderate to high rainfall.

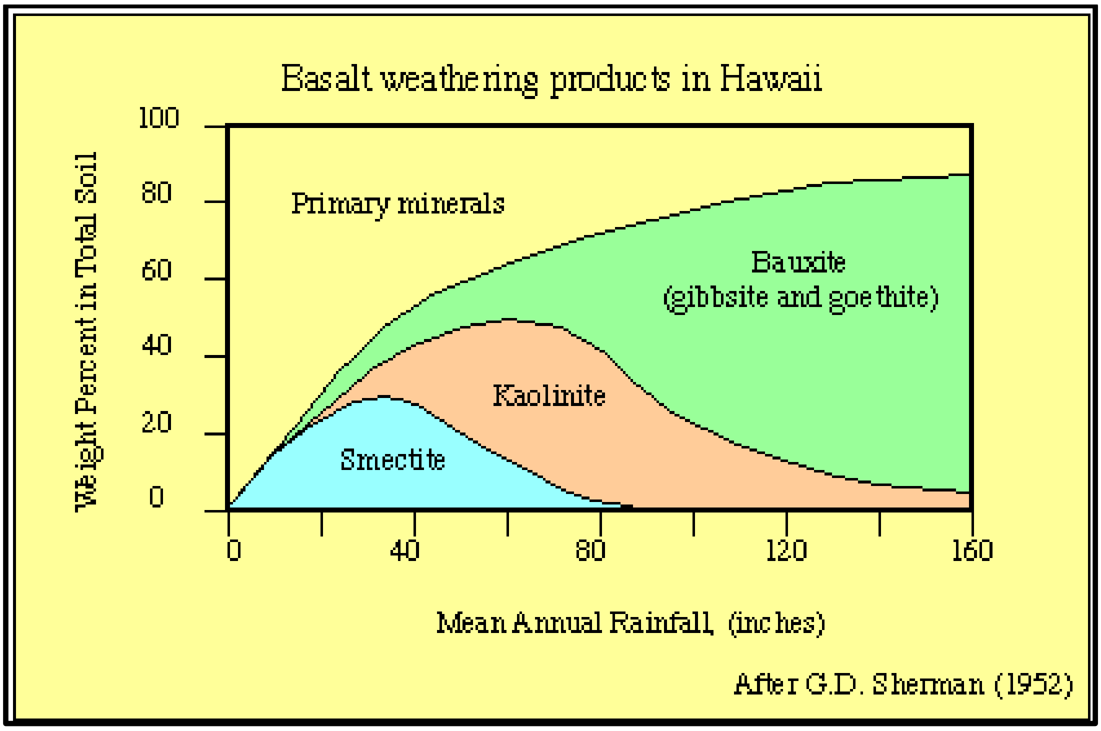

Figure 2.4: Stability diagram for weathering of rock in Hwaii

2.2.1 Structure

There are mainly three most common clay minerals i.e. kaolinite, smectite and illite. The structure of these three major clay minerals comprised of silicon tetrahedral and aluminium octahedral sheets as their basic building blocks. The composition as well as the arrangement of the octahedral and tetrahedral sheets are main reason for most of the differences in their chemical properties.

The important structural and chemical differences between the clay minerals are the basis for the individual mineral species names and the arrangement of the species in groups. The details of tetrahedral and octahedral sheets of clay minerals are discussed under the coming headings.

2.2.1.1 Tetrahedral structure

The tetrahedral structure is formed when four O^{-2} undergo ionic bonding with a central cation such as Si^{+4}. The size of the silicon is very bigger than that of oxygen.

Therefore, its co-ordination number allows bond with four O^{-2} ions instead of making a stable bond with two O^{-2}. This formation causes the overall structure to form a tetrahedral structure. A representative structure of silicon tetrahedral is shown in Figure 2.5. In a tetrahedral sheet, one single epical oxygen is carrying negative charge and all three basal oxygen atoms are shared to form a tetrahedral sheet (Holtz and Kovacs, 1981).

One tetrahedron of silicon has a net charge of -4. Therefore, it is in unstable form and it always form a layer of tetrahedra. As valency, if silicon is +4 and oxygen is -2. After one of two available electrons has been shared by O-2, the valency of Si +4 becomes zero, therefore the net charge over the structure becomes -4. This indicates that the formation tends to form further bonds Build up clay grid. The oxygen over the silicon cation is known as apical, while three under the silicon cation are basal oxygen (Al-Ani and Sarapaa, 2008).

2.2.1.2 Octahedral structure

When Al cations bonds with six hydroxyl anions. It forms an octahedron arrangement. This octahedron shares its corners with adjacent octahedron structures to form a sheet-like structure called as an octahedral sheet (Nelson, 2006). In this structure, O^{-2} or OH^{-} (Hydroxyl) cations surrounds a central anion. The anions could be divalent or trivalent e.g. Mg^{+2}, Al^{+3}.

In Octahedral configuration OH^{-} anions are utilized in the formation of the Al-O-Al chain while remaining OH^{-} anions are arranged in octahedral. Therefore, the net charge over isolated Al-OH octahedron is -3. This octahedral structure is shown in Figure 2.6.

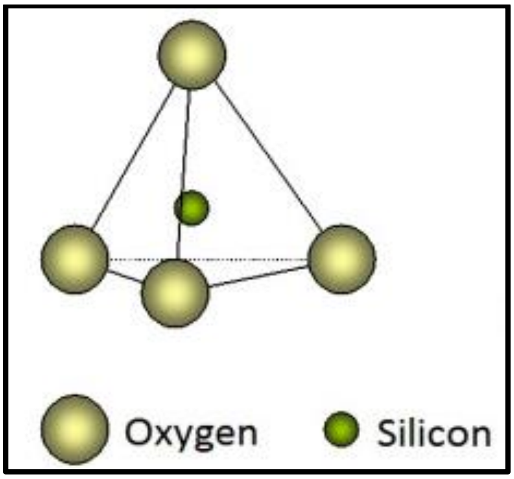

Figure 2.5: Structure of silicon tetrahedral

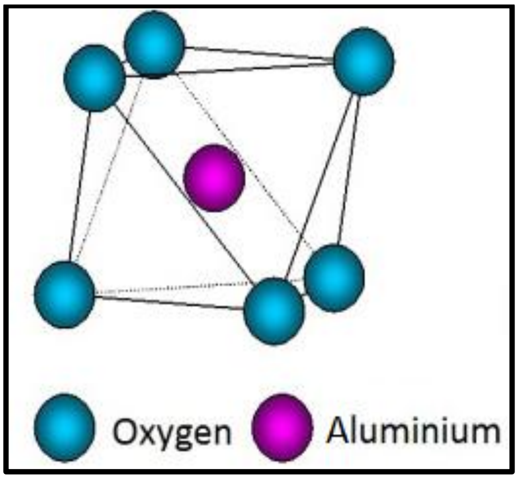

Figure 2.6: Structure of aluminium octahedral

The octahedral sheet is formed when all OH^- are shared with adjacent octahedrons. In an aluminium octahedral sheet, octahedrons are shared with six hydroxyls in upper and lower plane respectively.

2.2.2 Common Clay Minerals

2.2.2.1 Kaolinite

Kaolinite formed when 1:1 layer stacked in such an orientation the oxygen in tetrahedron face hydroxyl group in the octahedron. Each layer is about 7.2 Å thick, the interlayer cleavage is held together via hydrogen bonding between O in a tetrahedron and OH in octahedron group (Holtz and Kovacs, 1981). Due to strong interlayer hydrogen bond, this mineral doesn’t go hydration reaction and makes up large piles of the layer stack. Usually, each crystal of Kaolinite is made up of 70-100 layers thick.

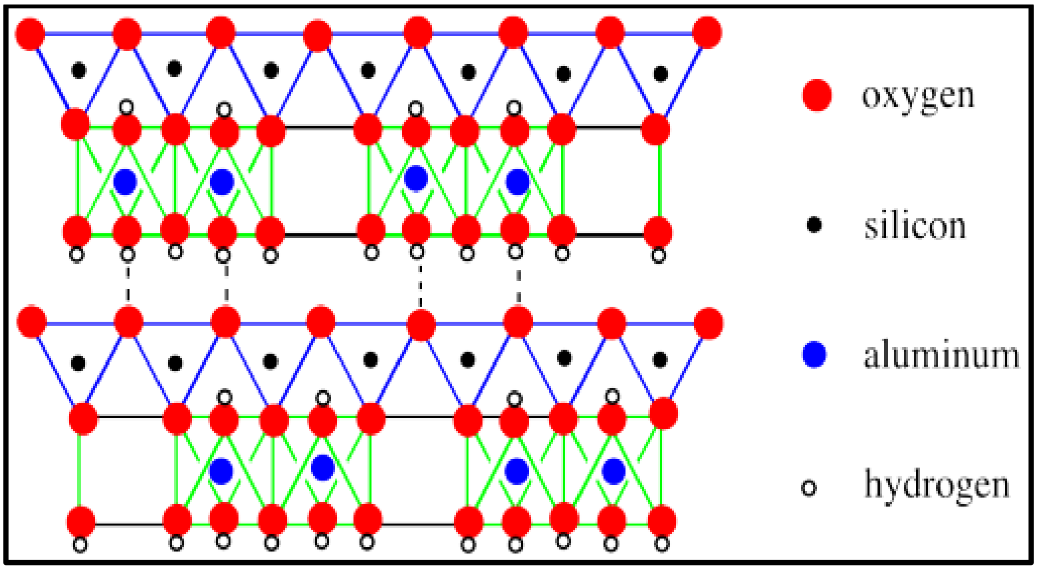

Figure 2.7: Structure of kaolinite

2.2.2.2 Montmorillonite

Montmorillonite belongs to a group called smectite. This mineral is the primary constituent of volcanic ash (Grim, 1953). It is 2:1 mineral and resembles micas, in this mineral the sheets are stacked over each other. All the tips of tetrahedral sheet face the OH of octahedral sheet, at this point the atoms common to both tetrahedral and octahedral layer become oxygen instead of OH (Grim, 1953). The stacking of layer over one and other brings O of tetrahedral face to face making excellent cleavage and allowing water or other cations to adsorb in between. The thickness of each layer is 9.6 Å.

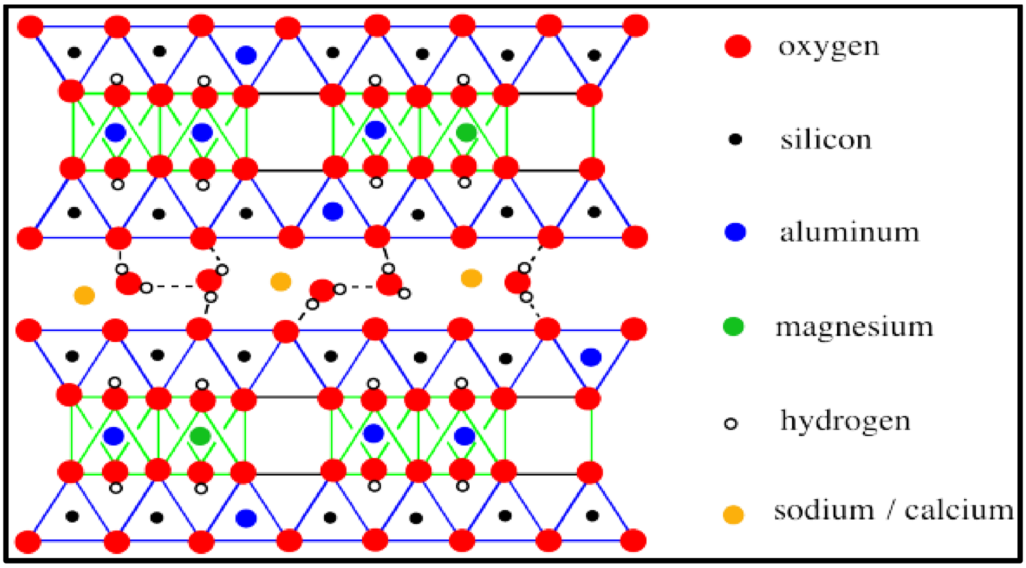

Figure 2.8: Structure of montmorillonite

2.2.2.3 Illite

Illite mineral was first discovered by Prof. Grim in Illinois, hence named Illite after Illinois. The general lattice structure of illite is like montmorillonite but main difference comes when Si in tetrahedral is partially replaced with Al creating charge imbalance (Grim, 1953). The overall lattice becomes negatively charged and this charge is balanced by K^+ cations via cation exchange in between layer cleavage.

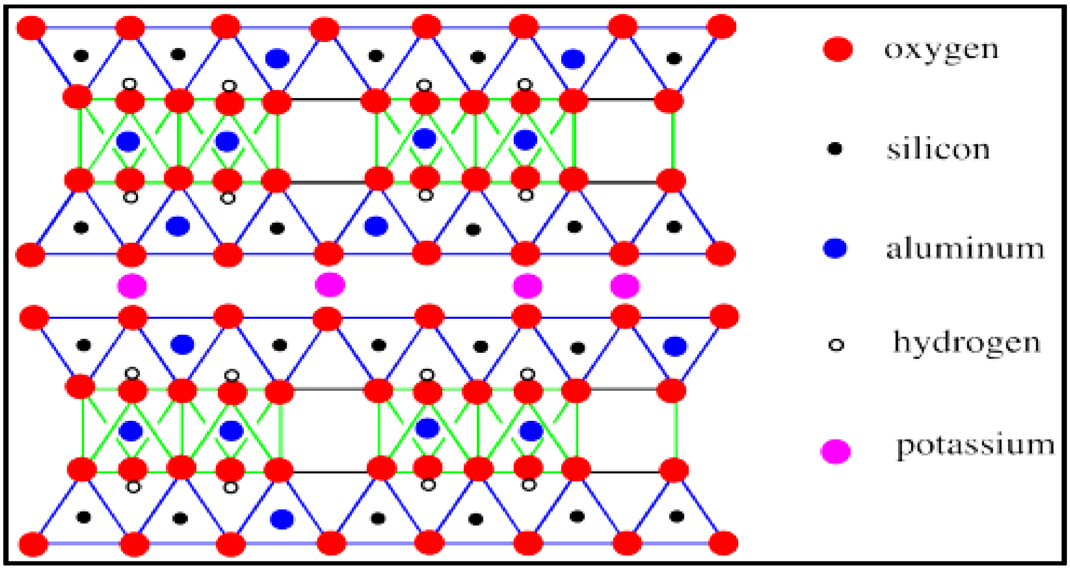

Figure 2.9: Structure of illite

2.2.3 Other Minerals Present in Soils

There are many other groups of minerals present in the soil mass like orthorhombic, zeolite, chlorite, calcite etc. These minerals influence the properties of parent clay minerals in different perspectives. Zeolite group of minerals consists of 1:1 structure (silicon tetrahedral and 1 aluminium octahedral sheet). The layers are arranged in such a way that they have a bigger size cation in between the layers. This group of minerals have very high values of cation exchange capacity than kaolinite and illte minerals.

Chlorite group of minerals have 2:1:1 structure (1 aluminium octahedral sheet surrounded by two silicon tetrahedral sheets like smectite or illite). There structures are further joined by octahedral sheets of Mg+ ions or similar cations. This group of minerals have cation exchange values in the range of illite. Orthorhombic and Quartz group of minerals are considered to be more stable minerals.

2.3 ACTIVITY AND THIXOTROPY

2.3.1 Activity

The volume change during shrinkage and swelling is considered to be the function of plasticity index and most importantly the colloidal clay present in the soil mass (Skempton, 1953). ‘Activity (A)’ is defined as the ratio of plasticity index to the percentage of clay-size fraction.

$$A = \frac{PI}{CF}$$

Where ‘CF’ is the percentage of clay-size particles which are less than 0.002mm (2μm).

Activity is derived conveniently from slope of straight line. The

Figure 2.10 gives plot for activity of clayey soils containing different clay minerals (kaolinite, illite, montmorillonite). A steeper slope represents greater activity. Sodium montmorillonite have much greater activity than that of kaolinite and illite.

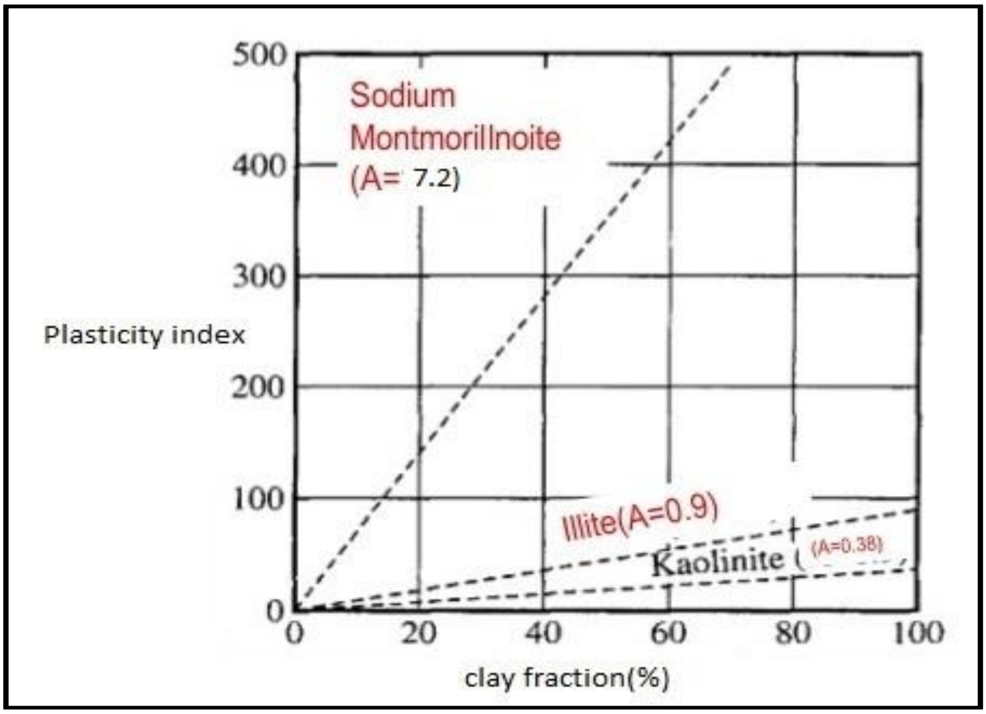

Figure 2.10: Activity of different clay minerals

Activity of a soil gives an idea about how much a soil may swell or shrink. Table 2.1 may be used to classify soils based on their activity.

Table 2.1: Soil classification based on activity

Activity	Classification
< 0.75	Inactive
0.75 – 1.25	Normal
>1.25	Active

The soils containing kaolinite will have lower activity as kaolinite is a stable clay mineral. Whereas, the soils containing montmorillonite, are a major problem as they undergo large volume changes depending on available water and have a very high activity value. Activity may be used as an index property to determine the swelling potential of expansive clays (Skempton, 1953).

2.3.2 Thixotropy

When clays with flocculent structure lose strength due to disturbance or remolding. The loss of strength is due to the permanent destruction of the structure and the reorientation of the molecules in the adsorbed layer. The loss of strength upon destruction of the structure cannot recover over time. But when the remolded soil, left undisturbed at the same water content, regains strength through gradual reorientation of the adsorbed water molecules. This phenomenon of strength loss and strength gain, with no change in volume or water content, is called thixotropy. Thixotropy is a combination of two Greek words thix, meaning ‘touch’ and tropein, meaning ‘to change’. This may also be said to be “a process of softening caused by remolding, followed by a time-dependent return to the original harder state”. Higher the sensitivity, larger thixotropic hardening. Extent of strength gain depends on type of the clay mineral. Mineral that absorb large quantity of water in lattice structure, such as Montmorillonite has greater thixotropic gain compared to other stable clay minerals (Skempton, 1953).

Figure 2.11 shows the gain in strength of soil due to thixotropic effect. Thixotropy has important applications in connection with pile-driving operations. The immediate frictional strength of thixotropic clay in driven piles is less compared to frictional strength after one month, because strength gain with passage of time.

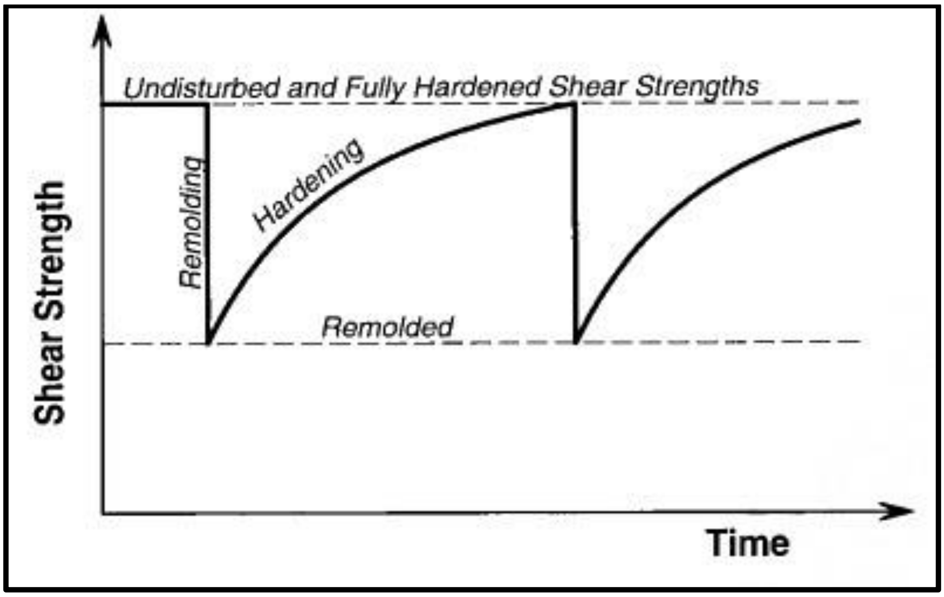

Figure 2.11: Strength loss and strength gain due to thixotropy

2.4 SHEAR STRENGTH

In almost all type of the problems in geotechnical engineering concerning the foundations of structures, slope stability analysis and excavations, the soils must resist the shearing stresses. The shearing stresses tends to displace a part of soil from interconnected soil mass. The ability of the soil mass to resist shearing stresses is its shear strength. It may also be explained as the maximum value of shearing stresses that can be mobilized within a soil mass prior to its failure. If this value comes equal to the shear stress on any plane or a surface, the failure in the soil mass will occur resulting in the movement of a portion of the soil in the soil mass along that specified plane or surface. At this failure surface, shearing stresses reaches the shear strength (τ) of the soil and sliding between the particles takes place. The soil slopes remain stable only if the shear strength of the soils is adequate such as sides of canals or rivers, hills or mountains, human induced cuts and fills etc.

The bearing capacity of a foundation, such as a footing that transfers loads from the superstructure to the underlying soil, also depends on the shear strength of the soil.

If the ground fails in shear and we use the term "shear fracture" also for the failure of the bearing capacity. Since water has no shear strength, it also has no bearing capacity. All the above is proving that shear strength is very basic property of the soil mass on which the pressure exerted by the soil and the pressure resisted by the soil depends. In fact, the entire body of Soil Mechanics is based on the fundamental fact that the characteristic strength of the soil is its shear strength (Babalola, 2016).

2.4.1 Mohr Coulomb's Envelop

The law for the shear strength of soil was first propounded as early as 1773 by Charles Augustine Coulomb, a French military engineer. In fact Coulomb's law for shear strength is considered the first milestone in classical soil mechanics (Murthy, 1989).

In its original form, the law states

$$s = c + \sigma \tan \phi \quad \text{......2.3.5.1}$$

where s is the shear strength (kN/m^2), c is the cohesion (kN/m^2), σ is the normal stress (kN/m^2) and ϕ is the angle of internal friction (degree).

In 1900, Mohr presented a theory for rupture in materials that contended that a material fails because of a critical combination of normal stress and shearing stress, and not from either maximum normal or shear stress alone. The failure envelope defined is generally a curved line. For most soil mechanics problems, it is sufficient to approximate the shear stress on the failure plane as a linear function of the normal stress.

The line satisfying the Equation 2.3.5.1 is called the Mohr-Coulomb failure envelope. It is shown that τ f is the maximum stress soil can take without failure under an applied vertical stress σ. The Mohr circle touches the failure envelope in case of a soil element taken from location of failure surface, whereas the Mohr circle of the soil element taken from other than the location of failure surface is situated below the failure envelope. Keeping σ3 (minor principal stress) constant, if vertical stress (σ_1) increases, the Mohr Circle becomes larger and, finally, it will touch the failure envelope, and failure will take place. The Mohr circle for total stress and effective stress condition is presented in Figure 2.5. The Equation (2.1) represents the shear strength in terms of total stress (σ). In terms of effective stress ($\sigma' = \sigma - u$), Where u is called the pore water pressure. The shear strength of the soil can be expressed as:

$\tau_f = c' + \sigma' \tan \phi'$……………………………………………………….…2.3.5.2

2.4.2 Shear Strength Parameters

The shear strength of a soil is derived from two parameters which are *inherent* properties of the soil. They are cohesion "c" and the angle of internal friction "ϕ".

2.4.2.1 Angle of internal friction

Angle of internal friction for a given soil is the angle on the graph (Mohr's Circle) of the shear stress and normal effective stresses at which shear failure occurs. Angle of internal friction of soil is generally denoted by "ϕ". The angle of internal friction is a function of the characteristics like particle size, compaction effort and applied stress level (Holtz and Kovacs, 1981). Angle of internal friction increases with the increase in particle size (Holtz, 1960). With an increase of density or decrease in void ratio, angle of internal friction increases. It was reported that angle of internal friction decreased with the increasing values of expansive mineral ratio (relative amount of expansive clay mineral to non-expansive clay mineral (Dahal *et al.*, 2009). The angle of internal friction ϕ', which corresponds to a more or less random arrangement of particles, is mainly a function of the clay mineral content and clay mineralogy of the composition. Among the pure clay minerals, sodium montmorillonite (consisting of filmy particles) has the lowest value of ϕ', whereas attapulgite (with interlocking fibers) exhibits the highest value. Typical values of ϕ' for soft clay, stiff clay, and shale constituents are in the range of 25^o to 35^o, 20^o to 35^o, and 15^o to 35^o, respectively (Terzaghi *et al.*, 1996).

2.4.2.2 Cohesion

Cohesion is one of the important components of shear strength soil mainly for fine materials. Cohesion is the attraction by which soil particles are united throughout the mass. Cohesion is the strength of soil which behaves like glue that binds the grains together. Cohesion of soil is usually denoted by "c". As maintained by Mohr-Coulomb equation, cohesion of a soil is defined as the shear strength at zero normal pressure on the surface of failure. Based on this definition, soil cohesion “c” is a constant parameter. In the classical soil mechanics, it is believed that cohesion is the relation and inter-connection between soil particles due to water polar molecules and soil polar particles. Since water has been always present and there has been no change in the particles and particle size distribution, it could be concluded that the polar molecules of water and polar particles of soil have no major role in the creation of cohesion. They are not the true factors that affect soil cohesion (Shahangian, 2011). there are other factors that can affect soil cohesion. These are briefly described below:

- Cohesion due to cementation (which exists, more or less in a large percentage of undisturbed native material)
- Cohesion due to thixotropy (Which is a reversible characteristic that exists in some highly plastic soils and is a result of interaction between soil polar particles),
- Cohesion due to negative capillary pressure (which is lost upon saturation),

- Cohesion due to negative pore pressure during undrained loading (which may be lost through time)
- Cohesion due to soil aging (which could be considered as a type of cementation) Cohesion due to osmotic pressure, Cohesion due to adhesion and interlocking of soil particles.

The type of interparticle bonds that make and affect soil cohesion may by classified in the three following categories:

- Chemical bonds (in cementation and aging),
- Electrostatic and electromagnetic bonds (in consolidation / compaction, capillary stresses and surface tension in non-saturated soils, thixotropy)
- Mechanical bonds (in adherence and interlocking of soil particles)

Each one of cohesion components react independently, may or may not be present and participate in soil shear resistance and the effect of each component of cohesion in soils may be measured accurately by simple laboratory tests.

Rock has a cohesion value of 10,000 kPa, whereas silt has 75 KPa and clay has 10 to 20 kPa. Depending on the stiffness of the clay soft to high, cohesion varies from 0 to 766 kPa. Natural minerals that have been leached into the soil, such as caliches and salts, can provide a very strong cohesion. Heat fusion and long-term overburden pressure will tend to fuse the soil grains together, producing significant cohesion.

2.4.3 Types of Shear Strength

For determining the shear strength of soils in the laboratory, it is a normal practice to use the shear stresses corresponding to the peak load with respect to the effective stresses. Experience has shown, however, that in some cases, especially natural and raised slopes in highly plastic clays, the shear strength may be lower than the values corresponding to the peak stresses. Instead, and depending on the particular slope and its history, the adequate shear strengths may be either the residual strength or the fully-softened strength, which are both lower than the peak strength.

2.4.3.1 Residual shear strength

The term residual strength was apparently first used by Skempton in 1964 to describe the shear strength that is ultimately developed after soil has experienced large strains under drained conditions. For many highly plastic clays the residual shear strength is significantly less than the peak shear strength, with a lower friction angle, ϕ' (ϕ'_r), and a small or negligible cohesion, c′ (c'_r). The residual shear strengths for London Clay, a heavily over-consolidated, stiff-fissured clay, and compared the strength to the strength that was apparently developed over time in the field. It was suggested that over time the residual shear strength would eventually develop and govern the design. However, subsequent studies over time eventually led to the conclusion that residual shear strengths probably only develop in slides that are a recurrence of a previous slide and/or similar large strains have been experienced in the past. Residual shear strengths are probably not applicable to slopes in general (Skempton, 1964).

2.4.3.2 Fully softened shear strength

Further studies by Skempton and his co-workers revealed that the shear strength in many slopes was lower than the peak strength, but higher than the residual value discussed in the previous section. This lower strength has been termed the fully-softened strength. It was observed that the fully-softened strength corresponded to the strength of the soil in “normally consolidated state.” The fully-softened strength can be measured in the laboratory by preparing samples of normally consolidated clay and then testing them. Usually samples are prepared by mixing the soil with water to form a slurry and then consolidating the slurry to various pressures for testing (Skempton, 1970).

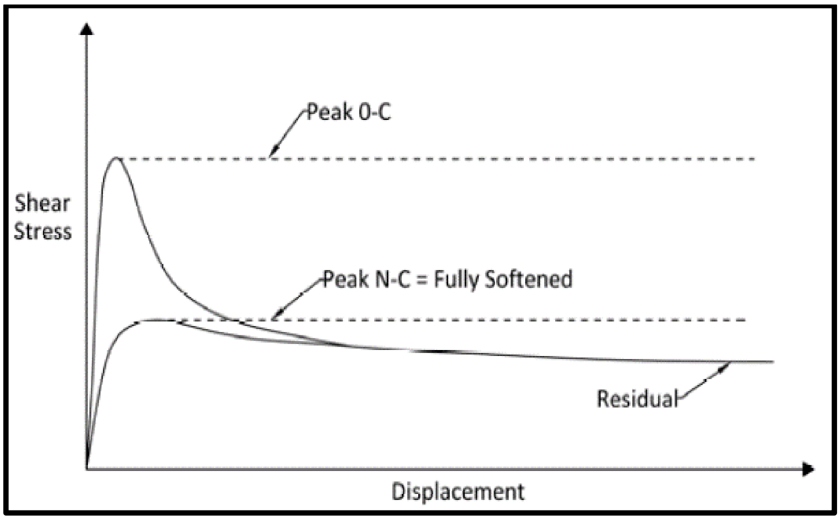

Figure 2.12: Shear strength characteristics of clays

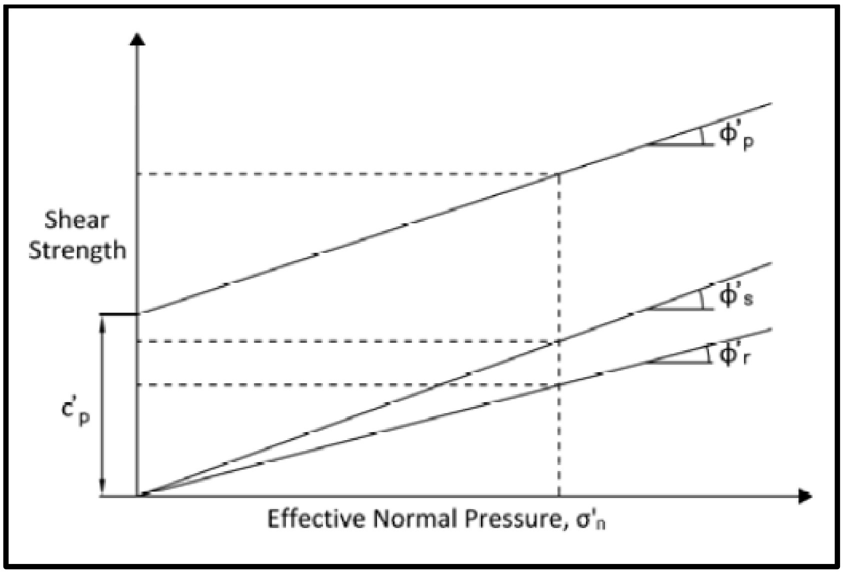

Figure 2.13: Description of fully softened shear strength

The term "fully-softened" strength is used to describe a drained shear strength, expressed in terms of effective stress shear strength parameters, c' and φ'.

Although there is also a softening and reduction in strength that occurs over time simply due to wetting and reduction in the effective stress (σ′), the term fully-softened is generally used in reference to the effective stress shear strength parameters, c′ and φ′, rather than the reduction in effective stress, σ′.

2.4.4 Empirical Correlations for Determination of FSSS

Clay-size fraction and plasticity index are the most common parameters to develop empirical correlations for drained residual and fully softened shear strengths (Mitchell and Soga, 2005; Skempton, 1964). It was also observed that, with the increase of clay fraction friction angle of the soil decreases based on the experimental results obtained from different soils (Skempton, 1964).

An empirical correlation incorporating effective normal stress, LL, and CF, was suggested which provides a good estimate of the friction angles, which was verified by the back analysis of landslide case histories (Stark *et al.*, 2005; Stark and Eid, 1997). These correlations were updated the correlation providing refined equations for three clay fraction groups (Stark and Hussain, 2012). They developed a set of three equations for the empirical correlation for drained fully softened secant friction angles of CF less than 20 percent and for LL values ranging from 30 percent to less than 80 percent. It is observed that, soil with same liquid limit and clay fraction will have lower value of friction angle at higher level of normal stress, which indicates the dependency of fully softened strength with normal stress.

$$(\phi'_{fs})_{\sigma'n=50\text{ kPa}} = 34.85 - 0.0709(LL) + 2.35 \times 10^{-4}(LL)^2 \quad \text{.......................2.3.1}$$

$$(\phi'_{fs})_{\sigma'n=100\text{ kPa}} = 34.39 - 0.0863(LL) + 2.66 \times 10^{-4}(LL)^2 \quad \text{.......................2.3.2}$$

$$(\phi'_{fs})_{\sigma'n=400\text{ kPa}} = 34.76 - 0.13(LL) + 4.71 \times 10^{-4}(LL)^2 \quad \text{.......................2.3.3}$$

A set of three equations was also developed for CF lies in between 25 percent to 45 percent and LL values ranging from 30 percent to 130 percent and is shown as in equation as under:

$(\phi'_{fs})_{\sigma'n=50\text{ kPa}} = 36.18 - 0.1143\,(LL) + 2.354 \times 10^{-4}(LL)^2$……………….....….2.3.4

$(\phi'_{fs})_{\sigma'n=100\text{ kPa}} = 33.11 - 0.107(LL) + 2.2 \times 10^{-4}(LL)^2$………………………..2.3.5

$(\phi'_{fs})_{\sigma'n=400\text{ kPa}} = 30.7 - 0.1263(LL) + 3.442 \times 10^{-4}(LL)^2$…………………......2.3.6

Second degree polynomial function was used to correlate the fully softened friction angle for clay fraction less than 45 percent but for the soil having clay fraction more than 50 percent third degree polynomial was used where the LL values ranging from 30 percent to 300 percent and is given as under.

$(\phi'_{fs})_{\sigma'n=50\text{ kPa}} = 33.37 - 0.11\,(LL) + 2.344 \times 10^{-4}(LL)^2 - 2.96 \times 10^{-7}(LL)^3$……2.3.7

$(\phi'_{fs})_{\sigma'n=100\text{ kPa}} = 31.17 - 0.142\,(LL) + 4.678 \times 10^{-4}(LL)^2 - 6.762 \times 10^{-7}(LL)^3$..2.3.8

$(\phi'_{fs})_{\sigma'n=400\text{ kPa}} = 28.0 - 0.1533\,(LL) + 5.64 \times 10^{-4}(LL)^2 - 8.414 \times 10^{-7}(LL)^3$….2.3.9

For shallow slope stability analysis, fully softened friction angle at normal stress of 50 kPa may be used but use of cohesion is not recommended. It is possible to calculate the shear strength under each normal stress level and form a failure envelope. Linear failure envelope was constructed for a given set of soils, having a liquid limit of 30 and 130. For each of the cases, linear Mohr coulomb envelope was observed with a value of cohesion ranging from 1.3 kPa to 7.9 kPa. But using a value of cohesion for shallow slope stability analysis with a slope which is subjected to drying and wetting will not provide adequate factor of safety (Khan, 2016b).

2.5 SHEAR STRENGTH AND MINEROLOGY

Shearing resistance in soils is the result of resistance to movement at interparticle contacts. Each contact can transmit normal force from one particle to another across an area that increases or decreases as the normal force increases or decreases. Bonds form across the contact areas and, together with any particle interlocking, resist tangential or sliding movements and thus create shearing resistance. The main bonding mechanism, the primary valence bond (in which surface atoms at interparticle contact are joined by sharing and transferring electrons), develops in response to the effective normal stress in the assemblage of particles. It is, therefore, of a physical nature. Other types of bonds may also contribute to the resistance; these include chemical bonds or cementation, which connect soil particles through a solid substance such as recrystallized calcium carbonate.

All these bonds increase with increasing interparticle contact area. Therefore, smectite or montmorillonite clay minerals having greater surface area will have more cohesion as compared to kaolinite and illite. But, as the size of montmorillonite is lesser than kaolinite and illite, it has lower values of angle of internal friction (Terzaghi *et al.*, 1996). Effective normal stress establishes the interparticle contacts at which bonds form; in general, an increase in effective normal stress produces an increase of interparticle contact area and thus an increase in shearing resistance. In some soils, if an increase in effective normal stress is followed by an equal decrease, the contact area may remain larger than the contact area before the stress changes took place. However, if the effective normal stress is reduced to zero, all physical and chemical interparticle bonds are broken, because the interparticle contact area reduces to zero. Chemical bonds or interparticle links, which develop at contacts after soil particles are brought together by an effective normal stress, break as a result of deformations at interparticle contact points when the effective normal stress decreases. Thus, chemical bonding is unlikely to survive an effective stress decrease to zero, and soils have no shearing resistance at zero effective normal stress.

The physio-chemical nature of the bonds at interparticle contacts is of engineering significance because of the insight it provides into the behavior of soils during shear. In practice, however, the behavior is related to more convenient indicators that integrate the physio-chemical effects and that can be measured more readily. The more important of these indicators and the way in which they reflect the influence of the bonding are discussed in the following paragraphs. Density is one important general indicator of shearing resistance. Porosity, void ratio, and water content reflect density for various types of soil.

Composition influences shearing resistance by controlling the densities attainable under normal geologic and construction conditions. For example, at an effective normal pressure of 300 kPa the shearing resistance of compacted rockfill composed of 0.6 to 200 mm angular particles of quarried basalt is 370 kPa, whereas at the other extreme the shearing resistance of a clay composed of sodium montmorillonite particles is only 33 kPa. The main reason for the difference in the shearing resistance of these two soils of extremely different compositions and those of the intermediate compositions is the difference in their void ratios (Khan, 2016a).

2.6 FACTORS EFFECTING THE SHEAR STRENGTH OF EXPANSIVE SOILS

Main factors which may affect the shear strength parameters of expansive soils includes clay content, clay mineralogy, plastic limits, moisture contents, dry density and strain rates. These factors are discussed below in detail.

2.6.1 Clay Content

The amount of clay content within the soil mass has serious effects on cohesion and angle of internal friction. At a water content slightly above the optimum water content, the increase in clay content improves the cohesion. This improvement cannot be achieved if the moisture content is well above the optimum water content. The angle of internal friction decreases with increasing clay content (Chowdhury and Hoque, 2013).

2.6.2 Clay Mineralogy

The presence of clay mineral reduces the shear strength of clay. Clay minerals are consistently weaker than natural rock flour made from crushed granite material. The montmorillonite which is most expandable clay is the weakest clay mineral. Swelling and shrinkage in expansive soils have two extremely opposite effects on shear strength. Shear strength is generally low for fully expansive clay, while dry shrinking clay can develop higher cohesion and a larger angle of internal friction (Chowdhury and Hoque, 2013).

2.6.3 Plasticity Index

As the Plasticity Index increases the shear strength reduces. Plasticity index is directly related with the water holding capacity of soils. Soils with smaller clay size have high plasticity index and lower value for angle of internal friction (Chowdhury and Hoque, 2013).

2.6.4 Moisture Content

The cohesion increases with increasing water content to the optimum water content, over which it decreases with increasing water content. The angle of internal friction decreases with increasing water content and approaches a constant value near the optimum water content. In general, the shear strength decreases with the increase of the water content since the contribution of the shear strength by suction decreases (Chowdhury and Hoque, 2013).

2.6.5 Dry Density

Increase in dry density indicates the compacted soils. The angle of internal friction and cohesion of the clays increases with the increasing the dry density. Therefore, shear strength of clays tends to increase with the increase of dry density (Chowdhury and Hoque, 2013).

2.6.6 Strain Rate

The impact of strain rate is very significant and depends on the test arrangements for drainage conditions and the type of soil being tested. In general, the rate of strain in clays is very low to allow proper dissipation of pore water pressure. A few days may be required to complete a single test. However, the drained shear strength obtained in a test at a rate of 1.2 to 1.3 mm/min may provide a better approximation for undrained shear strength of the samples (Loehr and Bowders, 2007). The increase in shear strain will give increase the undrained shear strength of clays. Therefore, the direct shear tests conducted with high strain rates will give overestimate of the undrained shear strength of clays (Boulanger and Idriss, 2007).

2.6.7 Clay Softening

Clay softening is basically the decrease in peak shear strength before failure. This is the main cause of slides in over-consolidated clays. Actually, clay softening happens due to increase in moisture content which may be due to change in state of stress (Chowdhury and Hoque, 2013).

Fissured over consolidated clays may experience a small reduction in shear strength by swelling caused by unloading. Then it was observe that the dissipation of negative excess pore water pressure may cause the reduction in shear strength parameters (Skempton, 1953). This can be easily clear from a simple example of an excavation. Excavation does not affect the shear strength properties but changing the effective state stress may cause the dissipation of negative excess pore water pressure resulting in the reduction shear strength. However, he did not exclude that during the long-lasting phase of pore pressure equalization, some decay of the shear strength properties can take place. It was also outlined that swelling can provoke a decrease in the dilative and brittle behavior of clay, causing a decrease in the shear strength through a loss of its component associated with over-consolidation: therefore, the long-term strength, the so called fully-softened strength, could be very close to the critical value (Picarelli *et al.*, 2006).

The mechanism of decrease in shear strength parameters in fissured stiff clays is due to the opening of fissures which is due to swelling of adjacent clay under zero confining stress or reconsolidation of clay under its own weight. However, this mechanism does not apply to all cases, especially to slightly fissured clay. Also, when consolidated undrained triaxial testing was done on non-fissured or slightly fissured over-consolidated clays under low confining stress, it was observed that swelling caused the reduction in the cohesion which is due to increase in moisture content (Takahashi *et al.*, 2005).

. However, cohesion does not completely vanish, and the soil behavior remains dilative due to over-consolidation. It is worth mentioning that softening, as described above, has some similarities with other phenomena that are responsible for time-depending decay of shear strength, such as weathering, slaking, i.e., soil de-structuration caused by cycles of wetting-drying or of freezing-thawing (Graham and Au, 1985). Through accumulated plastic strains, all these phenomena, generally concentrated in the most superficial soil layers, can determine a loss of that part of the shear strength that depends on interparticle bonding, causing a reduction in cohesion. Therefore, they affect only bonded clays.

It was only assumed that shrinkage and swelling may cause the de-structuration of the clay bonds. This fact is proved by excessive laboratory testing which is done on the undisturbed specimens of London clay (Takahashi *et al.*, 2005). All mechanisms mentioned above show how complicated is the interpretation of slope instability in stiff over-consolidated clay and clay shale, since more than one of them can contemporaneously act in the same slope at same time. Furthermore, strain-softening (progressive failure) and rate effects can play an additional and significant role. However, laboratory data and field observations on highly fissured plastic clay shales of Italian Apennines show that a decrease in the shear strength could be caused by chemical-physical processes provoked by exposure of soil to fresh water.

2.7 PREVIOUS STUDIES

Khan *et al*. (2016) reported the effect of wet and dry cycles on shear strength of two soil samples under low overburden stress. Wet and dry cycle consisted of 24 hour of wetting and 24 hours drying. The angle of internal friction and cohesion were reduced significantly after 5^{th} wet and dry cycle. this reduction was very significant in the initial cycles and it became constant after 5^{th} cycle. This decrease in shear strength parameters was due to soil disturbance after wet and dry cycles and increase in moisture content and void ratio. He proposed to further investigate the effect of wet and dry cycles on shear strength of low plastic clays under low plastic clays.

Khan *et al*. (2015) reported the importance of shallow slope stability analysis. The failure mechanism of two slopes subjected to excessive rain fall events were checked with the help of PLAXIS 2D. He noticed the depth of the compacted highway embankments from 3 ft to 6ft. the slopes were stabilized with the help of recycled plastic pins. The suitability of the plastic pins were also confirmed with the help of PLAXIS 2D. The back analysis of the failed slopes was done to compare the shear strength parameters of these slopes. The use of recycles plastic pins significantly reduced the settlement of the slopes and its stability against wet and dry cycles.

Castellanos (2014) proposed a method for determination of fully softened shear strength of clays. The peak shear strength of the normally consolidated clay sample was named as fully softened shear strength. Normally consolidated clay sample was prepared from mixture of disturbed oven dried soil sample and water content equal to twice the liquid limit of the clay sample. The clay sample is completely mixed with the help of mechanical blender to form a slurry. Excess water is removed with the help of filter paper. This slurry is shifted to the ring shear device with the help of spatula. He proposed a draft of ASTM standard for determination of fully softened shear strength from direct shear test.

Stark and Hussain (2012) updated the correlations for fully softened secant angle of internal friction and drained residual secant angle of internal friction by increasing data points. These correlations were divided into three main groups of clay fractions (CF ≤ 20 percent, 25 percent ≤ CF ≤ 45 percent and CF ≥ 50 percent) and liquid limits (30 percent ≤ LL ≤ 80 percent, 30 percent ≤ LL ≤ 180 percent, and 30 percent ≤ LL ≤ 300 percent). The correlations for drained residual shear strength were proposed for normal stresses of 50 kPa, 100 kPa, 200 kPa and 400 kPa. Similarly, the correlations for fully softened secant angle of internal friction were proposed for normal stresses of 50kPa, 100 kPa, and 400 kPa. He also gave a clue about self-healing in clays as a function of plasticity of the clays.

Lade (2010) presented the shallow slope failure mechanisms. The shallow slope failure is more in the semi-arid regions where the surface soil dried out completely followed by excessive rainfall. This rainwater infiltrates into the soil and cause its failures. In southern California, more than 1000 shallow slope failures were reported due to heavy rainfall in 1986. For low normal stress present in surficial slope failure events, it is not safe to use the Coulomb's failure criterion where the failure envelops were determined at higher normal stresses. He proposed a simple failure criterion for surficial slope stability analysis.

Wright *et al*. (2007) determined the fully softened shear strength of compacted high plastic clays of Texas region using triaxial testing procedures. He compared his fully softened shear strength with the correlations made by previous researchers to check their validity for high plastic clays. He did the slope stability analysis of different slopes in Texas region having high plastic clays. He concluded that wet and dry cycles reduced the shear strength of compacted high plastic clays to fully softened sear strength.

Stark and Eid (1997) investigated from several torsional ring shear tests that fully softened shear strength is stress dependent. They proposed empirical correlations to determine the fully softened shear strength from liquid limit, clay size fraction and normal stress. The correlations are made in three groups of clay fractions (CF ≤ 20 percent, 25 percent ≤ CF ≤ 45 and CF ≥ 50 percent) and for three normal stresses (50 kPa, 100 kPa and 400 kPa). The proposed location of fully softened shear strength was in between the residual shear strength and peak shear strength. They reported that the numerical difference between the residual shear strength and fully softened shear strength as a function of the clay minerology and normal stress.

Rogers *et al*. (1986) did extensive laboratory testing to check the impact of wet and dry cycles on shear strength of high plastic and low plastic clays under high overburden stress. A wet and dry cycle also consisted of 24 hour of wetting and 24 hours of drying. Soil samples were tested after wet and dry cycles of 1 to 30. A signification reduction in angle of internal friction and cohesion of both high plastic clays and low plastic clays was reported during the initial cycles. After 5^{th} cycle, this reduction in angle of internal friction was very less. He recommended further laboratory testing to investigate the effect of wet and dry cycles on long tern strength of embankments.

Wani *et al*. (1980) investigated the effect of wet and dry cycles on physical properties of different soil samples. A physical weathering of the soil mass is reported after each wet and dry cycle. Wet and dry cycles caused the formation of cracks and fissures in the soil mass reducing its shear strength. . A thixotropic hardening was also observed in the soils after wet and dry cycles. Further laboratory investigations were suggested to determine the degree of weathering caused by wet and dry cycles.

Skempton (1970) was the first one to use the term fully softened shear strength. Previously, he recommended to use the residual shear strength for safe slope stability analysis which was very uneconomical design. When the back analysis of several failed slopes was done, it was concluded that the angle of internal friction failure determined from the laboratory testing was less than the peak shear strength but greater than the residual shear strength. This shear strength at failure of these slopes was named as fully softened shear strength. It was defined as the peak shear strength of normally consolidated clay. He advised to use the fully softened angle of internal friction in the design of slopes.

Chapter 3

MATERIALS AND METHODOLOGY

3.1 GENERAL

This chapter includes the sampling techniques, testing programs, sample preparation methods, wetting and drying cycle applications in details. All the tests were performed in accordance with the ASTM standards at Geotechnical Engineering Laboratory in NUST Institute of Civil Engineering Islamabad.

3.2 MATERIALS

The materials used for this research were clays taken from natural slopes, excavations and road embankments from different locations near Islamabad. The details of soils and materials and locations are shown with the help of the Table 3.1.

Table 3.1: Details of sample locations

Sample ID	Location
Sample 1	Ballewala, Near Nandipur, District Gujranwala
Sample 2	Artificially prepared from Ballewala soil and 10% Bentonite
Sample 3	Artificially prepared from Ballewala soil and 30% Bentonite
Sample 4	Excavation site D-12, Islamabad
Sample 5	Natural slope, Shah Allah Ditta, Islamabad
Sample 6	Natural slope Burhan Near N-5
Sample 7	Excavation site, NUST institute of civil Engineering, Islamabad
Sample 8	Road embankment, Kashmir Highway
Sample 9	Natural slope, Sunny Bank, Muree
Sample 10	Natural Slope Hassan Abdaal near N-5

3.3 METHODOLOGY

The research work was divided into three main phases as follows:

- Phase-I: Characterization of soil samples
- Phase-II: Effect of wet and dry cycles
- Phase-III: Self-healing in soil after 6th wet and dry cycles

3.3.1 Phase I: Characteristics of Soil Samples

The soil samples are characterized on the index properties. Sieve analysis, hydrometer analysis, specific gravity, Atterberg limits, activity, swell index, maximum dry density and optimum moisture content.

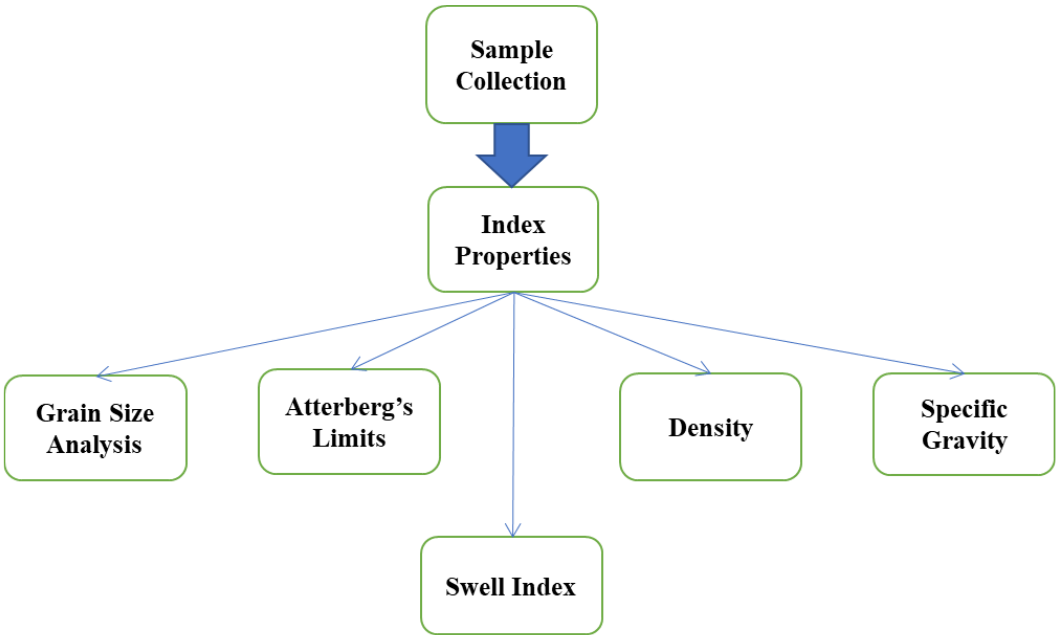

Figure 3.1: Flow chart for phase-I

3.3.1.1 Grain size analysis (GSA)

Full grain size analysis consists mainly of two tests i.e. sieve analysis and hydrometer analysis. 300 grams of oven dried representative soil sample was taken and pulverized. This soil mass is then sieved in standard set of sieves as recommended in ASTM (D422, 2016). For hydrometer analysis, 40 grams of oven dried soil is passed from sieve #200 and added in 12.5 percent solution of 4 percent sodium hexa-meta phosphate as recommended by ASTM (D7928, 2016).

3.3.1.2 Atterberg's limits

Atterberg limits test is performed as per ASTM (D4318, 2016) using Casagrande's apparatus. 250 grams of soil is passed through sieve #40 and is used for the test. Liquid limits and plastic limits for all the soil samples used in this research are determined to calculate the Plasticity Index (Lupini *et al.*) of the soil. Two values for liquid limits and plastic limit for each soil sample are calculated and average value is reported.

3.3.1.3 Specific gravity (G_s)

The specific gravity test was performed as per ASTM (D854,2014). 40 grams of oven dried soil mass passing sieve #16 and 250 ml flasks were used for this test. Two specific gravity values for each soil sample are determined and average value is reported.

3.3.1.4 Moisture-density relationship

Compaction behaviour of the soil after addition of moisture is determined by Standard Proctor Test. This test is performed as per ASTM (D698, 2012). 3500 grams of oven dried soil mass passing sieve #4 is used for this test.

3.3.1.5 One dimentional swell

The samples were compacted at 95% of maximum dry density and optimin moisture content in the rings used for one dimentional swell of soils. These rings were placed in the oedometer and submerged in water for complete saturation. Loading and unloading of the samples is done as per ASTM (D4546, 2014).

From the oedometer, swell index (C_S) value is determined which is the slope of unloading curve. Prediction of swelling nature of the soil from oedometer swell index is done by several researchers. Seed *et al.* (1962) presented the prediction of degree of expansion which is used by United States Bureau of Reclamation (USBR) is shown in Table 3.2.

Table 3.2: Prediction of degree of expansion from swell index

Degree of Expansion	Swell Potential (percent)
Very High	< 25
High	5 – 25
Medium	1.5 – 5
Low	0 – 1.5

The swelling behavior of the clay samples may also be predicted from the activity, plasticity index, liquid limit and clay fraction. Table 3.3 shows some criteria for prediction of swelling nature of the clays. This summary is presented by Yilmaz (2004).

Table 3.3: Summary of some criteria for identifying degree of expansion

Degree of Expansion	Chen (1963)	Seed et al. (1962)	Daksanamurthy and Raman (1973)	Holtz and Gibbs (1956)
Very High	LL > 60	PI > 35	LL > 70	CC > 28
High	40 < LL < 60	20 < PI < 35	50 < LL < 70	20 < CC < 31
Medium	30 < LL < 40	10 < PI < 20	35 < LL < 50	13 < CC < 23
Low	LL < 30	< 10	20 < LL < 35	CC < 13

Where, LL = Liquid limit (percent) , PI = Plasticity index (percent) , CC = Clay content (percent).

3.3.1.6 Shear strength parameters

Direct shear tests performed as per ASTM (D3080, 2014) on the automatic strain-controlled machine to determine the shear strength parameters of soils. Soil samples were compacted at 95 percent of MDD and OMC obtained from Standard Proctor Test in the rings used for direct shear test. The diameter and height of the rings were 6 cm and 2 cm respectively. The samples are compacted very carefully and transfered to the mould for direct shear test. Three compacted soil rings are prepared for each soil test and shear strength parameters were determined for low overburden stresses of 20 kPa, 30 kPa and 40 kPa as specified in the scope of this research.

3.3.1.7 XRD analysis

X-Ray Diffraction analysis of selected soil samples is done in Center for Advanced Studies (CAS), NUST Islamabad to determine the mineralogical configuration of selected soil samples to guess its behavior towards cyclic wetting and drying and self-healing. Soil samples are used in powdered form in the glass sample holder having 10 mm diameter and 4 mm height to get the crystallinity pattern and results are interpreted by a computer application MDI JADE 6.5.

3.3.2 Phase II: Effect of Wet and Dry Cycles

The effect of wet and dry cycles on shear strength parameters is determined after 0^{th}, 2^{nd}, 4^{th} and 6^{th} cycle along with other properties like void ratio, moisture content and density. The flow chart containing the detailed methodology of this phase is shown as under.

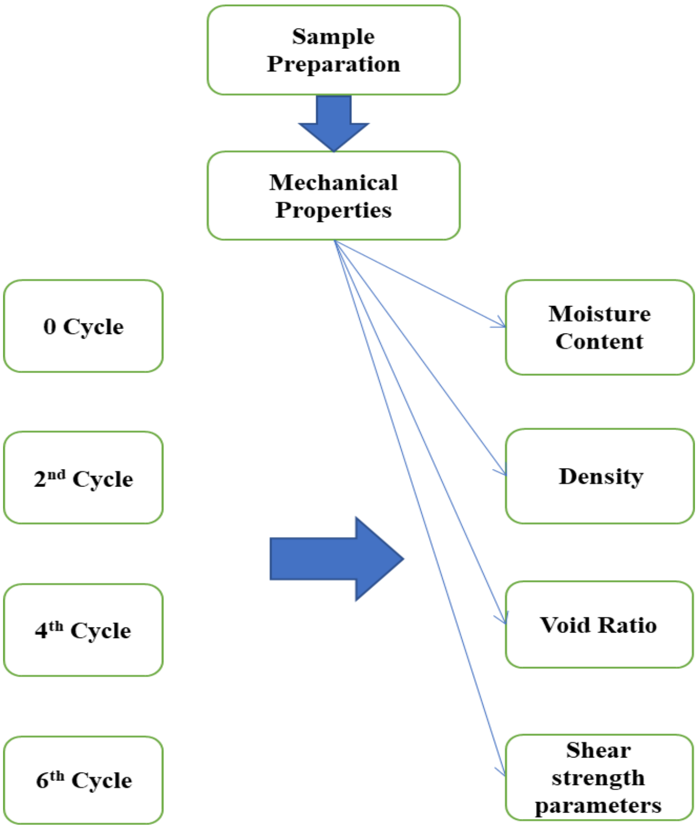

Figure 3.2: Flow chart for phase-II

3.3.2.1 Preparation of test samples

Samples were compacted in specially made PVC hard cylinders 2.5 inches (67 mm) diameter and 2.5 inches (67 mm) height at 95 percent MDD and OMC with 0.5 pounds hammer. Number of blows are adjusted to get standard compaction effort. These hard PVC cylinders were used for the application of wet and dry cycles as it is non corrosive material. Outer ring diameter for direct shear test was 64 mm. larger diameter PVC ring is selected to get a representative sample after trimming. Total 15 PVC rings are prepared for each soil sample at once and tested at 0^{th}, 2^{nd} and 6^{th} cycles of wetting and drying. The sample preparation may be shown in figures below. There is no standard method for the sample preparation. Same methodology was adopted by previous researchers (Khan, 2016a; Rogers and Wright, 1986). Soil loss is prevented by use of a filter paper and porous stone on top and bottom of the soil inside the cylinder as shown in the figure below.

Figure 3.3: Hard PVC cylinders

Figure 3.4: Sample compaction in the cylinders

3.3.2.2 Wetting and drying procedure

The prepared samples were immersed in water tank for 24 hours and then kept in the room temperature for 24 hours completing first cycle of wetting and drying. Thus 1st complete wet-dry cycle consists of wetting for 24 hours and drying for 24 hours at room temperature. These steps were repeated for further wet-dry cycles. Same methodology was adopted by previous researchers (Khan, 2016a; Rogers and Wright, 1986).

Figure 3.5: Sample preparation before applying wet and dry cycles

Figure 3.6: Submerged samples in water

3.3.2.3 Shear strength parameters after wet and dry cycles

After the application of wet and dry cycle, the sample is carefully extruded and trimmed to the size of the ring of shear test device. Digital direct shear device is used to performed DST on the samples after 24 hours drying. Other parameters like void ratio, moisture content and density were also calculated from the trimmed sample for 0^{th}, 2^{nd}, 4^{th} and 6^{th} cycle of wetting and drying. Two values for void ratio, moisture content and density are calculated, and average value is reported.

3.3.3 Phase III: Self Healing

Gain in strength after 6^{th} wetting and drying cycle is determined along with other properties like void ratio, density and moisture content variation. The flow chart showing the methodology of this phase is Figure 3.7.

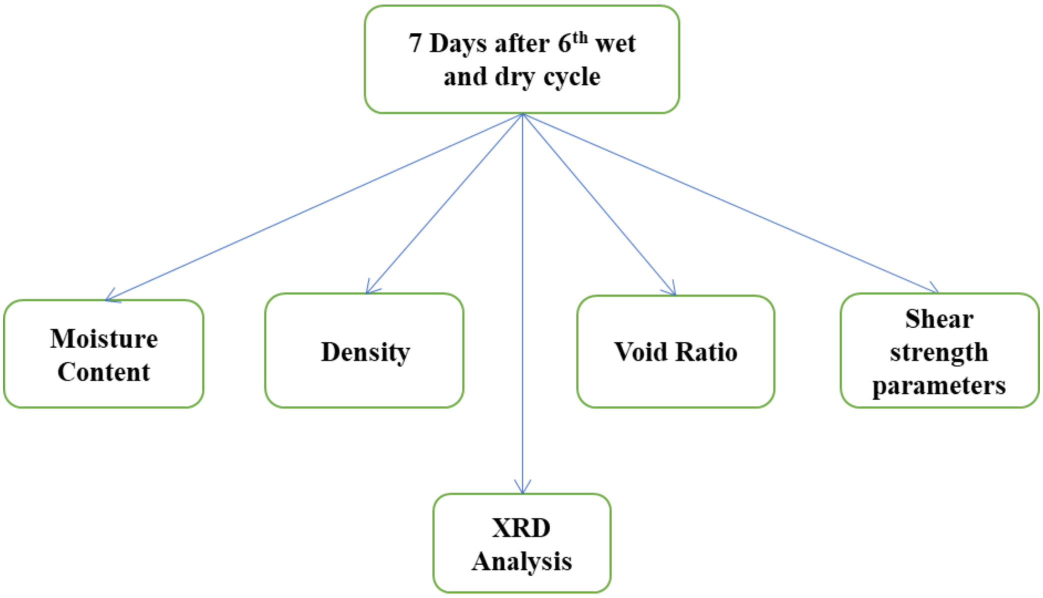

Figure 3.7: Flow chart for phase-III

3.3.3.1 Shear strength parameters 7 days after 6th wet and dry cycle

The PVC cylinders which are prepared on the same day of all prepared soil samples are placed at room temperature after 6th wetting and drying cycle. The moisture is allowed to evaporate simulating the same conditions in the actual in-situ conditions. These samples are tested 7 days after completion of 6th wet and dry cycle. The specimen is extruded and trimmed very carefully to the size of direct shear apparatus. The sample is testing in that dry state at strain rate of 0.01mm/sec immediately. Void ratios, moisture contents and densities of the trimmed sample are determined.

Chapter 4

RESULTS AND DISCUSSION

4.1 GENERAL

Characterization of the soil samples is based on its index properties. To classify the soils, sieve analysis, Hydrometer analysis, specific gravity, swell index and Standard Proctor Test were carried out in this research. A series of DST was done for each soil sample at optimum moisture content, 2^{nd}, 4^{th}, and 6^{th} wet and dry cycles for determining the effect of wet and dry cycles on shear strength parameters of the soils. Also, the variation in void ratio and moisture content was determined after each wet and dry cycle to justify the reduction in shear strength of clays. Shear strength parameters shows different trends after each wet and dry cycles for low plastic and high plastic clays.

4.2 PHASE-I: CHARACTERIZATION OF CLAY SAMPLES

Disturbed soil samples were collected from different sites at depth ranging from 3 ft to 6 ft depth. The soil was pulverized completely for performing the initial testing for the index properties of clays to classify the clays as per the Unified Soil Classification System (USCS). The results are discussed below in details.

4.2.1 Grain Size Analysis (GSA)

Grain size analysis includes the sieve analysis for determining the percentage passing sieve #200 and hydrometer analysis for determining the clay size particles in each clay sample. Table 4.1 showed the details of sieve and hydrometer analysis.

Table 4.1: Grain size analysis of samples

Sample No.	Passing sieve #200 (percent)	Clay fraction (percent)
1	98	19
2	97	22
3	96	23
4	98	18
5	93	19
6	95	18
7	94	18
8	95	18
9	92	18
10	96	12

The percent passing sieve #200 was above 90 percent for all the clays and percent clay fraction was ranged from 12 percent to 23 percent.

4.2.2 Atterberg's Limits

Liquid limits of clay samples ranged from 24.5 percent to 61.25 percent and plasticity index limit ranged from 13.47 percent to 35.53 percent. The details of liquid limits, plastic limits and plasticity index are shown in Table 4.2.

Table 4.2: Atterberg limits of clay samples

Samples	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index (percent)
1	48.75	24.65	24.1
2	57.25	25.72	31.53
3	61.25	25.72	35.53
4	28.5	14.45	14.05
5	25.25	11.09	14.16
6	31.5	14.95	16.55
7	26	11.66	14.34
8	24.5	11.03	13.47
9	27	11.72	15.28
10	29	16.31	12.69

4.2.3 Specific Gravity

As all the soil samples used in this study were clays so the determined specific gravity ranged from 2.65 to 2.71. Table 4.3 shows the details of specific gravity of the clay samples.

Table 4.3: Specific gravity of clay samples

Soil Samples	Specific Gravity
1	2.68
2	2.7
3	2.71
4	2.67
5	2.66
6	2.65
7	2.68
8	2.66
9	2.67
10	2.68

4.2.4 Soil Classification

Based on the above index properties soil samples are classified. After plotting the Liquid limits and plastic limits of all soil samples on plasticity chart, The soil samples are categorized as CL and CH as shown in the figure 4.2.

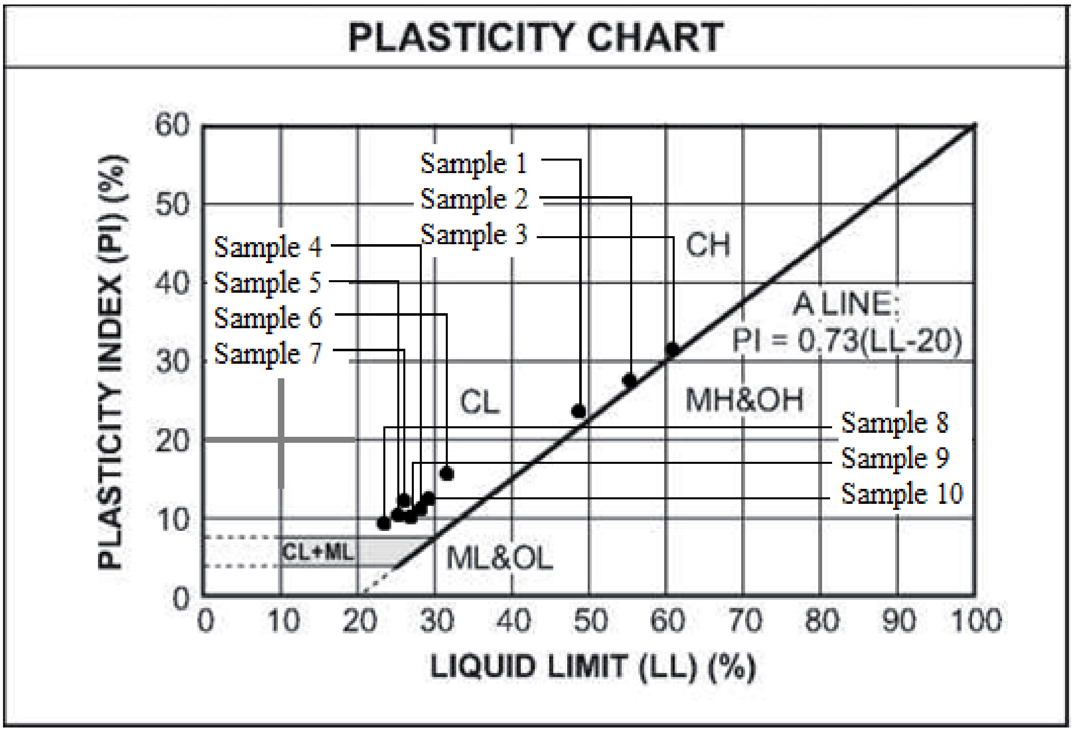

Figure 4.1: Plasticity chart showing soil types

4.2.5 Moisture-Density Relations

Standard Proctor test was performed for all soil samples. The maximum dry density varied from 1.699 g/cm^3 to 1.96 g/cm^3 and optimum moisture content varied from 11 percent to 24 percent. The details of MDD and OMC values for all samples are shown in Table 4.4.

Table 4.4: Moisture-density relations of the samples

Sample No.	Maximum dry density (g/cm^3)	Optimum moisture content (percent)
1	1.71	17
2	1.7	21
3	1.699	25
4	1.88	13.5
5	1.925	11
6	1.81	15
7	1.87	14
8	1.82	15
9	1.96	12
10	1.83	16

4.2.6 Activity and Swell Potential

Activity of soil samples was determined from the Plasticity Index and percent clay size fractions in the clay samples. Some clay samples showed high activity up to 1.54. The one-dimensional swell index was determined from the one-dimensional swell test. Details of activity and one-dimensional swell soils as shown in Table 4.5. Sample 1, 2, 3, 6 and 7 showed very high activity. But sample 4, 5, 7, 8 and 9 showed normal to medium activity.

The one dimensional swell calculated was in the range of 2.03 to 15.98 percent. Based on the Seed *et al*. (1962) and Yilmaz (2004) criteria, as discussed in Chapter 3, sample 1, 2, 3, 6 and 10 are high swelling but rest of the samples are medium swelling.

Table 4.5: Activity and one dimentional swell of soil samples

Sample No.	Clay Type	Activity	One dimensional Swell (percent)	Degree of expansion (Seed et al., 1962)	Degree of expansion (Yilmaz, 2004)
1	CL	1.27	8.66	High	High
2	CH	1.43	10.31	High	High
3	CH	1.54	15.98	High	High
4	CL	0.78	3.02	Medium	Medium
5	CL	0.75	2.11	Medium	Medium
6	CL	0.92	7.34	High	High
7	CL	0.80	3.45	Medium	Medium
8	CL	0.75	3.12	Medium	Medium
9	CL	0.85	2.03	Medium	Medium
10	CL	1.06	6.76	High	High

XRD Analysis of sample 1, 3, 6 and 7 was done to understand the activity and swelling nature of the soil samples. Sample 1 is mainly composed of muscovite, quartz and caysichite mineral. Caysichite is a member of orthorhombic group of minerals. Muscovite is a member of illite group of clay minerals, which have very higher specific surface area and high cation exchange capacity. Therefore, it has thick double defused layer resulting in high swell potential when subjected to water. Figure 4.3 shows the mineralogical composition of sample 1.

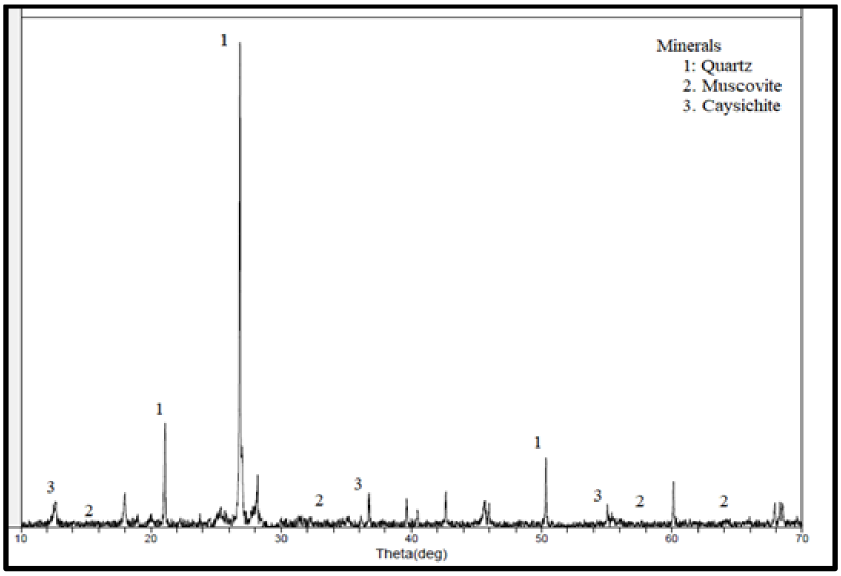

Figure 4.2: XRD Analysis of sample 1

Sample 3 composed of illite, muscovite, gismondine, Quartz, Clinochlore and Chlorite minerals. Illite and muscovite belongs to the Illite clay minerals. Gismondine belongs to zeolite group of minerals and clinochlore belongs to chlorite group of minerals. Illite, muscovite, gismondine and chlinocholre minerals have very high cation exchange capacity, attracting more water ions and having much greater water carrying capacity. There minerals resulted in very high swelling index of sample 3. Mineralogical configuration of sample 3 is shown in Figure 4.3.

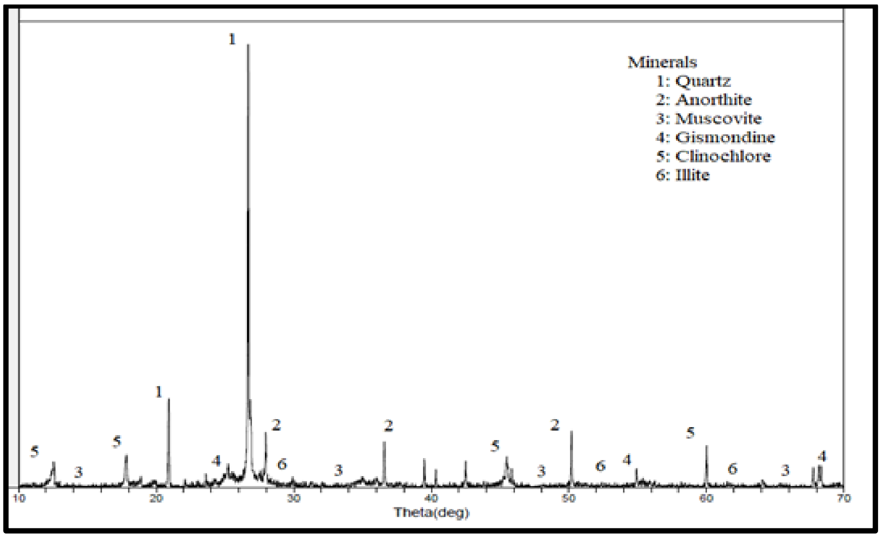

Figure 4.3: XRD Analysis of sample 3

Sample 6 composed of quartz, calcite, anorthite, gismondine and nagashimalite. Anorthite belongs to kaolinite group of minerals and nagashimalite belongs to orthorhombic group of minerals. Gismondine (zeolite) is responsible for high swell index of the sample due to smaller surface area and very high cation exchange capacity.

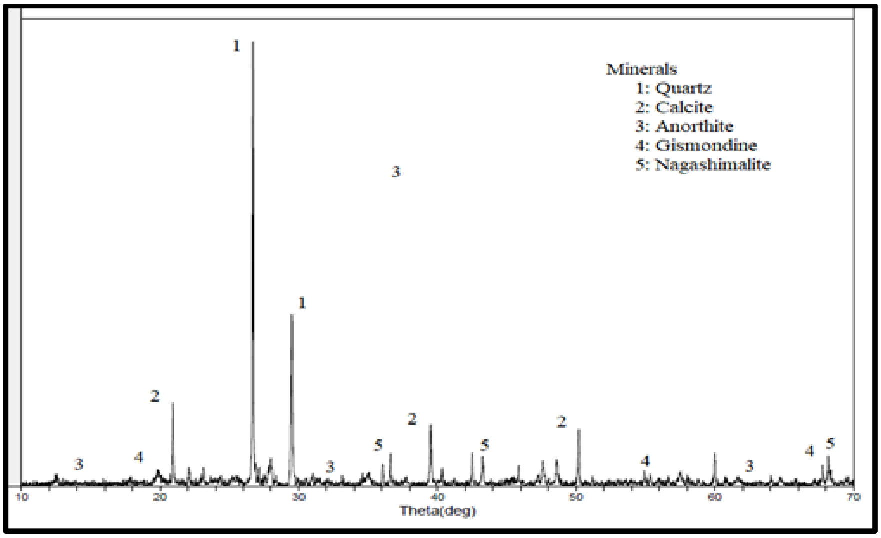

Figure 4.4: XRD analysis of sample 6

Sample 10 composed of quartz, calcite, anorthite, nagashimalite, albite and gismondine minerals. Albite belongs to kaolinite group of minerals. As similar to sample 6, gismondine is responsible for the high swell index of sample 10 because of very high cation exchange capacity.

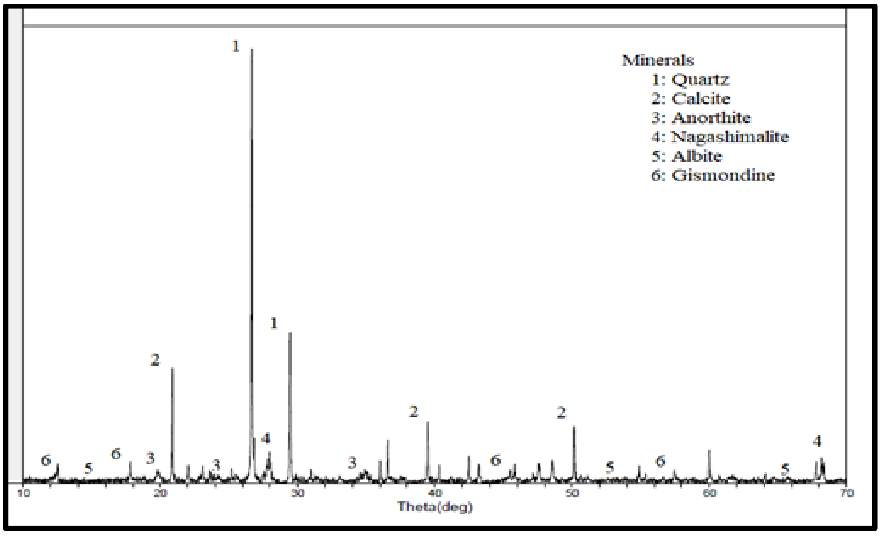

Figure 4.5: XRD analysis of sample 10

Zeolite minerals have cation exchange capacity values in the range of 230 meq. /100g (Mordenite) to 530 meq. /100g (Natrolite). These values are very high as compared to kaolinite and illite group of minerals. Figure 4.6 shows the physical properties of the soil samples.

4.2.7 Summary of Soil Characteristics

Detailed summary of all soil properties asa shown in the the Table 4.6.

Table 4.6: Summary of soil characteristics

Sr. No.	Sample No.	Physical properties									
		Passing sieve # 200 (percent)	CF (percent)	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index (percent)	Maximum Dry density (g/cm^3)	Optimum Moisture Content (percent)	Soil Type	Activity	Swell Index (percent)
1	1	98	19	48.75	24.65	24.1	1.71	17	CL	1.27	8.66
2	2	97	22	57.25	25.72	31.53	1.7	21	CL	1.43	10.31
3	3	96	23	61.25	25.72	35.53	1.699	25	CH	1.54	15.98
4	4	92	18	28.5	14.45	14.05	1.88	13.5	CH	0.78	3.02
5	5	93	19	25.25	11.09	14.16	1.925	11	CL	0.75	2.11
6	6	95	18	31.5	14.95	16.55	1.81	15	CL	0.92	7.34
7	7	94	18	26	11.66	14.34	1.87	14	CL	0.80	3.45
8	8	95	18	24.5	11.03	13.47	1.82	15	CL	0.75	3.12
9	9	92	18	27	11.72	15.28	1.96	12	CL	0.85	2.03
10	10	96	12	29	16.31	12.69	1.83	16	CL	1.06	6.76

4.3 PHASE-II: EFFECT OF WET AND DRY CYCLES

Clay samples were tested in the digital Direct shear testing machine in geotechnical engineering laboratory at NUST. These samples are tested at optimum moisture content, after 2^{nd}, 4^{th}, and 6^{th} wet and dry cycles and shear strength parameters were determined along with the void ratio and moisture content calculations. The soils are divided in two categories, i.e., low plastic clays and high plastic clays.

4.3.1 Low Plastic Clays

Low plastic clays showed a decrease in cohesion of the soil along with a decrease in angle of internal friction. Main reason for reduction in cohesion of the soil sample is the disturbance due to introduction of wet and dry cycles. Decrease in the angle of internal friction may be related with the variation in the effective porosity of the soil sample. The reduction in angle of internal friction, and increase in void ratio were very high in the initial cycles but this variation becomes smaller after 4^{th} and 6^{th} cycle. Wet and dry cycles caused the physical weathering of the soil mass and particle rearrangement. As angle of internal friction is directly related with the relative density of the sample. A slight variation in the void ratio results in very high variation in relative density. Therefore, angle of internal friction was reduced due to increase in void ratio. Variation in the cohesion was very drastic proving it a not reliable parameter and it must be consider as zero in the long-term design of any civil engineering g structure. Angle of internal friction was reduced up to 32 percent for low plastic clays. The details of variation in shear strength parameters and other properties is discussed one by one.

4.3.1.1 Sample 1

At maximum dry density, the cohesion and angle of internal friction of the clay sample were 54.5 kPa and 34.58^o respectively. Soil cohesion was reduced to 1.65 kPa and angle of internal reduced to 21.45^o after 6th wet and dry cycle. This reduction in shear strength parameters is due to increase in void ratio and moisture content of the soil samples to 68 percent and 27.51 percent respectively after 6th wet and dry cycles. This decrease was very prominent in the start and values become constant after 6th wet and dry cycle. The variation in the shear strength parameters and other parameters are shown with the help of the Table 4.7.

Table 4.7: Effect of wet and dry cycles on sample 1

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0th Cycle	54.50	34.58	17	0.57
2nd Cycle	4.65	25.51	25.76	0.65
4th Cycle	2.12	22.98	27.19	0.67
6th Cycle	1.65	21.45	27.51	0.68

4.3.1.2 Sample 4

At maximum dry density, the cohesion and angle of internal friction of the clay sample were 70.12 kPa and 34.17^o respectively. Soil cohesion was reduced to 3.83 kPa and angle of internal reduced to 23.42^o after 6th wet and dry cycle. This reduction in shear strength is due to increase in void ratio and moisture content of the soil samples to 55 percent and 22.43 percent respectively after wet and dry cycles.

The variation in the shear strength parameters and other parameters are explained by the Table 4.8.

Table 4.8: Effect of wet and dry cycles on sample 4

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	70.12	34.17	13.5	0.43
2^{nd} Cycle	35.89	27.17	20.63	0.52
4^{th} Cycle	6.78	24.31	21.44	0.53
6^{th} Cycle	3.83	23.42	22.43	0.55

4.3.1.3 Sample 5

At maximum dry density, the cohesion and angle of internal friction of the clay sample were 84.43 kPa and 28.73^{o} respectively. Soil cohesion was reduced to 4.71 kPa after 2^{nd} wet and dry cycle and a slightly increased to 13.55 kPa after 4^{th} wet and dry cycle. It is further reduced 4.89 kPa after 6^{th} wet and dry cycles. this variation in the cohesion is due to particle rearrangement and physical weathering of the soil. Angle of internal reduced to 24.05^{o} after wet and dry cycles. The reduction in shear strength parameters is due to increase in void ratio and moisture content of the soil samples to 48 percent and 22.56 percent respectively after wet and dry cycles. The variation in the shear strength parameters and other parameters are described in the Table 4.9.

Table 4.9: Effect of wet and dry cycles on sample 5

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	84.438	28.73	11	0.39
2^{nd} Cycle	4.7139	24.93	18.77	0.45
4^{th} Cycle	13.552	24.32	20.34	0.47
6^{th} Cycle	4.8907	24.05	22.56	0.48

4.3.1.4 Sample 6

At maximum dry density, the cohesion and angle of internal friction of the clay sample were 57.98 kPa and 29.74^o respectively. Soil cohesion was reduced to 15.143 kPa after 2^{nd} wet and dry cycle and then slightly increased to 25.14 kPa. After 6^{th} cycle, this value is further decreased to 1.82 kPa. This variation is due to physical weathering and particle rearrangement of clay samples upon wetting and drying. The angle of internal reduced to 22.95^o after 6^{th} wet and dry cycle. This reduction in shear strength parameters is due to increase in void ratio and moisture content of the soil samples to 49 percent and 20.76 percent respectively after wet and dry cycles. The variation in the shear strength parameters and other parameters are shown in the Table 4.10.

Table 4.10: Effect of wet and dry cycles on sample 6

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	57.981	29.74	15	0.41
2^{nd} Cycle	15.143	24.11	20.89	0.47
4^{th} Cycle	25.15	23.45	21.34	0.48
6^{th} Cycle	1.8266	22.95	20.76	0.49

4.3.1.5 Sample 7

At maximum dry density, the cohesion and angle of internal friction of the clay sample were 26.63 kPa and 28.19^{o} respectively. Soil cohesion was reduced to 16.08 kPa after 2^{nd} wet and dry cycle and then increased to 23.805 kPa. After 6^{th} cycle, this value is further decreased to 22.09 kPa. This variation is due to physical weathering and particle rearrangement of clay samples upon wetting and drying. The angle of internal reduced to 23.86^{o} after 6^{th} wet and dry cycle. This reduction in shear strength parameters is due to increase in void ratio and moisture content of the soil samples to 51 percent and 22.03 percent respectively after wet and dry cycles. The variation in the shear strength parameters and other parameters are shown in the Table 4.11.

Table 4.11: Effect of wet and dry cycles on sample 7

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	26.633	28.19	14	0.43
2^{nd} Cycle	16.086	24.86	20.66	0.49
4^{th} Cycle	23.805	24.21	21.21	0.51
6^{th} Cycle	22.096	23.86	22.03	0.52

4.3.1.6 Sample 8

At maximum dry density, the cohesion and angle of internal friction of the clay sample were 36.76 kPa and 29.55^{o} respectively. Soil cohesion was reduced to 4.30 kPa after 2^{nd} wet and dry cycle and then increased to 22.921 kPa. After 6^{th} cycle, this value is further decreased to 6.24 kPa. This variation is due to physical weathering and particle rearrangement of clay samples upon wetting and drying. The angle of internal reduced to 24.98^{o} after 6^{th} wet and dry cycle. This reduction in shear strength parameters is due to increase in void ratio and moisture content of the soil samples to 53 percent and 22.44 percent respectively after wet and dry cycles. The variation in the shear strength parameters and other parameters are shown in the Table 4.12.

Table 4.12: Effect of wet and dry cycles on sample 8

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	36.768	29.55	15	0.44
2^{nd} Cycle	4.3014	25.73	21.58	0.50
4^{th} Cycle	22.921	25.73	22.12	0.52
6^{th} Cycle	6.2459	24.98	22.44	0.53

4.3.1.7 Sample 9

At maximum dry density, the cohesion and angle of internal friction of the clay sample were 40.95 kPa and 33.46^{o} respectively. Soil cohesion was reduced to 18.914 kPa after 2^{nd} wet and dry cycle and then increased to 24.39 kPa after 6^{th} cycle. This variation is due to physical weathering and particle rearrangement of clay samples upon wetting and drying. The angle of internal reduced to 23.25^{o} after 6^{th} wet and dry cycle. This reduction in shear strength parameters is due to increase in void ratio and moisture content of the soil samples to 50 percent and 18.93 percent respectively after wet and dry cycles. The variation in the shear strength parameters and other parameters are shown in the Table 4.13.

Table 4.13: Effect of wet and dry cycles on sample 9

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	40.952	33.46	12	0.37
2^{nd} Cycle	18.914	27.36	16.59	0.44
4^{th} Cycle	22.273	23.98	17.45	0.47
6^{th} Cycle	24.394	23.25	18.93	0.50

4.3.1.8 Sample 10

At maximum dry density, the cohesion and angle of internal friction of the clay sample were 54.63 kPa and 33.46^{o} respectively. Soil cohesion was reduced to 9.36 kPa after 2^{nd} wet and dry cycle and then slightly increased to 21.64 kPa after 4^{th} cycle. After 6^{th} cycle, this value is further reduced to 3.064 kPa. This variation is due to physical weathering and particle rearrangement of clay samples upon wetting and drying. The angle of internal reduced to 23.21^{o} after 6^{th} wet and dry cycle. This reduction in shear strength parameters is due to increase in void ratio and moisture content of the soil samples to 55 percent and 24 percent respectively after wet and dry cycles. The variation in the shear strength parameters and other parameters are shown in the Table 4.14.

Table 4.14: Effect of wet and dry cycles on sample 10

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	54.632	33.46	16	0.46
2^{nd} Cycle	9.3689	27.31	22.89	0.51
4^{th} Cycle	21.64	26.55	23.32	0.52
6^{th} Cycle	3.064	23.21	24	0.55

4.3.2 High Plastic Clays

High plastic clays showed very similar trend. Major decrease in cohesion of the soil along with a considerable decrease in angle of internal friction was observed in the soil samples when subjected to wet and dry cycles. The reduction in angle of internal friction, and increase in void ratio were very high in the initial cycles but this variation becomes smaller after 4^{th} and 6^{th} cycle. Angle of internal friction was reduced up to 30 percent for high plastic clays. The details of variation in shear strength parameters and other properties are discussed one by one.

4.3.2.1 Sample 2

At maximum dry density, the cohesion and angle of internal friction of the clay sample were 36.82 kPa and 33.46^{o} respectively. Soil cohesion was reduced to 0.94 kPa after 6^{nd} wet and dry cycles. This variation is due to physical weathering and particle rearrangement of clay samples. The angle of internal friction reduced to 19.63^{o} after 6^{th} wet and dry cycle. This reduction in shear strength parameters is due to increase in void ratio and moisture content of the soil samples to 77 percent and

34.11 percent respectively after wet and dry cycles. The variation in the shear strength parameters and other parameters are shown in the Table 4.15.

Table 4.15: Effect of wet and dry cycles on sample 2

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	36.827	33.46	21	0.58
2^{nd} Cycle	5.8924	23.18	28.39	0.72
4^{th} Cycle	4.7728	20.53	31.21	0.74
6^{th} Cycle	0.9428	19.63	34.11	0.77

4.3.2.2 Sample 3

At maximum dry density, the cohesion and angle of internal friction of the clay sample were 32.87 kPa and 32.74^{o} respectively. Soil cohesion was reduced to 2.00 kPa after 6^{nd} wet and dry cycles. This variation is due to physical weathering and particle rearrangement of clay samples upon wetting and drying. The angle of internal friction reduced to 19.32^{o} after 6^{th} wet and dry cycle. This reduction in shear strength parameters is due to increase in void ratio and moisture content of the soil samples to 77 percent and 48.28 percent respectively after wet and dry cycles. The variation in the shear strength parameters and other parameters are shown in the Table 4.16.

Table 4.16: Effect of wet and dry cycles on sample 3

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	32.879	32.74	24	0.60
2^{nd} Cycle	6.0691	22.31	37.13	0.71
4^{th} Cycle	4.1247	21.43	44.93	0.74
6^{th} Cycle	2.0034	19.32	48.28	0.77

4.3.3 Summary of Reduction in Cohesion

The cohesion of the clay samples reduced up to 98 percent. It is concluded that cohesion of the soil is very sensitive parameter.

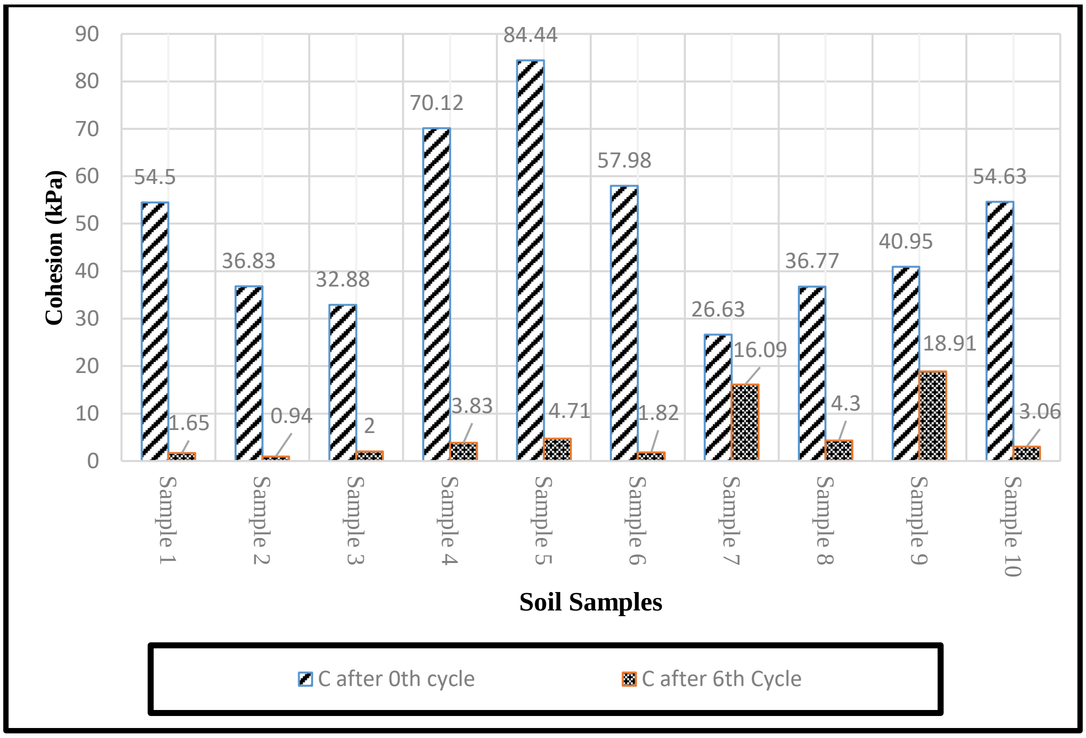

Figure 4.6: Summary of reduction in soil cohesion of different soil samples

4.3.4 Summary of Reduction in Angle of Internal Friction

The angle of internal friction was reduced up to approximately 30 percent for low plastic and high plastic clays as shown in Figure 4.8. This reduction was less in some samples. Sample 6 have kaolinite minerals which have larger grain size as compared to illite and muscovite. Therefore, the reduction of angle of internal friction was less in samples 6.

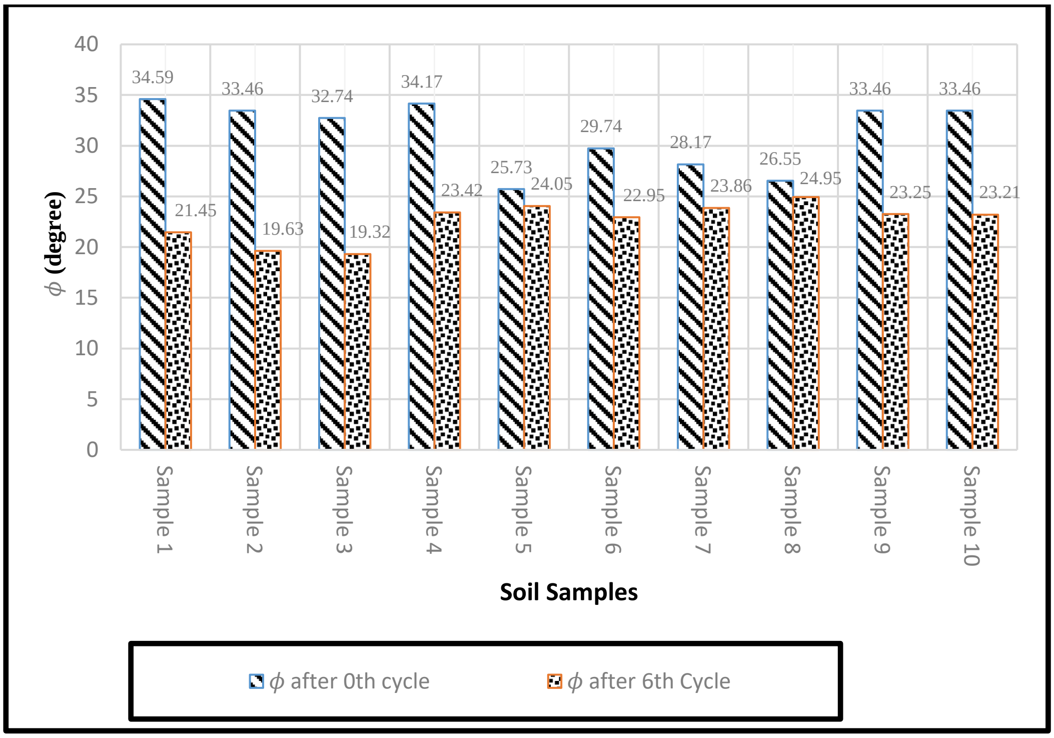

Figure 4.7: Summary of reduction in angle of internal friction of different soil samples

4.3.5 Comparison with Stark and Hussain (2012) Correlations

Skempton (1970) recommended the use of fully softened shear strength as the design strength for slope stability analysis. Therefore, the reduced values for angle of internal friction were plotted against liquid limits of all the samples. The values of angle of internal friction were determined from the updated correlations by Stark and Hussain (2012) for fully softened angle of internal friction for overburden stress of 50 kPa, 100 kPa and 400 kPa. It is clear from the Figure 4.8 that the reduced angle of internal friction after 6th wet and dry cycles is lesser than the fully softened angle of internal friction proposed by Stark and Hussain (2012). Therefore, these correlations are not valid for low over burden stresses (<50 kPa) and there was a need to propose a correlations for shallow slope stability analysis.

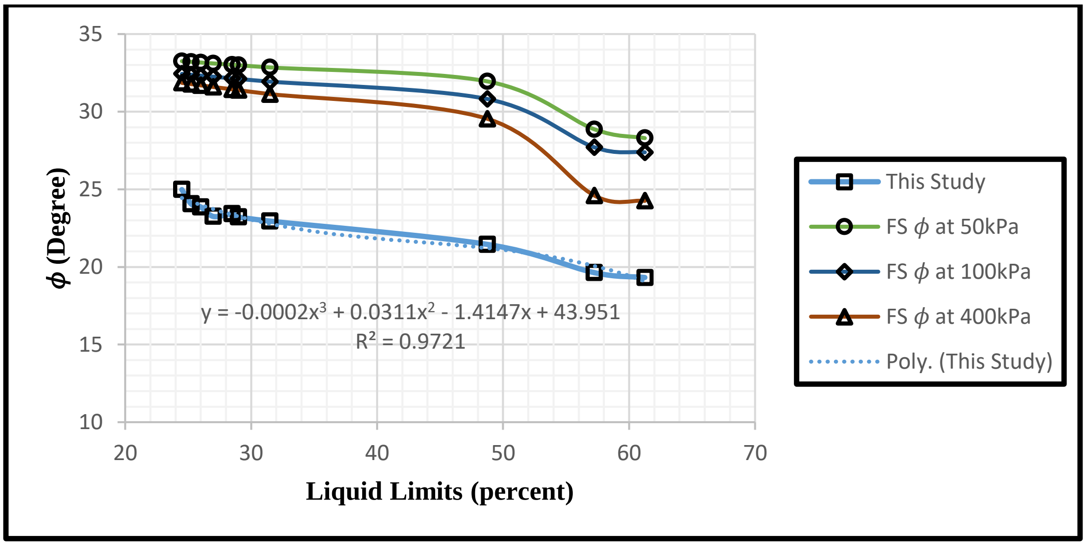

Figure 4.8: Comparison of reduced values of angle of internal friction with Stark and Hussain (2012) fully softened angle of internal friction

Based on the above conclusions, a correlation is proposed between the reduced angle of internal friction and liquid limit for clays having clay fraction less than 20 percent. The R^2 = 0.971, which is very close to 1 showing a strong correlation. The correlation is as under:

$$\phi_{Reduced} = 43.951 - 1.4147\,(LL) + 0.0311\,(LL)^2 - 2 \times 10^{-4}\,(LL)^3$$

4.4 PHASE-III: SELF HEALING

All the clays showed a gain in shear strength parameters when tested seven days after 6th wet and dry cycle. Some soil samples showed a tremendous increase in the cohesion and angle of internal friction and in some soil samples this increase was minor. To understand the gain in strength phenomenon, soil samples are divided in two types. First type consists of those soil samples in which the increase in the soil cohesion and angle of internal friction is lesser than the shear strength parameters at maximum dry density (0^{th} Cycle). In second group of soils, the increase in soil cohesion and angle of internal friction was higher than the shear strength parameters at 0^{th} cycle. Increase in shear strength parameters was due to thixotropic gain due to aging of the soil and shrinkage of the soil samples. Shrinkage took place due to loss of moisture. Densification of soil matrix and particle rearrangement are the governing parameters of gain in soil cohesion and angle of internal friction of soil.

4.4.1 Group-I Soils

Group 1 consisted of sample 1, 2, 3 and 5. XRD analysis of sample 1 and 3 showed the presence of muscovite, illite, gismondine and clinochlore, which have higher activity. These details of moisture loss and reduction in void ratio is shown with the help of the tables given below for each soil sample.

4.4.1.1 Gain in strength after 6^{th} wet and dry cycle in sample 1

At maximum dry density, the cohesion and angle of internal friction were 54.5 kPa and 34.58^{o} respectively. After 7 days of wet and dry cycles, gain was observed in cohesion and angle of internal friction from 1.646 kPa to 39.06 kPa and 19.62^{o} to 33.46^{o} respectively. Soil was highly active so shrinkage in the sample increased its density. Shear strength envelopes elaborating the increase in the shear strength parameters are shown in Figure 4.9.

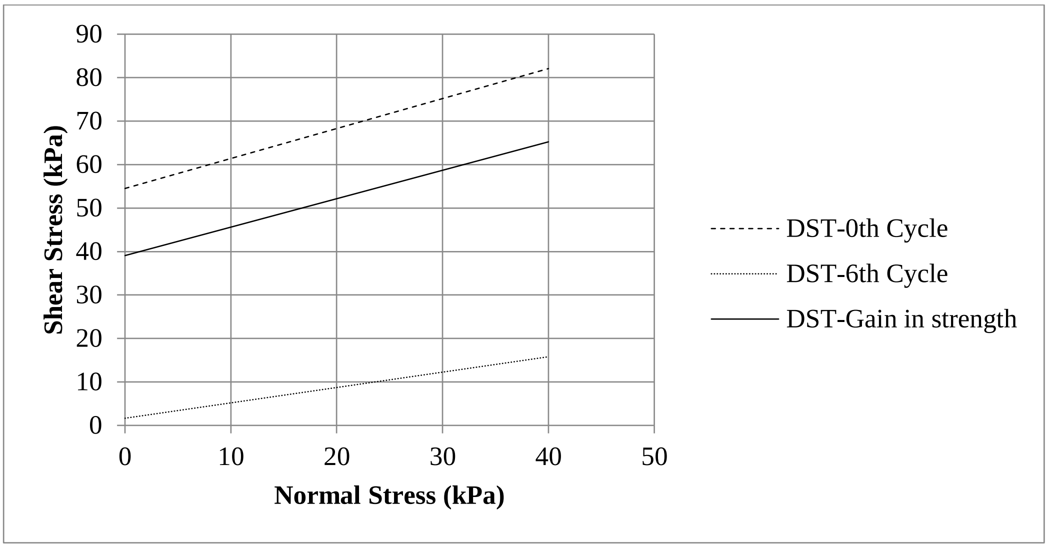

Figure 4.9: Gain in strength in sample 1

Variation in void ratios and shear strength parameters is also shown in Table 4.17 for sample 1. Void ratio of the sample decreased showing the soil densification.

Table 4.17: Gain in strength in sample 1

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	54.50	34.58	17	0.57
6^{th} Cycle	1.65	19.63	29.51	0.68
7 Days after 6^{th} Cycle	39.06	33.46	21.87	0.61

4.4.1.2 Gain in strength after 6^{th} wet and dry cycle in sample 2

At maximum dry density, the cohesion and angle of internal friction were 36.8 kPa and 33.46^o respectively. After 7 days of wet and dry cycles, gain in strength was observed in cohesion and angle of internal friction from 0.94 kPa to 22.74 kPa and 19.6^o to 31.26^o respectively. Soil was highly active so shrinkage in the sample increased its density. Shear strength envelopes elaborating the increase in the shear strength parameters are shown in Figure 4.10.

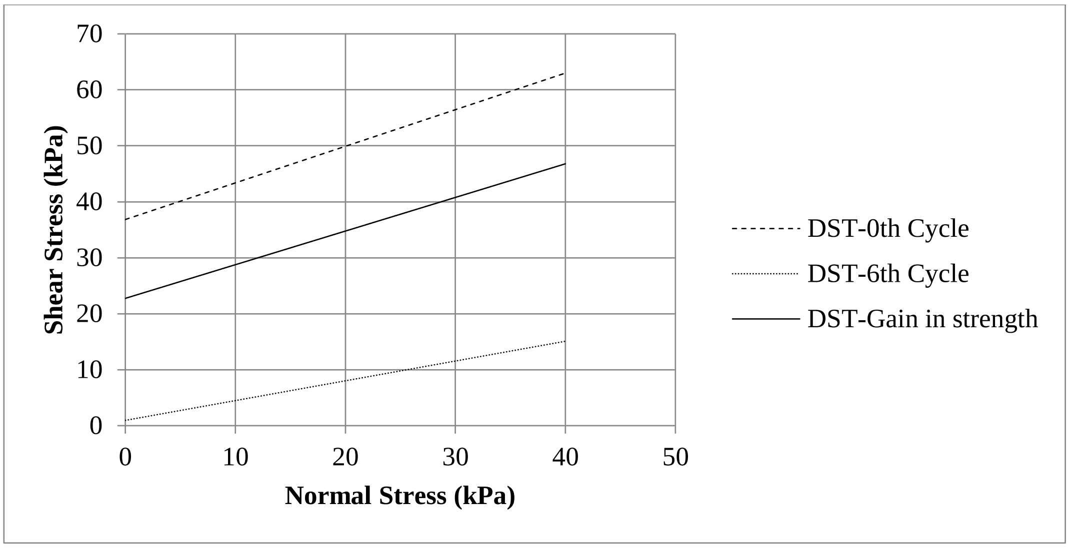

Figure 4.10: Gain in strength in sample 2

Variation in void ratios and shear strength parameters is also shown in Table 4.18 for sample 2. Void ratio of the sample decreased showing the soil densification.

Table 4.18: Gain in strength in sample 2

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	36.827	33.46	21	0.58
6^{th} Cycle	0.9428	19.63	34.11	0.77
7 Days after 6^{th} Cycle	22.74	31.26	29.67	0.70

4.4.1.3 Gain in strength after 6^{th} wet and dry cycle in sample 3

At maximum dry density, the cohesion and angle of internal friction were 32.87 kPa and 32.74^{o} respectively. After 7 days of wet and dry cycles, gain in strength was observed in cohesion and angle of internal friction from 2.00 kPa to

14.673 kPa and 19.62° to 30.55° respectively. Soil was highly active so shrinkage in the sample increased its density. Shear strength envelopes elaborating the increase in the shear strength parameters are shown in Figure 4.11.

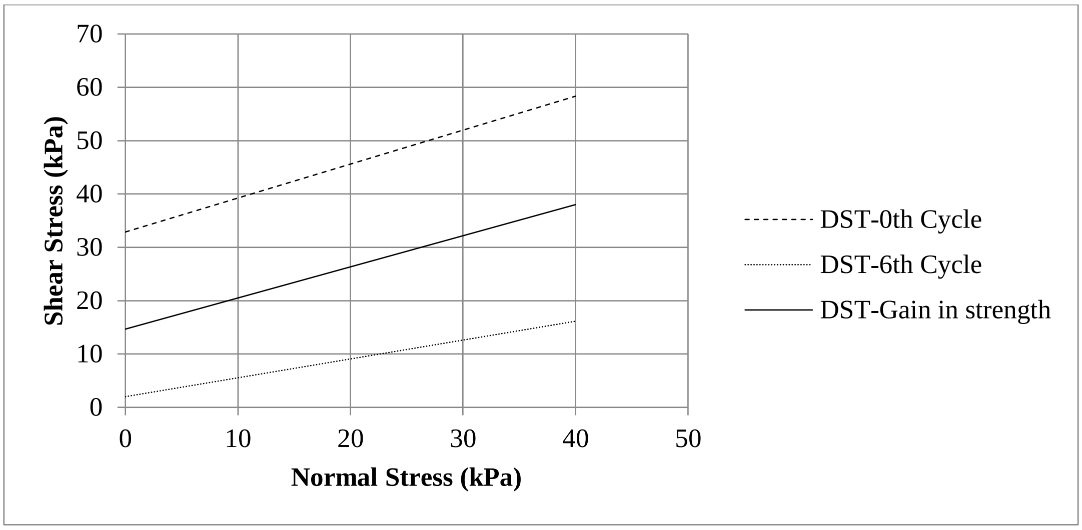

Figure 4.11: Gain in strength in sample 3

Variation in void ratios and shear strength parameters is also shown in Table 4.19 for sample 3. Void ratio of the sample decreased showing the soil densification.

Table 4.19: Gain in strength in sample 3

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0th Cycle	32.879	32.74	24	0.60
6th Cycle	2.0034	19.63	48.28	0.77
7 Days after 6th Cycle	14.673	30.51	39.99	0.69

4.4.1.4 Gain in strength after 6th wet and dry cycle in sample 5

At maximum dry density, the cohesion and angle of internal friction were 84.43 kPa and 25.72° respectively. After 7 days of wet and dry cycles, gain in strength was observed in cohesion and angle of internal friction from 4.71 kPa to 66.34 kPa and 21.42^{o} to 25.64^{o} respectively. Soil was slightly active so shrinkage in the sample increased its density. Shear strength envelopes are shown in Figure 4.12.

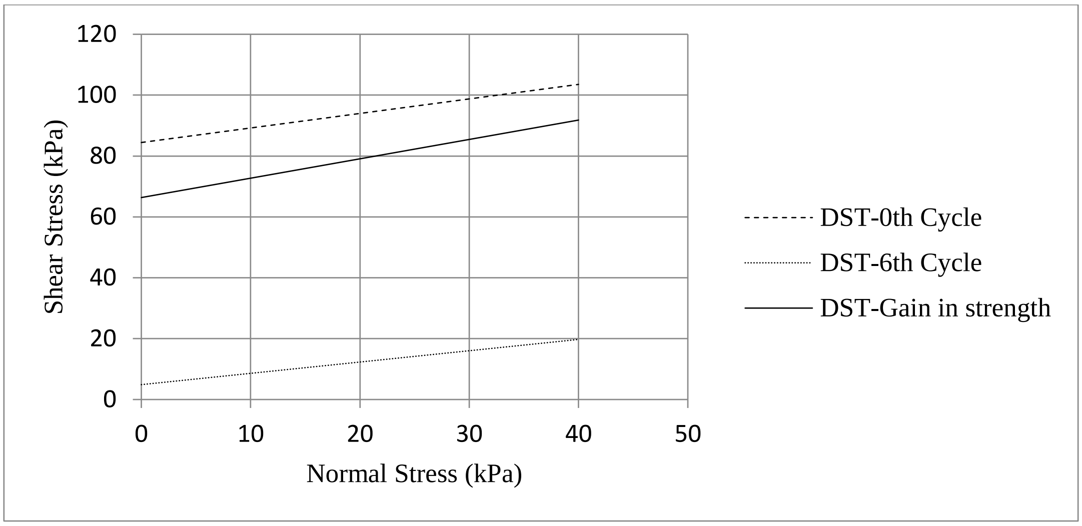

Figure 4.12: Gain in strength in sample 5

Variation in void ratios and shear strength parameters is also shown in Table 4.20 for sample 5. Void ratio of the sample decreased showing the soil densification.

Table 4.20: Gain in strength in sample 5

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	84.438	25.73	11	0.39
6^{th} Cycle	4.7139	21.43	18.77	0.50
7 Days after 6^{th} Cycle	66.34	25.64	12.35	0.46

Summary of the increase in soil shear strength parameters of group-I soils is shown in Table 4.21

Table 4.21: Summary of gain in strength in group-I soils

Soil Samples	Soil Type	C (kPa)			ϕ (Degrees)		
		At MDD	Reduced cohesion after 6th cycle	Gain in Cohesion after 7 days	At MDD	Reduced ϕ after 6th cycle	Gain in ϕ after 7 days
1	CL	54.5	1.649	39.06	34.59	19.46	33.46
2	CH	36.83	0.943	22.75	33.46	19.63	31.26
3	CH	32.88	2.00	14.67	32.74	19.63	30.51
5	CL	84.44	4.71	66.35	25.73	19.63	32.74

4.4.2 Group-II Soils

This group consisted of sample 4, 6, 7, 8, 9 and 10. XRD analysis sample 6 and sample 10 showed the presence of calcite and gismondine. Gismondine being very active caused the shrinkage of the soil sample upon loss of moisture. Also, calcite is calcium carbonate (limestone), which have very cementitious nature in the presence of water. The densification, thixotropic gain and particle cementation were resulted in high strength gain.

4.4.2.1 Gain in strength after 6th wet and dry cycle in sample 6

At maximum dry density, the cohesion and angle of internal friction were 57.98 kPa and 29.74^o respectively. After 7 days of wet and dry cycles, gain in strength was observed in cohesion and angle of internal friction from 1.82 kPa to 203.64 kPa and 20.53^o to 34.17^o respectively. Soil was highly active so shrinkage in the sample increased its density. Shear strength envelopes elaborating the increase in the shear strength parameters are shown in Figure 4.13.

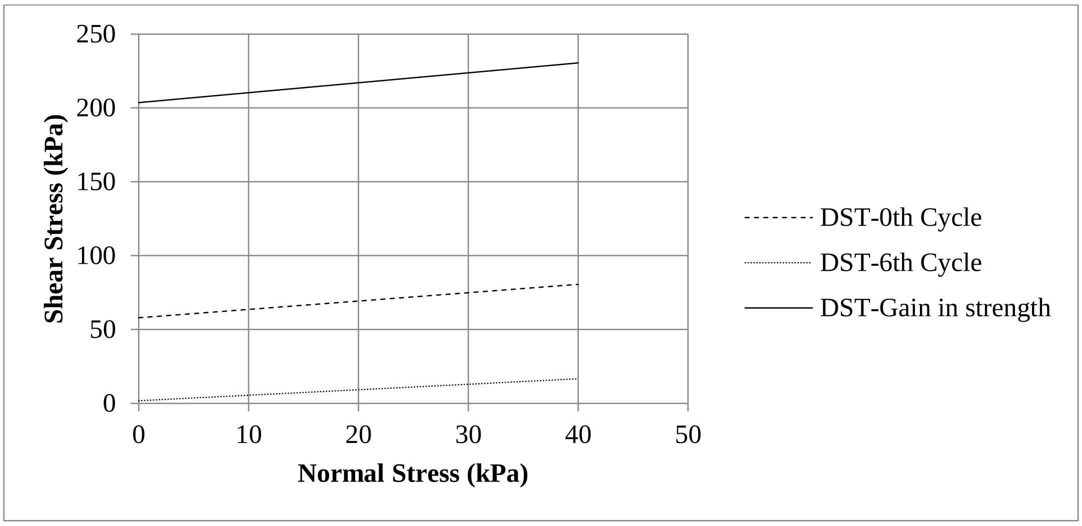

Figure 4.13: Gain in strength in sample 6

Variation in void ratios and shear strength parameters is also shown in Table 4.22 for sample 1. Void ratio of the sample decreased showing the soil densification.

Table 4.22: Gain in strength in sample 6

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	57.981	29.74	15	0.41
6^{th} Cycle	1.8266	20.53	20.76	0.49
7 Days after 6^{th} Cycle	203.64	34.17	11.33	0.44

4.4.2.2 Gain in strength after 6^{th} wet and dry cycle in sample 10

At maximum dry density, the cohesion and angle of internal friction were 54.63 kPa and 33.46^o respectively. After 7 days of wet and dry cycles, gain in strength was observed in cohesion and angle of internal friction from 3.06 kPa to 135.41 kPa and 21.42^o to 32.74^o respectively. Soil was highly active so shrinkage in the sample increased its density. Shear strength envelopes elaborating the increase in the shear strength parameters are shown in Figure 4.14.

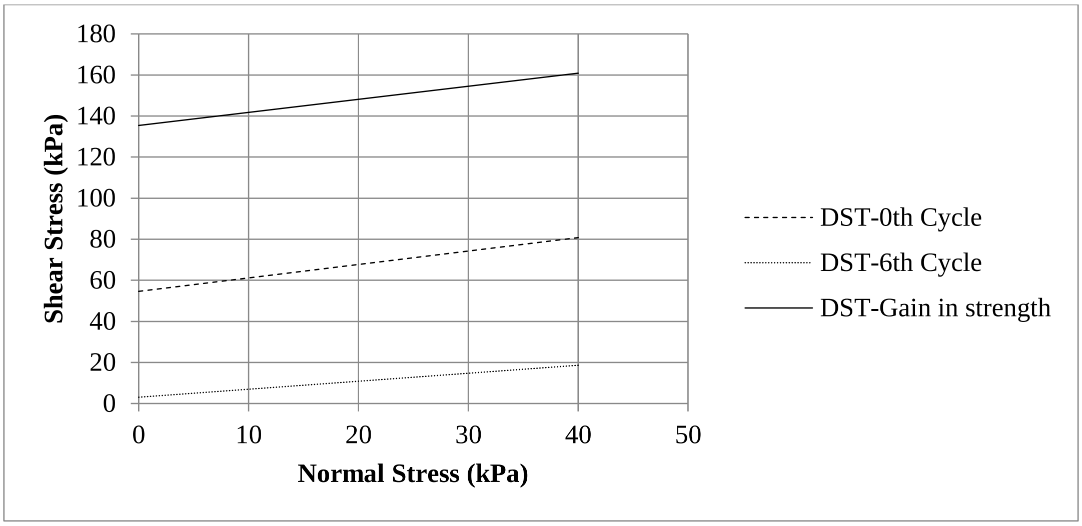

Figure 4.14: Gain in strength in sample 10

Variation in void ratios and shear strength parameters is also shown in Table 4.23 for sample 1. Void ratio of the sample decreased showing the soil densification.

Table 4.23: Gain in strength in sample 10

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	54.632	33.46	16	0.46
6^{th} Cycle	3.064	21.42	23	0.55
7 Days after 6^{th} Cycle	135.41	32.74	15	0.51

4.4.2.3 Gain in strength after 6^{th} wet and dry Cycle in sample 4

At maximum dry density, the cohesion and angle of internal friction were 70.11 kPa and 34.16^o respectively. After 7 days of wet and dry cycles, gain in strength was observed in cohesion and angle of internal friction from 3.83 kPa to

95.57 kPa and 20.53° to 33.46° respectively. Soil was slightly active so shrinkage in the sample increased its density. Shear strength envelopes are shown in Figure 4.15.

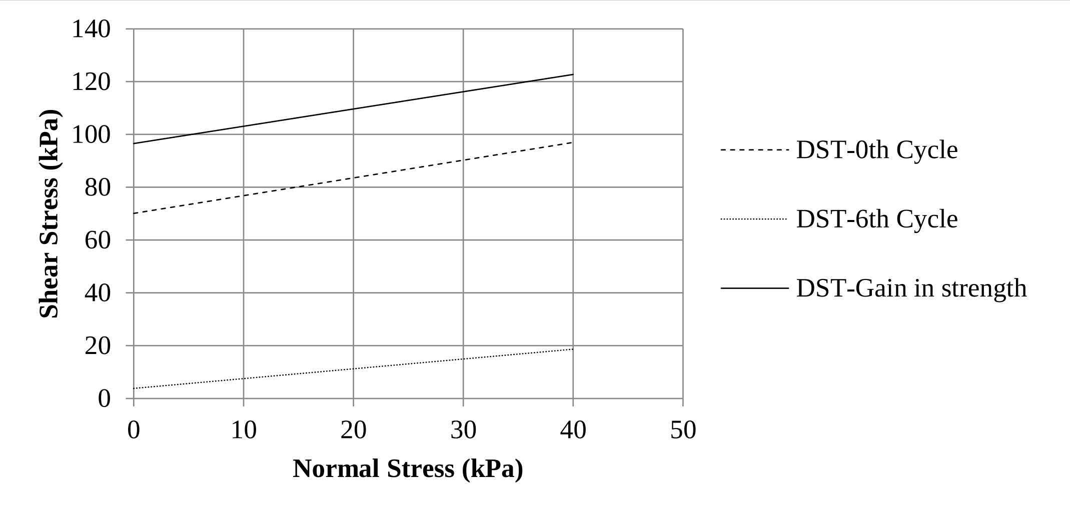

Figure 4.15: Gain in strength in sample 4

Variation in void ratios and shear strength parameters is also shown in Table 4.24 for sample 1. Void ratio of the sample decreased showing the soil densification.

Table 4.24: Gain in strength in sample 4

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	70.119	34.17	13.5	0.43
6^{th} Cycle	3.83	20.53	21.43	0.55
7 Days after 6^{th} Cycle	95.57	35.46	12.46	0.48

4.4.2.4 Gain in strength after 6^{th} wet and dry cycle in sample 9

At maximum dry density, the cohesion and angle of internal friction were 40.95 kPa and 33.46° respectively. After 7 days of wet and dry cycles, gain in strength was observed in cohesion and angle of internal friction from 18.914 kPa to

53.62 kPa and 27.36° to 32.74° respectively. Soil was slightly active so shrinkage in the sample increased its density. Shear strength envelopes are shown in Figure 4.16.

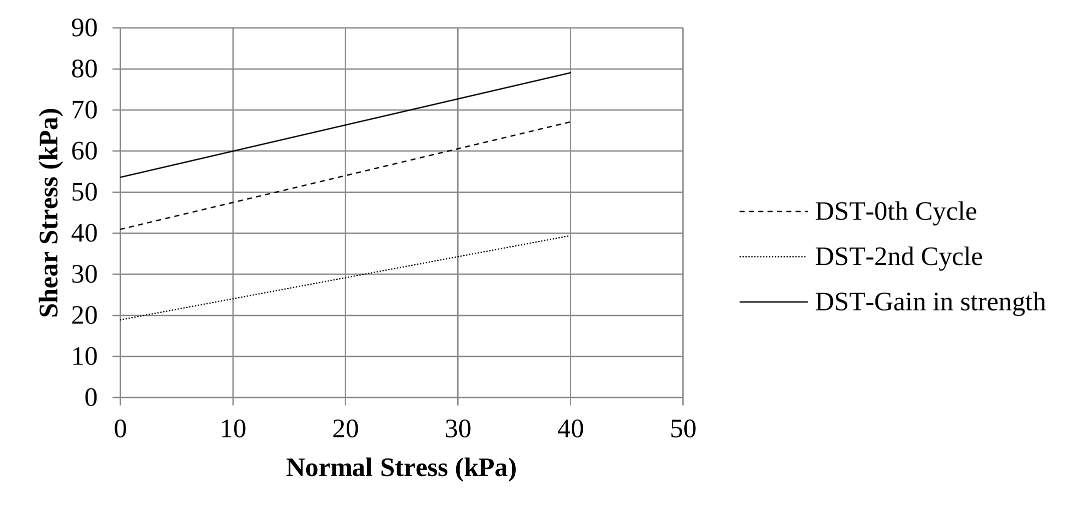

Figure 4.16: Gain in strength in sample 9

Variation in void ratios and shear strength parameters is also shown in Table 4.25 for sample 1. Void ratio of the sample decreased showing the soil densification.

Table 4.25: Gain in strength in sample 9

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	40.952	33.46	12	0.37
6^{th} Cycle	18.914	27.36	16.59	0.50
7 Days after 6^{th} Cycle	53.62	34.74	11.22	0.41

4.4.2.5 Gain in strength after 6^{th} wet and dry cycle in sample 7

At maximum dry density, the cohesion and angle of internal friction were 26.63 kPa and 28.16° respectively. After 7 days of wet and dry cycles, gain in strength was observed in cohesion and angle of internal friction from 16.08 kPa to

64.22 kPa and 20.53° to 35.55° respectively. Soil was highly active so shrinkage in the sample increased its density. Shear strength envelopes elaborating the increase in the shear strength parameters are shown in Figure 4.17.

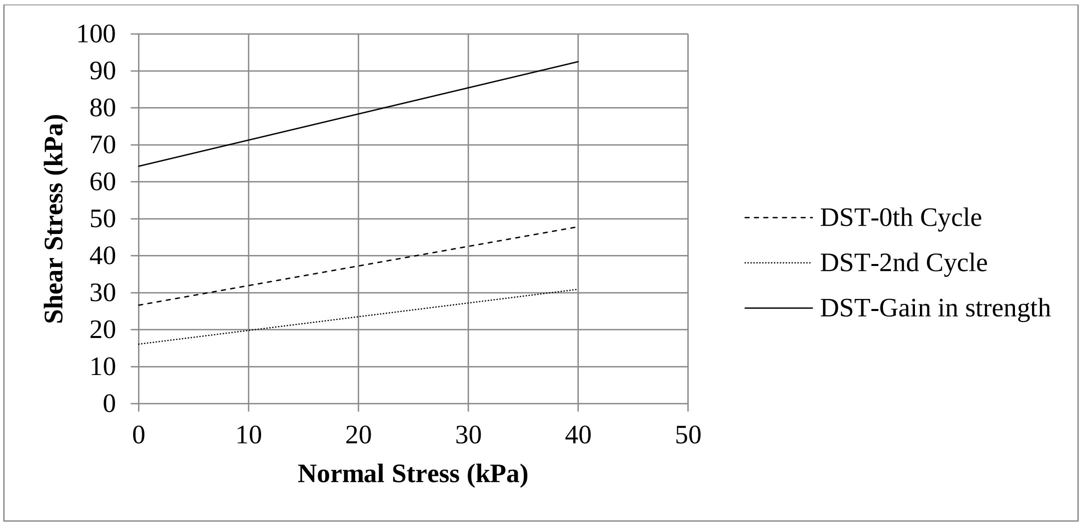

Figure 4.17: Gain in strength in sample 7

Variation in void ratios and shear strength parameters is also shown in Table 4.26 for sample 1. Void ratio of the sample decreased showing the soil densification.

Table 4.26: Gain in strength in sample 7

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	26.633	28.17	14	0.43
6^{th} Cycle	16.086	20.53	20.66	0.51
7 Days after 6^{th} Cycle	64.22	35.55	13.87	0.48

4.4.2.6 Gain in strength after 6^{th} wet and dry cycle in sample 8

At maximum dry density, the cohesion and angle of internal friction were 36.76 kPa and 26.55° respectively. After 7 days of wet and dry cycles, gain in strength was observed in cohesion and angle of internal friction from 4.30 kPa to 56.15 kPa and 25.72° to 33.46° respectively. Soil was highly active so shrinkage in the sample increased its density. Shear strength envelopes elaborating the increase in the shear strength parameters are shown in Figure 4.18.

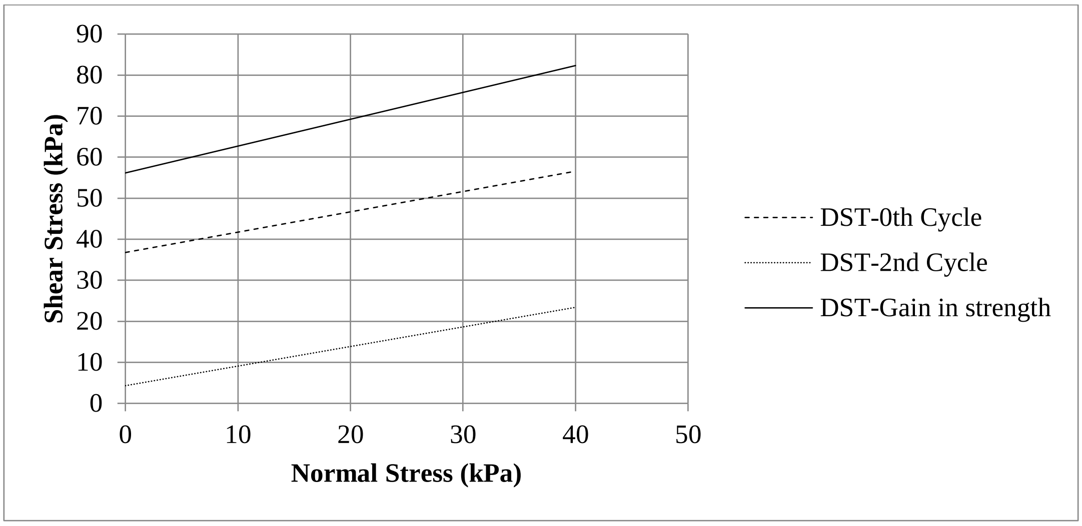

Figure 4.18: Gain in strength in sample 8

Variation in void ratios and shear strength parameters is also shown in Table 4.27 for sample 1. Void ratio of the sample decreased showing the soil densification.

Table 4.27: Gain in strength in sample 8

Wet and Dry Cycles	Cohesion (KPa)	Angle of Internal Friction (Degree)	Moisture Content (percent)	Void Ratio
0^{th} Cycle	36.768	26.55	15	0.44
6^{th} Cycle	4.3014	25.73	23.58	0.52
7 Days after 6^{th} Cycle	56.15	33.46	13.21	0.48

Summary of gain in shear strength parameters in group-II soils is shown in Table 4.28.

Table 4.28: Summary of gain in strength in group-II soils

Soil Samples	Soil Type	C (kPa)			ϕ (Degrees)		
		At MDD	Reduced cohesion after 6th cycle	Gain in cohesion after 7 days	At MDD	Reduced ϕ after 6th cycles	Gain in ϕ after 7 days
6	CL	57.98	1.822	203.6	29.74	20.53	34.17
7	CL	26.63	16.09	64.23	28.17	16.86	35.56
8	CL	36.77	4.301	56.15	26.55	25.73	33.46
10	CL	54.63	3.064	135.4	33.46	21.43	32.74
4	CL	70.12	3.83	95.58	34.17	20.53	33.46
9	CL	40.95	18.91	53.62	33.46	23.18	32.74

Chapter 5

CONCLUSION AND RECOMMENDATIONS

5.1 CONCLUSION

Wet and dry cycles reduced the shear strength parameters of the low plastic and high plastic clays. Following are the conclusions made from study.

- Percent decrease in the cohesion of the clays was as high as 97 percent for both low plastic and high plastic clays proving the cohesion of the clays as non-reliable parameter. Therefore, cohesion of the soil must be considered as zero for long term design of any civil engineering structure.
- The cohesion of the high plastic clays showed a continuous decrease with increasing wet and dry cycles. But in case of low plastic clays, the cohesion of some clays showed an anomaly in the trends. The cohesion of the clays decreased after 2^{nd} wet and dry cycle then increased after 4^{th} wet and dry cycle. This decrease and increase in the trends were due to the soil disturbance. Also, the cohesion of the clays not only depends on the inter-connection between soil particles (due to water molecules) but also on the compaction and interlocking of the soil particles (Shahangian, 2011).
- Angle of internal friction was decreased up to 31 percent for both low plastic and high plastic clays after wet and dry cycles and lesser for some samples. The reduced values for angle of internal friction were compared with fully softened angle of internal friction calculated from the updated correlations of Stark and Hussain (2012). It is concluded that the correlations are not valid for shallow slope stability analysis of slopes. Therefore, a correlation is presented to determine the reduced angle of internal friction from liquid limit.

- The void ratio of the clay samples increased after wet and dry cycles for both low plastic and high plastic clays. Wet and dry cycles increased the effective porosity of the soil mass after each cycle (Khan, 2016b). Some variation in void ratios was observed for low plastic clays showing the weathering of soil clay particles by wet and dry cycles which decreased the void ratio.
- Both low plastic and high plastic clay samples showed a gain in strength when tested seven days after 6^{th} wet and dry cycle. This gain was very high for some clays and less for the others. This was due to shrinkage of the clay samples and thixotropic gain in soils with aging. Also, XRD analysis showed the presence of muscovite, illite and gismondine which resulted in high activity. Also, the presence of calcite in sample 6 and 10 caused the particle cementation. Therefore, clay minerology is proven very prominent factors for gain in strength in clay samples.

5.2 RECOMMENDATIONS

Impact of wet-dry cycle was observed for ten different liquid limits. Soils with different liquid limit may be used for further analysis to observe the impact. Based on this research, recommendations for the future studies are summarized below:

- The plasticity of the clays specially bentonite continuously decreases with increasing the wet and dry cycles when tap water was used for wetting (Lin and Benson, 2000). The presence of salts in water may change not only the soil composition but also the soil minerology. So distilled water may be used for wetting.

- Different chemical stabilization methods are in in practice including the use of cements, lime, gypsum etc. The behavior of the treated soils towards wet and dry cycles must be check before their application.

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