EVALUATION OF HYDRO-MECHANICAL BEHAVIOR OF COLLAPSIBLE SOIL

By

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THIS THESIS IS

DEDICATED

TO

MY BELOVED PARENTS

&

MY WIFE & MY SON

(For their unending love, support, and inspiration)

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ABSTRACT

Collapsible soils are generally found in arid and semi-arid regions of the world, which undergo large deformation upon loading and wetting, showing significant decrease in void ratio. These soils are quite spread in Pakistan, in addition to other regions of the world. Research shows that cracks are generally developed in the structure, built on such types of soils during or after certain period of construction work which ultimately results in the structure failure, leading to human and financial losses. These changes in the performance of these soils are associated with changes in their hydraulic and mechanical properties due to moisture content variations over time. So, in this study, the hydraulic and mechanical properties of untreated and treated collapsible soils were investigated for various moisture contents of the compaction curve. The grain size distribution, Atterberg limits, standard Proctor, direct shear, oedometer and hydraulic conductivity were performed in this regard. The additives, such as ground granulated blast furnace slag (GGBS) and bagasse ash (BA) were used to improve the engineering behavior of collapsible soil in this study. GGBS and BA are the by-products of iron and sugarcane. The test results show that the Kallar Kahar soil falls in the category of low plastic clay (CL) and the collapsible potential of treated soil is about 50-60%less than untreated soil. The dry unit weight decreases and optimum moisture content increases up to a certain threshold of BA and GGBS, showing that there is no efficacy to use additives beyond a certain threshold. The cohesion and angle of internal friction are relatively higher for dry of optimum than wet of optimum side. Furthermore, the treated soil shows lower hydraulic conductivity than untreated soil due to their lower void ratios and higher consolidation potential.

CHAPTER 01

INTRODUCTION

1.1. Background

The construction of infrastructure projects is taking place quite fast throughout the world due to rapid increase in population. Engineers and policy makers are making their best to innovate approaches in the construction filed to build safe structures on economic grounds. The recent trend is to build high rise buildings to accommodate more population on a lesser space. The load of the super structure is carried by the foundation resting on underneath soil. Generally, the sandy soils are a good option to use as foundation materials in any circumstances. In certain situation, the higher capacity soils are not available at the project site, and there is no alternative rather than to use low bearing capacity soil (clayey soil). The structural and textural configuration of soil varies from place to place due to differences in geological factors, such as soil composition, mineralogy, weathering and climate change etc.

Collapsible soils are considered one of the low bearing capacity soils, found in arid and semi-arid regions of the world. These soils undergo large deformation upon loading, wetting or by both, showing significant decrease in void ratio. These soils are also quite spread in Pakistan, in addition to other regions of the world. According to(Haq, 1984), the cracks were developed in canal lining in the reach RD246+765 within couple of months after the completion of right bank Chashma canal, and the investigations revealed that the cracks were developed in the strata due to saturation of collapsible soils. These soils are also found in Kallar Kahar region - Motorway (M-2), Chakwal, Pakistan. Figure 1.1 shows the structure of the collapsible soil. According to(Howayek et al., 1994), the collapsible soils undergo considerable changes in volume upon partial or full saturation due to their open structure, geological young, low inter-particle strength, and high sensitivity potential.

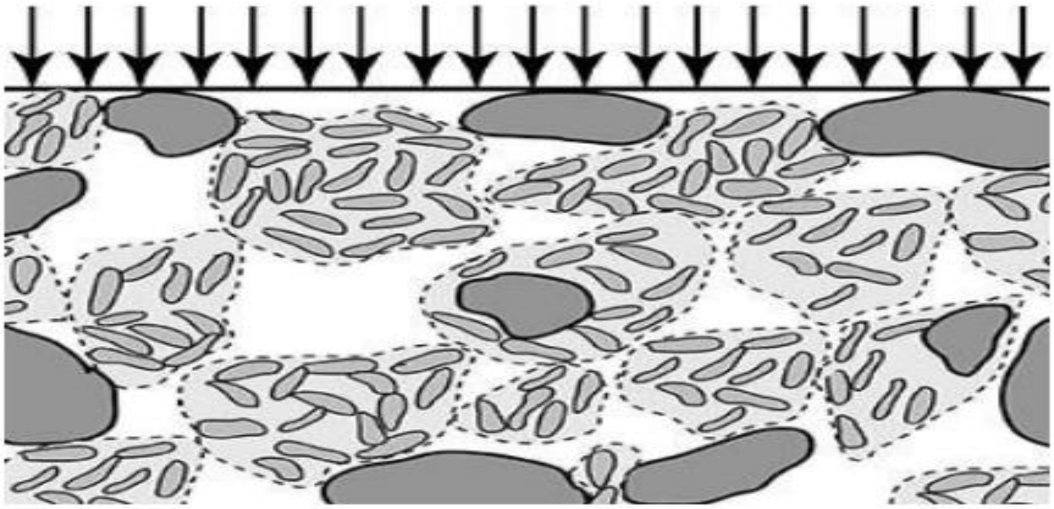

Figure 1. 1: Structure of the collapsible soil (Anwar & Sarbast, 2019)

Collapsible soils are relatively stiff and show higher strength in natural dry conditions but lose strength and undergo significant volume change upon wetting. In natural dry conditions, different types of inter-particle bonds exist in collapsible soils, which act as a cementing agent to hold the soil grains together in order to keep the soil in stable conditions. These bonds break immediately upon wetting, causing substantial reduction in strength due to volume change (Clemence and Finbarr 1981). These soils also contain appreciable percentage of air in their voids. There are several cases of foundation failures due to excessive settlements of collapsible soils upon wetting (Ibrahim, 2016). Laboratory and field tests are available to estimate the collapsible potential of collapsible soils, quantitatively. With the reinforcement, engineering properties of these soil can be improved to encounter structural requirements.

Several studies are available in the literature reporting various approaches to improve the engineering properties of collapsible soils. (Fattah et al., 2014) used acrylate liquid with various percentages of gypsum contents to stabilize the collapsible soil and concluded that the treated samples showed 60–70% less compressibility than untreated soil. (Alawaji, 2001) reinforced the collapsible soil with geo-grid reinforcement and showed that the use of geo-grids increased the load carrying capacity reducing wetting induced collapse potential. AlShaba et al., (2018) treated the collapsible soil with iron powder for various percentages and reported that the collapsible potential reduced from 12.1% to 8.7%, 9.3% to 6.9%, and 6.7% to 5.5%. The study further reported that the collapsible potential was greatly associated with the relative density. (Ali et al., 2015) improved the bearing capacity of collapsible soil with partial replacement of sand. (Ziani et al., 2019) used granulated slag and natural pozzolan to reduce the collapse potential of collapsible soils.

The ground granulated blast furnace slag and bagasse ash were used in this study to reinforce the soil. The ground granulated blast furnace slag (GGBS) and bagasse ash (BA) are natural fiber residues, obtained from the refining processing of steel and sugarcane mills, respectively. Both binders contain silica, which shows significant potential to lessen the swelling potential of expansive soils, enhancing the stability of soils with their pozzolanic activities. The bagasse is the remains of fibrous waste after extraction of sugar juice from cane. This fibrous waste is used as a fire fuel to heat up the boilers and the remains of this fire fuel is Sugarcane Bagasse Ash (SCBA). The SCBA is a pozzolanic material because of high amount of silica and alumina (Payá et al., 2002). Ground granulated blast furnace slag (GGBS) is a by-product in the production of steel and pig iron. According to ACI -116R, GGBS can be defined as “nonmetallic product consisting essentially of calcium silicates and other bases that is developed in a molten condition simultaneously with iron in a blast furnace”. GGBS mainly consists of silicates such as, aluminum silicates and calcium alumina silicates, and its composition includes Cao (40 %)

silica (35%), and aluminum oxide (12%) (Sinalkar et al., 2020). The soil used in this study was obtained from Kallar Kahar, Punjab, well known in the region as a problematic soil. The cracks are generally developed in the structures, built in this region, after certain period of construction work. According to (Tung & Fragkiadaki, 2017), these cracks are associated with changes in the behavior of collapsible soils over time. Bagasse ash used in this study was obtained from Sheikho sugar mill Kot Addu Muzaffargarh and GGBS from Agha steel mill Karachi, Pakistan. The uniqueness of this research work involves examining the hydro-mechanical behavior of untreated and treated collapsible soil for different moisture contents of the compaction curve. Different data points were selected for optimal, dry of optimum and wet of optimum sides of the compaction curve. To attain the set objectives, index properties, direct shear, hydraulic conductivity and oedometer tests were performed in the laboratory. The test results show that the liquid limit and plastic limit decrease with an increase in the percentage of additives, which is due to the changes in the behavior of soil from clayey to silty soil. The soil falls in the category of low plastic clay (CL) and A-4 group. The collapse potential of soil reduces from 50-60% using additives. The dry unit weight decreases and optimum moisture content increases up to a certain threshold of additives, showing that there is no efficacy to use additives beyond a certain threshold. The cohesion and angle of internal friction are comparatively higher for dry of optimum side than wet of optimum side. The treated soils show lower hydraulic conductivity than untreated soil due to their lower void ratio and higher consolidation potential. The compression index and swelling index profiles show lower peaks for treated soils than untreated soil, which show their higher consolidation potential.

1.2. Problem statement:

Collapsible soils are metastable soils which can withstand high pressure in their natural dry condition but undergo significant volume changes when subjected to wetting or additional loading. The water seeps into the soil beneath the foundation, the bond between the soil grains breaks and the soil loses strength, which alternately results in the collapse of structure. The collapsible soil is a big threat to civil infrastructures, constructed in arid and semi-arid regions, as cracks initiates to propagate after certain period of construction in these structures.

1.3. Justification of the study

The study is quite useful in context of Pakistan as most of the structures get cracked during or after the construction work in Kallar Kahar region, Pakistan. The soil in the region apparently seems hard, but the buildings constructed on this soil gets cracked after certain period of time, even though they are only up to two storey, and the reality of this fact is still not understood. So, in this study, the authors analyze the hydro-mechanical behavior of fiber treated collapsible soil, following various laboratory testing approaches. For the research work, the soil is collected from Kallar Kahar, Pakistan.

1.4. Objectives of the study

- To investigate the hydraulic and mechanical behavior of fiber reinforced collapsible soils.

1.5. Thesis outline

The chapters in this thesis are outlined as;

1. Chapter 1 highlights the research background, problem statement, scope of the project, objectives, and the justification of the study.
2. Chapter 2 represents a detailed literature review, relating to the research work.
3. Chapter 3 elaborates the materials used and the methods adopted to conduct the research study.
4. Chapter 4 reports the results and discussions, drawn from the research work.
5. Chapter 5 represents the conclusions with few key recommendations, drawn from the study.

CHAPTER 02

LITERATURE REVIEW

2.1. General

Collapsible soils, also known as sensitive soils can bear very high pressure without showing significant changes in volume in dry conditions, however upon wetting, they undergo large and sudden reduction in volume changes, which alternately results in the collapse of the structure (Gaaver, 2012). There is a sudden change in weight and size of the soil with an increase in moisture contents for saturation ratio is > 50% (Anwar & Sarbast, 2019). Figure 2. 1 highlights the collapse of a house, constructed on collapsible soil without proper preliminary investigations(Houston et al., 2001)

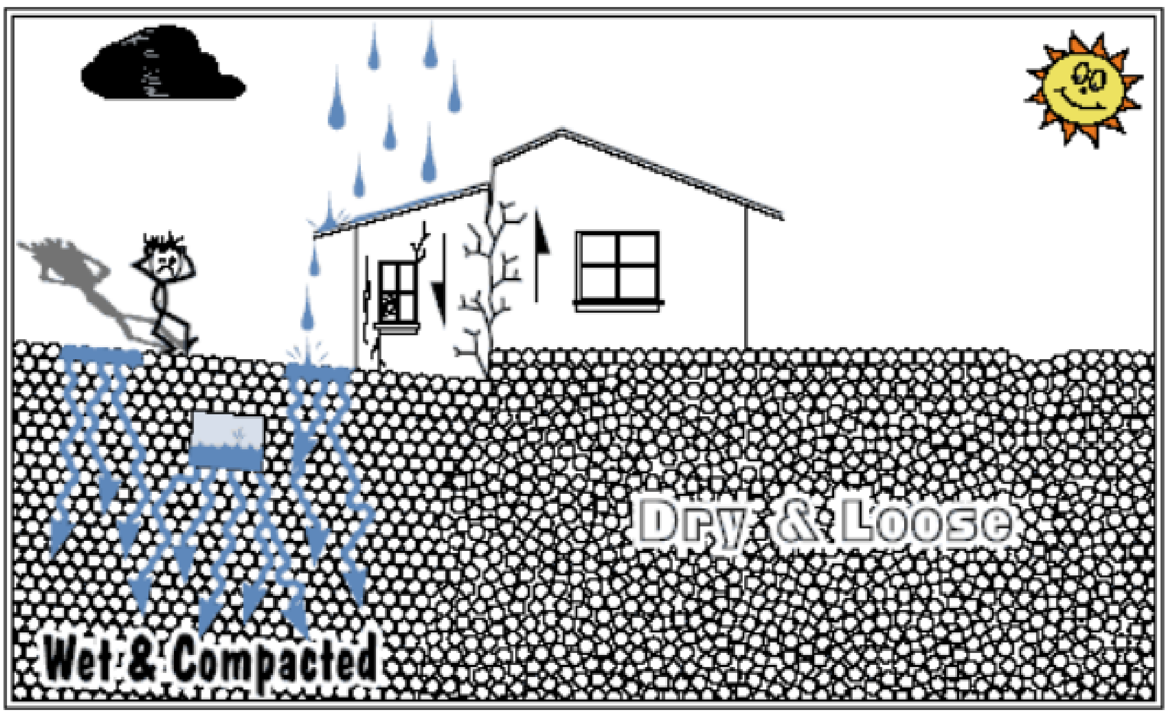

Figure 2. 1: House constructed on collapsible soil without early investigation (Houston et al., 2001)

2.2. Soil Characterization

2.2.1. Grain size distribution

Grain size distribution provides significant information about engineering behavior of soil. The Czechslovak standard as in Klukanova and Frankovaska(1994) highlights six parameters to classify the collapse potential of soil. According to (Howayek et al., 1980), a soil is highly collapsible with silt contents > 60%, clay contents < 15%, liquid limit < 32% and porosity > 40%. Handy (1973) concluded that the soil is susceptibility to; higher collapse potential for clay contents between 16 and 24%, moderate for clay contents between 24 and 32%, and usually safe for clay contents > 32%.

2.2.2. Atterberg limits

Atterberg limits are basic parameters to define the engineering behavior of soil. Various criteria are available in the literature for the measurement of collapse potential of fine grained soils on the basis of Atterberg limits. Denisov (1951) used coefficient of subsidence to classify the collapsible potential which is ratio of in situ void ratio to void ratio at liquid limit. The study reported that the soils are highly collapsible for subsidence ratio between 0.5 and 0.75, and generally not expected to collapse for ratio > 1.5. Priklonski's (1952) related liquidity index with collapsible potential and showed that the soils for LI >0 provided higher collapsible potential due to their more susceptibility to water infiltration. However, for liquidity index > 0.5, the soils are less likely to collapse.

2.2.3 Shrinkage limit

Shrinkage limit of collapsible soils is another important property to define their engineering behavior as these soils are volume sensitive and show permanent distortion in volume due to wetting rather than swelling as in expansive soil (Kakoli & Hanna, 2011). The volume of these soils reduces with an increase in moisture content under internal stresses, and even without any external stress (Iranpour & haddad, 2016). The main characteristics of the collapsible soil are associated with their porous and metastable structure, due to which the strain induced within the soil matrix with an increase in water content results in collapse of the soil.(Almahbobi et al., 2018)examined the volume change and shear strength behavior of collapsible soil, composed of 40% silt 40% sand and 20% kaolin and reported that the soil showed similar behavior to other soils, such as Aeolian soil deposits, known as loess by performing saturated and unsaturated tri-axial tests. The result indicated that the reduction in total volume was observed due to decrease in matric suction and volume change become more significant at lower suction values. The shrinkage limit was found to be 10.7%.

2.2.4. Compaction

Standard and modified Proctor tests are generally used to determine the moisture density relationships. At a given initial water content, the collapse potential decreases with an increase in compaction energy (Lahmadi et al., 2012).. Figure 2.2 shows variations in collapse potential for increase in compaction energy.

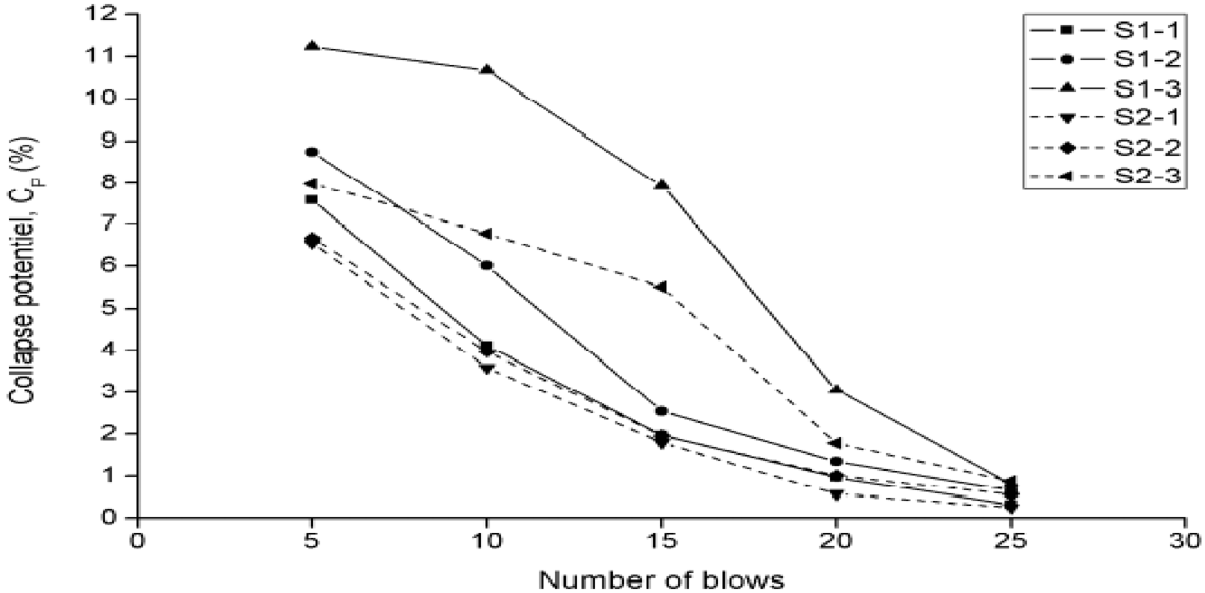

Figure 2. 2: Collapse potential variation versus energy of compaction (Lahmadi et al., 2012)

For a specified compaction energy, the collapse potential decreases with an increase in water content. (Lahmadi et al., 2012)reported that a soil shows higher collapsible potential with lower initial water content and in loose condition Figure 2.3 shows collapse potential variation with water content for specified compaction energy.

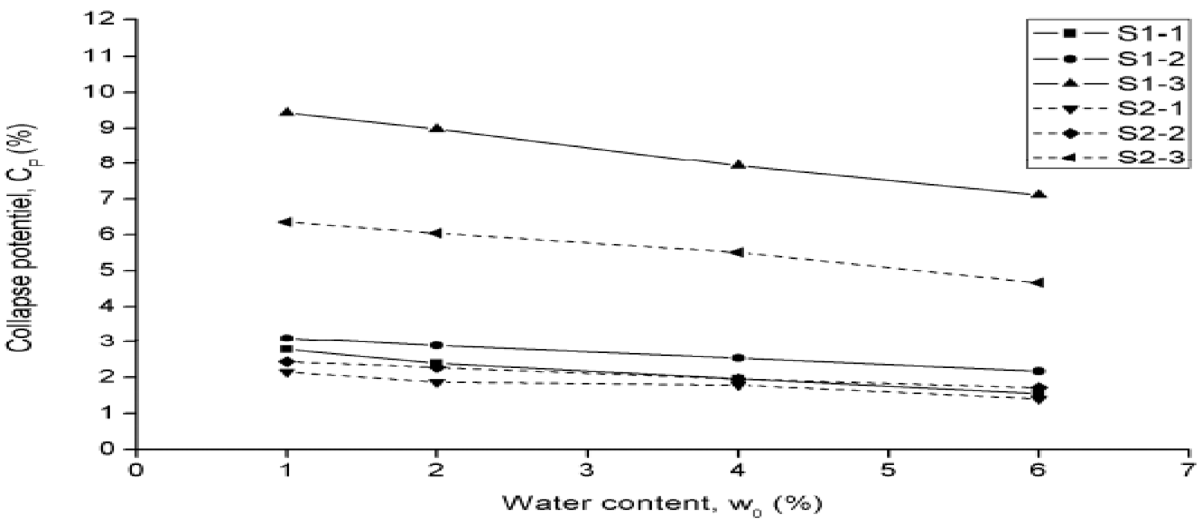

Figure 2. 3: Collapse potential vs water content for compaction energy at 15 blows).(Lahmadi et al., 2012)

Furthermore, as in Figure 2.4, the dry density decreases with a decrease in compaction energy, irrespective of soil type and water content (Lahmadi et al., 2012). Figure 2.5 shows the relationship between initial water content and dry density which depicts that dry density increases with an increase in water content. The increase in dry density with respect to moisture content is only up to certain limit and after that limit the dry density start decreasing.

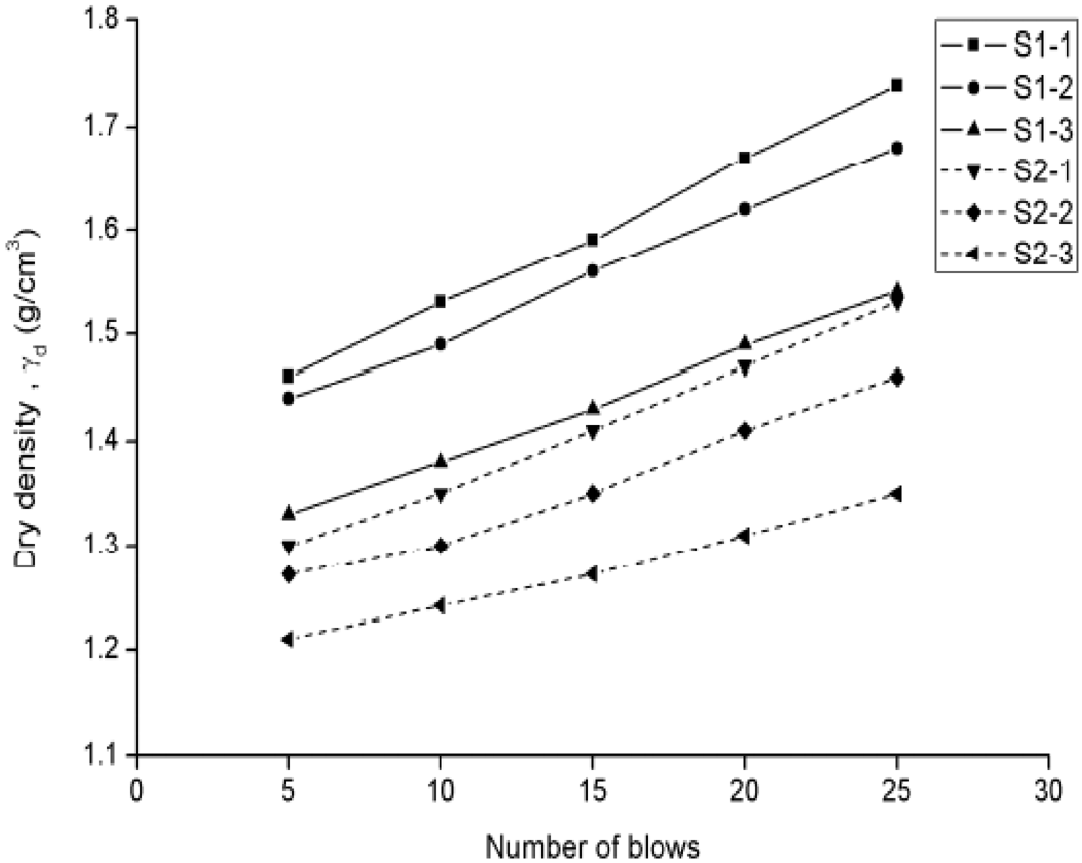

Figure 2. 4: Dry density variations versus compaction energy (w_0 = 4%).(Lahmadi et al., 2012)

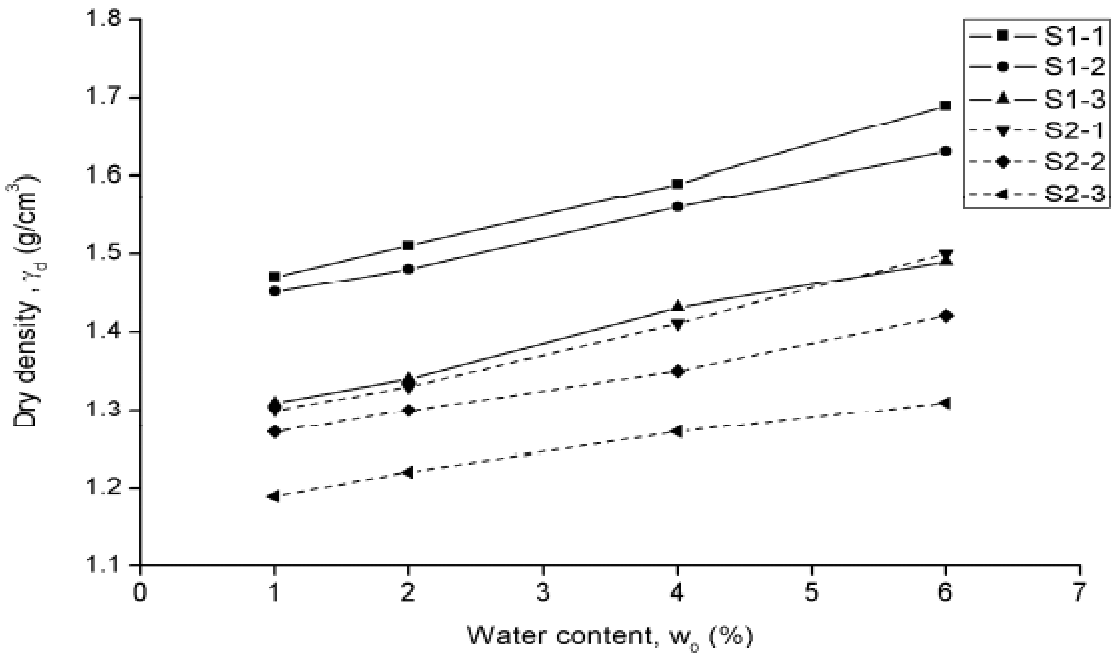

Figure 2. 5: Dry density versus water content (Energy of compaction = 15 blows).

2.2.5. Specific Gravity

Specific gravity is another important as it is used to estimate other important soil properties such as porosity, void ratio, compressibility and others of soils. It classifies the heaviness of a material with respect to water. The specific gravity of collapsible soil generally varies from 2.65 to 2.70, and it is determined following the guidelines as in ASTM D5550-14 standard.Yu et al., (2021)investigated the specific gravity of stabilized clay for different drying methods (Figure 2.6) and concluded that specific gravity decreased with an increase in curing time due to hydration. Figure 2.6 shows specific gravity of a particular soil, determined with different drying methods.

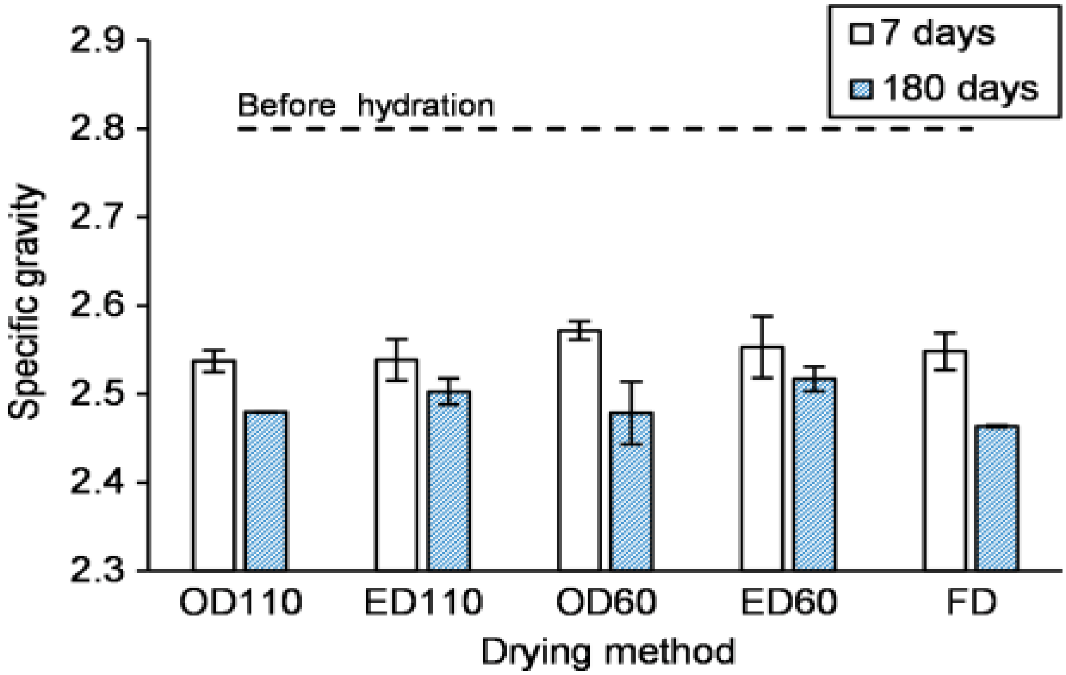

Figure 2. 6: Specific gravity of stabilized clay for different drying methods (Yu et al., 2021).

2.3. Collapsible soil behavior

Collapsible soils undergo a sudden volume change (reduction) upon wetting under a constant stress. A wide range of soils, such as alluvial flood plains, residual soils, mud flows, colluvial deposits and wind-blown deposits are generally categorized as collapsible soils. (Sultan, 1969) stated that a collapsible soil undergoes a considerable amount of volume changes upon wetting, load application, or with a combination of both. According to (Jennings and Knight 1975), "additional settlement due to the wetting of a partially saturated soil, normally without any increase in applied pressure occurs in collapsible soil". Figure 2.7 shows classification of different types of collapsible soils.

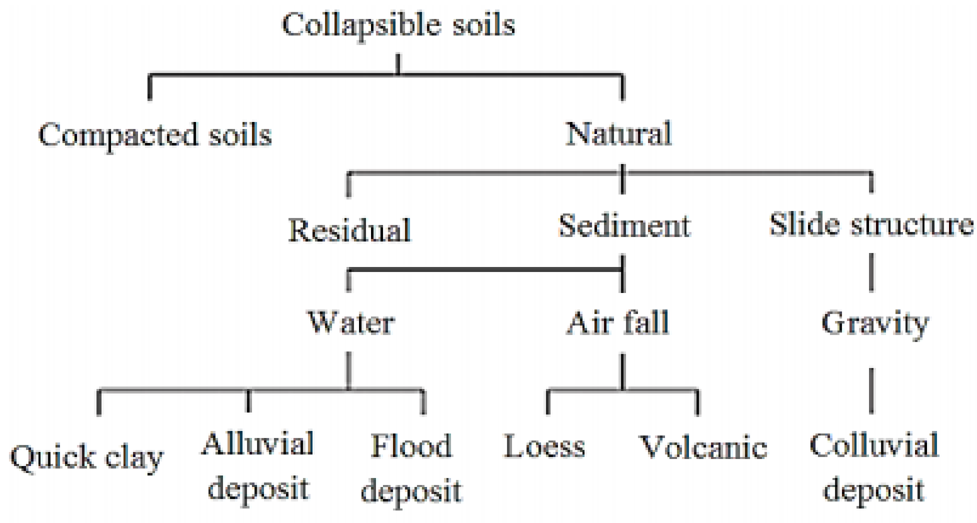

Figure 2. 7: Classification of different types of collapsible soils (Rogers, 1995).

According to (Clemence and Finbarr 1981), different types of inter-particle bonds exist in collapsible soils in natural dry conditions, which act as a cementing agent and hold the soil grains together, keeping the soil in stable conditions (Figure 2.8).These bonds break immediately upon wetting, which alternately results in significant loss of strength and volume.

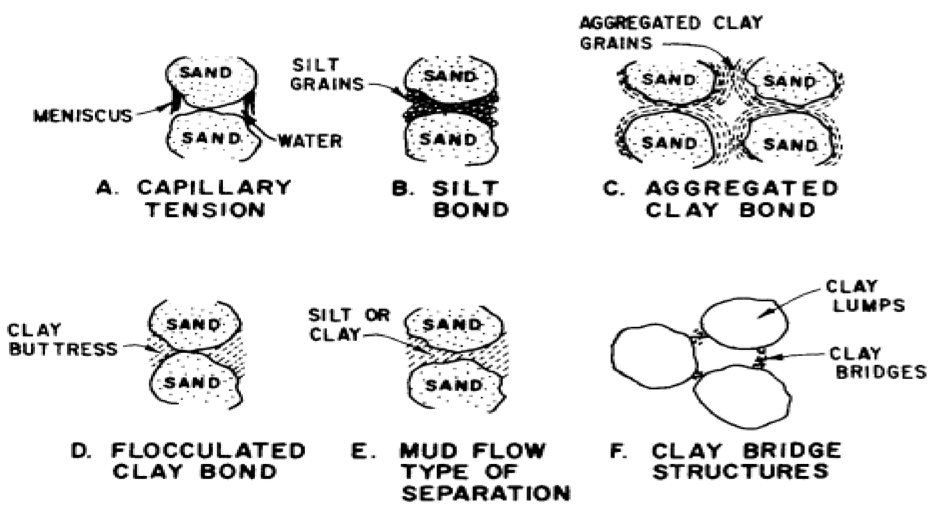

Figure 2. 8: Collapsible soil mechanisms (Clemence and Finbarr 1981).

2.3.1. Identification and classification of collapsible soils

Knight (1963) introduced a factor, known as collapse potential to classify the severity of soil collapse and suggested to perform oedometer for direct evaluation of collapse potential. The process involves placing a soil sample in a consolidometer ring at its natural moisture content and a stress up to 200 kPa is applied, gradually. The sample is left under load for 24 hours in saturated conditions. The e-logP curve of oedometer test results was plotted. The collapse potential is the ratio of change in heights of consolidated specimen to the initial specimen, and it is expressed in percentage.

$$Cp = \frac{\Delta e_o}{1+e_o} = \frac{\Delta Hc}{\mathrm{Ho}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{Equation 2.1}$$

In Equation 2.1, e_o is void ratio of soil at the natural state, Δe_o= change in void ratio upon wetting, ΔHc is the change in height of soil specimen upon wetting and Ho is the initial height of soil specimen. Figure 2.9 show a typical plot for the determination of collapse potential (Clemence and Finbarr et al. 1981).

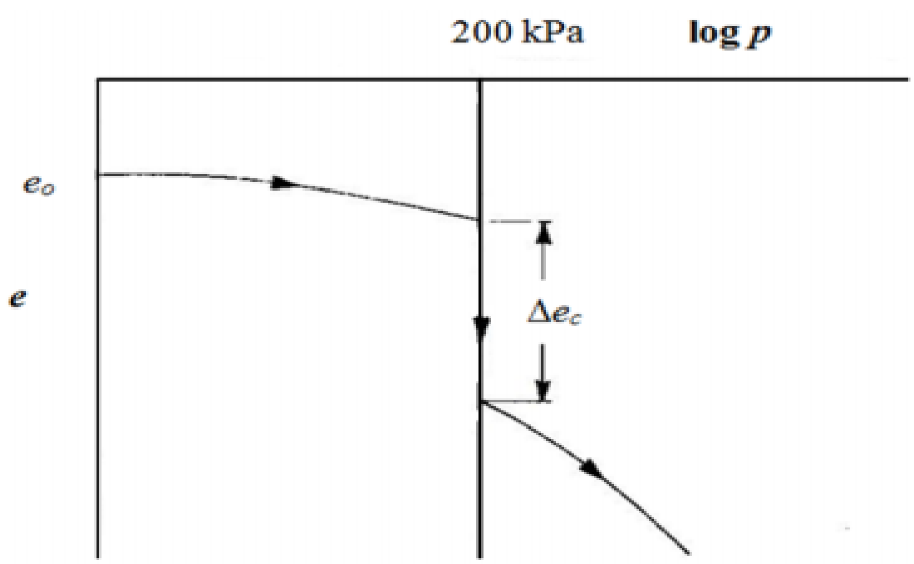

Figure 2. 910: Typical collapse potential results

Furthermore, Jennings and Knight (1975) reported a criteria to estimate the severity of the collapse potential (Cp) of a soil on the basis of oedometer test data (Table 2.1).

Table 2. 1: Severity of problems with respect to collapse potential

C_p (%)	Severity of problem
0-1	No problem
1-5	Moderate trouble
5-10	Trouble
10-20	Severe trouble
>20	Very severe trouble

2.3.2. Factors affecting the behavior of collapsible soil

Experimental and field studies show that several parameters, such as compaction effort, soil type, stress-level, percent clay, and initial water content influence the collapse potential of a soil(Likos, 2010). Percent fines also affect the collapse potential as the clay particles acts as a cementing agent in a clay-sand admixture. Ho et al. (1988) concluded from experimental test results that the collapse potential increased with an increase in the percentage of fines. The magnitude of the collapse potential is proportional to the clay contents until it reaches at a certain threshold, and beyond this point, the soil swells rather than collapse due to further increase in clay contents (Adnan and Edril 1992). El-Ehwany and Houston (1990) examined the collapse potential of partially wetted sandy silt from the experimental work and concluded that partial wetting resulted in partial collapse, however a substantial collapse occurred for degree of saturation between 60 and 75 percent (Figure 2.10).According to (Houston et al., 2002), the lateral migration of water in the subgrade, constructed on collapsible soil resulted in differential settlement and pavement distress

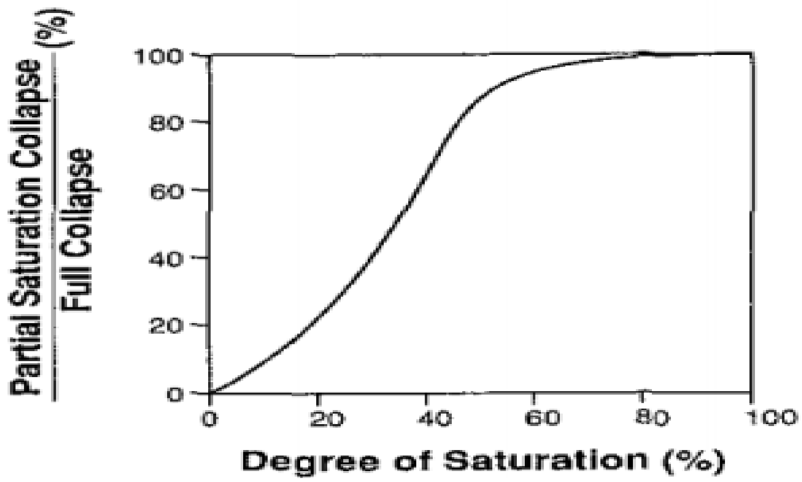

Figure 2. 110: Partial collapse due to partial wetting (El-Ehwany and Houston 1990)

2.3.3. Mitigation measures for foundations on collapsible soils

Mitigation measures may broadly be defined as approaches or designs that lessen or solve the collapsing problem (Houston et al., 2001). Several methods are available in literature which minimizes the effect of collapse, stabilizing the collapsible soils. (Fattah et al., 2014) used acrylate liquid with various gypsum contents to reduce the collapse potential. The study showed that the acrylate liquid reduced the compressibility of the collapsible soil more than 60–70%. This was attributed to the acrylate liquid film coating on the gypsum particles. The treated soil also showed higher shear strength than untreated soil. (Gaaver, et al., 2012) densified the collapsible soil of Western desert, Borg El-Arab region, Cairo region with compaction to examine the bearing capacity in relation to collapse potential. The undisturbed block samples were collected from four different sites and tested under both soaked and unsoaked conditions. The test results showed that the bearing capacity decreased up to 50% due to soaking process. The study suggested to use 2 x the factor of safety to take into account the soaking effect in collapsible soils.

Alawaji, (2001) used geo-grid reinforcement for in-situ stabilization of collapsible soils, and reported that the geo-grids reinforcement resulted in significant decrease in excessive settlement of weak and soft soils. Reinforcement mechanism was basically due to the geogrid-sand interaction. When the sand layers were compacted with geo-grid, they partially penetrated to each other, creating an interlocking between sand and geo-grid. These interlock enabled the grid to resist the horizontal shear stresses and increased the bearing capacity of the soft subsoil. Similarly, mechanical interlock produced a flexural stiff platform which distributed the vertical pressure evenly and alternately reduced the wetting induced collapse settlement and differential settlement as well.

AlShaba et al., (2018)reinforced the collapsible soil with different percentages of iron powder (4%, 5%, 6%, 8% and 10). Oedometer tests were conducted to examine the consolidation behavior of treated and untreated soils. Iron powder is a by-product of iron processing. Initially, the test was conducted to estimate the collapse potential (C.P) of collapsible soil with single oedometer tests and then the collapse potential was measured, wetting the soil with rainfall water. The test results showed that that the collapse potential reduced from severe category to less severe category for changes in concentration of iron powder from 12.1% to 8.7%. It was further added that the relative density (Dr) also influenced the collapse potential.

Ali et al.,(2015) reinforced the collapsible soil integrating it with sand layers of different thickness, and reported that the bearing capacity increased up to 40% with partial replacement of collapsible soil with sand which was due to the fact that replaced sand provided more uniform stress distribution within the soil matrix than compacted collapsible soil. Resultantly, the partially replaced collapsible soil with sand layers reduced the footing settlement and increased bearing capacity by a factor of 1.8.(Tiwari & Sharma, 2013) stabilized the dune sand with lime and cement, and reported that the California Bearing Ratio (CBR) increased from 7% to 20%, 28%, 40%, and 80% with an addition of 2%, 4%, 6% and 8% of hydrated lime. Ziani et al., (2019) used different %ages of granulated slag and natural pozzolan to reduce the collapse potential of collapsible soil and concluded that collapse potential was significantly less for treated than untreated soil.

2.4. Shear strength

The cohesion and angle of internal friction are important shear strength parameters, considered in the design of foundations, slope stability and retaining walls, etc. The collapsible soils lose strength upon wetting due to loss of apparent cementation. The addition of water reduced the shear strength and increased the compressibility of the soil (Houston et al., 2001). Sarsam et al., (2016) determined the shear strength parameters of asphalt stabilized collapsible soil and concluded that soaking process decreased the shear strength by about 25 and 33%for untreated and asphalt stabilized soil, respectively. The angle of internal friction showed an increase of 3.7 and 8.3 degrees in soaked and dry conditions, respectively for treated soil.

Khalid et al., (2015) showed that the angle of internal friction of collapsible soil increased from 7% to 17% with an addition of 2 to 4% of the nano-soil. Garakani et al., (2018) stabilized the collapsible soil with three different types of salts and showed that the treated soil provided more strength and stiffness than untreated soil. The study also reported that beyond a certain threshold of treatment, the shear strength initiated to decrease. Ayeldeen et al., (2017) used two biopolymers to increase the bearing capacity of collapsible soil and concluded that the treated soil provided higher shearing resistance than untreated soil. However, the cohesion increased and friction angle slightly reduced with an increase in the treatment concentration.

2.5. Consolidations

Consolidation test is performed to determine the collapse potential, compression index, swelling index, initial and final void ratio. (Bakir et al., 2017) performed oedometer consolidation tests on soil sample containing 80% sand and 20 % kaolin. It was concluded that the soil collapsibility reduced with an increase in moisture content. Garakani et al., (2019)performed oedometer tests to examine the consolidation behavior of 15 different collapsible soils, treated with 1 and 3% of lime and formulated the constitutive models and stress – strain relationships on the basis of test data.

Bakir et al., (2017) performed oedometer consolidation tests on soil samples containing 80% sand and 20 % kaolin treated with gypsum. It was concluded that the soil collapsibility reduced with an increase in moisture content. The study also showed that the compressibility significantly reduced with an increase in the compaction efforts. Furthermore, due to soaking effects, the compression index and swelling index increased from 0.011 to 0.221 and from 0.0016 to 0.0054, respectively.(Fattah et al., 2012) treated the collapsible soil with dynamic compaction using three weights and different number of blows and concluded that the compression index increased for unsoaked conditions and decreased for soaked conditions. Similarly, the swelling index increased and decreased with an increase in number of blows for unsoaked and soaked conditions, respectively, up to 20 blows. Furthermore, less change in Cc and Cs were noted after 20 blows for both conditions.

2.6. Hydraulic conductivity

Hydraulic conductivity of soil is considered one of the key parameters in the sustainable design of specific geo-structures and water resources management. (Sadeghi & AliPanahi, 2020)tested the hydraulic conductivity of collapsible soils, performing field and laboratory tests under different conditions and reported that the horizontal conductivity dominated the vertical conductivity.

Ayeldeen et al., (2017) estimated the hydraulic conductivity of both untreated and treated collapsible soils. The soil was treated using two bio polymers. The test data showed that the treated samples showed lower hydraulic conductivity, impeding the water flowing through the specimen Mohamed & El Gamal, (2012) investigated the hydraulic conductivity of sulphur cement treated collapsible soils and reported that the hydraulic conductivity of treated soil varied between1.46×10^{-13} and 7.66×10^{-11} m/s. Furthermore (Houston et al., 2002) showed that the collapsible soil generally provided higher hydraulic conductivity than the parent base materials at similar water contents.

CHAPTER 03

MATERIALS AND METHOD USED

3.1. Introduction

This Chapter describes the materials and methods used to attain set objectives of the research work

3.2. Materials

The soil was collected from Kallar Kahar, Chakwal Pakistan, which is well known in the region for its problematic behavior. The additives, such as bagasse ash and ground granulated blast furnace slag was obtained from Sheikho Sugar mills, Muzaffargarh and Agha steel mills, Multan Pakistan, respectively.

3.3. Soil characterization

3.3.1. Grain size distribution

The mechanical analysis of soil was carried out performing sieve analysis and hydrometer tests. For these tests, the sample was initially oven dried for 24 hours and then pulverized. For sieve analysis, the soil was pulverized until no soil grain was adhered to each other. Hydrometer test was performed on soil passing through sieve # 200, following the procedures as discussed in ASTM D 422-63 standard. For these tests, the dispersing agent, sodium hexametaphosphate was used for dispersion of soil grains. The test data of sieve analysis and hydrometer test were then combined for grain size distribution profile. Figure 3.1 shows the scheme of hydrometer test in the laboratory.

Figure 3. 1: Apparatus for hydrometer analysis

3.3.2. Atterberg limits

Liquid limit and plastic limit were performed following the guidelines as discussed in ASTM D423-54T and ASTM D424-54T standards, respectively. These tests were performed on oven-dried samples passing through sieve # 40. For shrinkage limit, the soil sample was prepared at its liquid limit. The shrinkage dish was slightly coated with petroleum jelly to prevent sticking of the soil particles. The sample was spread in a standard dish in three layers. For each layer, the dish was taped from the base until the soil layer gets smooth and free from air bubbles. The specimen was then air-dried for 6 hours and then placed in an oven at 105 to 110° C for 12 hours. The oven dried sample was replaced with the same mass of mercury in the dish. The oven dried specimen and the mass of mercury were then weighed to measure the shrinkage limit. Figure 3.2 shows the test arrangement of shrinkage limit test. Finally, Equation 3.1was used to measure the shrinkage limit of specimens.

$$S.L = \frac{(Vi - Vf)\rho w}{ws} x100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{Equation 3.1}$$

Figure 3. 2: Shrinkage limit test arrangement in the laboratory

3.3.6. Specific gravity tests

Specific gravity test of natural soil was performed following the guidelines as in ASTM D 854-14 standard. Figure 3.3 shows the specific gravity test progression in the laboratory.

Figure 3. 3: Specific gravity test progress in the laboratory

3.3.3. Moisture-density relationship

Maximum - density relationships were obtained following the guidelines as in ASTM D698 - 12 standard. For preparation of samples, the water was added to the soil in an increment of 2%, and mixed thoroughly to ensure soil water uniformity. The prepared sample was then transferred to the compaction mold, and the test was completed following the guidelines as in the standard. All data points for compaction curve were obtained in this fashion for untreated soil. Similarly, the compaction tests of treated soils for several concentrations of BA and GGBS were performed following the guidelines as set out in ASTM D 698 –12 standard. For these tests, 1 %, 2% and 3% of GGBS were trialed with various concentrations of BA, such as from 4 % to 12%. Duplicate samples were prepared to ensure the repeatability of the test data. With an increase in the %age of additives, the dry unit weight decreased, and water content increased until they bounced back towards higher dry unit weight and lower water contents, and which was the change point for this soil. Finally, the moisture – density relationships were drawn to do a comparative evaluation of all these curves. Figure 3.4 shows a few pictorial views of the test scheme. For treated soil, the best compaction curve was selected for further analysis.

Figure 3. 4: Few compaction test arrangements in the laboratory

3.3.4. Direct shear test:

The consolidated undrained direct shear tests were performed following the guidelines as discussed in ASTM D3080-11 standard to determine the shear strength parameters of natural and treated soils for both saturated and unsaturated conditions. The tests were carried out for different data points, for wet of optimum, dry of optimum and optimal point of the compaction curve for natural and treated soils, and the samples were directly prepared in the direct shear box of size, 6.032 x 6.032x 2.58 cm^3. The samples were sheared at a shear rate of 0.029mm/m under normal loads of 50,100 and 200 kPa. The oedometer test was performed following the procedure as discussed in ASTM D 2435-11 standard to estimate the shearing rate of the soil specimens. For saturated conditions, the sample remained soaked in the shearing box, during the consolidation and shearing processes. Finally, the stress – strain and shear strength parameters relationships were plotted from the test data for untreated and treated soils. Figure 3.5 shows the scheme of direct shear tests in the laboratory.

Figure 3. 5: Direct shear test arrangements in the laboratory

3.3.5. Hydraulic conductivity tests

Falling head test method was followed to determine the hydraulic conductivity of the natural and treated soils. The tests were performed following the guidelines as in ASTM D2434-68 standard. A permeator with 6 cm dia., and 4 cm depth was used in these tests (Figure 3.5). The samples were prepared at similar points of the compaction curve as in direct shear tests. The water soil mixture was transferred to the permeator with the help of a spatula, and compacted with a small tamping rod for desired dry unit weight. The number of blows of tamping rod was adjusted with trial and error method, prior starting the test with original samples. Figure 3.6 shows the arrangement of falling head test in the laboratory to determine the hydraulic conductivity of fine grained soil. Equation 3.2 was used to determine the hydraulic conductivity of the soil. In Equation 3.2, A and a = x-sectional areas of specimen and stand pipe respectively, L = length of the specimen, t = elapsed time during test, h_1 and h_2 are heads at the beginning and end of the test. Similarly, the tests were performed for other data points of the compaction curve for both natural and treated soils to develop relationships between k and dry unit weights / optimum moisture contents

$$k = 2.3 \frac{aL}{At} log \frac{h1}{h2} \text{ ……………………………………Equation 3.2}$$

Figure 3. 6: Falling head method for determination of hydraulic conductivity of fine grained soil

3.3.7. Collapse potential test

Collapse potential of the soil was determined following the criteria as discussed by Jennings and Knight (1975). The prepared soil sample was placed in a consolidometer ring and a stress up to 200 kPa was applied on the specimen, gradually. The sample was left under this stress for 24 hours in saturated conditions. Equation 3.3 was used to determine the collapse potential of the soil after 24 hours.

$$C.P = \Delta e/1+e_o \quad \text{..Equation 3.3}$$

Similarly, the collapse potential for best compaction curve of treated soil was determined. For both untreated and treated soils, the samples were prepared at different data points of the compaction curve as in direct shear and hydraulic conductivity tests. Finally, the collapse potential was compared for both untreated and treated soil for various water contents.

3.3.8. Consolidation test

The consolidation tests of both treated and natural soils were performed, following the guidelines as set out in ASTM D 2435/2435M-11 standards. The specimens were prepared at different data points of the compaction curve in the consolidometer ring of 6 cm diameter and 2 cm depth in the study. The load was applied in a load incremental ratio (LIR) of 2, i.e., 25, 50, 100, 200 and 400 kPa, and the specimen were consolidated until it reached its maximum consolidation capacity. Similarly, for unloading conditions, the specimen was unloaded for each loading after it reached its maximum swelling potential. A seating load of 1 kPa is applied on each sample, initially. The consolidation tests for both treated and untreated soils were performed under the soaked condition, in which the consolidometer ring was filled with water during the test. After unloading, the sample was subjected to re loading condition, and the load was now applied in a LIR of 2, i.e., 25, 50, 100, 200, 400, 800, 1600 kPa. For each load, the reading was taken at different time intervals, until the specimen reached to its maximum consolidation capacity. Figure 3.7 shows the progression of consolidation test in the laboratory.

Figure 3. 7: Consolidation test in saturated condition

From the oedometer consolidation tests, the initial void ratio, final void ratio, pre-consolidation pressure, compression index and re-compression index parameters were determined for untreated and treated soils. Figure 3.8 shows other pictorial views of consolidation test in the laboratory.

Figure 3. 8: Consolidation test progression in unsaturated condition

CHAPTER 04

RESULTS AND DISCUSSION

4.1. General

This chapter reports the results and discussion on the basis of test data drawn from the research work.

4.2. Soil characterization

4.2.1 Grain size distribution

The grain size distribution profile of Kallar Kahar soil is shown in Figure 4.1, which indicates that the soil contains 75.34% silt, 11.66% sand and 13% clay content, and it falls in the category of collapsible soil as per Czechslovak standard. According to this standard, a soil with silt contents >60%, clay content < 15% and liquid limit <32% shows significant tendency for collapse.

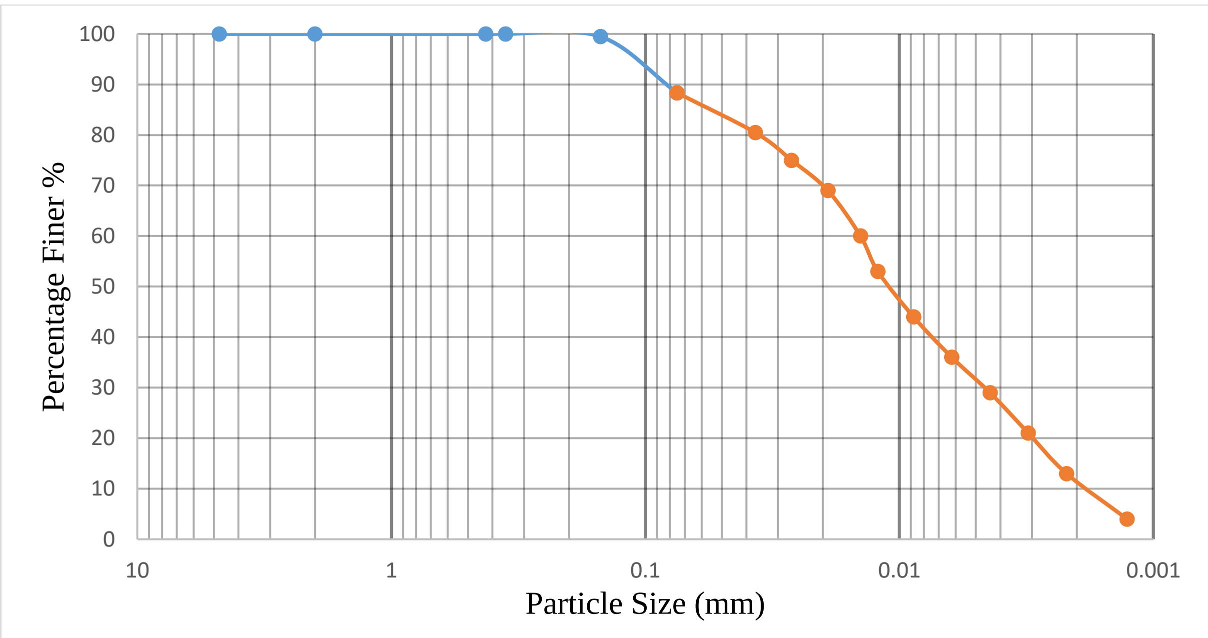

Figure 4. 1: Particle size distribution curves of Kallar Kahar soils

4.2.2. Atterberg limits

The soil shows liquid and plastic limits of 28.67 and 21 respectively with the plasticity index of 7.55. Figure 4.2 shows the plot between moisture content and no. of blows for liquid limit determination. The shrinkage limit of the soil is estimated as 11.54%.

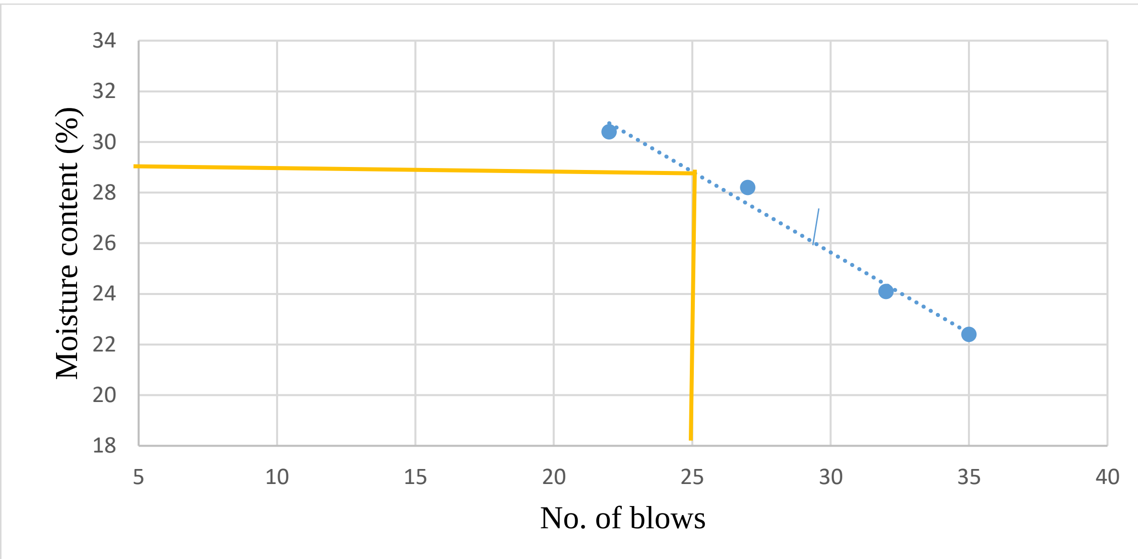

Figure 4. 2: Liquid limit test results of the Kallar Kahar soil

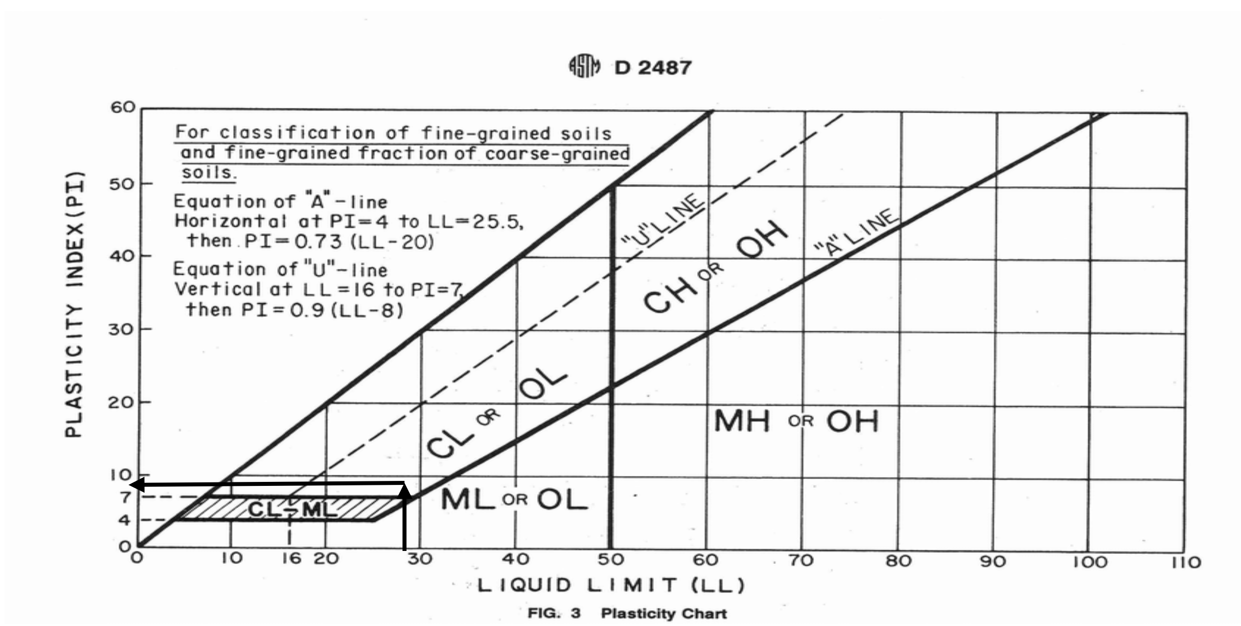

Figure 4. 3: USCS classification - Plasticity chart

4.3. Specific gravity

The specific gravity of soil comes out to be 2.67

4.4. Moisture – density relationships

Figure 4.4 shows the moisture density relationship of untreated soil which shows that the soil provides maximum dry unit weight of 18.74 kN/m^3 and optimum moisture content of 14.8%. Figures 4.5 and 4.6 depict the test results for treated soils. It can be seen that with an increase in the %age of additives, the peaks of the compaction curves are shifting towards right, showing a gradual increase and decrease in the optimum moisture content and maximum dry unit weight, respectively. However, this trend continues up to certain concentrations of additives, such as 4% GGBS and 8% BA. After this concentration, the compaction curve bounces back, showing an increase in the dry unit weight and decrease in the optimum moisture content. This is the change point at which it is presumed that this is the best compaction curve with 4% GGBS and 8% BA, which will be further used for hydraulic and mechanical investigations in this study. This compaction curve provides a maximum dry unit weight of 17.7 kN/m^3, which is relatively lower than the untreated soil (18.74 kN/m^3). However, the optimum moisture content (16.2%) for this curve is higher than untreated soil (14.8%). Dry unit weight of the soil decreased due the used of additives which has less density compared to soil. So the dry unit weight of the soil decreases with the addition of additives. Meanwhile the moisture content of the soil increases due to increase in surface area of the material. The more fine material the more is the specific surface area. Hence more water is needed to for large specific area of the material.

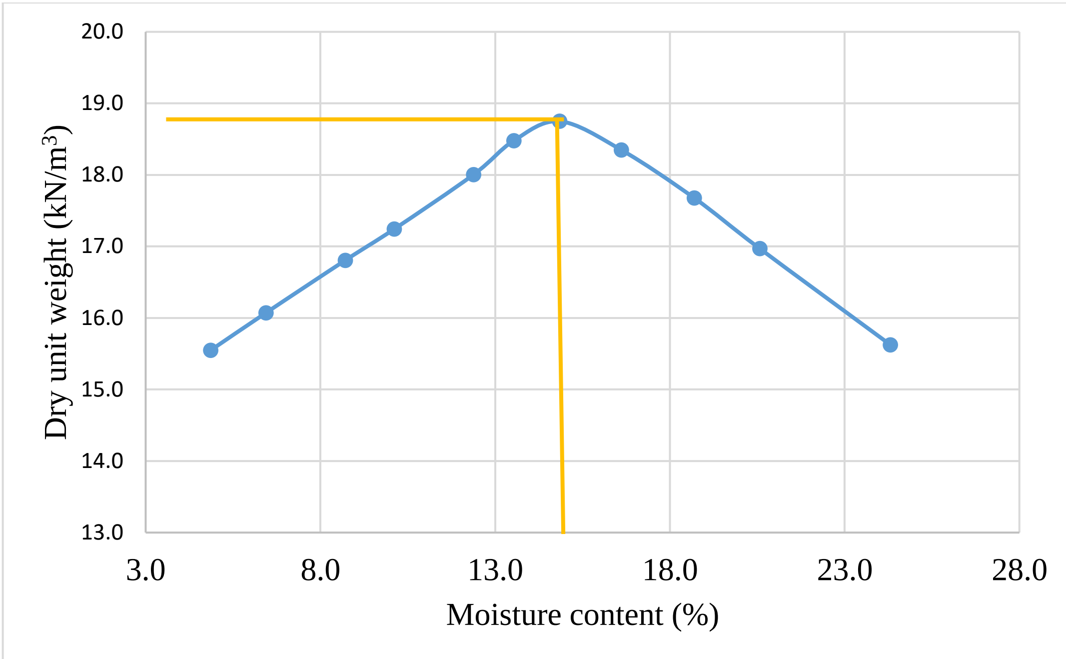

Figure 4. 4: Compaction curve of untreated soil

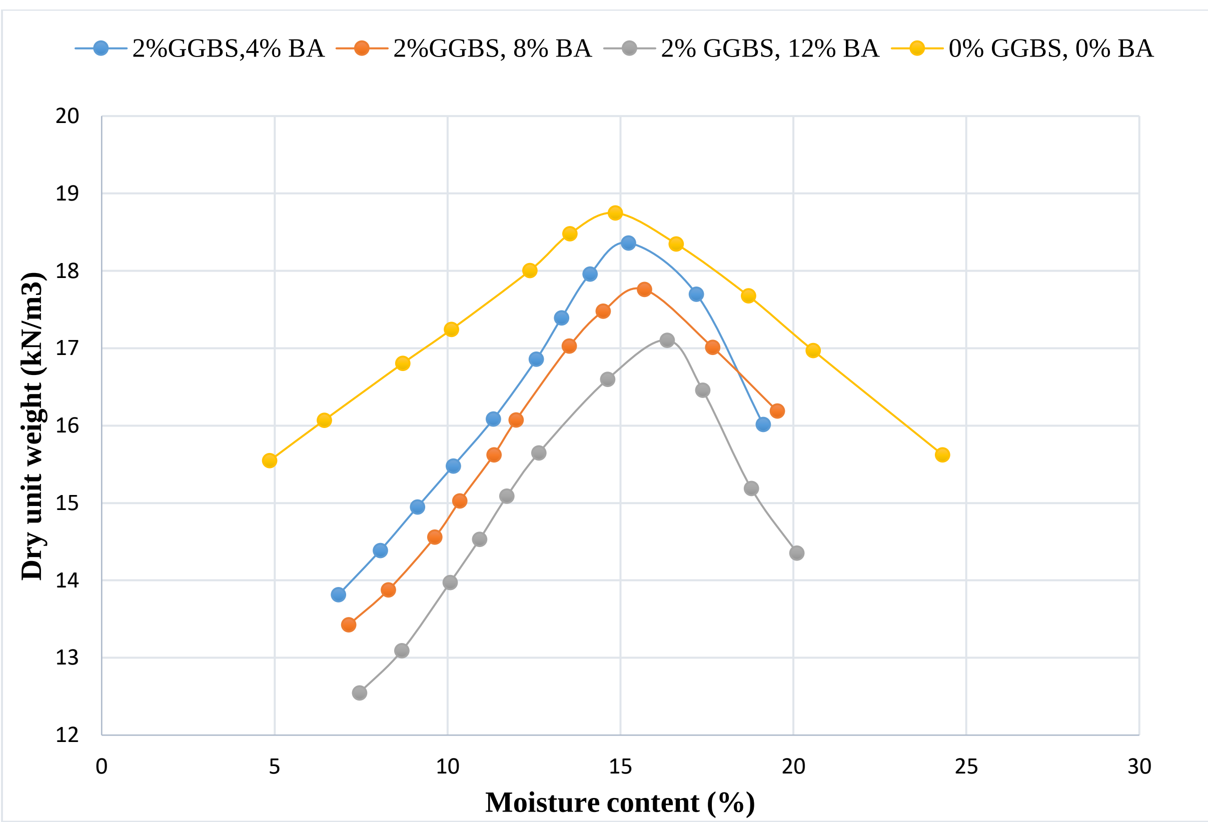

Figure 4. 5: Compaction curves of treated collapsible soil

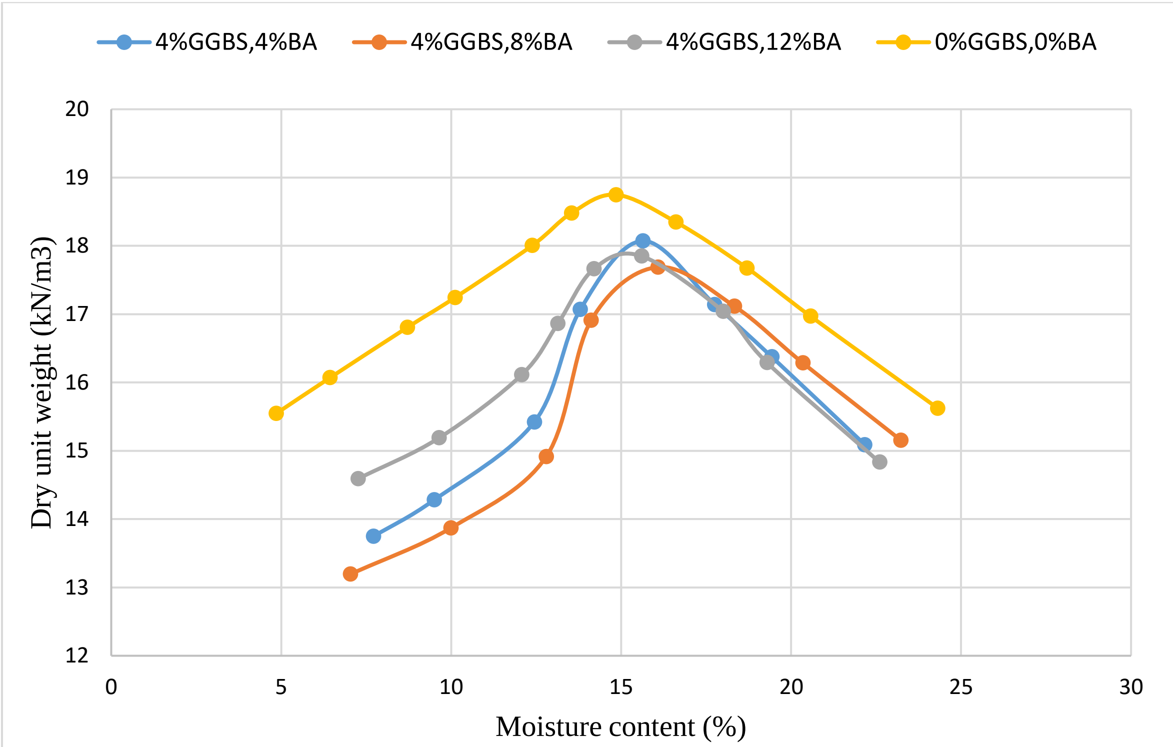

Figure 4. 6: Compaction curves of treated collapsible soil

4.5. Direct shear test results:

The relationships between shear stress and displacement are shown in Figures 4.7 – 4.12, which show that shear stress deformation profiles behave in a different fashion for different data points of the compaction curve. For dry of optimum and wet of optimum, the specimens relatively fail at smaller displacements than at optimal point. For optimal point, the sample fails at a shear displacement of 0.007 m, which is quite higher than wet of optimum (0.002 m) and dry of optimum (0.002 m).

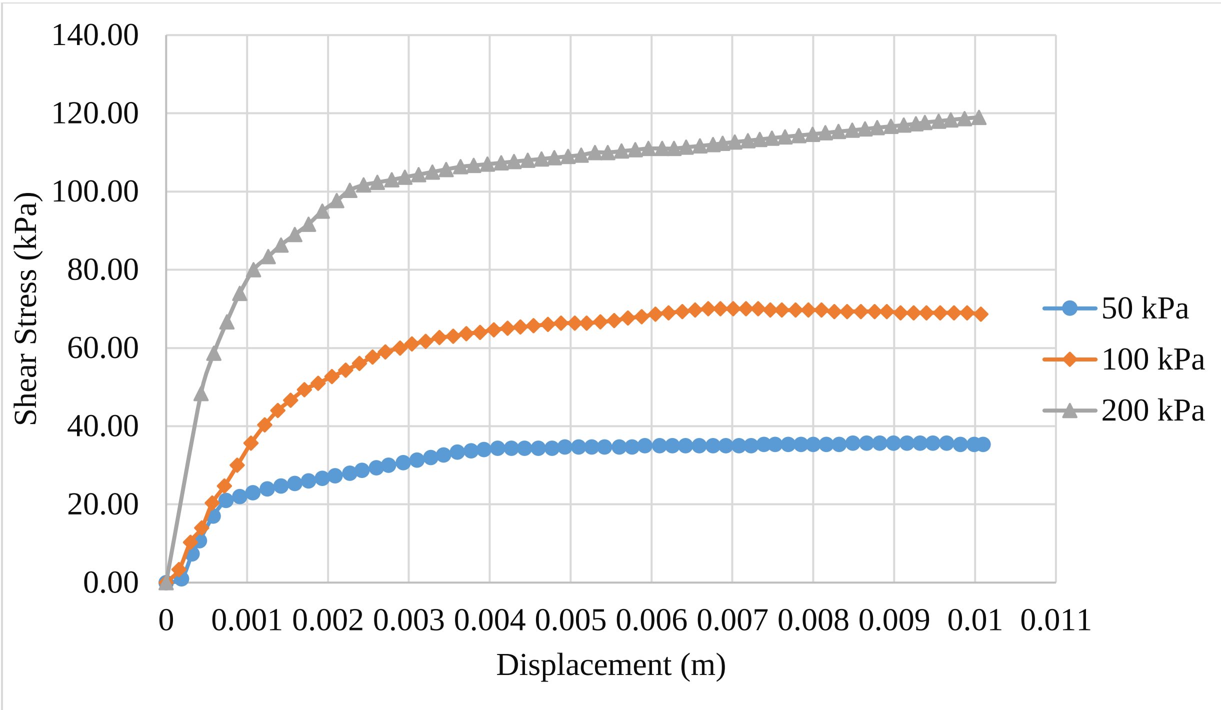

Figure 4. 7: Shear stress and displacement relationships for 1st dry unit weight (dry of optimum side) of compaction curve (untreated - unsaturated condition)

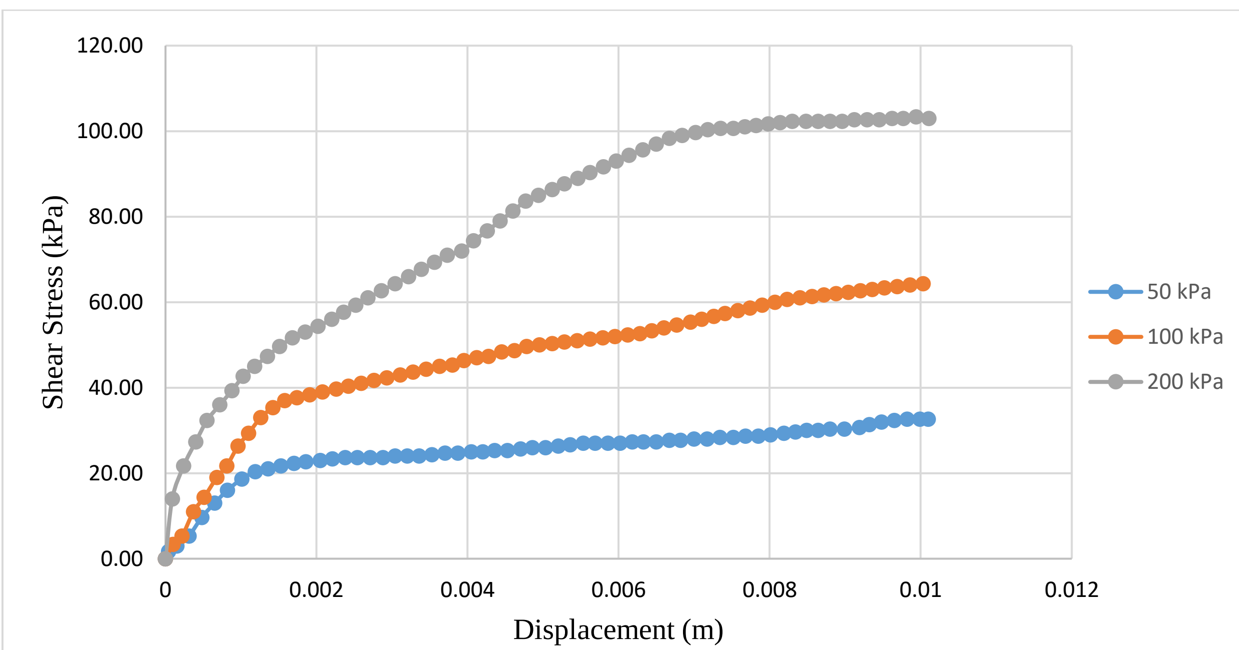

Figure 4. 8: Shear stress and displacement relationships for 3rd dry unit weight (dry of optimum side) of compaction curve (untreated - unsaturated condition)

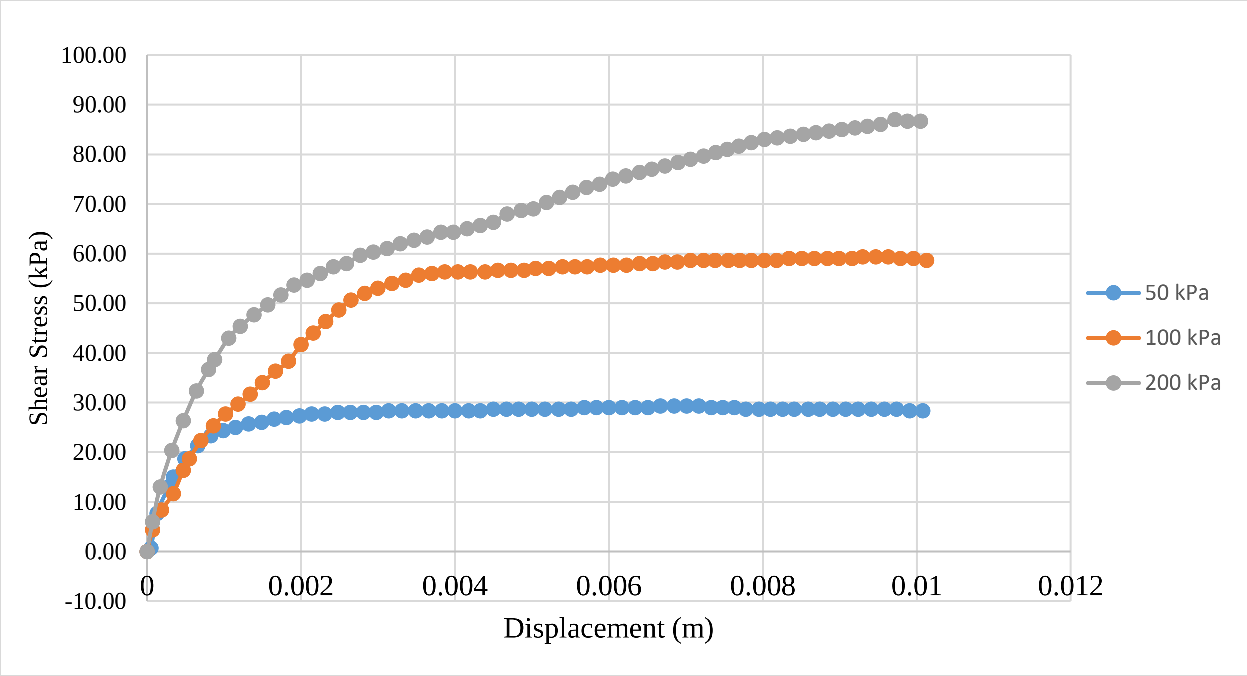

Figure 4. 9: Shear stress and displacement relationships for 5th dry unit weight (dry of optimum side) of compaction curve (untreated - unsaturated condition)

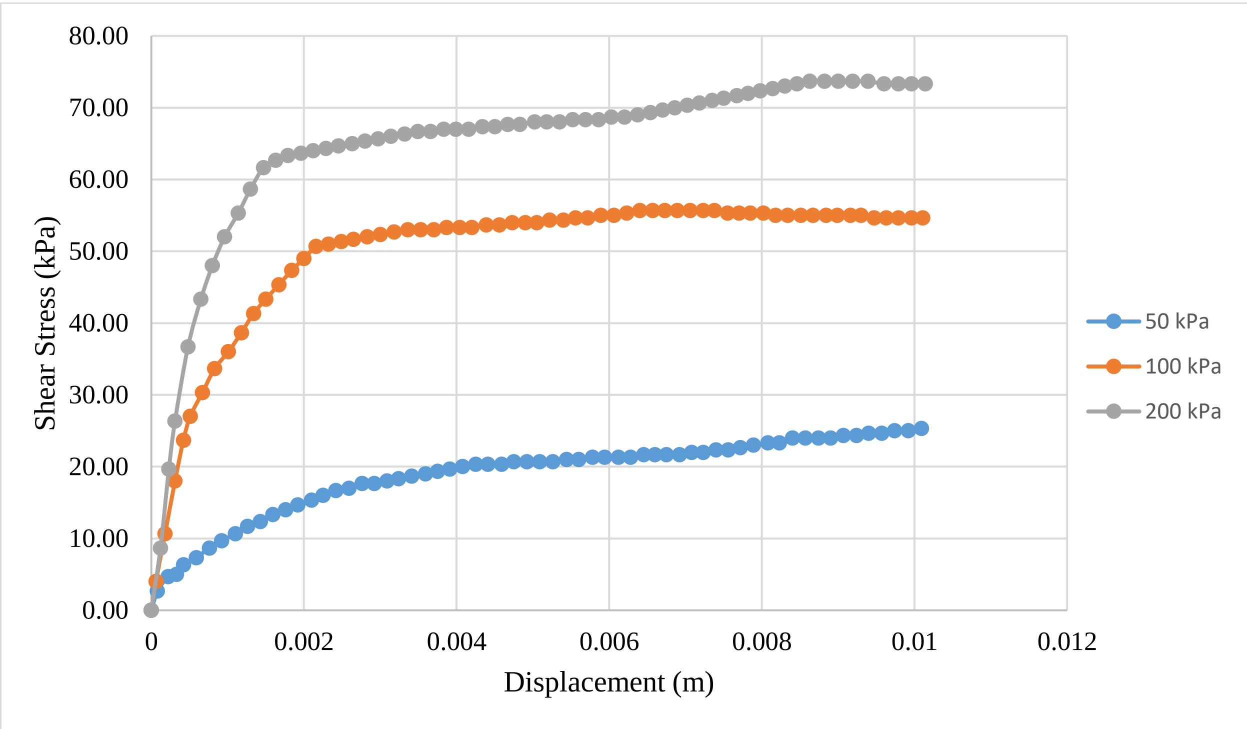

Figure 4. 10: Shear stress and displacement relationships for optimal dry unit weight of compaction curve (untreated - unsaturated condition)

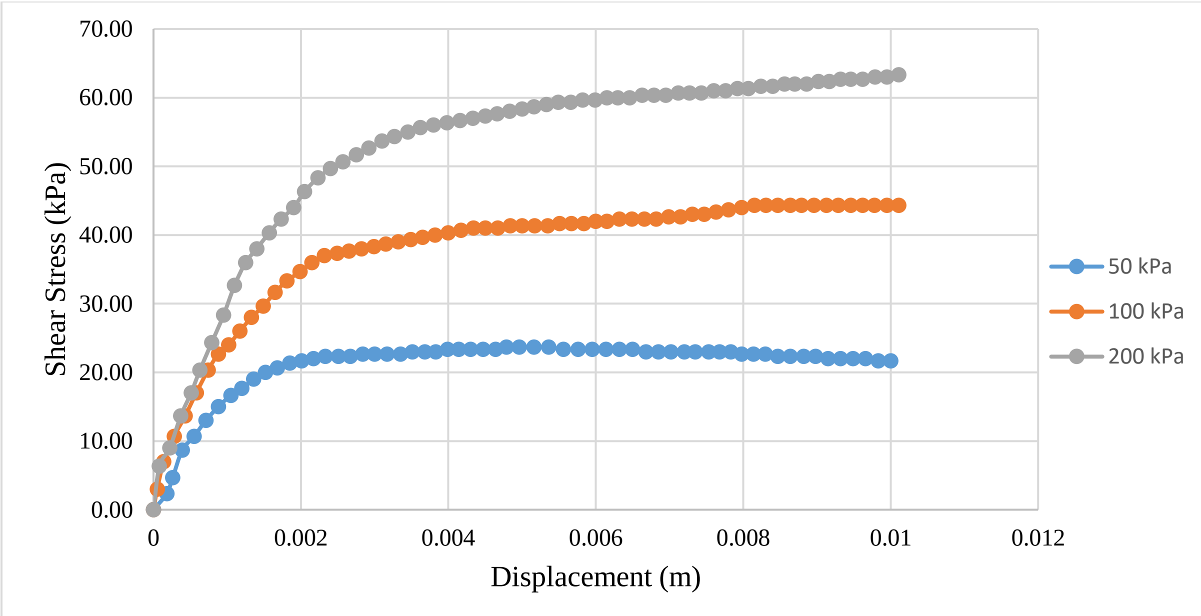

Figure 4. 11: Shear stress and displacement relationships for 9th dry unit weight (wet of optimum side) of compaction curve (untreated - unsaturated condition)

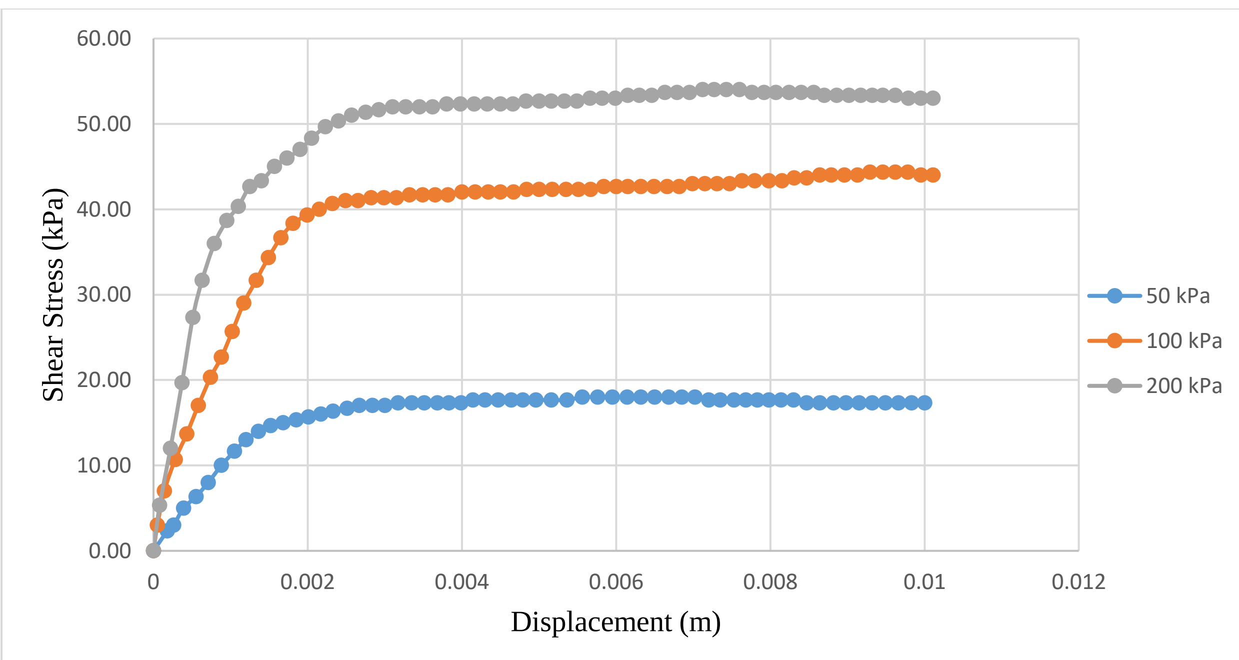

Figure 4. 12: Shear stress and displacement relationships for 11th dry unit weight (wet of optimum side) of compaction curve (untreated - unsaturated condition)

Figure 4.13 shows the cohesion and angle of internal friction at different dry unit weights of compaction curve in unsaturated conditions. The test data shows that the untreated soil provides angle of internal friction and cohesion of 16.3° and 16.8 kPa, respectively at an optimal point. It can be seen that the angle of internal friction increases and cohesion decreases with an increase in dry unit weight or moisture content up to optimum moisture content. After this point, both angle of internal friction and cohesion initiate to decrease. According to(Tiwari & Sharma, 2013), the angle of internal friction is a function of initial void ratio. The increase in cohesion and internal friction is due to the relative increase in dry unit weight which holds the particles together up to optimum moisture content but after that with the further addition of moisture content, the bonds between the soil grains loosen, which alternately results in a decrease of cohesion and angle of internal friction as well.

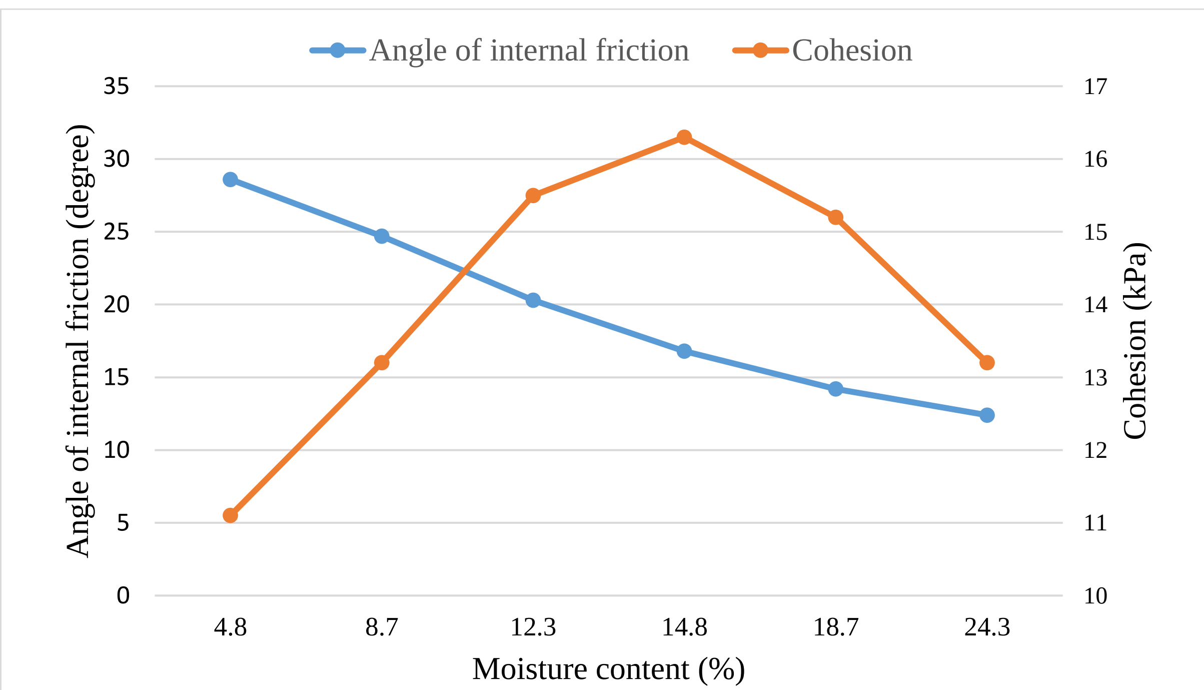

Figure 4. 13: Shear strength parameters for changes in moisture contents of compaction curve (unsaturated untreated soil)

Similar to that of unsaturated condition, the cohesion and angle of internal friction profiles show similar trends for different data points of the compaction curve in saturated conditions. As expected, the shear strength parameters provide lower peak in saturated conditions than unsaturated conditions for all data points (Figures4.13 and 4.20). For saturated conditions, the bonds between soil grains further loosens due to addition of water, which alternately results in the reduction of friction angle and cohesion. The soil provides an angle of internal friction and cohesion of 15.67^{o} and 15.8 kPa, respectively at an optimal point, which are relatively lower than the unsaturated condition. The shear stress and displacement relationships for various data points of the compaction curve are shown in Figures 4.14 – 4.19. It can be seen from Figure 4.13 and 4.20 that the differences in shear strength parameters are not so significant for saturated and unsaturated conditions.

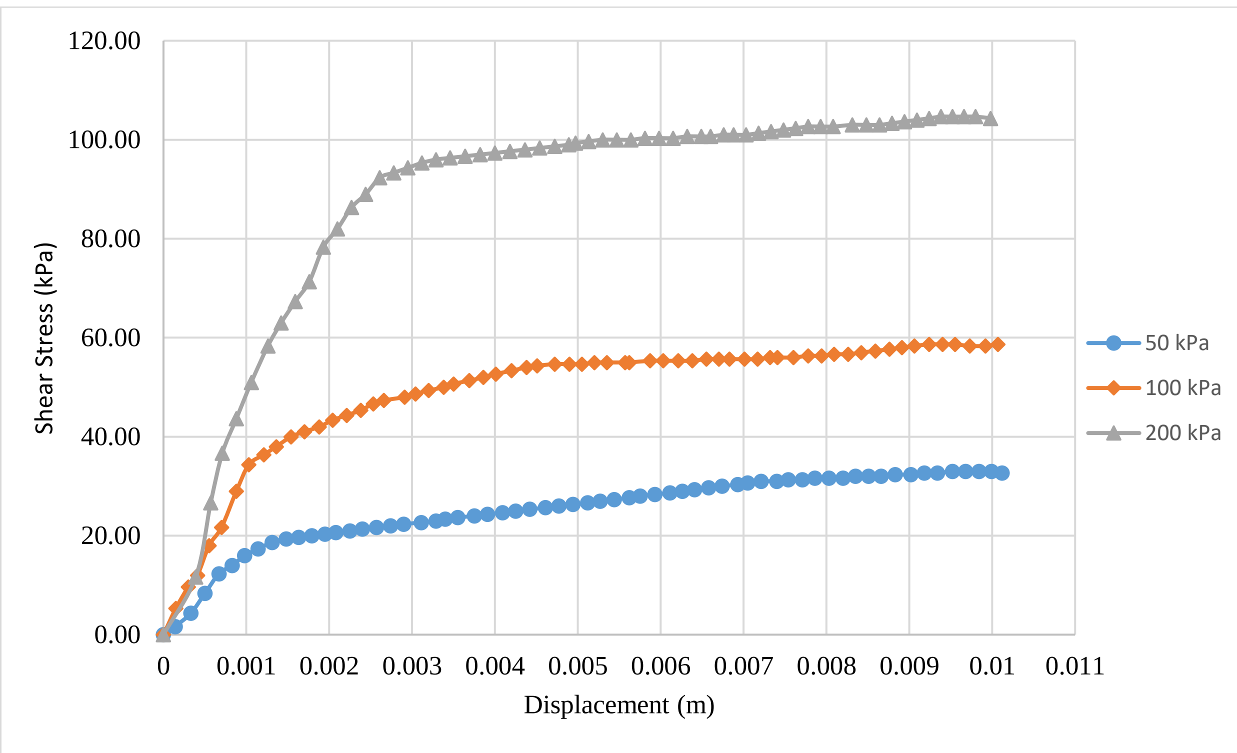

Figure 4. 14: Shear stress and displacement relationships for 1st dry unit weight (dry of optimum side) of compaction curve (untreated - saturated condition)

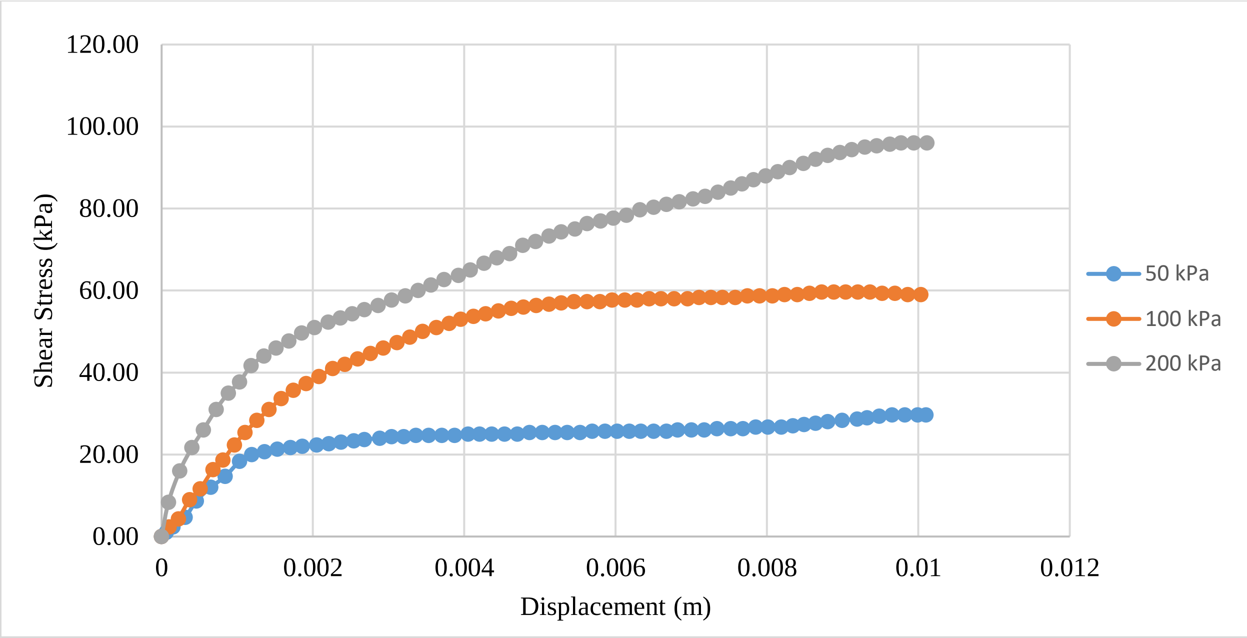

Figure 4. 15: Shear stress and displacement relationships for 3rd dry unit weight (dry of optimum side) of compaction curve (untreated - saturated condition)

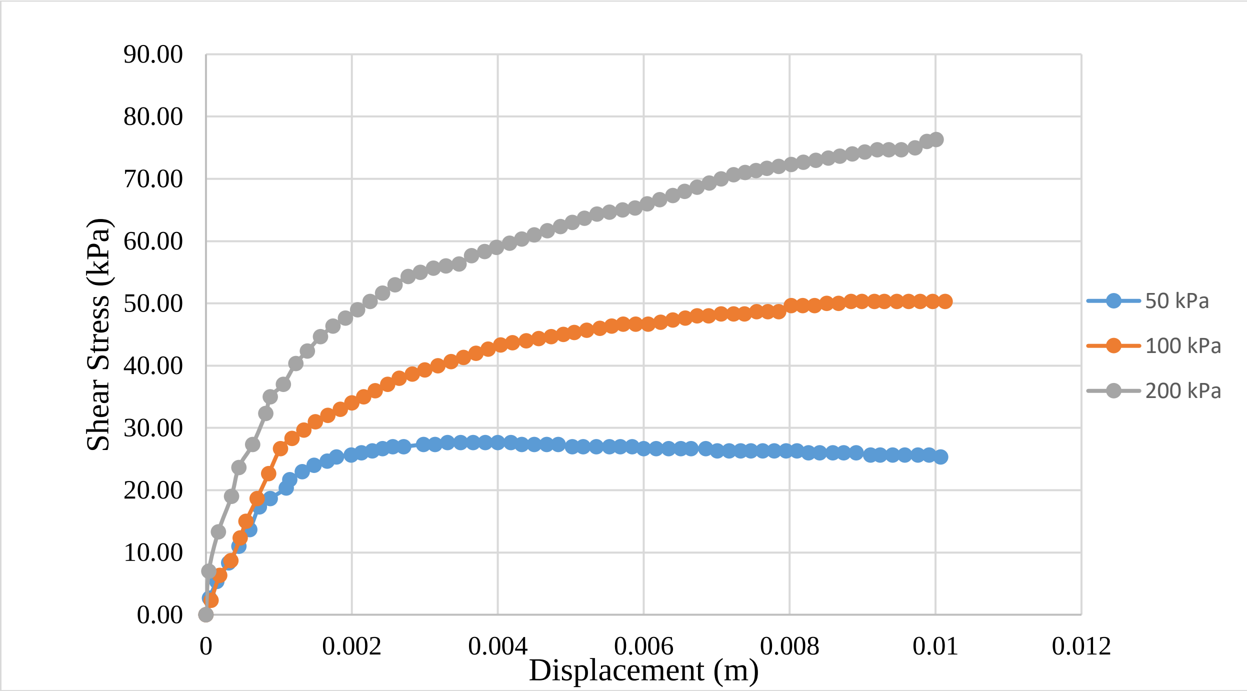

Figure 4. 16: Shear stress and displacement relationships for 5th dry unit weight (dry of optimum side) of compaction curve (untreated - saturated condition)

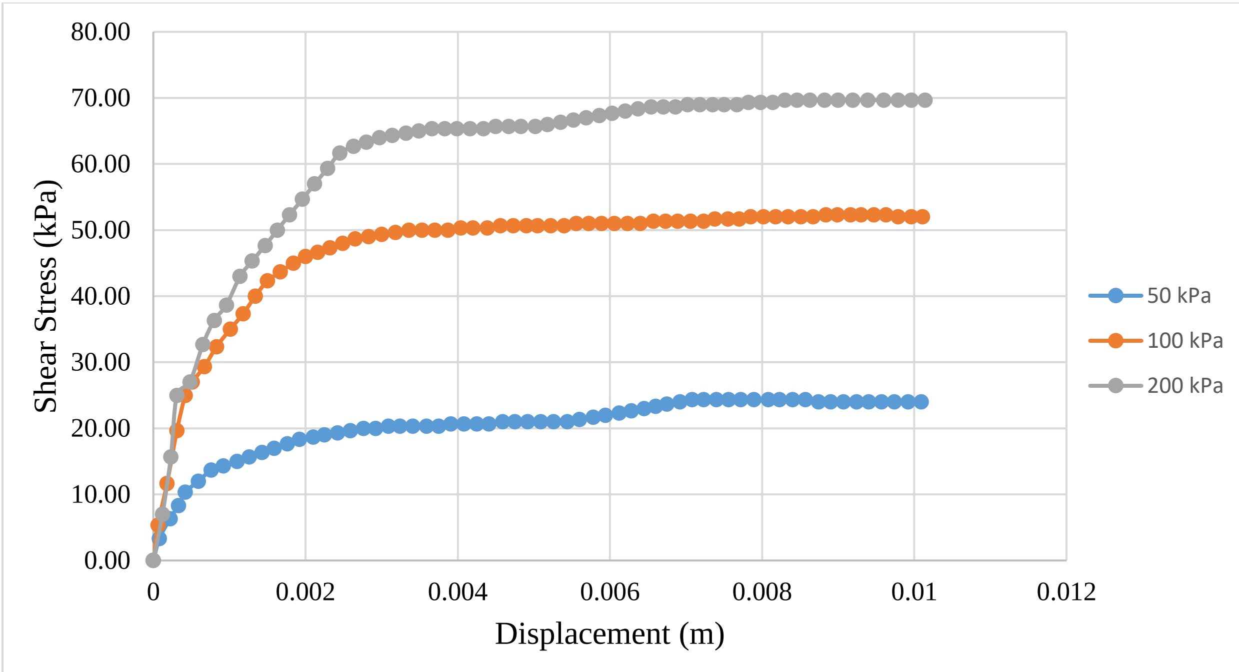

Figure 4. 17: Shear stress and displacement relationships for optimal dry unit weight of compaction curve (untreated - saturated condition)

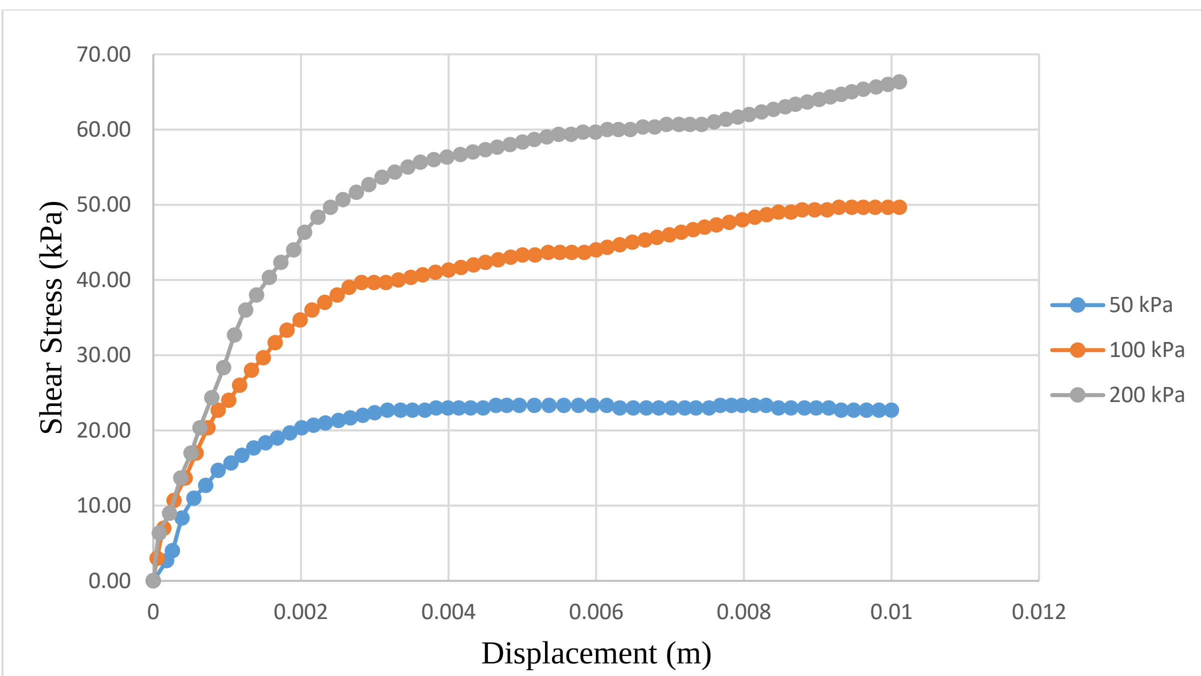

Figure 4. 18: Shear stress and displacement relationships for 9th dry unit weight (wet of optimum side) of compaction curve (untreated - saturated condition)

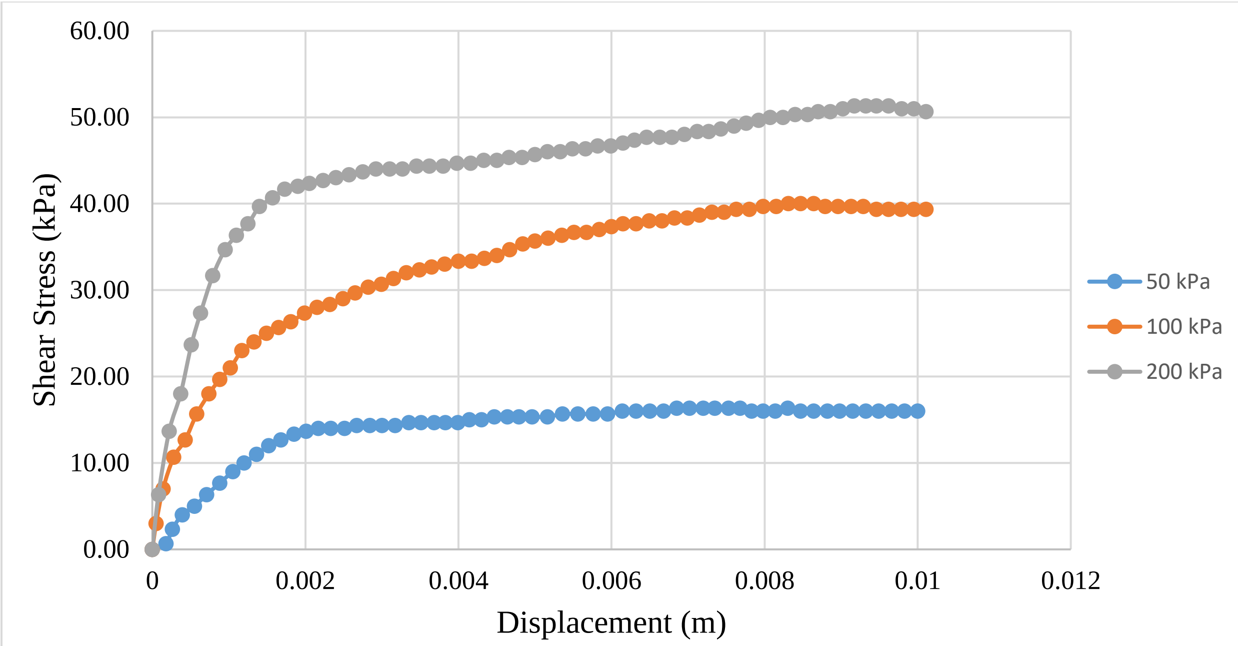

Figure 4. 19: Shear stress and displacement relationships for 11th dry unit weight of (wet of optimum) compaction curve (untreated - saturated condition)

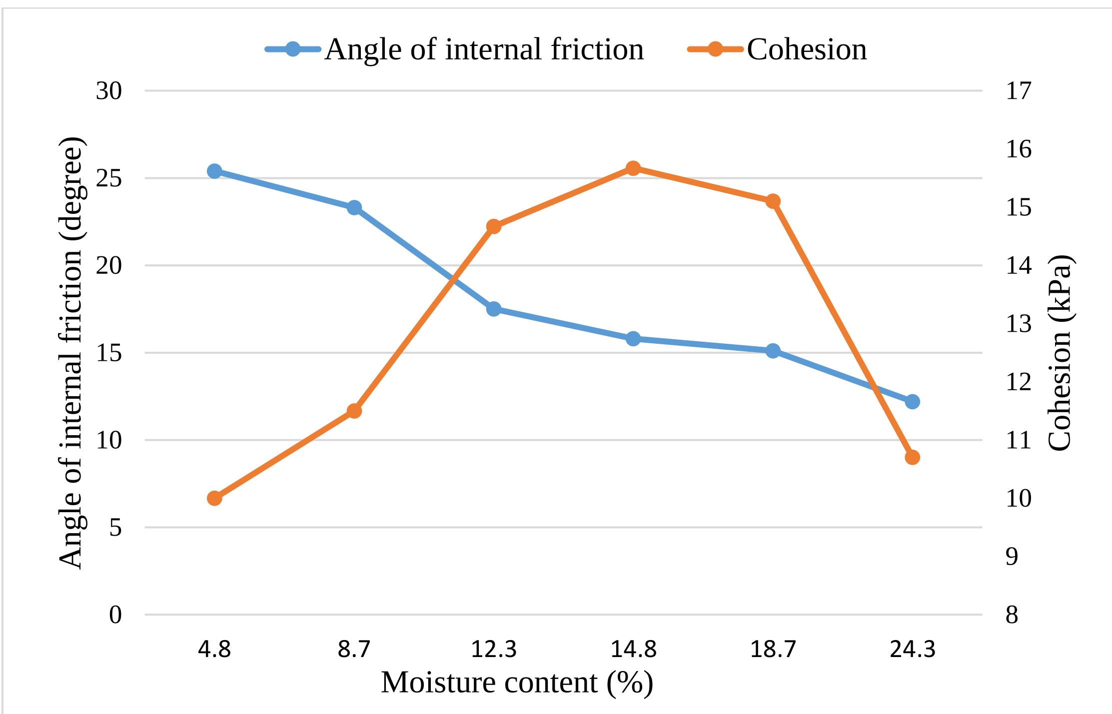

Figure 4. 20: Shear strength parameters for moisture contents in saturated condition

Figures 4.21 - 4.25 present the shear stress displacement relationships of treated soil in unsaturated conditions which depict that the specimens fail at relatively larger displacement than untreated specimens as in Figures 4.7 - 4.12. It can be seen from Figure 4.10 and 4.23 that the shear stress -displacement profiles provide different trends at various normal stresses. For treated soil, the curves appear more consistent with relatively higher peaks than untreated soil, and as in Figure 4.26, the angle of internal friction is higher, and correspondingly, the cohesions lower at optimal water content. The treated soil provides an angle of internal friction of 20.1^0 and cohesion of 12 kPa (Figure 4.26). Similarly, the shear stress- displacement relationships for treated soil in saturated conditions are presented in Figures 4.27 – 4.31. Figure 4.32 shows relationships between shear strength parameters for various moisture contents of the compaction curve. Comparatively, the shear strength parameters of treated specimens provide lower peaks at optimal moisture content saturated condition as expected. The saturated specimen provides an angle of internal friction of $18.7\ ^o$ and cohesion of 9.5 kPa (Figure 4.32), which is relatively lower than the unsaturated conditions (Figure 4.26). For all conditions, the cohesion increases up to optimal moisture content and then initiates to decrease on wet of optimum side, and conversely, angle of internal friction gradually decreases with an increase in the moisture contents of compaction curve for all conditions. Resultantly, the angle of internal friction is noted maximum at dry of optimum side and cohesion at optimal water content. Furthermore, the cohesion is relatively higher on wet of optimum side than dry of optimum side. These differences in shear strength parameters are due to the pozzolanic action of BA and GGBS, which provide cementitious effects in combination with soil colloids, and alternately change the shear strength parameters.

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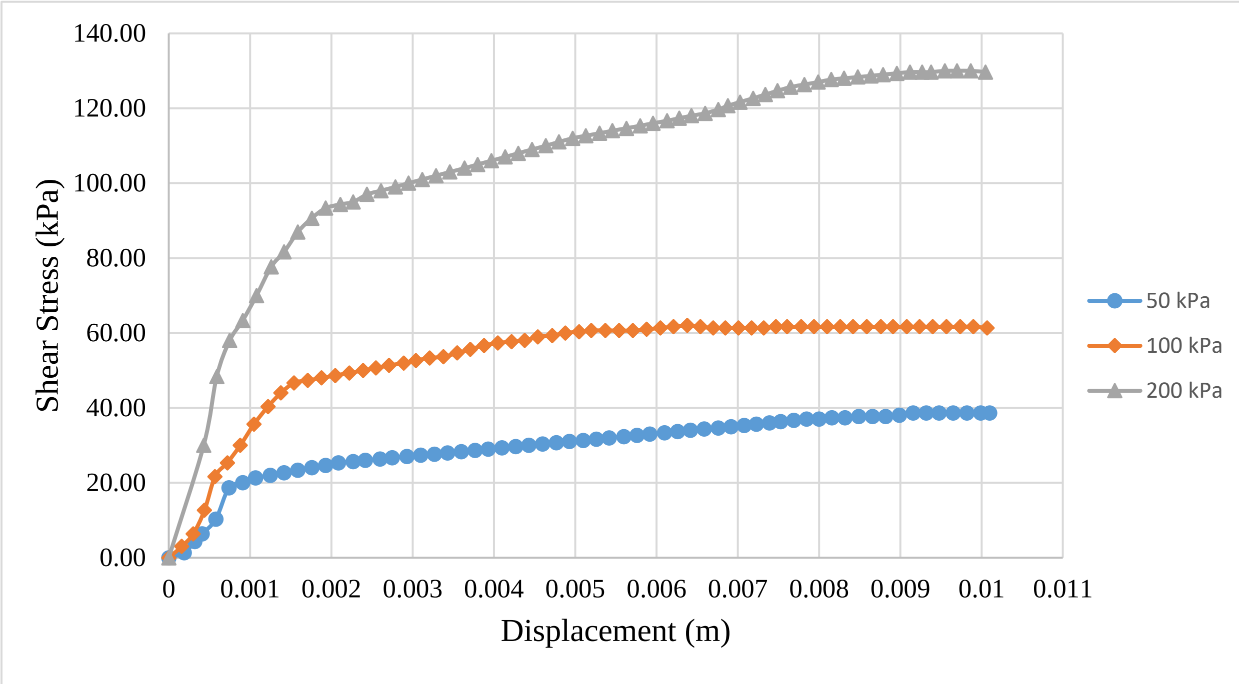

Figure 4. 21: Shear stress and displacement relationships for 1st dry unit weight (dry of optimum side) of compaction curve (treated - unsaturated condition)

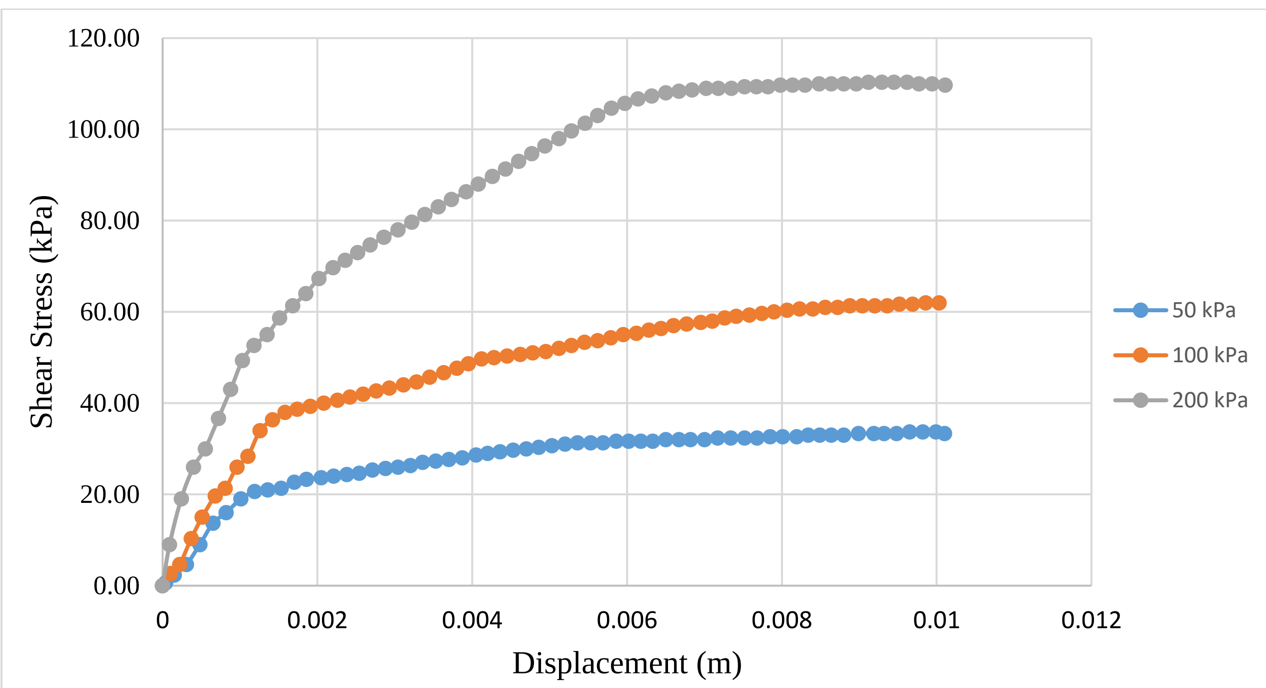

Figure 4. 22: Shear stress and displacement relationships for 3rd dry unit weight (dry of optimum side) of compaction curve (treated - unsaturated condition)

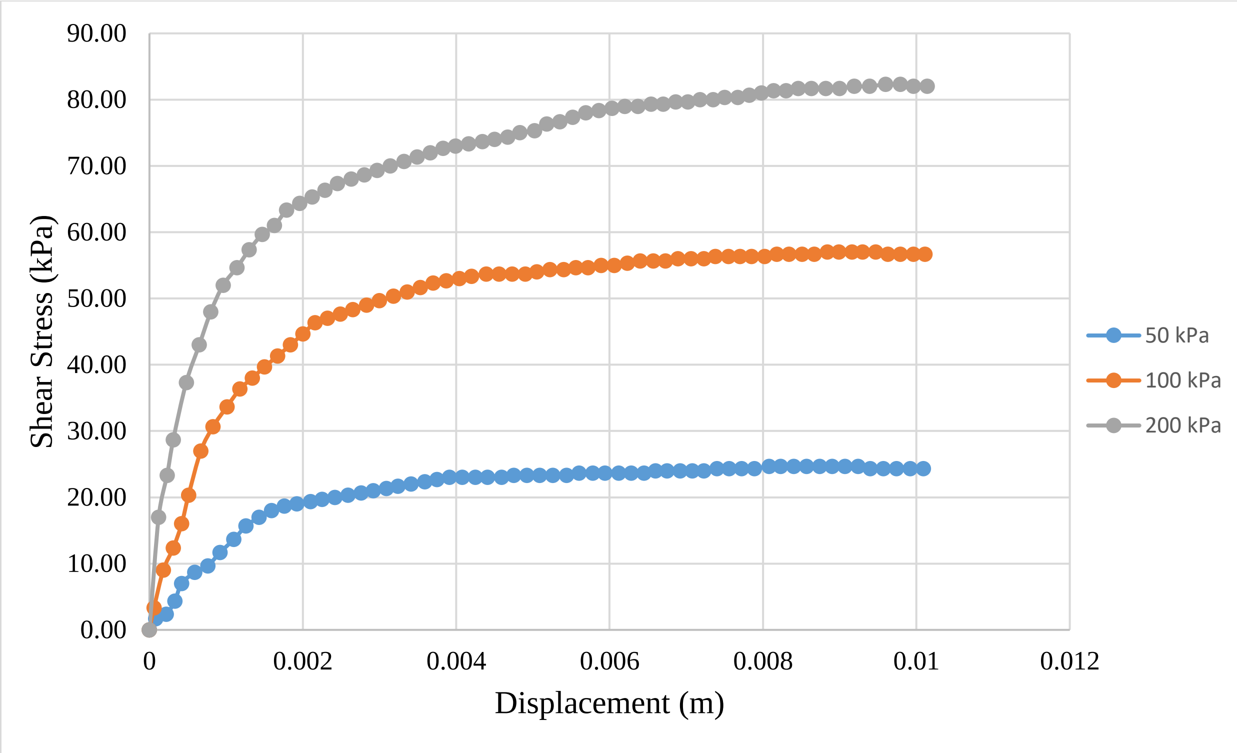

Figure 4. 23: Shear stress and displacement relationships for optimal dry unit weight of compaction curve (treated - unsaturated condition)

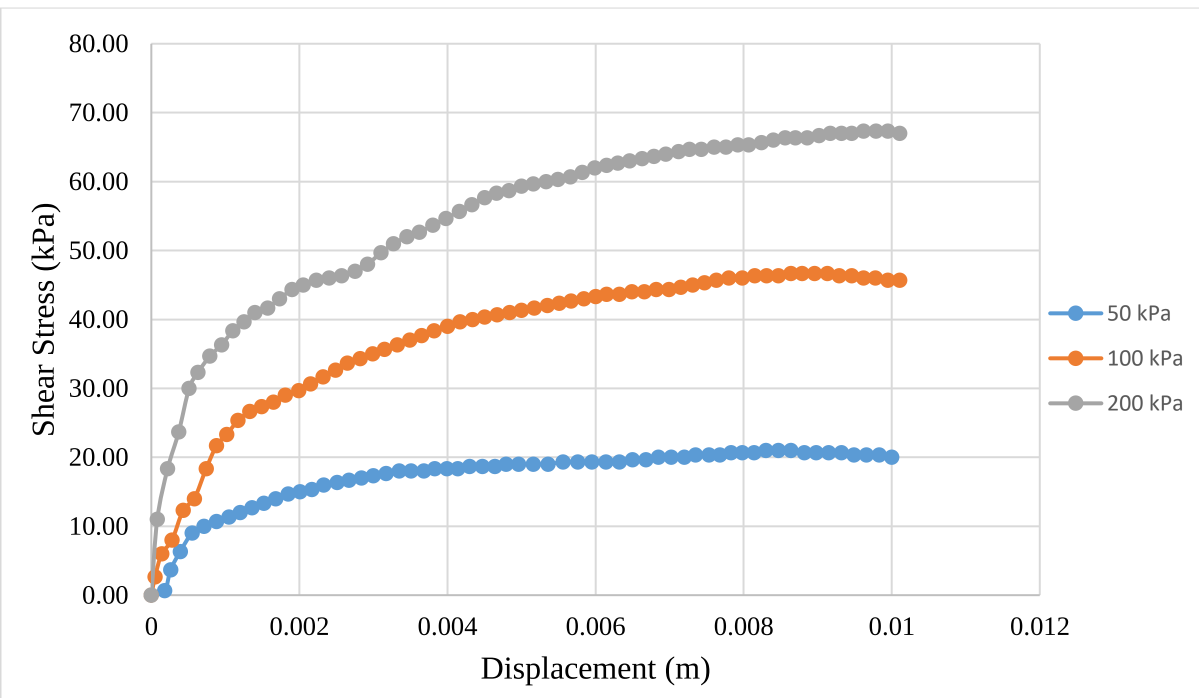

Figure 4. 24: Shear stress and displacement relationships for 6th dry unit weight (wet of optimum side) of compaction curve (treated - unsaturated condition)

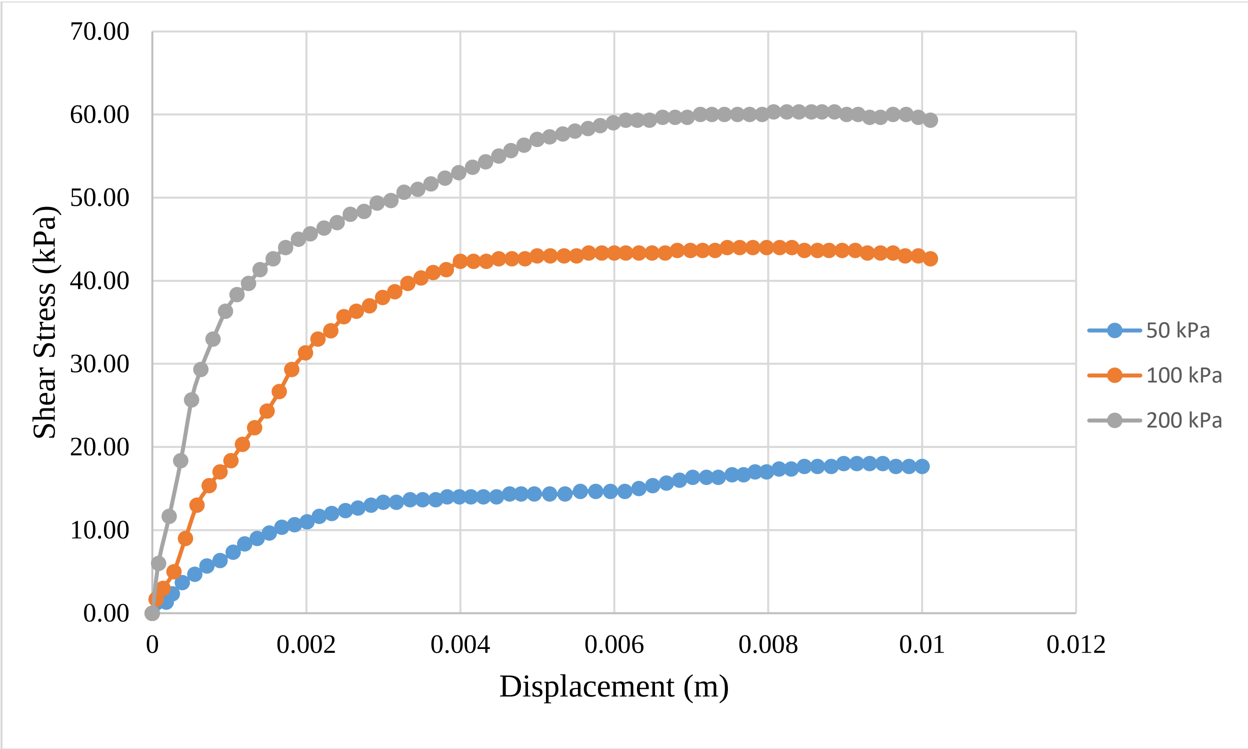

Figure 4. 25: Shear stress and displacement relationships for 7th dry unit weight (wet of optimum side) of compaction curve (treated - unsaturated condition)

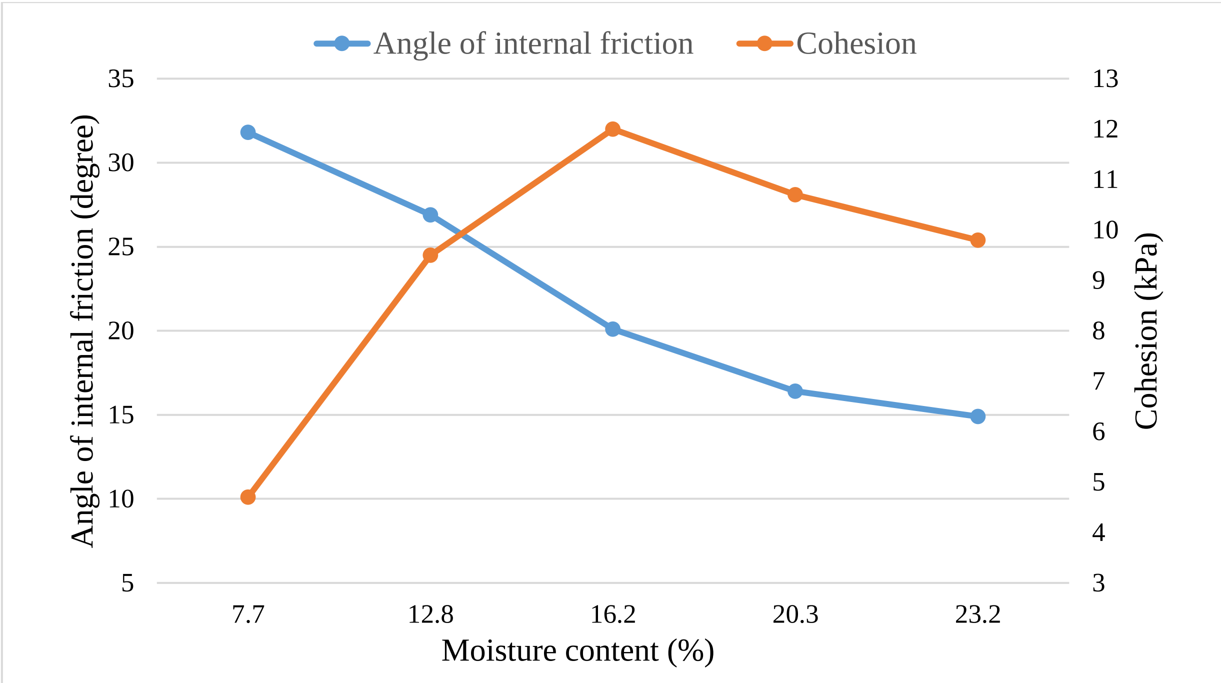

Figure 4. 26: Shear strength parameters of reinforced soil against moisture content in unsaturated condition

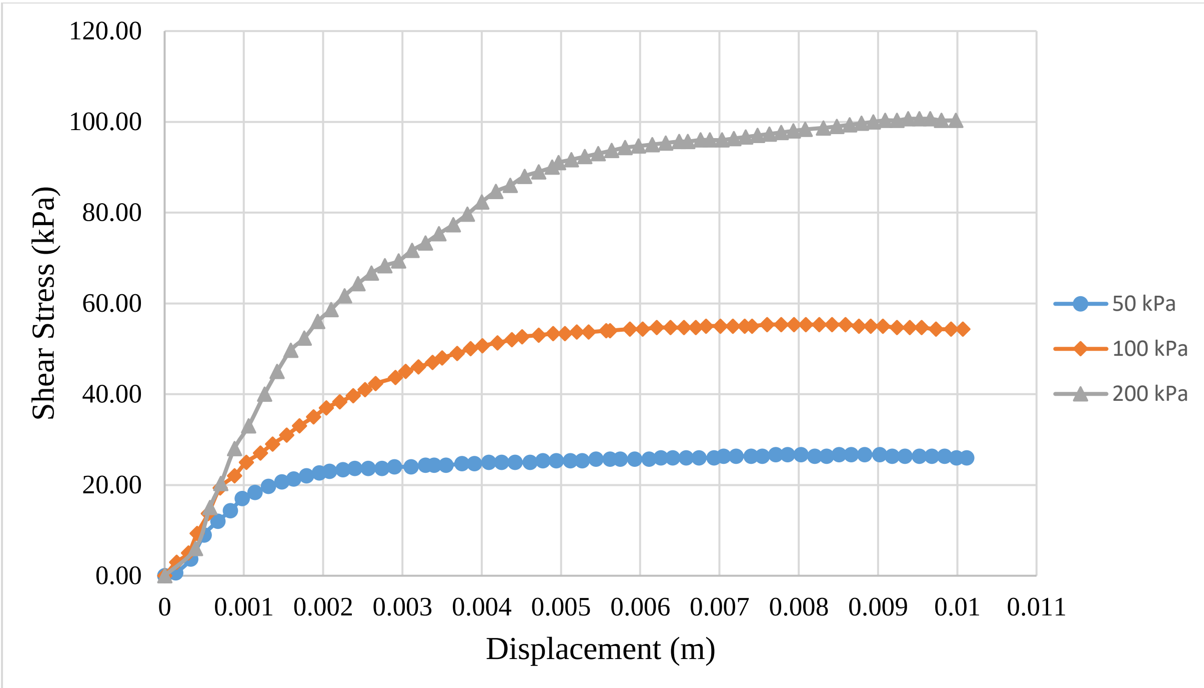

Figure 4. 27: Shear stress and displacement relationships for 1st dry unit weight (dry of optimum side) of compaction curve (treated - saturated condition)

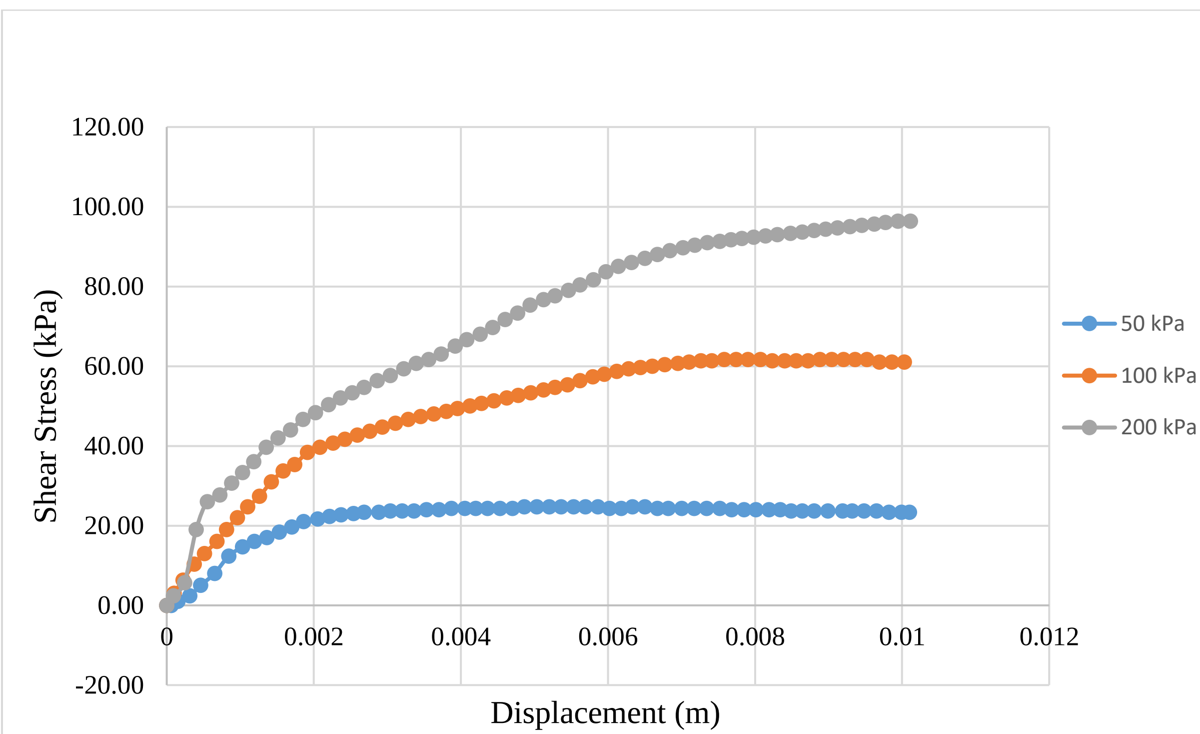

Figure 4. 28: Shear stress and displacement relationships for 3rd dry unit weight (dry of optimum side) of compaction curve (treated - saturated condition)

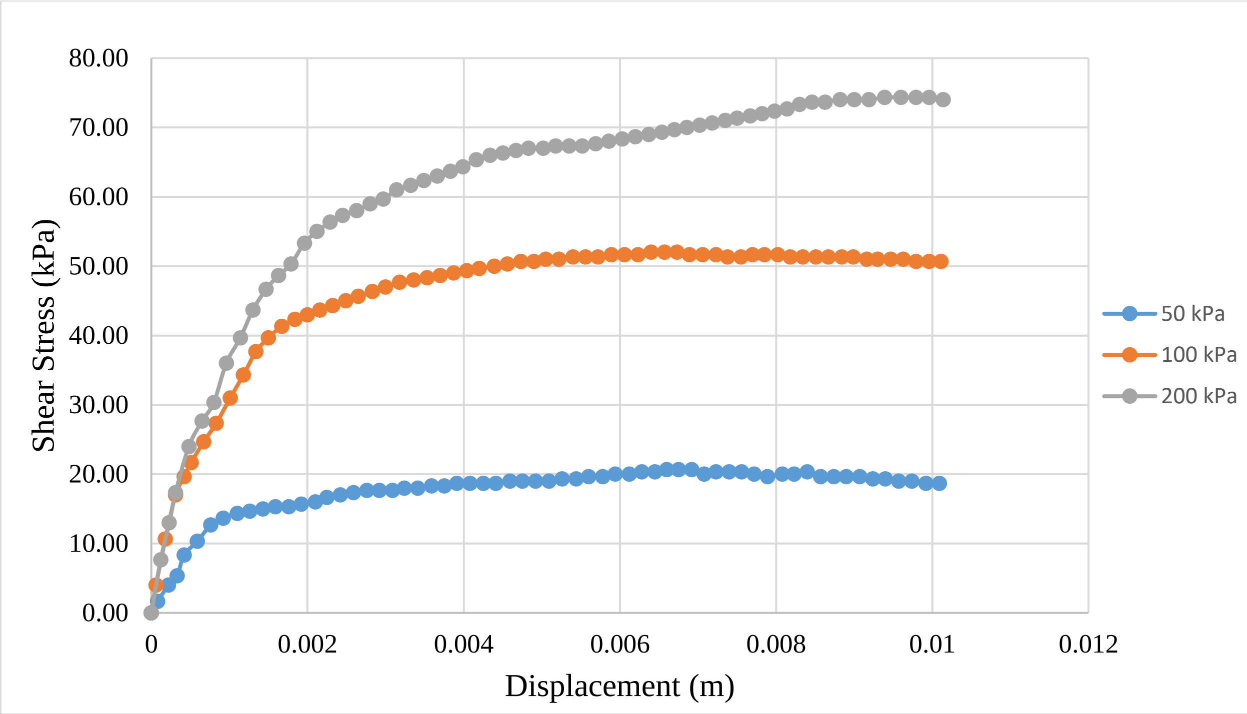

Figure 4. 29: Shear stress and displacement relationships for optimum dry unit of compaction curve (treated - saturated condition)

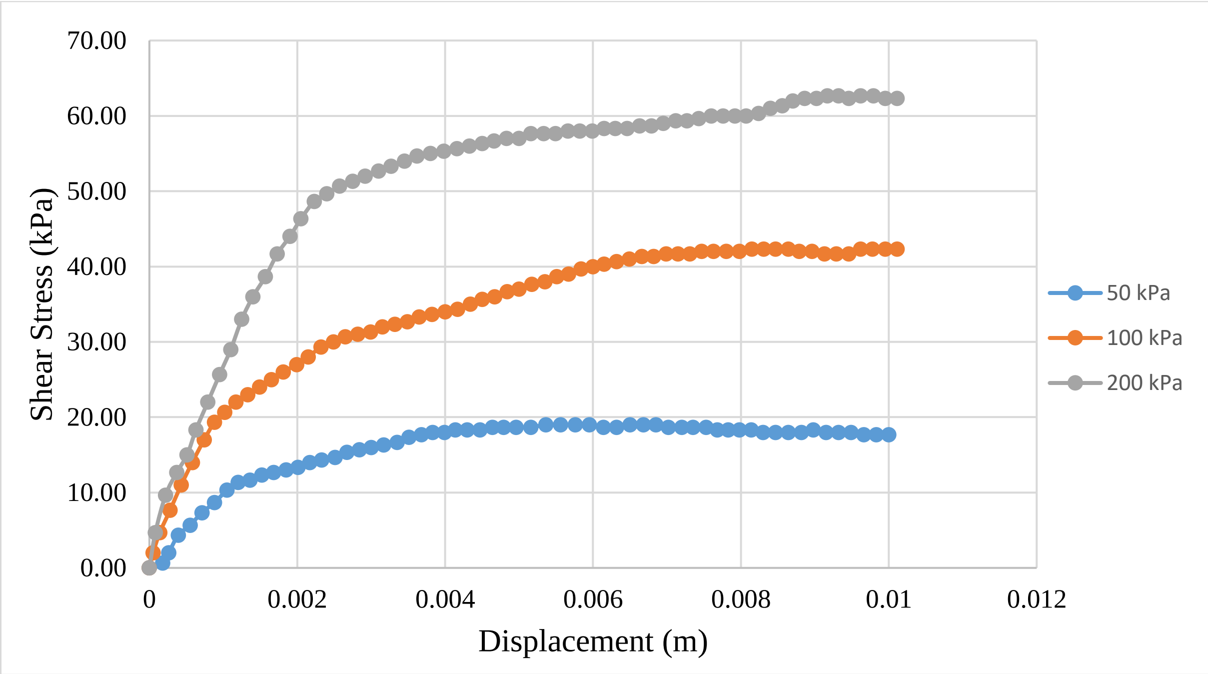

Figure 4. 30: Shear stress and displacement relationships for 6th dry unit weight (wet of optimum side) of compaction curve (treated - saturated condition)

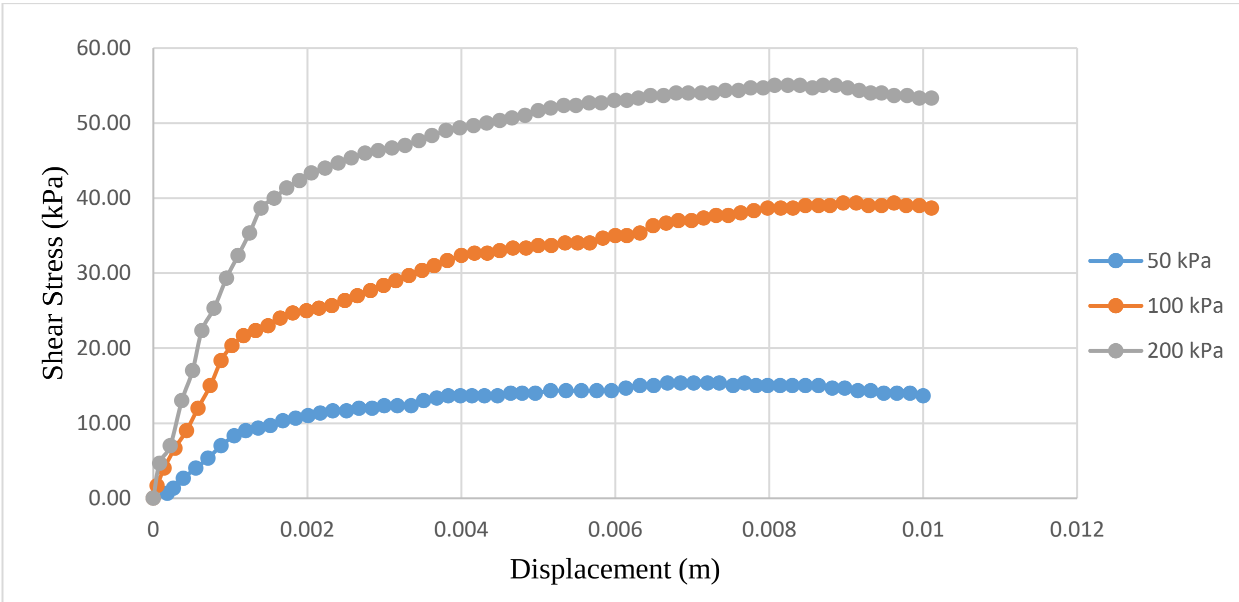

Figure 4. 31: Shear stress and displacement relationships for 7th dry unit weight (wet of optimum side) of compaction curve (treated - saturated condition)

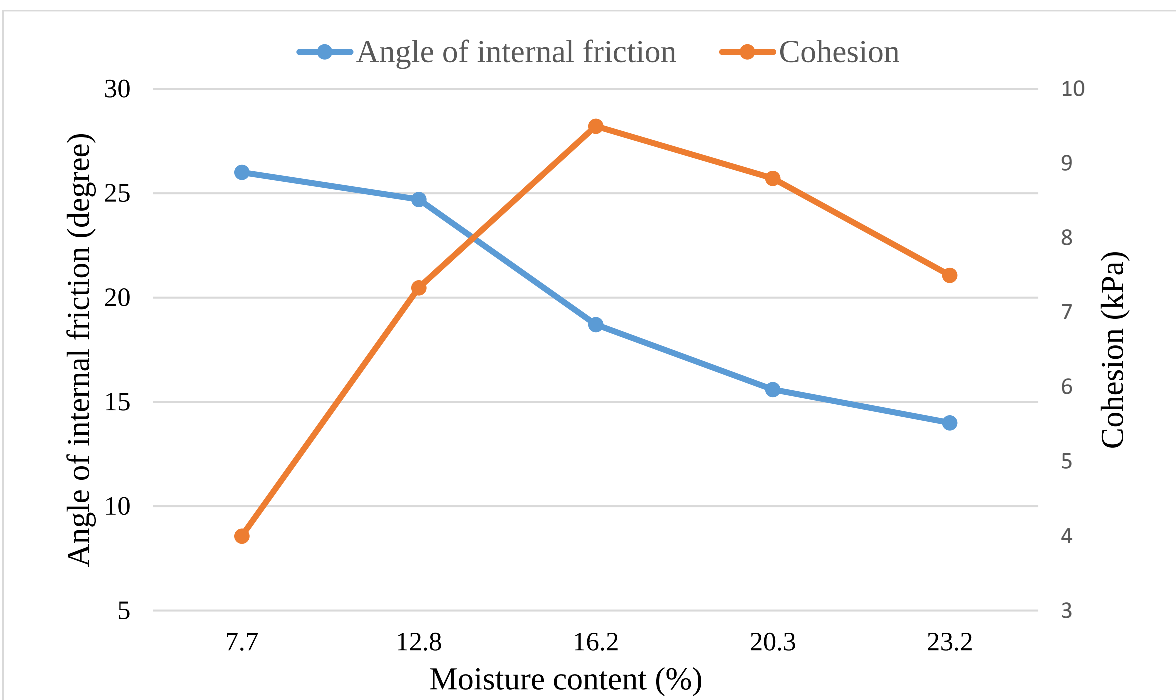

Figure 4. 32: Shear strength parameters of reinforced soil against moisture content

4.6. Hydraulic conductivity

Tables 4.1 and 4.2 show the hydraulic conductivity test results at different dry unit weights and moisture contents of untreated and reinforced soils, respectively. Table 4.1 shows that the hydraulic conductivity of untreated soil is higher for wet of optimum than dry of optimum, and it provides maximum k at dry unit weight of 17.7 kN/m^3. The differences in hydraulic conductivity is due to differences in void ratio and porosity. At wet of optimum, i.e., Yd = 16 kN/m^3 , the k was not estimated due to higher water content and the soil sample was collapsed while fixing in the permeator. As in Table 4.2, similar to that of untreated soil, the treated soil provides maximum k for wet of optimum side which is again due to differences in void ratio / porosity. The untreated soil relatively provides higher hydraulic conductivity than treated soil, which is due to the fact that the treated soil shows less void ratio than untreated soil which is due to its more packed structure as a result of pozzolanic action of additives with soil.

Table 4. 1: Hydraulic conductivity of untreated soil for different unit weights

Dry unit weight (kN/m^3)	15.5	16.8	18	18.7	17.7	16
Hydraulic conductivity (cm/sec)	2.09*10^{-6}	2.43*10^{-6}	2.72*10^{-6}	2.95*10^{-6}	3.42*10^{-6}	Fail

Table 4. 2: Hydraulic conductivity of treated soil for different unit weights

Dry unit weight (kN/m^3)	14.2	16.2	17.7	16.2	15.1
Hydraulic conductivity (cm/sec)	1.12*10^{-7}	1.37*10^{-7}	1.70*10^{-7}	1.88*10^{-7}	2.22*10^{-7}

4.7. Collapse potential

Figure 4.33 shows the collapse potential for both untreated and treated soils which depicts that collapse potential varies from 5.5% to 10.13% and from 2.83 to 4.14% for untreated and treated soils, respectively with minimum at optimal point. (Jennings and Knight 1975) reported a criteria to determine the severity of the soil in relation to collapse potential, and according to this criteria, the soil satisfies the trouble to severe trouble category. As in Figure 4.33, the addition of additives reduces the collapse potential up to 50-60%, and the maximum change occurs at the OMC. After the OMC, it can be seen from Figure 4.33 that the collapse potential initiates to increase, again.

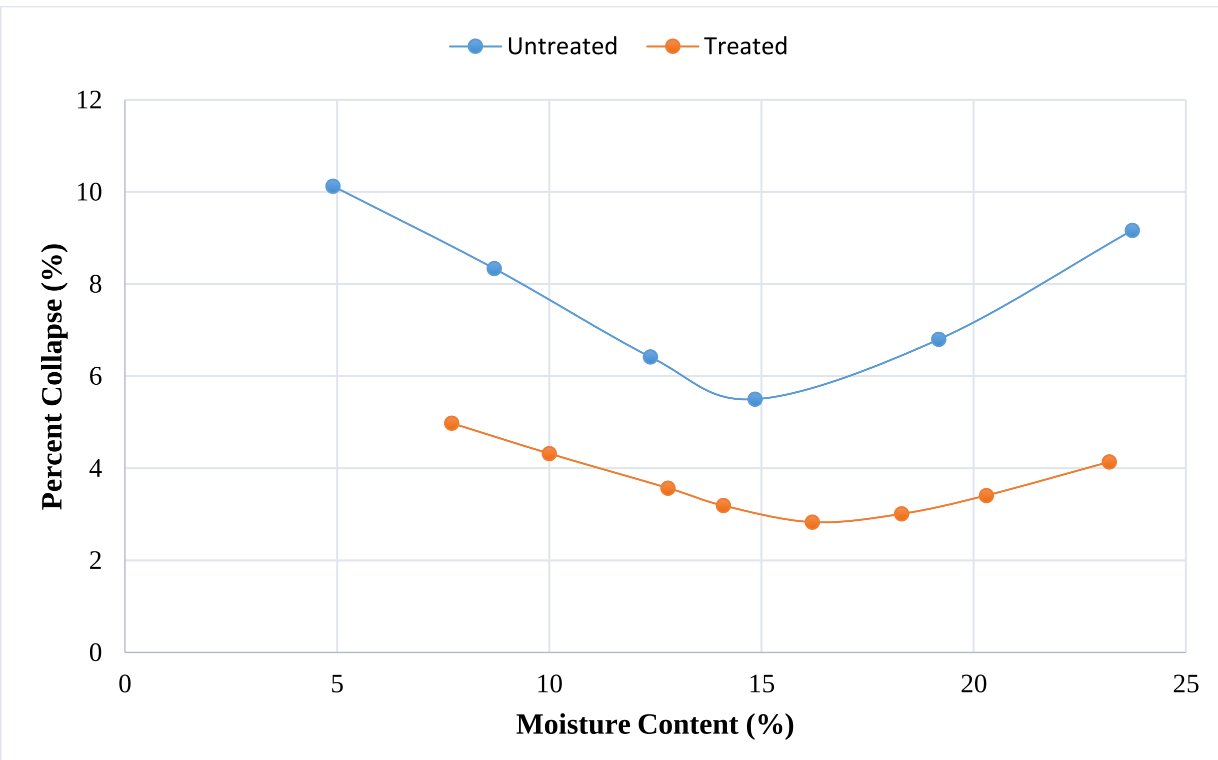

Figure 4. 33: Collapse potential vs moisture content

4.8. Oedometer consolidation test

4.8.1. Initial and final void ratios profile

Consolidation tests were performed at different data points of the compaction curve for both treated and untreated soils. As in Figure 4.34, the initial and final void ratios of untreated soils vary from 0.81 to 0.46 and from 0.36 to 0.22 at different moisture contents. Similarly, as in Figure 4.35, the initial and final void ratios of treated soils vary from 0.73 to 0.42 and from 0.32 to 0.20 at different moisture contents. The treated soil relatively shows lower void ratio than untreated soil, and as expected, the void ratio is minimum at optimal water content. As in Figures 4.34 and 4.35, with an increase in moisture content, the initial and final void ratios decrease up to optimum moisture content but after this point initiates to increase.

After optimum moisture content, the increase in initial and final void ratios is due to the breakage of bonds between the soil grains which results in an increase in void ratio. The void ratio is relatively lower for wet of optimum side than dry of optimum side with minimum at the optimal point, due to which the strength is maximum at this point. Furthermore, the lower void ratio of treated soil than untreated soil (Figures 4.34 and 4.35) is due to the reason that additives fill the pore spaces between the soil grains, due to which inter-particle contact increases, which alternately increases the cohesion as discussed in the proceeding section.

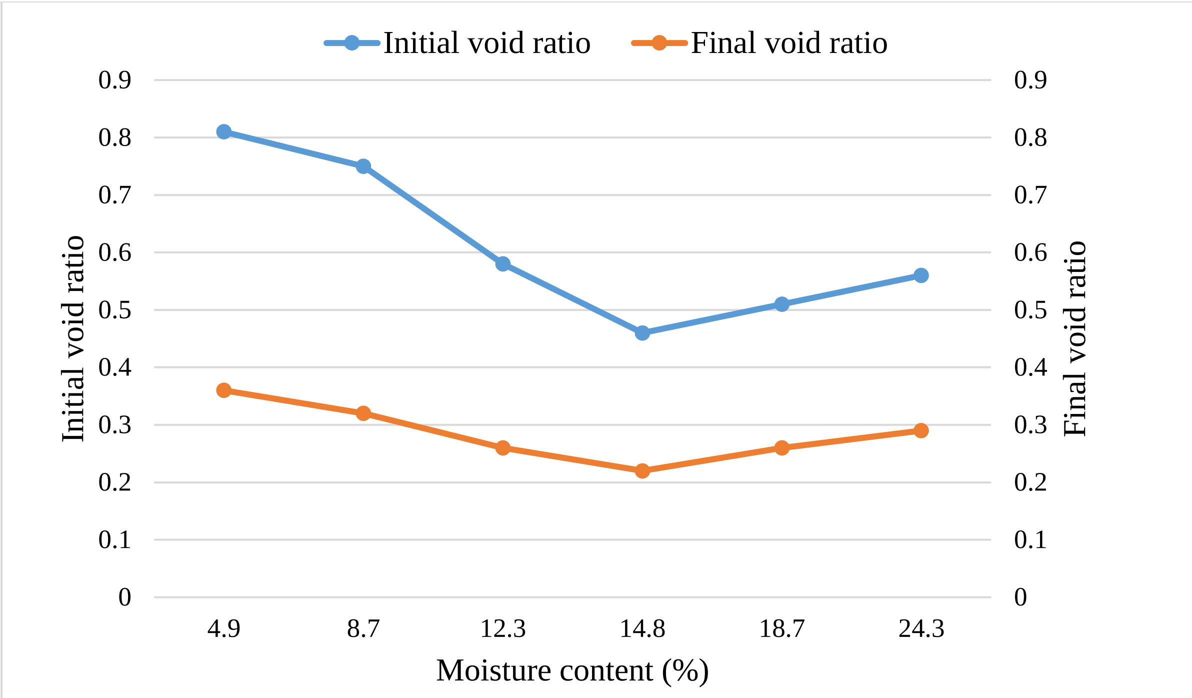

Figure 4. 34: Initial and final void ratio vs moisture content of untreated soil

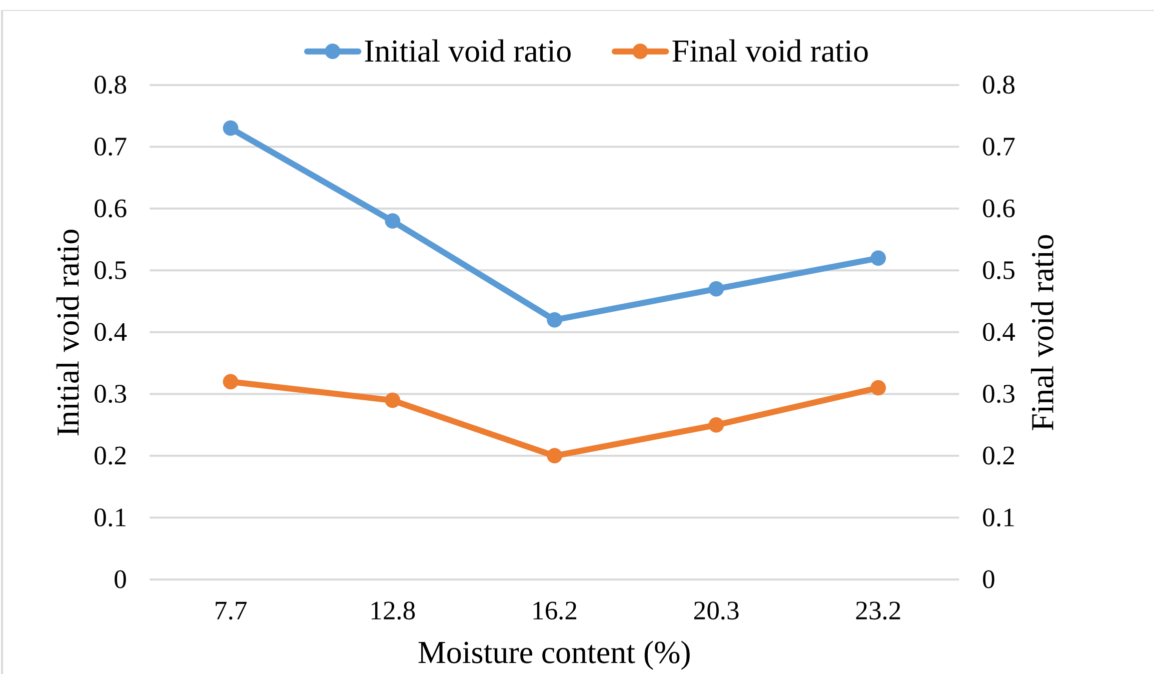

Figure 4. 35: Initial and final void ratios vs moisture content of treated soil

4.8.1. Compression and swelling indices

Figure 4.36 and 4.37 shows the test results of compression and swelling indices for both untreated and treated soils which depict that the compression and swelling indices (Cc) increase with an increase in the moisture content. The rate of increase is more for wet of optimum side than dry of optimum side for both these parameters. The compression index and swelling index vary from 0.086 to 0.27 and from 0.0074 to 0.021 for untreated soil and from 0.072 to 0.18 and from 0.007 to 0.014 for treated soils, respectively. The comparative evaluation of this test data shows that the treated soil provides lower peaks than untreated soils, and due to this higher consolidation potential, the treated soil provides higher strength than untreated soil.

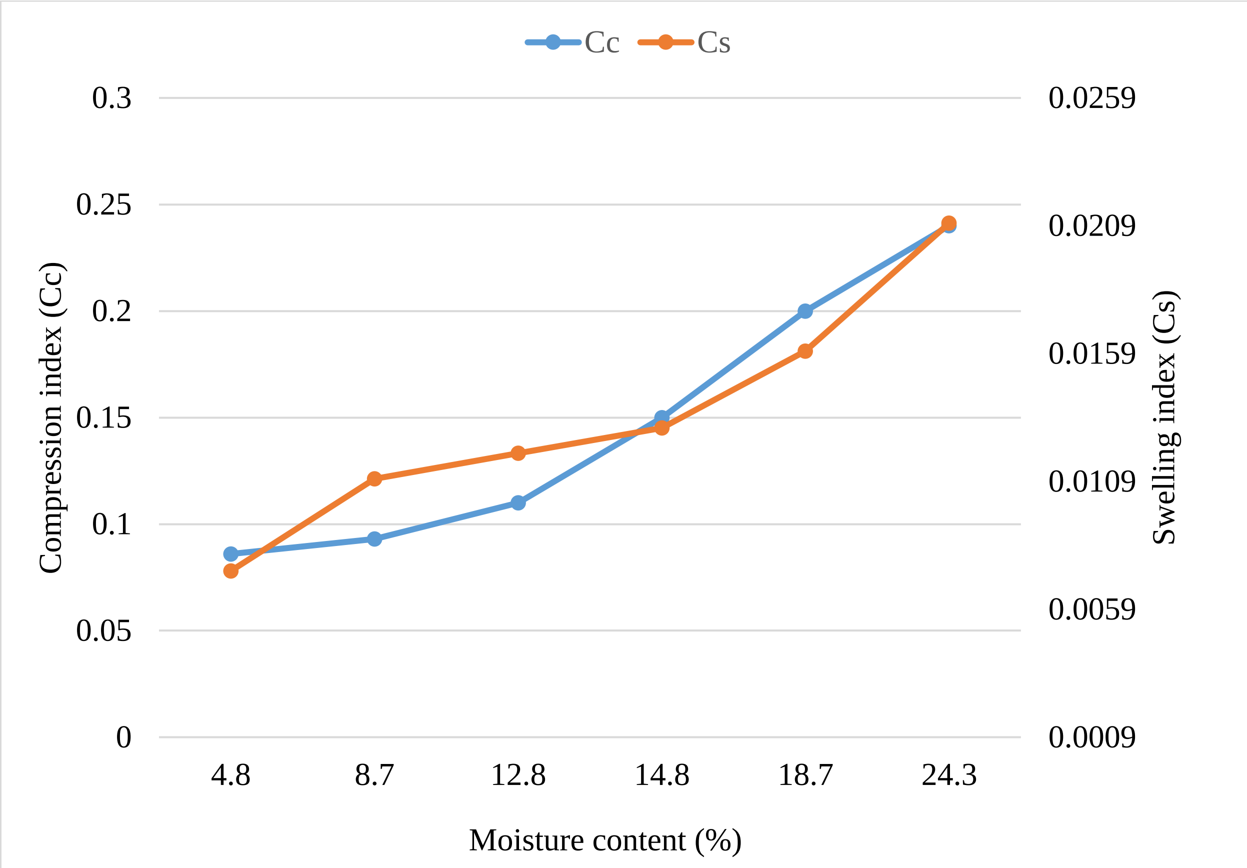

Figure 4. 36: Compression and swelling indices vs moisture content for untreated soils

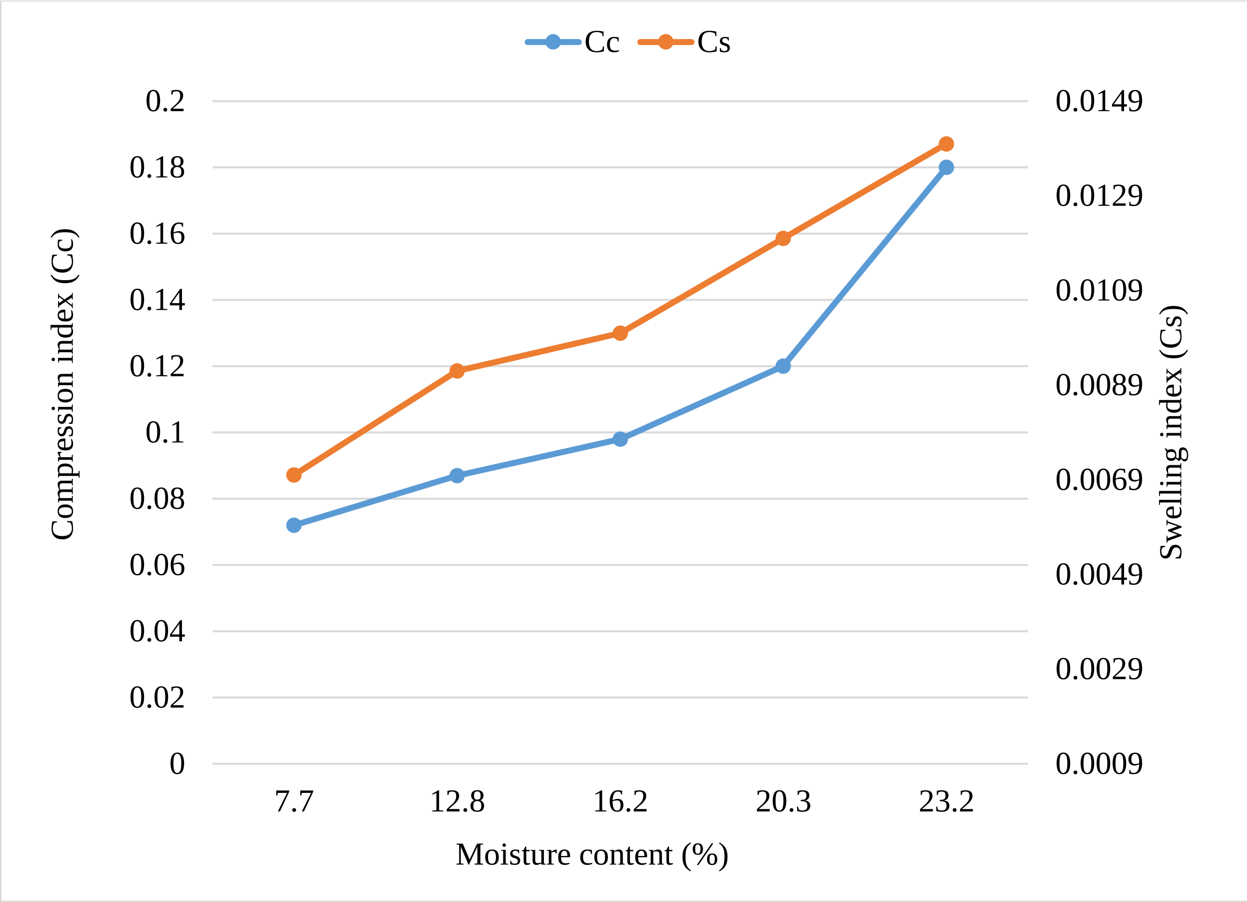

Figure 4. 37: Compression and swelling indices vs moisture content for treated soils

4.8.2. Pre consolidation pressure

It can be seen from Figure 4.38 that pre-consolidation pressure increases in both cases for an increase in moisture contents, and the treated soil provides relatively higher peaks than untreated soil for all data points of the compaction curve. Generally, for treated soils, the rate of increase in the consolidation pressure is relatively higher for dry of optimum side than wet of optimum side, similar to that of void ratio profiles (Figures 4.34 and 4.35). At optimal moisture content, the treated soil provides quite high pre-consolidation pressure (350 kPa) than untreated soil (220 kPa), which alternately results in different initial void ratios.

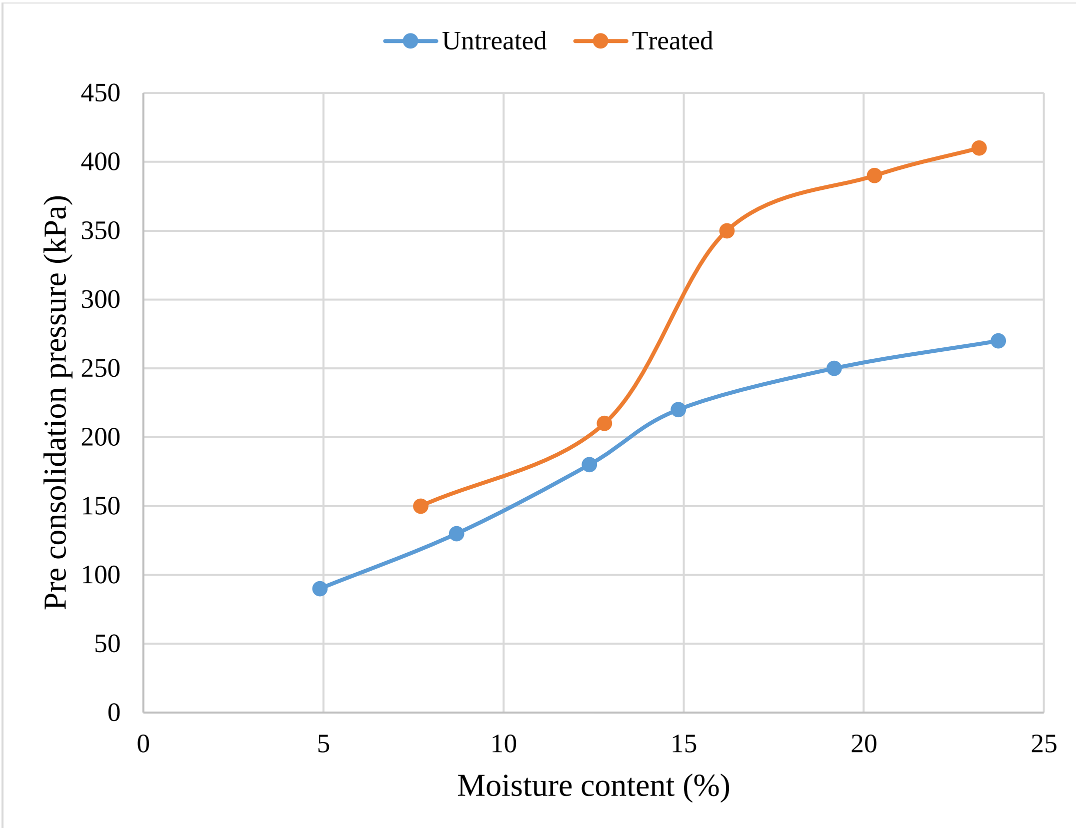

Figure 4. 38: Pre-consolidation pressure profiles for treated and untreated soils

CHAPTER 05

CONCLUSIONS AND RECOMMENDATIONS

5.1. General

This chapter reports the key conclusions and recommendations obtained from the data sets of this study

5.2. Conclusions

After the detail analysis of the soil taken from Kallar Kahar, it is concluded that soil spread around this region seems quite hard in its natural dry condition but are potentially collapsible when gets saturated. Furthermore, according to field and laboratory test data, the soil shows significant collapse potential.

As per USCS and AASHTO systems, the soil sample fell in the category of low plastic clay (CL) and (A-4), respectively.

The collapse potential of treated soil reduced from 50-60%, which alternately changed the soil behavior from sever trouble to moderate conditions.

The dry unit weight decreased and optimum moisture content increased up to a certain threshold of BA and GGBS, and after then, the compaction curve reversed, showing decrease in moisture contents and increase in unit weights.

The addition of additives minimized the collapse potential, changing the properties of soil from clayey to silty soils.

In both unsaturated and saturated conditions, for dry of optimum, cohesion increased and angle of internal friction decreased up to optimum moisture content and both cohesion and angle of internal showed a decreasing trend for wet of optimum.

The hydraulic conductivity increased from dry of optimum to wet of optimum side gradually for both treated and untreated soils and treated soil relatively showed lower hydraulic conductivity than untreated soils, which was due to differences in their void ratios

The treated soil showed more consolidation potential than untreated soil, due to which the compression index and swelling index of treated soil showed lower peaks than untreated soil.

5.3. Recommendations

The detail investigation shows that though additives modifies the collapsible potential from severe to moderate conditions but unable to provide significant changes in its hydro-mechanical for different dry unit weight, so this study suggests to use an alternative method to improve the engineering behavior of Kallar Kahar collapsible soils.

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