EVALUATION OF MECHANICAL BEHAVIOR OF REINFORCED EXPANSIVE SOIL

By

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DEDICATED

TO

MY BELOVED PARENTS,

MY YOUNGER SISTER MARIA ISMAIL

&

MY LATE GRANDMOTHER

ACKNOWLEDGEMENT

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Abstract

Expansive soils undergo swell upon wetting and shrink upon drying, and the roads and the buildings constructed on these foundation soils often undergo differential settlements after certain period of time, which ultimately results in the collapse / failure of these structures. It is unavoidable in certain situations to use such types of soils, so, in this scenario, It is critical to improve these soils' performance properties. Different researchers used different additives such as jute fiber, palm fiber, lime fly ash silica fume, rubber powder, wheat straw ash, coconut fibers and ground nutshell etc., to enhance the performance properties of these soils. Despite this, this area still needs significant attention by engineers and scientists, innovating new technologies and methods to stabilize these soils on economic ground. 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 has the ability to reduce the swelling potential of expansive soils while also improving soil stability through pozzolanic activities. So, the novelty of this research work involves examining the mechanical behavior of reinforced expansive soil. The bagasse ash and ground granulated blast furnace slag (GGBS) were used in the study. To examine the behavior of reinforced expansive soils, various testing approches such as index properties, direct shear, unconfined compressive strength, and oedometer tests were used. For a steady rise in the proportion of reinforcement up to a particular threshold, i.e., up to 3% of GGBS and 12% of BA, optimum moisture content increased, and generally maximum dry unit weights decreased. The optimum moisture content increased from 23.2 % to 26.78 % and maximum dry unit weight decreased from 15.84 kN/m^2 to 15.34 kN/m^2. In both soaked and unsoaked conditions, the cohesiveness reduced, and the angle of internal friction increased as the percentages of reinforcement increased. Similarly UCS increased from 98.4 kPa to 246.34 kPa

with a decrease in swelling potential from 9.75 to 3.75 and it was maximum at 12% of BA and 3% of GGBS. The treated expansive soil's strength increased due to changes in soil behavior from clayey to silty, as well as a decrease in swelling potential and PI as well. The results of the tests revealed that ground granulated blast furnace slag and bagasse ash are effective additions for improving soil mechanical behavior.

Chapter 1

Introduction

1.1 General

Expansive soil is considered as a problematic soil as it undergoes shrinkage and swelling upon drying and wetting, respectively. This behavior of expansive soil is generally associated with its mineralogy or chemical composition. The mineral montmorillonite generally dominates the network structure of this soil, which has the tendency to undergo large swelling and shrinkage. Expansive soil shows great tendency to absorb water and alternate significant expansion due to the presence of the fine grains and week bonding between the particles. Expansion of clay is far more damageable than the natural hazards. It is twice more damageable than the damage caused by the natural hazards in combination **(Jones & Holtz, 1973)**. Due to this property of the expansive soil, it is generally considered unsuitable for construction purposes.

Due to their shrinkage and swelling potential behavior upon their drying and wetting respectively. These soils which alternatively cause differential settlement in foundation soil. In the connection with the development projects, the acquiring of land is increasing and the soils with such properties are not acceptable for the development projects. With the reinforcement, engineering properties of these soil can be improved to encounter structural requirements. Additives such as, ground granulated blast furnace slag and bagasse ash were used in this study to reinforce the soil. The GGBS and 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 such soils, enhancing their stability through pozzolanic activities. Bagasse is the fibrous residue left over from the extraction of sugar cane juice. This fibrous waste is used as a fire fuel to heat the boilers, with Sugarcane Bagasse Ash (SCBA) as the byproduct. Because of its high silica and alumina content, SCBA is a pozzolanic material **(Payá et al., 2002)**. GGBS is a by-product obtained in the production of steel and pig iron. According to ACI -116R, GGBS can be defined as "nonmetallic product of calcium silicates and other bases , 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 Nandipur, Gujranwala, which is well known in the region as a problematic soil. The moisture changes in these soils results in the severe movement of the mass, and any structure built on these soils experiences cracking and progressive damage due to differential settlement. Bagasse ash was collected for this study from Sheikho sugar mill Kot Addu Muzaffargarh and GGBS from Agha steel mill Karachi, Pakistan. The novelty of this research work involves examining the mechanical behavior of fiber reinforced expansive soil.

Regarding this, several laboratory testing approach such as index properties tests, direct shear test, unconfined compressive strength tests and oedometer tests were carried out to get the set objectives. The results of the tests revealed that the addition of additives decreased liquid limit and plastic limit, which was due to the changes in the behavior of soil from clayey to silty soil. The optimum moisture contents increased, and relatively maximum dry unit weights decreased for a gradual increase in the percentage of reinforcement up to a certain threshold, i.e., 3% GGBS and 12% BA. The UCS increased from 98.4 kPa to 246.34 kPa, and maximum at 12% of BA and 3% of GGBS. Similar to this, the cohesion of soil decreased, and angle of internal friction increased with the reinforcement of the soil with maximum shear strength at 12% BA and 3% GGBS in both soaked and unsoaked conditions. The compression index and swelling index of the soil decreased with an increase in percentage of additives, too. The test results showed that GGBS and BA are suitable additives, which can be used to increase the mechanical behavior of expansive soil.

1.2 Problem statement

Scientists, engineers and policy makers spend billions of dollars each year throughout the world for an effective management of domestic and commercial wastes, but despite this, the growing volume of these wastes is a big headache due to swift growth of population and modern industrialization. The fact is that our landfills are filling up, the planet is being polluted and the existing nonrenewable resources will not last longer. As a result, there is a strong need to develop environmental acceptable materials, prompting various studies in recent years to investigate the use of natural fibers for soil reinforcement. The term "eco-composite" emphasizes the value of natural fiber in the modern industry.

1.3 Research objectives

The main objective of this research is as under:

To investigate the mechanical behavior of reinforced expansive soils.

1.4 Justification of the research

Pakistan is an agricultural country. Agriculture contributes about 24% to the GDP of this country, and the annual production of sugarcane is about 64.77 million tons (Pakistan Economic Survey 2019-2020), resulting in the production of huge quantity of BA, that needs to be properly dumped or used in an efficient manner to lessen its environmental impacts. The best approach is to add it in expansive soils to improve their properties to use in highways, runways and other foundation structures. Similarly, Pakistan produces ground granulated blast furnace slag in huge quantity each year due to its extensive steel mills network, and it also needs to be managed properly to mitigate its environmental impacts So, the study has two-folds objectives, i-) BA and GGBS are used to amplify the stability of expansive soil, ii-) it will put a positive impact on the environment, using these traditional wastes in a productive way rather than to throw away as garbage.

1.5 Thesis outlines

The chapters in this thesis are outlined as.

1. Chapter 1 highlights the research background, problem statement, objectives, and the justification of the research work.
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, in lieu of experimental work.
5. Chapter 5 represents the conclusions with few key recommendations, drawn from the study.

Chapter 2

Literature Review

2.1 General

Expansive soils have the potential to undergo a change in volume with the change in moisture content. Due to the lift-up potential of these soils, they can cause potential damage to infrastructures due to the differential settlement. Due to differential settlement, moisture changes in expansive soil cause considerable movement of the mass, causing cracking and progressive destruction to any structure built on these soils., e.g., a road in Sudan showed a differential settlement of up to 15%, when the expansive soil beneath the road got wetted due to infiltration of rainwater **(Zumwari, 2015)**. The swelling and shrinkage effects of the soils on the road are represented in Figure 2.1.

Figure 2.1. Pavement appearance constructed on expansive soil subgrade

The swelling potential of these soils is significantly associated with the clay mineral, i.e., montmorillonite, which has a great potential to absorb water compared to smectite and illite. The composition of expansive soil is chiefly composed of montmorillonite, illite, and smectite. The soil composed of montmorillonite is generally classified as highly plastic soil, which tends to swell and shrink upon wetting and drying. Expansive soil has a structure like montmorillonite which will affect the soil to swell or shrink with changes in moisture level. So, it is essential to enhance the performance properties of these soils, while used as a construction material to counterfeit the infrastructures' failures, taking place due to their differential settlement. These soils' performance attributes can be improved through soil stabilization, and different methods such as mechanical, chemical, and biological are employed in this regard.

Biological soil stabilization is a new technology for soil stabilization that is both ecologically benign and effective. Due to their metabolic mechanisms, several microorganisms found in the natural soil environment can form cementations (binding material). To see if biological treatment had any effect on the shear strength of swelling, soil samples were treated with bacillus sphaericus **(Saffari et al., 2017)**. As the bacterial concentration increased, shear strength surged as well.

Mechanical stabilization, such as compaction, is another method to stabilize soft soils. Different types of mechanical stabilization techniques such as dewatering, soft soil replacement, and compaction with various mechanical machinery are also used to stabilize the soil.

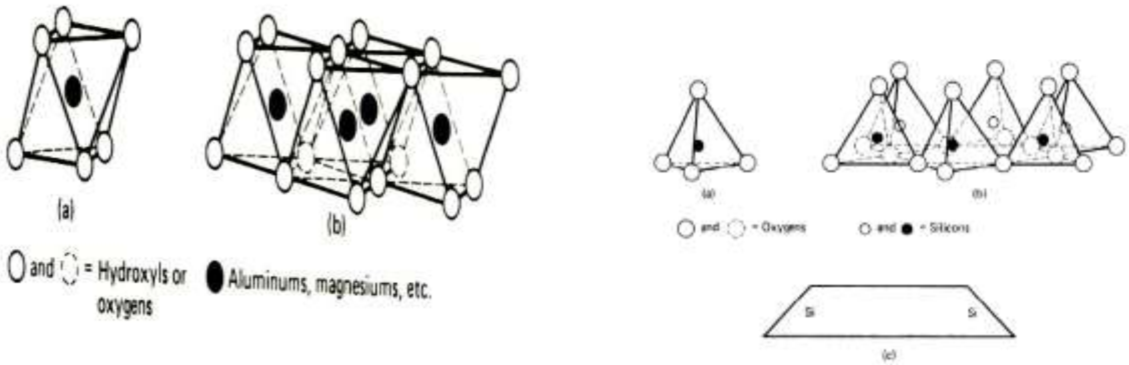
Stabilization by chemical means is a simple process. Chemical stabilization uses a variety of chemicals in varying percentages to stabilize expansive soils. Rice husk, bagasse ash, wheat straw ash, and nutshell, among other natural byproducts, are readily available chemical stabilizers. The proper disposal of these traditional wastes is essential; otherwise, they adversely impact the environment. Various researchers have been using bagasse ash, wheat straw ash, rice husk, lime, and other cementitious products in different combinations to enhance the strength properties of the soils. Each method has its merits and demerits, such as it is challenging to produce bacteria in the lab for biological stabilization. There is a higher cost associated with mechanical stabilization. Comparatively, traditional chemical stabilizers are readily available, and their use as a reinforcing agent may also help reduce the greenhouse gas load from the environment.

2.2 Clayey soil

The term clay generally refers to the size and mineralogy of the soil. The soil particles having a size smaller than 0.002mm are known as clayey soil. The clay mineral generally provides net negative electric charge, plasticity, and higher resistance to the weathering. The clayey soil is generally formed due to the weathering of rock, which alternately defines the chemical composition of clayey soils. These types of soils are low permeable soil, and the nature of clayey soil is plastic. The permeability and swelling potential of the clayey soil is always higher than the sandy soils, and comparatively, their strength is low.

2.2.1 Clayey structure

Clay minerals are formed due to the chemical weathering of the rocks, and the rock type generally defines the chemical composition of the clayey soil **(Barton & Karathanasis, 2002)**. The basic structural units of clay minerals include alumina and silica. The silica (tetrahedral) is composed of four oxygen atoms, each connected to a silicon atom. Similarly, alumina (Octahedral) is made up of six atoms of oxygen, directly related to either aluminum, iron, magnesium, or other atoms **(Holtz & Kovacs, 1981)**. Figure 2.2 shows a single unit of octahedral mineral.



(a) (b)

Figure 2.2. Single unit; (a) tetrahedral mineral, (b) octahedral mineral

2.2.1.1 Kaolinite

Kaolinite is also known as 1:1 clay mineral, consisting of one octahedral (Al-O) and one tetrahedral sheet (Si-O). $Al_2Si_2O_5(OH)_4$ is the stoichiometric formula of kaolinite. The hydrogen bonding unites different crystals of kaolinite **(Kotal Bowmick, 2015)**. Figure 2.3 shows structural unit of kaolinite.

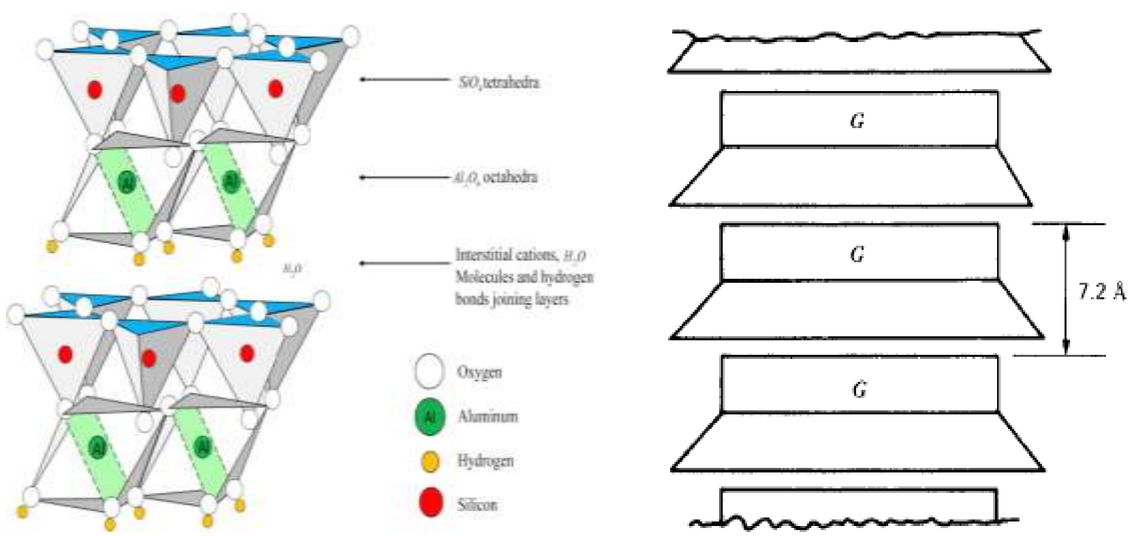


Figure 2.3. Structure of kaolinite

2.2.1.2 Montmorillonite

Montmorillonite, also known as a nanolayered structure, occurs in stacked layers with a thickness of 1nm. This clay mineral is generally composed of two tetrahedral sheets (O-Si-O) and one (O-Al (Mg)-O) octahedral sheet. The montmorillonite is also known as smectite. Generally, the Vander Waals forces contribute to hold the silica with oxygen in a tetrahedral sheet and magnesium with oxygen in an octahedral sheet. Montmorillonite minerals have a greater surface area and a high aspect ratio. Figure 2.4 shows the structure of montmorillonite.

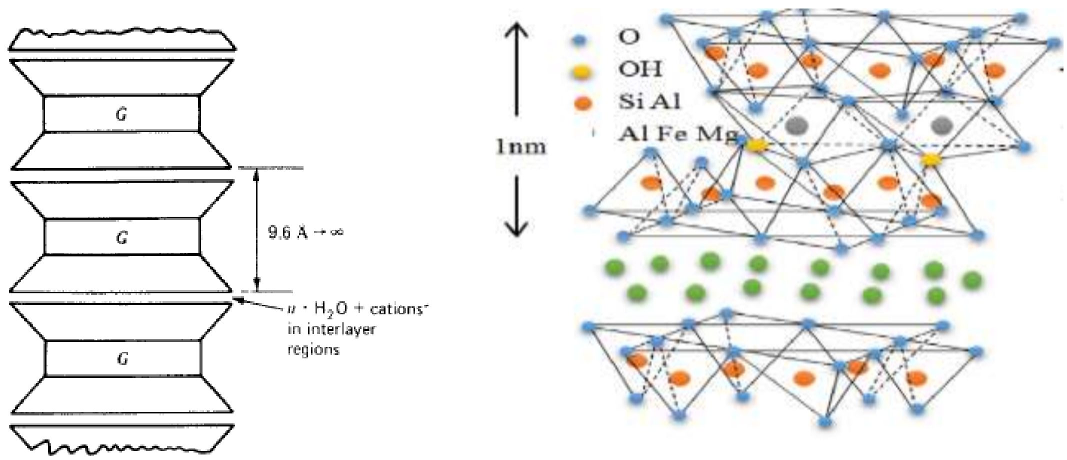


Figure 2.4. Structure of montmorillonite

2.2.1.3 Illite

The illite structure is similar to montmorillonite structure, which is composed of 2:1 clay mineral, and the interlayers are bonded together with a potassium atom. The crystal structure of illite resembles the mica minerals, but comparatively, due to less potassium concentration, these minerals are chemically more active than micas. Figure 2.5 shows the structural composition of the illite mineral.

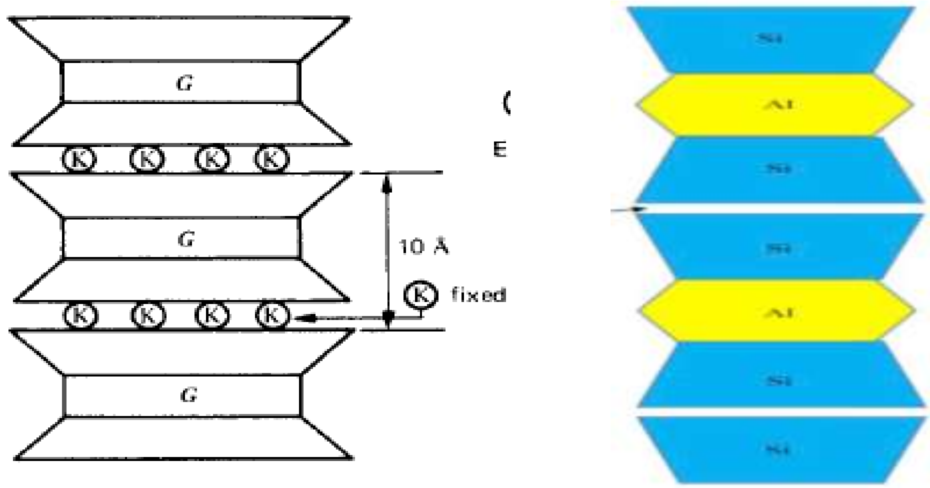


Figure 2.5. Structure of illite

2.2.2 Properties of clayey soil

Clayey soils are well known for their lower bearing capacity and higher compressibility, shear strength, shrinkage & swelling potential, and plasticity. A brief introduction of some valuable properties of clayey soil is as follows.

2.2.2.1 Grain size distribution

The percentage of distinct particle sizes in a soil sample is known as particle size or grain size distribution. The grain size distribution of soil particles substantially impacted all soils' engineering behavior **(Tyler & Wheatcraft, 1992)**. To classify the grain size distribution of soils, sieve analysis, and hydrometer tests are commonly used.

2.2.2.2 Atterberg's limits

The presence of water shows a substantial impact on the engineering behavior of fine-grained soils. Different soils show different Atterberg limits due to their unique water absorption capacity. The liquid limit and plastic limits are essential properties used to estimate activity, liquidity index, and plasticity index, critical parameters to categorize soil behavior. The classification of fine-grained soil with liquid limit is shown in Table 2.1.

Table 2.1. Classification of fine-grained soil with liquid limit

Soil Type	Liquid limit
High plastic clay	LL > 50
Medium plastic clay	30<LL>50
Low plastic clay	LL<30

2.2.2.3 Cation exchange capacity (CEC)

Cations that neutralize the net negative charge on soil particle surfaces are easily exchanged with other cations in water. In the exchange reaction, the electrovalence of cations and the relative concentration of cations in water are both critical considerations. The cation exchange capacity of soil, which is measured in milliequivalents per 100 g, is the measure of soil particle net negative charge induced by isomorphous substation and broken bonds at the margins. Table 2.1 shows the cation exchange capacity values for the most common clay minerals **(Mesri, 1996)**.

Table 2.2. Cation exchange capacity of clay minerals

Minerals	Cation exchange capacity (meq/100 g)
Kaolinite	0.03-0.1
Illite	0.2-0.3
Chlorite	0.2-0.3
Attapulgite	0.2-0.35
hydrated halloysite	0.4-0.5
Montmorillonite	0.8-1.2

2.2.2.4 Compaction characteristics

Proctor tests, both standard and modified, are commonly used to assess the maximum dry density and optimum moisture content of soil **(Juran & Guermazi, 1988)**. The compaction of the soil removes air which alternately increases the soil strength. The OMC & MDD are related to the compaction curve **(Linderbug, 2012),** which alternately defines soil behavior as each soil has a unique compaction curve. Table 2.2 represents the dry unit weight and optimum moisture content for soil types, which shows that higher unit weight and lower water content represent good soil, and similarly, less unit weight and water content represent poor soil.

Table2.3. Typical Values of Maximum dry unit weight and Optimum moisture content

Soil (USCS)	**Maximum Dry unit weight (lb./ft^3)**	**Optimum Moisture content (%)**
Well Graded Gravel	125-135	8-11
Poorly Graded Gravel	115-125	11-14
Silty Gravel	120-135	8-12
Clayey Gravel	115-130	9-14
Well Graded Sand	110-130	9-16
Poorly Graded Sand	100-120	12-21
Silty Sand	110-125	11-16
Clayey Sand	105-125	11-19
Non-Plastic Silt	95-120	12-24
Medium Plastic Clay	95-120	12-24
High Plastic Silt	70-95	24-40
High Plastic Clay	75-105	19-36
Organic Clay	65-100	21-45

2.2.2.5 Swelling potential

The swelling potential is an important property to determine the engineering behavior of soil. The higher the swelling potential, the more will be the sensitivity of the soil, which would alternately result in the poor performance of the soil. Equation 2.1 shows a relationship between swelling potential and plasticity index **(Seed et al., 1962)**, and Table 2.3 shows the swelling potential with respect to the sensitivity of the soil.

$$\mathbf{S = K\,(M)\,(PI)^{2.44}} \qquad \textbf{Equation 2.1}$$

Here, K = 3.6 x 10^{-5-} , M = 60 for natural soil and 100 for artificial soil, and PI= plasticity index. According to the swelling potential of soil, the author classified soil as given below,

Table 2.4. Characterization of soil sensitivity for swelling potential

Swelling Potential	Nature of soil sensitivity
>25	Very high
5-25	High
1.5-5	Medium
<1.5	Low

Equation 2.2 represents an empirical approach between liquid limit and cation exchange capacity of soil to classify swelling potential as discussed in **(Yilmaz, 2004)**.

$$\mathbf{CEC = e^{(2.63+0.02LL).}} \qquad \textbf{Equation 2.2}$$

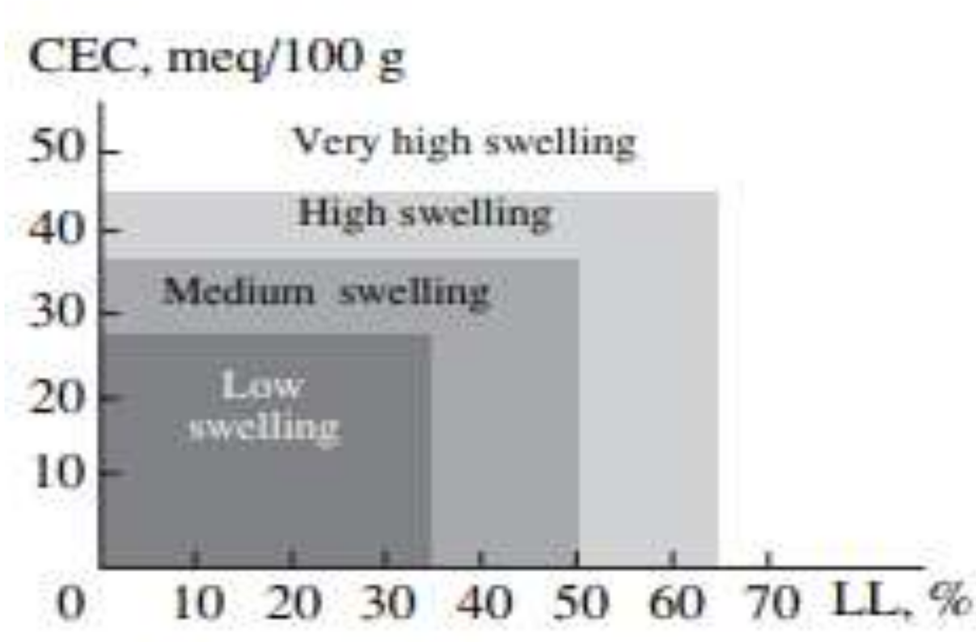


Figure 2.6. Swelling potential for liquid limit and cation exchange capacity relationships

Figure 2.7 shows the swelling behavior concerning dry density and liquid limit as proposed by **(De Nelovi, 1964)**. The swelling potential of soil is defined by a strong link between dry density and liquid limit. As in table 2.3, a soil with swelling potential > 25 and less than 1.50 classifies it as a very highly sensitive and lower sensitive soil, respectively. Similarly, soils with liquid limits > 50 and < 30 represent the high plastic and low plastic soils, respectively.

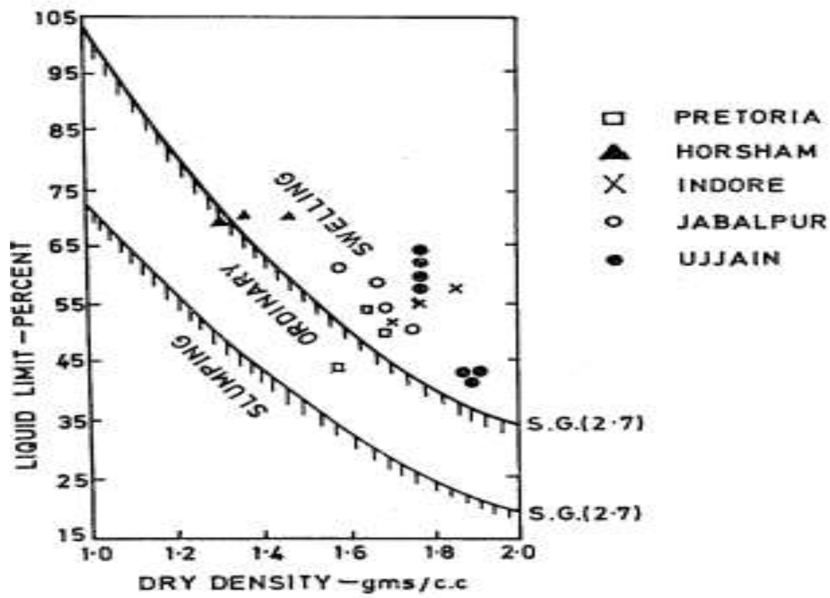


Figure 2.7. Swelling behavior of different soils

2.3 Soil reinforcement

Soil stabilization is commonly used to enhance the engineering properties such as shear strength, plasticity, bearing capacity, swelling potential, and consolidation property of the expansive soil, and the mechanical, chemical, and biological methods are employed in soil stabilization. Mechanical stabilization techniques include compaction, drainage, pre-loading, and others to increase the engineering qualities of soil. In addition, in chemical stabilization, various synthetic and traditional chemicals reinforce the soft soils. These additives develop pozzolanic activities due to their cementitious nature. Generally, the clay particles are negatively charged. These negatively charged particles repel each other, causing soil dispersion. The positively charged cations can hold the negatively charged soil particles together, which results in flocculation or agglomeration of soil particles.

Several conventional approaches, like mechanical stabilization and chemical admixture stabilization, are utilized in a wide range of situations to improve the geotechnical engineering qualities of soils on site. Mechanical and chemical stabilization processes increase the bearing capacity and strength of the soil while lowering its swell potential, hydraulic conductivity, liquid limit (LL), and plastic limit (PL). Mechanical stabilization is more common in Pakistan and requires less time than chemical stabilization. For soil stabilization, cement, fly ash, bagasse ash, lime, and other chemical stabilizers are widely utilized. Due to the hydrolysis process, the addition of a chemical stabilizer reduces the moisture content of the soil, lowering the plastic limit while increasing bearing capacity **(Dang et al., 2016)**. Several critical parameters such as stabilizer content and type, water content, temperature, mixing time, and curing period play an essential role in soil stability.

2.4 Bagasse ash as a pozzolans

2.4.1 Pozzolans

Separately, pozzolan is a siliceous and aluminous substance with minimal or no cementation properties. Still, in combination with water in powdered form, at room temperature, it interacts chemically with calcium hydroxide to generate other compounds, possessing sufficient cementations properties. According to ASTM C 618-05 standard, pozzolans are classified into different classes such as Class C, Class F, and Class N fly ash. Class C and Class F fly ashes relate to sub-bituminous and bituminous materials, respectively. Class N pozzolans are classified as natural or raw pozzolans. **(Abdullahi, 2007)** according to the report, Sugar cane bagasse ash meets the physical and chemical standards of Class N fly ash, as defined by ASTM C 618-05, which contains at least 3% moisture, 4% sulfur-trioxide (SO3), and silicon dioxide (SiO2), aluminum oxide (Al_2O_3), and iron oxide (Fe_2O_3) make up 70% of its composition.

2.4.2 Sugarcane bagasse as a pozzolans

The fibrous residue left over after sugar cane juice is removed is known as bagasse. The residues of this fibrous trash are Sugarcane Bagasse Ash, which is utilized as a fire fuel to heat the boilers (SCBA). Because of its high silica and alumina content, SCBA is a pozzolanic material **(Payá et al., 2002)**. The study further concluded that the factors including, degree of crystallinity of silica and bagasse ash's reactivity is generally controlled by the impurities in the ash. Furthermore, the pozzolanic reactions also greatly depend upon the temperature and moisture conditions **(Little et al., 1987).** The pozzolanic reaction occurs when calcium hydroxide reacts with silica and alumina. Because of the heterogeneous nature of fly ash and soil, these chemical reactions are complicated.

2.4.3 Potential of bagasse usage in Pakistan

Pakistan is an agricultural country that stands 5th in sugarcane production globally, producing more than 64.77 million tons of sugarcane every year **(Wade, 2004)**. Sugarcane is used in the sugar industry to the extent of 81 percent **(Akbar et al., 2006)**. Each ton of sugarcane yields about 26% bagasse and 0.62 percent residual ashes (at a moisture substance of half) **(Cordeiro et al., 2004)**. This percentage is at 50% moisture content. Pakistan produces approximately 11 million tons of bagasse and 0.26 million tons of bagasse ash from sugarcane cultivation. In Pakistan, BA is extensively utilized in concrete as a partial replacement for cement, soil reinforcement, and low-cost mud blocks in the construction of buildings.

Khan et al., (2018) SCBA can be utilized efficiently in road construction, according to the paper; nevertheless, its employment is contingent on the project site's financial resources and transportation expenses. (**Amin, 2010)** worked on replacing Portland cement with SCBA and concluded that well-burnet BA could be replaced with Portland cement, and 20% high strength can be achieved on the required properties of concrete. The replacement of BA showed several advantages over the cement usage in improving water permeability, early strength gain, and more resistance to diffusion and chloride penetration. **(Akram et al., 2007**) reported that bagasse ash combined with lime increased the UCS of soil strength up to 64%. The study further added that CBR of the soil was noted 4.5 times more when BA was used in combination with lime. Furthermore, a decrease in swelling potential was also reduced from 2.5 to almost close to zero.

2.4.4 Effect of bagasse ash on soil properties

2.4.4.1 Grain size distribution

Sharma & Sivapullaiah, (2016) carried their study on expansive soil with a 1% sand content, a 29% silt content, and a 70% clay content. **(Aamir et al., 2019)** investigated the properties of the sustainable soil with reinforcement using waste materials. 2 percent sand, 41 percent clay, and 57 percent silt were found in natural soil. According to the AASHTO classification, the soil was classified as A7-6. **(Irfan et al., (2018)** worked on the reinforcement of the expansive soil having USCS soil classification CH and CL. In CH sand percentage was 7%, silt percentage 41% and clay in 52%. **(Dang et al., 2016)** checked the behavior of expansive soil stabilized with hydrated lime and bagasse fibers. There is 0.1 percent gravel, 18.3 percent sand, and 81.6 percent fine-grained soil in this soil.

2.4.4.2 Atterberg's limit

Ashish et al., (2015) used bagasse ash to reinforce medium plastic clay and reported that with the addition of 10 % of bagasse ash, the liquid limit decreased from 35 percent to 26 percent, which alternately decreased the plasticity index from 13 percent to 9 percent. **(Gandhi, 2012)** BA to the soil reduced the liquid limit from 72 to 52 and the plasticity index from 42 to 27. Furthermore, the study also noted a decrease of 21 to 15% in the PI.

2.4.4.3 Compaction characteristics

Rajeswari et al., (2018) strengthened the sub-grade soil with bagasse ash and stated that maximum dry density decreased from 2.04 g/cm^3 to 1.92 g/cm^3 with the addition of 3% of BA. **(Chhacchia & Mittal, 2015)** for 28 percent bagasse ash, clayey soil showed a maximum rise in OMC and a reduction in MDD. The OMC was increased from 22.42 to 27.9 percent, and MDD was decreased from 1.82 g/cm^3 to 1.34 g/cm^3. **(Ashish et al., 2015)** also studied the effects of bagasse ash with medium plastic clay for compaction characteristics and concluded that for 10 percent BA, the OMC was increased from 15.3 to 18 percent, and the MDD was adjusted from 1.793 g/cm3 to 1.692 g/cm3.

2.4.4.4 Unconfined compressive strength of soil

Osinubi et al., (2009) stabilized lateritic soils with bagasse ash and reported that the unconfined compressive strength of the soil increased from 366 KN/m^2 to 943 KN/m^2 for 2% of additive when the soil was remained soaked for 28 days. The study highlighted the reason for this increase in strength: a cementitious compound formed between the soil and the additive. (**Sabat, 2012)** stabilized lateritic soils with bagasse ash and showed that the UCS of the lateritic soil increased from 301 KN/m^2 to 904 KN/m^2 for 2% of additive with 28 days of soaking.

2.4.4.5 Swelling potential

Ahmed et al., (2015) showed that with an addition of BA, the swelling potential decreased from 11.47 to 4.85 percent, which was observed maximum for 9% bagasse ash. **(Chhacchia & Mittal, 2015)** also examined the influences of bagasse ash on the swelling potential of the clayey soil and reported that the swelling potential decreased from 12.4 percent to 6.8 with an increase in the %ages of BA up to 24%, however after this treatment, a decrease in swelling potential was noted, i.e., from 6.8 to 7.6. **(Gandhi, 2012)** reported that for ten percent of the bagasse ash, a maximum decrease in the swelling index was noted from 150 to 80 percent.

2.5 Ground granulated blast furnace slag

Ground granulated blast furnace slag (GGBS) is a by-product obtained 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 other metal oxides such as sodium oxide, sulphur oxide, iron oxide and potassium oxide exist in GGBS, but their %ages are too low. Pozzolanic and physical properties of slag are significantly associated with the cooling process of slag. Due to the presence of silicates, the GGBS has the binding property in combination with other materials. Recently, GGBS is significantly used in building construction as a replacement for cement.

2.5.1 Potential of GGBS in Pakistan

Pakistan produces 2.89 million metric tons of steel every year. From 1 ton of basic steel, 0.2 tons of slag is produced. Pakistan steel mills generate 1.1 million metric tons of the total 2.89 million metric tons **(Ahmed et al., 2018)**. The per capita consumption of steel in 2020 is 36 kg in Pakistan, which is relatively low as per capita steel consumption in the world (229 kg). In the construction sector, GGBS is currently utilized to partially replace cement and aggregate. Ash obtained during this process is dumped in open spaces in Pakistan, which has a dangerous effect on the environment.

2.6 Effect of GGBS in combination with the reinforcing agent on soil properties

2.6.1 Specific gravity

Kumar et al., (2014) revealed that when 25 percent GGBS was mixed to the soil, the specific gravity increased from 2.56 to 2.63. **(Sinalkar et al., 2020)**, when the percentage of GGBS was increased by 10%, the specific gravity of laterite soil decreased from 2.92 to 2.57.

2.6.2 Atterberg's limit

Kumar et al., (2014) reported that as the percentage of GGBS increased, the PI of the soil declined, resulting in a 17 percent reduction in PI. (**Sinalkar et al., 2020)** showed that 25% GGBS, the PI and L.L of the soil decreased 16.67 and 25%, respectively, with 10% GGBS.

2.6.3 Compaction characteristics

The moisture content decreased from 14.33 to 12.46 and MDD increased from 1.71 g/cm^3 to 1.765 g/cm^{3}with an increase in the percentage of GGBS from 0 to 25%. The study further added that the increase in MDD was due to increased grain sizes, which alternately reduced the PI of the soil **(Kumar et al., 2014)**. **(Shamshad et al., 2019)** worked on two distinct types of soils and found that a maximum rise in dry density was recorded in both soils for 6 percent GGBS, and that this increase in MDD was related to a drop in the percentage of clay fraction.

2.6.4 Direct shear test

Soil shear strength characteristics are crucial in identifying the strength behavior of various soils. (**Salman, 2011)** in soaked conditions, the shear strength parameters were evaluated, and the soil cohesiveness was found to be 2.5 times that of the original, with a smaller drop in the angle of internal friction. (**Saravanan et al., 2017)** to try to stabilize expansive soil, a combination of GGBS and lime was utilized. As the proportion of binder grew, the angle of internal friction increased, and the cohesiveness of the soil reduced. A maximum increase in the angle of internal friction was seen with 1 percent lime and 4 percent GGBS. (**Sinalkar et al., 2020)** the effect of GGBS on laterite soil was studied, and it was discovered that with 10% GGBS, the angle of internal friction rose from 23.75 to 34.45 degrees., but after this treatment, it reduced. Similarly, the soil's cohesion reduced from 0.13 kg/cm2 to 0.0145 kg/cm2 up to 15% GGBS and then showed an increasing trend.

2.6.5 Unconfined compressive strength

Shamshad et al., (**2019)** The effects of GGBS on the UCS of two types of red mud soils were investigated, and it was revealed that the unconfined compressive strength of both soils rose as the amount of GGBS increased. For 24 percent GGBS, a rise of 260.93 percent in UCS was recorded after 28 days of curing. **(Sabat, 2012)** The compressive strength of expansive soil varied from 55 kN/m2 to 98 kN/m2 for 30 percent of the waste ceramic dust, according to the engineering properties of expansive soil. (**Agarwal & Kaur, 2014)** stabilized black cotton soil with terrazyme for various curing periods. The unconfined comspressive strength of soil increased up to almost 200 kPa for 1ml/kg of soil with optimum percentage of terrazyme for the curing period of 1 to 7 days.

The BA and GGBS were selected as reinforcing agents in the present study to strengthen the expansive soil, which is in contrast to the other studies in which authors have been utilizing bagasse ash in combination with other waste materials to improve the mechanical behavior of expansive soil.

Chapter 3

Materials and Methodology

3.1 General

A laboratory investigation was carried out to examine the treated mechanical behavior of expansive soil. Various laboratory testing approaches were used in this regard. The materials used and methods adopted in the research are described in this chapter.

3.2 Materials used

The materials such as expansive soil, bagasse ash (BA) and ground granulated blast furnace slag (GGBS) were used in the research work.

3.2.1 Soil

The soil used in this study was obtained from Nandipur, Gujranwala, which is well known in the region as a problematic soil. This soil is generally used for the preparation of cricket pitches in Pakistan, but in contrary to this, this soil is not suitable for the construction and agriculture purposes. The nature of Nandipur soil is expansive due to its swelling and shrinkage nature.

3.2.2 Swelling behavior of soil

First of all, swelling behavior of the soil was checked. For this check liquid limit and compaction tests were performed on the natural soil. Liquid limit of the natural soil was calculated as 62 and MDD of soil was determined as 1.615 g/cm^3. Upon checking on graph selected soil for research was found to be in swelling range. So, reinforcement of the soil is necessary.

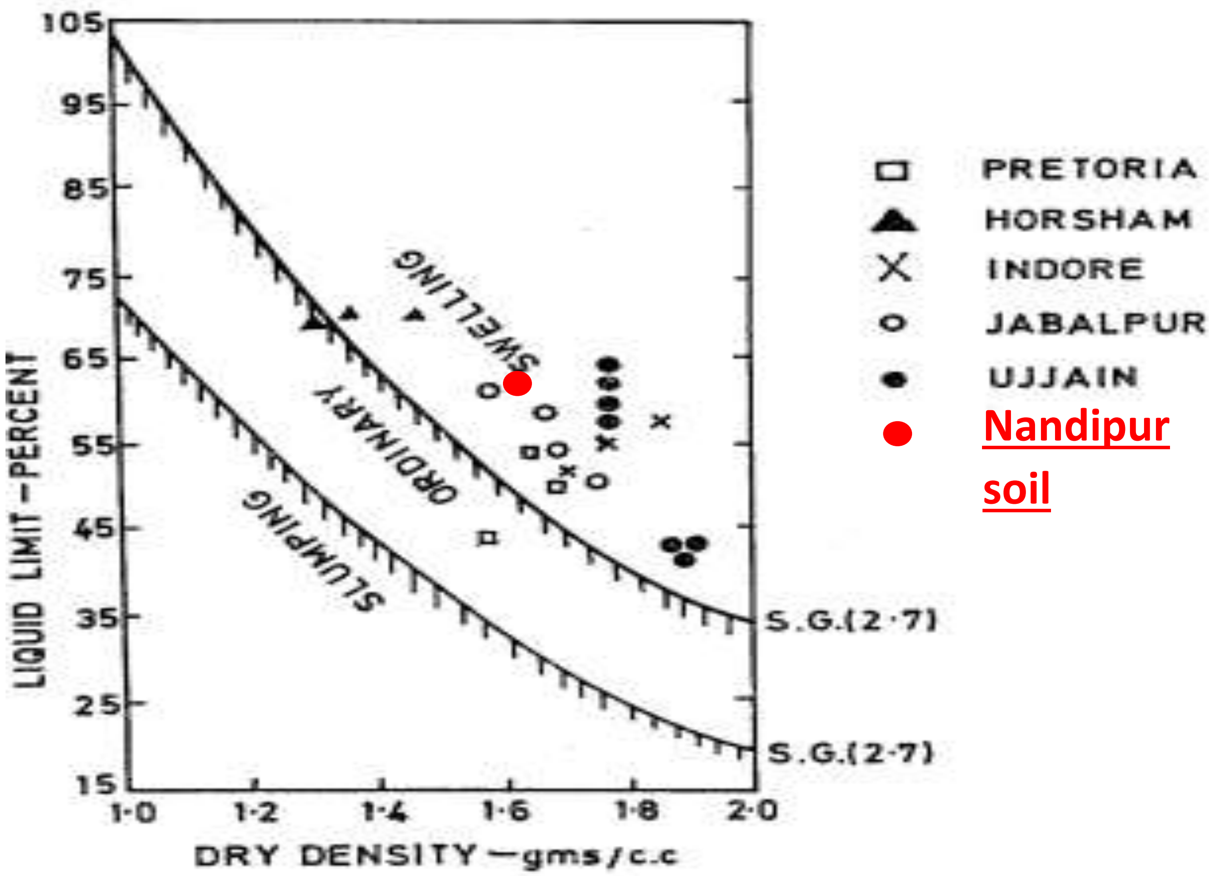


Figure 3.1. Swelling check for Nandipur soil

3.2.3 Bagasse ash

Residue obtained from the sugarcane industry from burring of bagasse in the boilers is called bagasse ash. After extracting juice from the sugarcane stocked, remaining fiber residue is known as bagasse. This sugarcane bagasse is then burnt in boilers to produced steam which is then utilized in the production of the electricity. Ash obtained in this process is known as bagasse ash. Production process of bagasse ash is shown in Figure 3.2. Bagasse ash is silica dominate product with some percentage of aluminum and iron oxides. Bagasse ash is generally consisting of 70 % silica which gives it pozzolanic behavior. After obtaining sugar from sugarcane, bagasse is obtained which is then powdered into ash by burning. Bagasse ash has pozzolanic nature and now a days it is used in place of cementitious materials (BA). the Bagasse ash used in this study was taken from Sheikho sugar mill Kot Addu district Muzaffargarh.

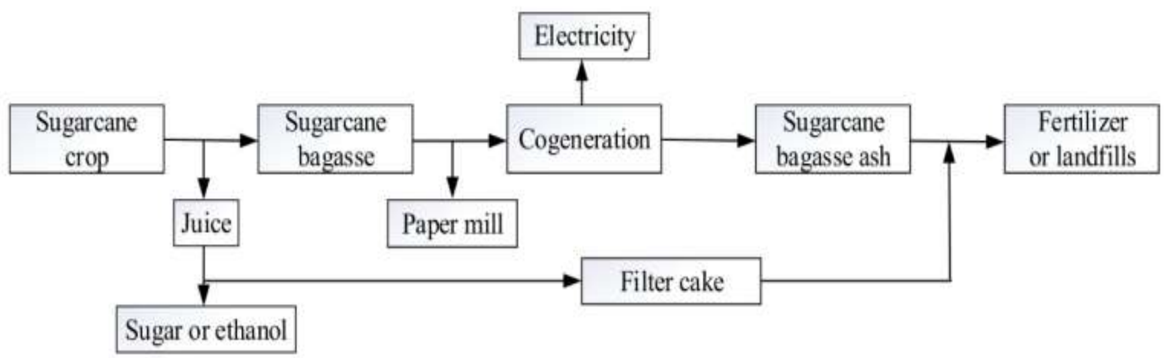


Figure 3.2. Process of production of bagasse ash

Bagasse ash is generally dumped in open air, which shows its detrimental impact on the environment. The Punjab province report sixty five percent of area of the sugarcane area from which 46.725-million-ton sugarcane produced. Total production of sugarcane in the Pakistan is 71.725 million ton **(Shafiq et al., 2020)**.

Figure 3.3. Pictorial view of Bagasse ash (BA)

3.2.3 Ground granulated blast furnace slag

As a by-product, the iron and steel industry produce ground granulated blast furnace slag. Blast furnaces are used in the manufacturing of iron and steel. Iron is melted at 1500^o in blast furnaces. The molten iron slag is then quenched using water or steam. A crystalline structure is generated as a result of this process. After this rapid cooling of crystalline structure is done and then it is grounded in fine powder. Grinding of slag is done in rotating ball mill. This fine powder obtained from the grinding process is called ground granulated blast furnace slag (GGBS). GGBS is a highly cementitious by product which has high percentage of calcium hydrate. GGBS used in this study was obtained from Tarbela dam where it has been used for the construction purpose. GGBS used in Tarbela dam was obtained from Agha steel mill Karachi.

Ground granulated blast furnace slag is the byproduct of the iron obtained from blast furnaces which are used for the manufacturing of iron. Blast furnaces which function at 1500^o are reinforced with iron and limestone. In blast furnaces molten slag and molten iron is obtained in this process. The molten slag usually consists of alumina and silicates. After this slag is cooled in high pressure water jet. After cooling then grinding of slag is done in rotating ball mill. After this white powder is obtained called as ground granulated blast furnace slag (GGBS). GGBS used in this study was obtained from Tarbela dam where it has been used for the construction purpose. GGBS used in Tarbela dam was obtained from Agha steel mill Karachi.

Figure 3.4. Pictorial view of ground granulated blast furnace slag (GGBS)

3.3 Methods used

In the research methodology, different type of tests on the natural soil and reinforced soil were performed in laboratory following the guidelines of different ASTM standards. Different test performed on natural and reinforced soil is given below,

3.3.1 Grain size distribution

The mechanical analysis of soil was carried out performing the sieve analysis and hydrometer tests, following the guidelines as set out in ASTM standards. First of all, a soil sample was dried in the oven for 24 hours. After drying the soil, it was pulverized with hand and hammering. First, the sieve analysis was performed following the procedure as discussed in ASTM Standard 422. The hydrometer test was then performed following the guidelines as set out in ASTM D 7928-16 standard, and for which the soil passing through # 200 sieve was used. For hydrometric analysis, a dispersing agent like sodium hexa-meta-phosphate was used for dispersion of soil particles. Figure 3.5 shows that the hydrometer tests is in progress.

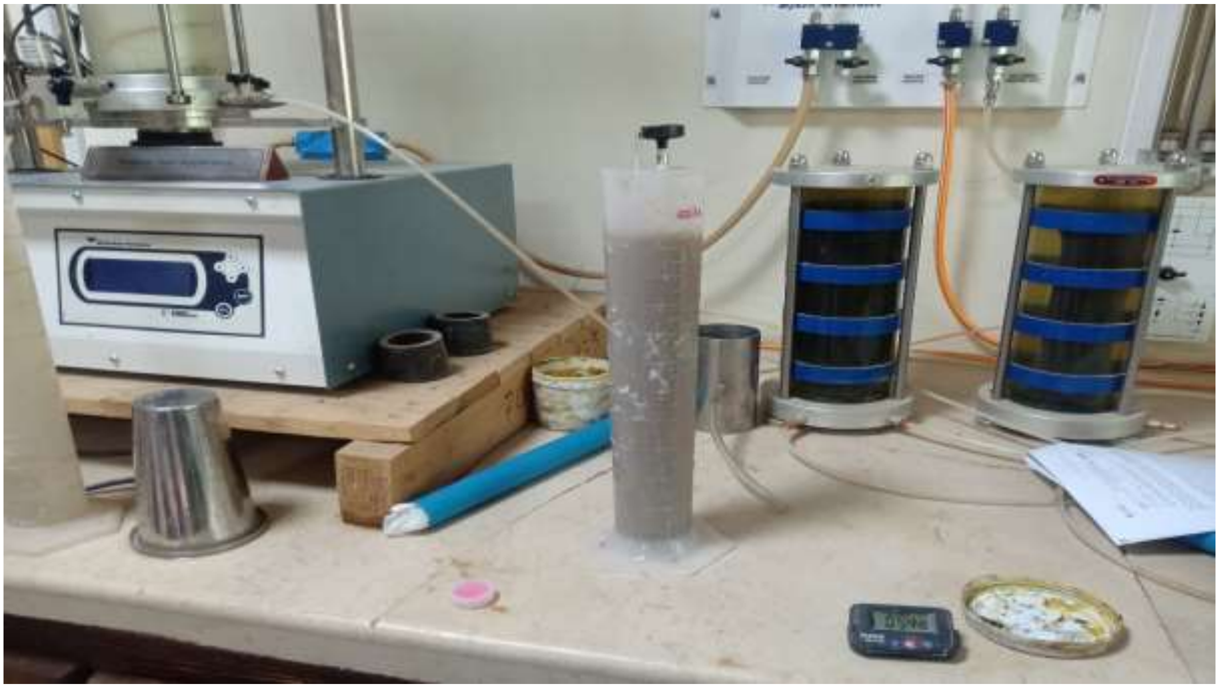


Figure 3.5. Hydrometer analysis in progress

3.3.2 Atterberg's limit

The Atterberg limit tests of both natural and treated soils were performed following the guidelines as set out in ASTM D 4318 standard. First of all, fifty grams of natural soil passing through sieve # 40 was carried out for the determination of liquid limit. When water was added to the soil to make the test paste, the soil stuck to the finger and had to be scraped off using a sharp knife.

In the next stage, the liquid limit was determined of amended soil with various concentrations of reinforcement agents such as BA and GGBS. The LL of treated soil was examined to test the changes in its performance behavior with various dosages. In first instance, the percentage of ground granulated blast furnaces slag was kept fixed as 1 % and percentage of bagasse ash was varied as: 4%, 8%, 12% and 16%. Similarly, the %age of GGBS was changed to 2% and 3% for different concentrations of BA. In this approach, the samples for determining the liquid limit of reinforced soil with various percentages of additives were prepared. It was concluded that once the additives were added, soil particles were unable to attach to fingers, demonstrating behavior that differed from that of natural soil.

Following the parameters outlined in the ASTM D 4318 standard, the plastic limit of both treated and untreated soils was obtained. For each sample, duplicate samples were made, and the values were averaged to obtain valid test data. The difference between the liquid and plastic limits for both treated and untreated soils was used to calculate the plasticity index. The plasticity index is an important index property since it indicates the soil's ability to swell.

3.3.3 Specific gravity

The ASTM D 854 –14 standard was used to calculate the specific gravity of natural soil.

Figure 3.6. Specific gravity test in progress

3.3.4 Compaction tests

The moisture – density relationships were figured out by performing the standard compaction tests for both natural and reinforced soils, and regarding these guidelines as discussed in ASTM D 698-12 standard was followed. Initially, a tray was filled with 04 kg of soil that had passed through sieve #4, and water was carefully added to the soil sample, ensuring soil-water uniformity. While mixing water with the soil, the soil particles got stick with fingers, so a knife was used to clean the soil form the fingers. After ensuring soil water uniformity, the mixture was transferred to the compaction mold, and the test was completed following the guidelines as mentioned in the standard. Similarly, the compaction curve for natural soil was completed. Duplicate samples were prepared for each data point to get the test accurate results.

Similarly, the compaction tests of treated soils for several concentrations of BA and GGBS was performed following the guidelines as set out in ASTM D 698 –12 standard. For these tests, 1 %, 2% and 3% of GGBS was trialed with various concentrations of BA, such as from 4 % to 16%. Duplicate samples were prepared for each water content to get the accurate test results. There was a total of 13 compaction tests performed on both natural and treated soils. With variations in additive concentrations, the dry unit weight fell, and the water content increased until it bounced back to the greater dry unit weight, which was the transition point for this soil. Finally, the moisture – density relationships were plotted to do a comparative evaluation of all these curves.

Figure 3.7. Standard proctor test in progress

3.3.5 Direct shear test

Direct shear tests for both treated and untreated soils were conducted according to ASTM D 3080-03 guidelines to obtain shear strength characteristics such as angle of internal friction and cohesion. Both soaked and unsoaked consolidated undrained tests were performed in this regard, for consolidated undrained conditions. First of all, the unsoaked and soaked samples were prepared at 95 percent of the compaction curve's maximum dry density, and after then, transferred to the shearing box. Size of the box used in direct shear test was 6.032 x 6.032x 2.58 cm^3. The shear strength parameters of treated soils for several concentrations of BA and GGBS were determined. For these tests, 1 %, 2% and 3% of GGBS was trialed with various concentrations of f BA, such as from 4 % to 16%. Duplicate samples were prepared for each water content to get the accurate test results. IA total of 26 direct shear tests for natural and reinforced soils were performed. Initially, the samples were consolidated at 50 kPa, until they reached their maximum consolidation potential, and then sheared at normal stresses of 50, 100 and 200 kPa. A shearing rate of 0.05 mm/min was applied to shear the specimens. The shearing rate of this soil was assessed by performing an oedometer test, as described in the ASTM D 2435-11 standard. For soaked conditions, the samples were remained soaked in the shearing box, during the consolidation and shearing processes. Normal stress and displacement measurement were noted after each 20 seconds. Finally, the stress – strain and shear strength parameters relationships were plotted from this test data. It was observed while preparing the samples that the wetter soil particles got stick with fingers, and a knife was used in this respect. The stickiness of natural soil was found to be significantly higher than that of reinforced soil. The pictorial view of the direct shear test is shown in Figure 3.7.

Figure 3.8. Direct shear test in progress

3.3.6 Unconfined compressive strength tests

The unconfined compressive strength tests for both natural and reinforced soils were performed following the guidelines as set out in ASTM D 2166 / D2166 M-16 standard. A material testing machine named as BESMAK was used to do these tests. The samples were prepared at 95 percent of the maximum dry density of the compaction curve, similar to direct shear tests. The diameter to height ratio of the soil sample was kept as 1:2, as per ASTM standard requirements. A specimen with 4 cm dia. and 8 cm depth was used in these tests. The shear strength parameters of treated soils for several concentrations of BA and GGBS were determined.

Figure 3.9. Sample preparation for unconfined compressive strength test

For these tests, 1 %, 2% and 3% of GGBS was trialed with various concentrations of f BA, such as from 4 % to 16%. Duplicate samples were prepared for each water content to get the accurate test results. For both natural and treated soils, a total of 13 UCS tests were performed. After preparing the sample, it was transferred to UCS test machine, and the normal stress was applied until, the specimen failed. Figure 3.9 shows the pictorial view of the sample preparation for UCS test. Finally, several stress strain relationships were plotted to examine the behavior of both treated and untreated soils.

Figure 3.10. Unconfined compression test in progress

3.3.7 Oedometer tests

3.3.7.1 Swelling potential

Swelling potential test was performed for both treated and natural soil following the guidelines mentioned in ASTM D 4546-15. The samples were prepared to a maximum dry density of 95%, which was identical to the compressive strength tests conducted in direct shear and unconfined compression. The specimens were prepared in the consolidometer ring with 6 cm dia., and 2 cm depth in the study. The swelling was noted at the start of the test and after 24 hours, and similar procedures were adopted for all samples. The tests were performed with percentage of BA from 4 to 16 % changing the percentage of GGBS from 1% to 3%.

3.3.7.2. One dimensional consolidation test

After performing the swelling potential tests, the consolidation behavior of both reinforced and natural soil was examined following the guidelines as set out in ASTM D 2435/2435M-11 standard, for both loading and unloading conditions. The samples were prepared at 95 percent of maximum dry density, much like in direct shear and unconfined compressive strength tests. The specimens were prepared in a consolidometer ring with a diameter of 6 cm and a depth of 2 cm as part of the study. The load was applied in 25, 50, 100, 200, 400 800, and 1600 kPa, and the specimen was consolidated for at least 24 hours for each loading. Similarly, for unloading conditions, the specimen was unloaded for each load after 24 hours. Here, 1 kPa is considered as a seating load. The consolidation tests for both treated and untreated soils were performed under the soaked condition, in which the consolidometer ring was filled with water until the completion of the test.

Figure 3. 11. One dimensional consolidation test in progress

After performing the tests with natural soil, the oedometer tests were performed with treated soils with various percentages of additives, similarly. For treated soils, 1 %, 2% and 3% of GGBS was trialed with various concentrations of f BA, such as from 4 % to 16%. Similarly, for treated soils, samples were prepared at 95 percent of the MDD of the compaction curves. In total 13 of consolidation tests were carried out for both natural and treated soils. Finally, load – deformation plots were used to analyze the consolidation behavior of both treated and untreated soils.

Chapter 4

Result and Discussion

4.1 General

This chapter describes the results and discussions of the various properties for treated and untreated expansive soil. The below test data explicitly describes the behavior of natural soil in relation to treated soil.

4.2 Results and discussion

The results of natural and reinforced expansive soil are given below:

4.2.1 Basic index properties

Figure 4.1 shows that the %age passing through sieve number 200 was 92 %, showing the studied material falls in the category of fine-grained soil with clay dominancy as the percentage passing through sieve # 200 is more than 50%. Figure 4.1 further shows that the expansive soil utilized in the study is made up of 48 percent clay, 44 percent silt, and 8% sand. This soil's particle size distribution behaves similarly to that of (**Irfan et al., 2018)** and **(Sharma & Sivapullaiah, 2016)** in which the clayey contents were 52 and 70 percent, respectively.

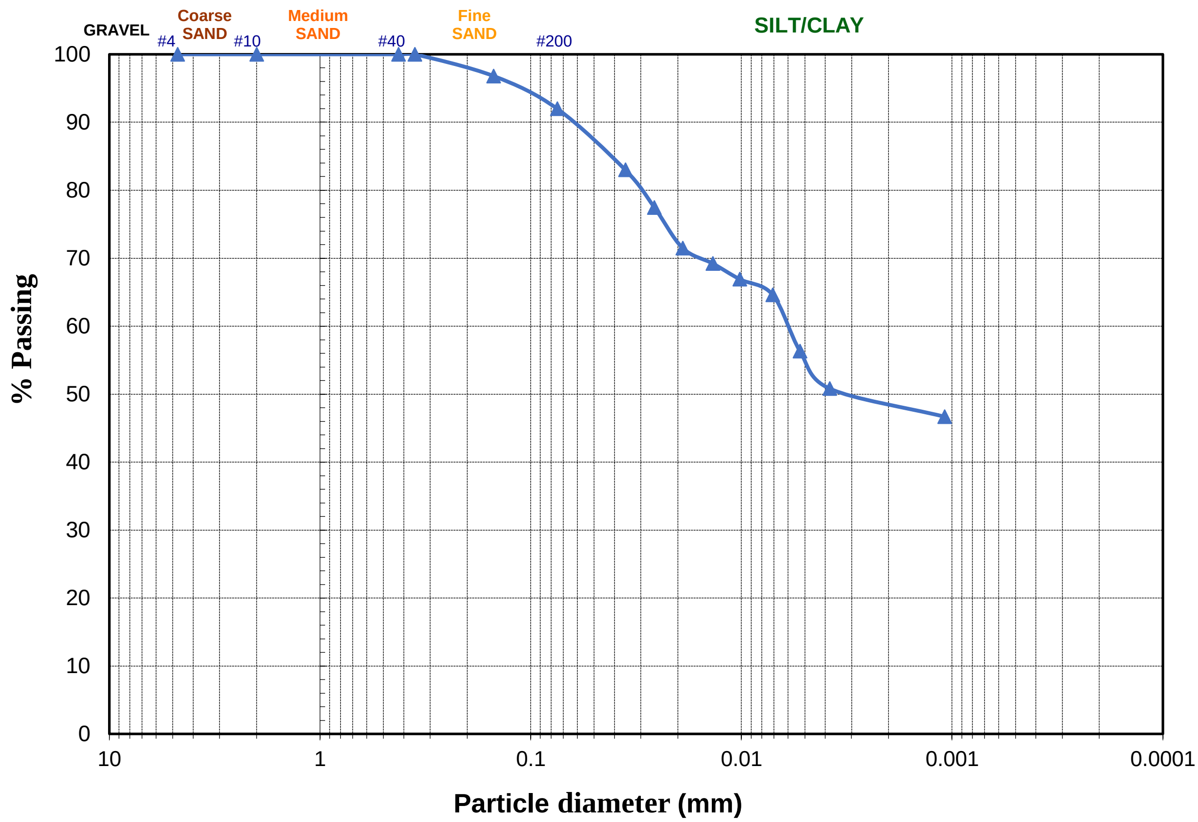


Figure 4.1. Grain size distribution curve

The soil has a liquid limit of 62 percent and a plastic limit of 21 percent, according to Atterberg's limit test data, with a plasticity index of 41. The plasticity index is an important feature since it determines how a soil will swell. According to the unified soil classification system (USCS), the soil belongs to the CH or OH (Figure 4.2) categories, and it belongs to the A-7-6 group in the American Association of State Highway and Transportation Officials (AASHTO) system.

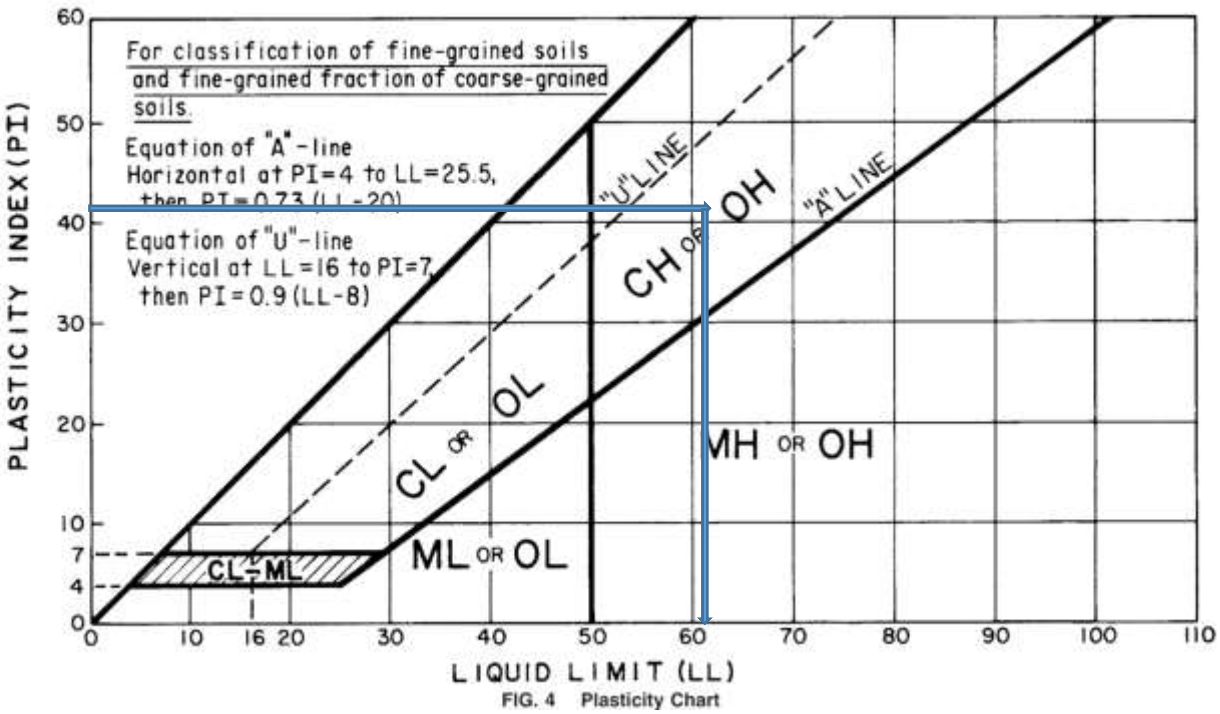


Figure 4.2. USCS classification of soil

Figure 4.3 clearly shows that an increase in the percent ages of additives results to a drop in both the liquid and plastic limits. The liquid limit dropped from 62 to 59.4 percent with 1% GGBS and 4% BA, while the plastic limit dropped from 21 to 20.2 percent. A notable decrease in liquid limit and plastic limit was observed with the reinforcement of GGBS and BA in expensive soil. The reduction in plasticity index is due to soil particle flocculation and agglomeration, which alternatively increases the particle size of soil. The liquid limit decreased from 62 to 47 whereas plastic limit decreased from 21 to 16, which alternately reduced the PI from 41 to 31, which showed that the soil has changed from high swelling potential to medium swelling potential as per criteria, as discussed in **(Williams, 1957)** and which is also shown in Figure 4.4. Similar observations ware observed in the study of **(Khan, 2019)**, **(Ghandi, 2012)** and **(Ashish et al., 2015),** showing that LL of the expansive soil reduced from 65 to 50 at 15% of gypsum and 6% of the bagasse ash. **(Gandhi, 2012)** showed that liquid limit decreased from 72 to 52 with 10% of ash. Table 4.1 shows the consistency limits of expansive soil for addition in additives. In addition, the soil's specific gravity was determined to be 2.71, which is nearly identical to the soil classed as CH, as stated in the study of **(Khan, 2019)** and the effects of BA and GGBS on plasticity of the soil are shown in Figure 4.3.

Table 4.1. Atterberg's limit of natural and reinforced soil

Natural Soil	**Liquid Limit**		**Plastic Limit**	**Plasticity Index**
	62		21	41
Reinforced Soil				
1% GGBS	**4% BA**	59.41	20.2	39.21
	8% BA	58.25	19.61	38.64
	12% BA	57.08	19.02	38.06
	16% BA	56.7	18.72	37.98
2% GGBS	**4% BA**	58.52	19.55	38.97
	8% BA	57.41	19.03	38.38
	12% BA	55.32	18.26	37.06
	16% BA	43.43	17.48	25.95
3% GGBS	**4% BA**	56.39	19.18	37.21
	8% BA	53.2	18.24	34.96
	12% BA	51.31	17.31	34
	16% BA	47.19	16.39	30.8

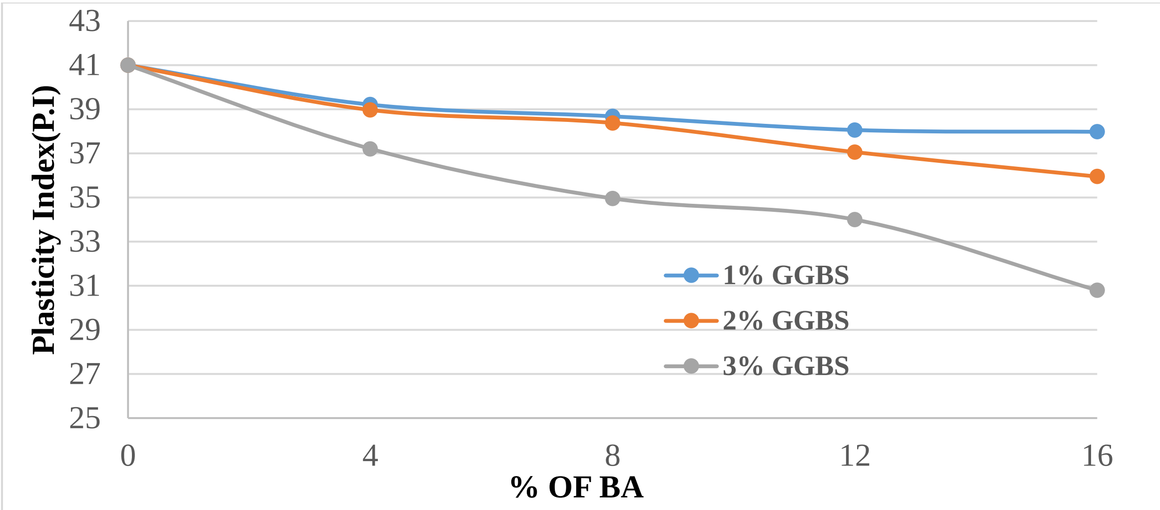


Figure 4. 3. Variation in P.I with different % of additives

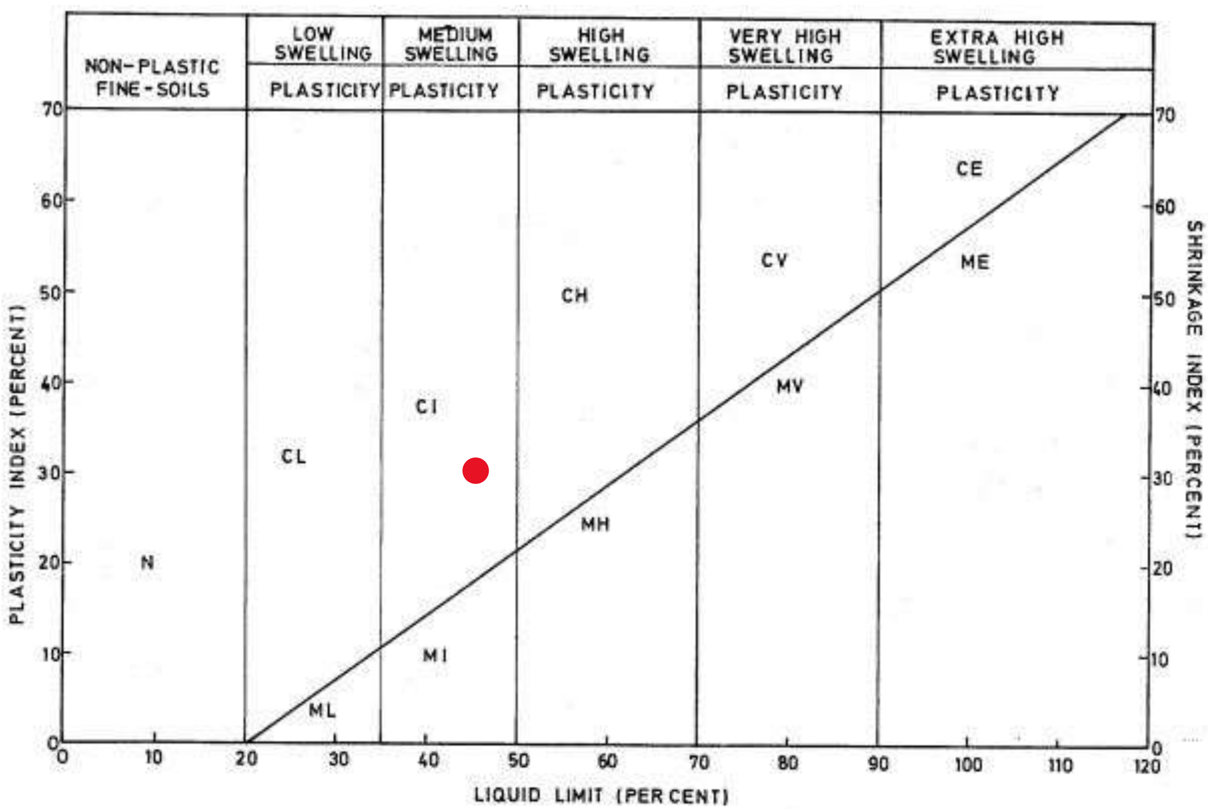


Figure 4.4. Change in swelling behavior of soil w.r.t LL and P.I with reinforcement

4.2.2 Compaction characteristics of soil

The natural soil shows the maximum unit weight 15.84 kN/m^3 and optimum moisture content 23.2 %, shown in Figure 4.5. With the addition of additives, the maximum dry unit gradually decreases while the optimum moisture content of the soil increases. The flocculation and agglomeration of fine-grained soil particles causes this drop in maximum dry unit weight. These flocculated particles occupy larger spaces, which alternately reduce the dry density of soil **(Khan, 2019)**. It is also due to the development of coating of soil particles by admixtures which forms large sized particles. Furthermore, the specific gravity and density of BA is also less as compared to soil, which is also another reason for reduction in this dry unit weight. In contrast, when the percentage of GGBS and BA increases, the optimum moisture content of the soil increases. This is due to the reason that the GGBS and BA particles are finer than the soil, which alternately results in higher specific area. Now, it is quite natural that the larger surface area needs more water for the lubrication of colloid particles, which alternately increases the water contents.

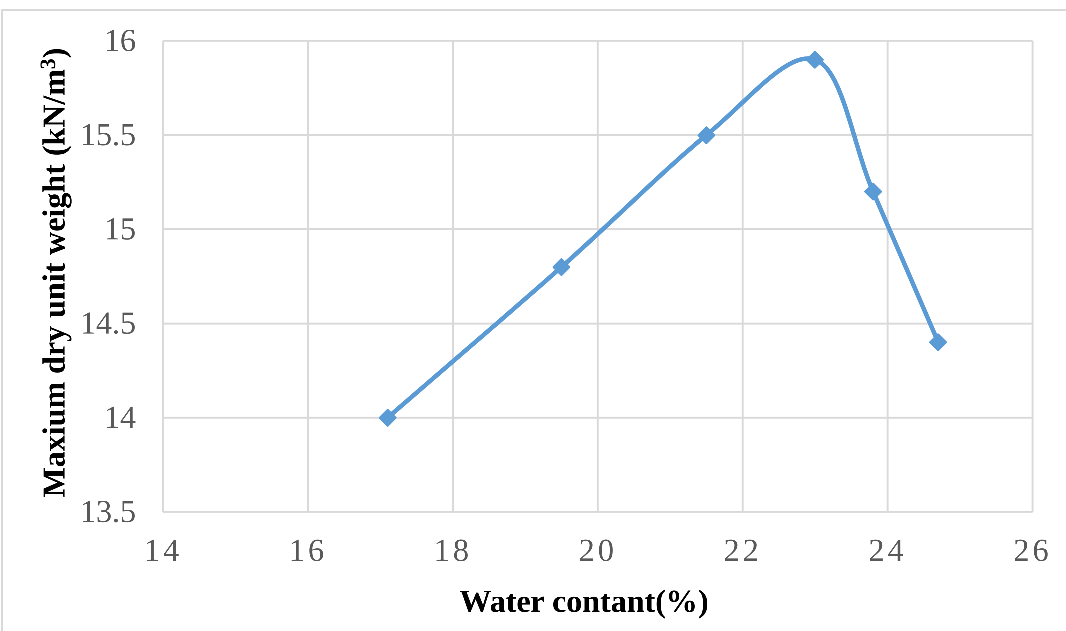


Figure 4.5. Compaction curve of natural soil

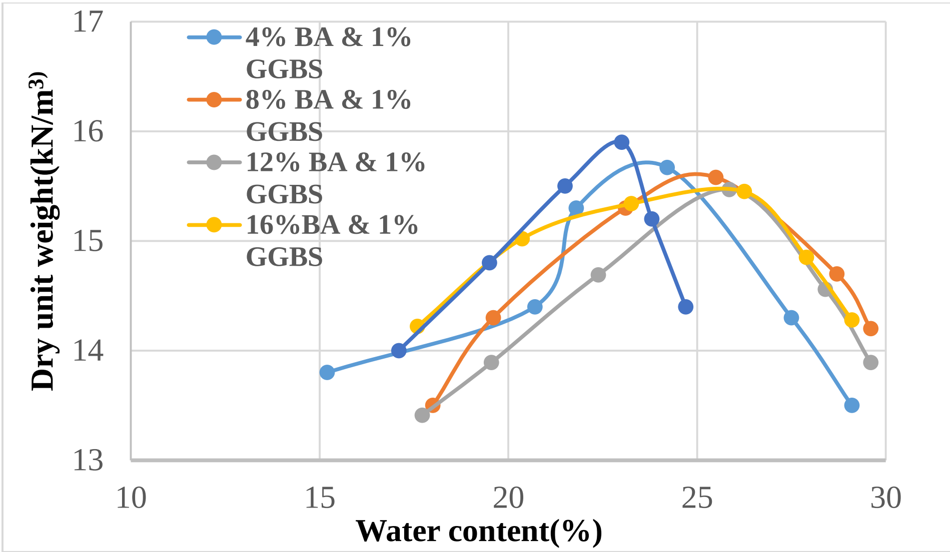


Figure 4.6. Compaction curves with 1% GGBS

The pozzolanic activity involving ground granulated blast furnace slag, bagasse ash, and soil particles is also responsible for the rise in water content.

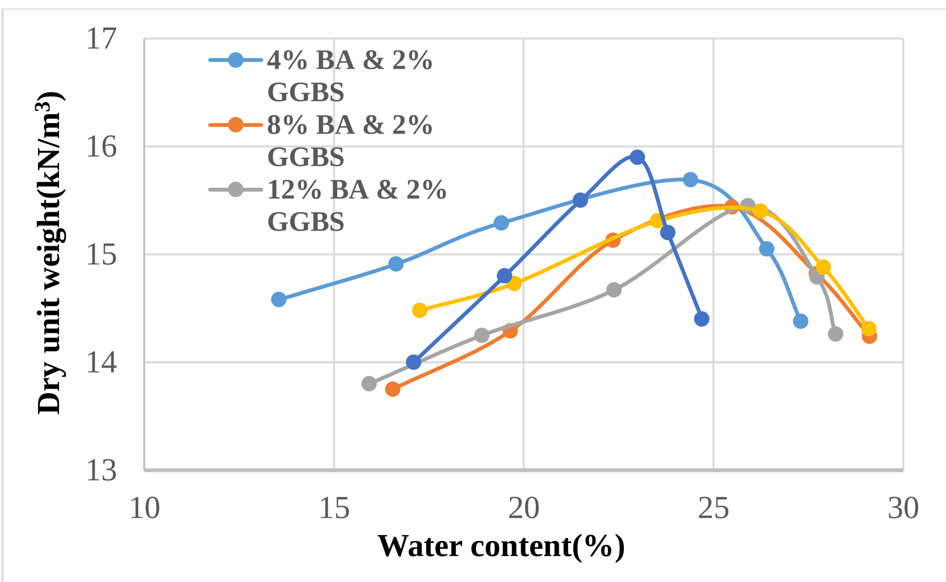


Figure 4.7. Compaction curves with 2% GGBS

It is noted that optimum moisture increases, and maximum dry unit weight decreases up to certain threshold of treatment, but after this point, they get reversed, which was due to the increase in size of the particles. The maximum change in OMC and MDD of the studied material is noted for 3% GGBS and 12 % BA. Figures 4.9 and 4.10 show the correlations between maximum dry unit weight and optimum water contents for varied percent ages of additions. The dry unit weight and optimum water content altered in a pattern that was almost identical to those of previous research (**Chechia & Mittal, 2015), (Rajeswari et al., 2018), (Ashish et al., 2015)** and **(Khan 2019),** the maximum dry unit weight dropped from 16.97 kN/m3 to 15.6 kN/m3 in general. This shift in maximum dry unit weight indicates that the soil has transitioned from high swell to ordinary soil, as evidenced by the criteria stated by **(De Nilov, 1964)**, and the proposed criteria is also shown in Figure 4.11.

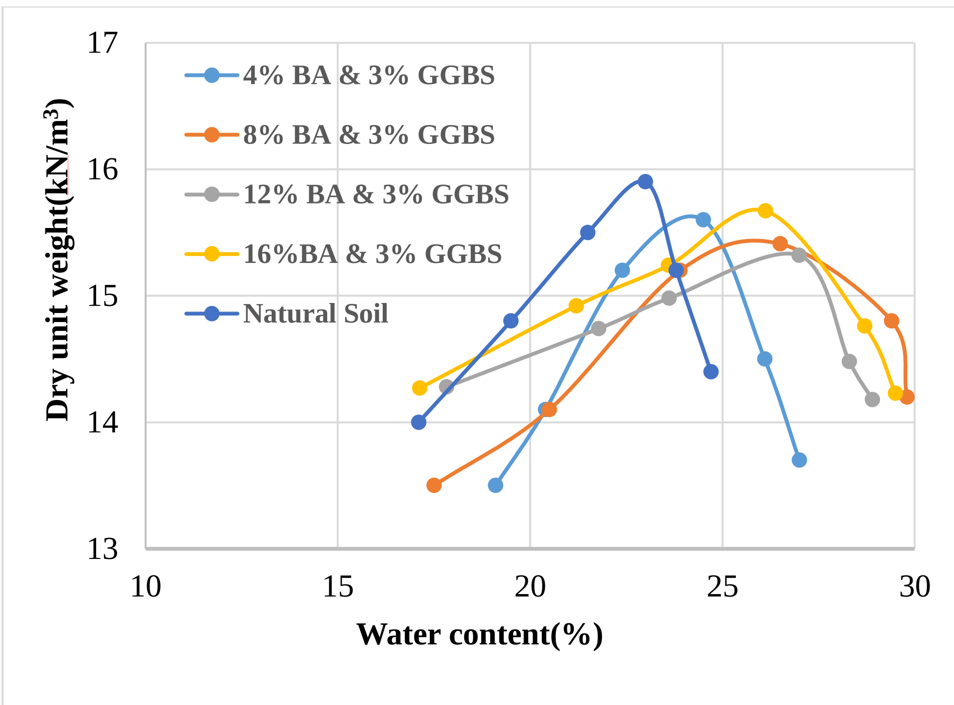


Figure 4.8. Compaction curves with 3% GGBS

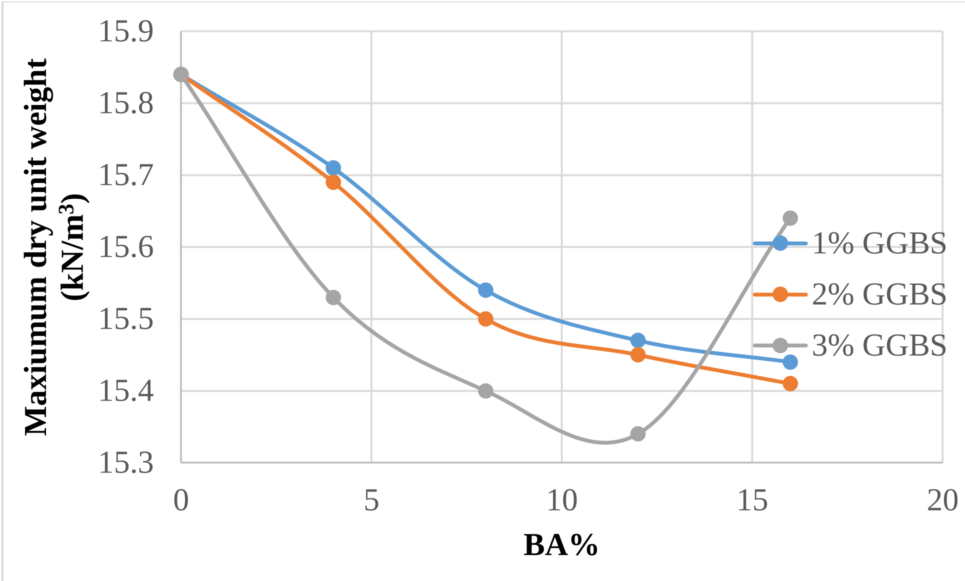


Figure 4.9. Variation in maximum dry unit weight with different percentage of additives

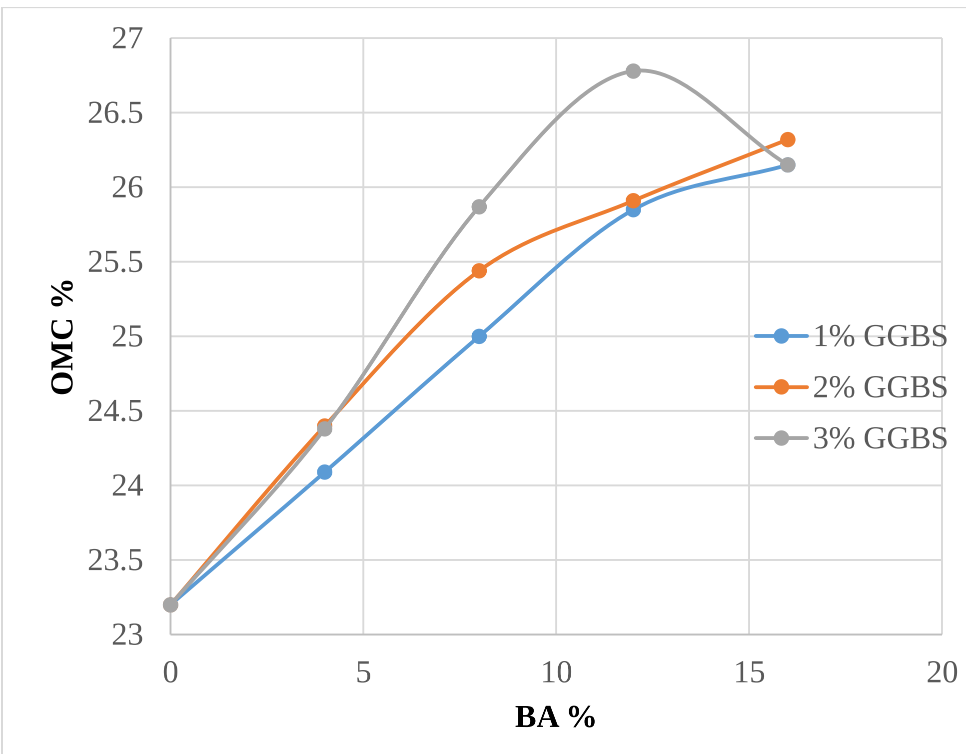


Figure 4.10. Variation in OMC with different percentage of additives

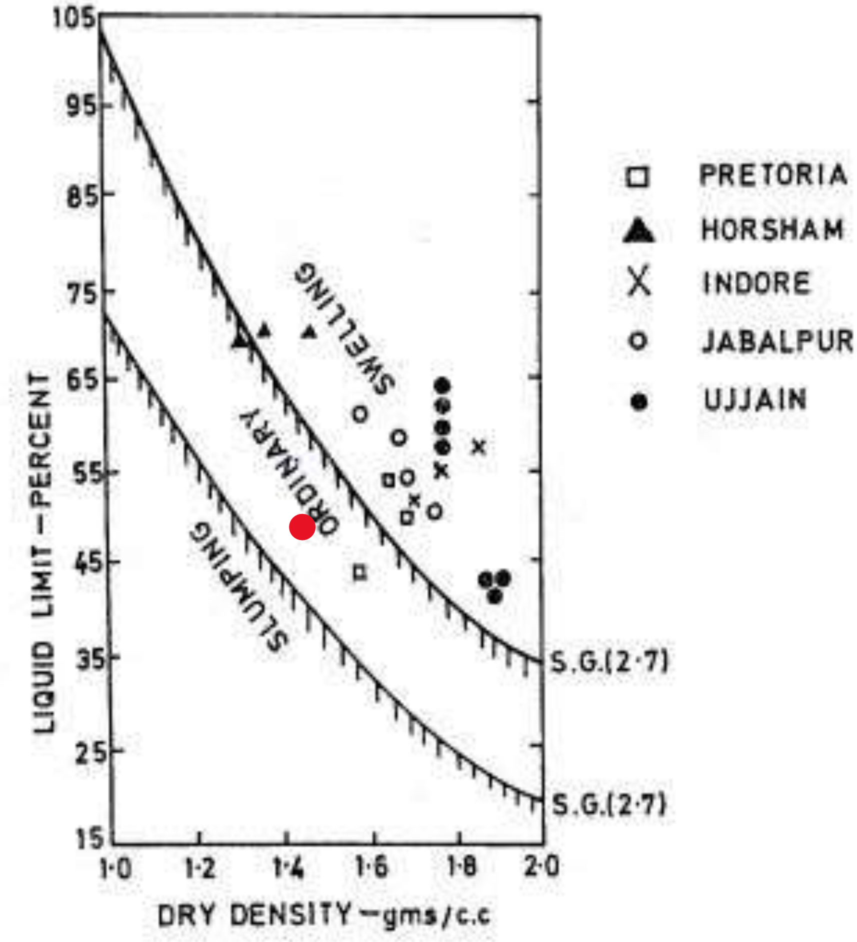


Figure 4.11. Change in soil at 3% GGBS and 12 % BA w.r.t L.L and MDD

4.2.3 Direct shear test

4.2.3.1 Direct shear test - Unsoaked

Figure 4.12 shows the direct shear test results for natural soil which indicates that the soil shows a cohesion of 24.5 kPa and angle of internal friction of 5.42 degrees in unsoaked conditions.

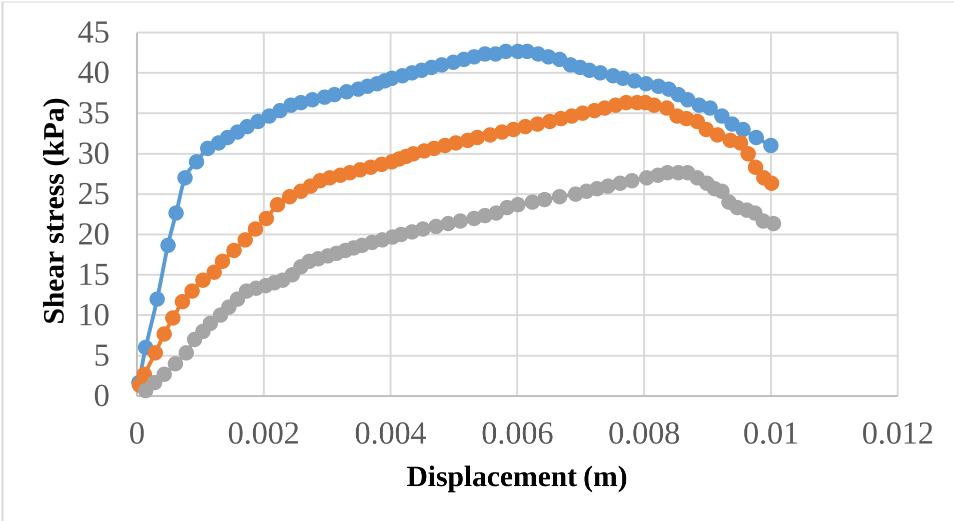


Figure 4.12. Direct shear test on natural soil

Figure 4.13 – 4.16 shows the direct shear test results for treated soil. It can be seen from these relationships that with the addition of GGBS and BA, a gradual decrease in cohesion and increase in angle of internal friction was observed. This change in behavior in soil is due to the change in the composition of the soil. As discussed earlier, that with the addition of additives, the PI of the soil reduces which alternately reduces the clay contents in the soil. As in Figure 4.13, for 1% GGBS and 4% of BA, less changes in both cohesion and angle of internal friction are noted.

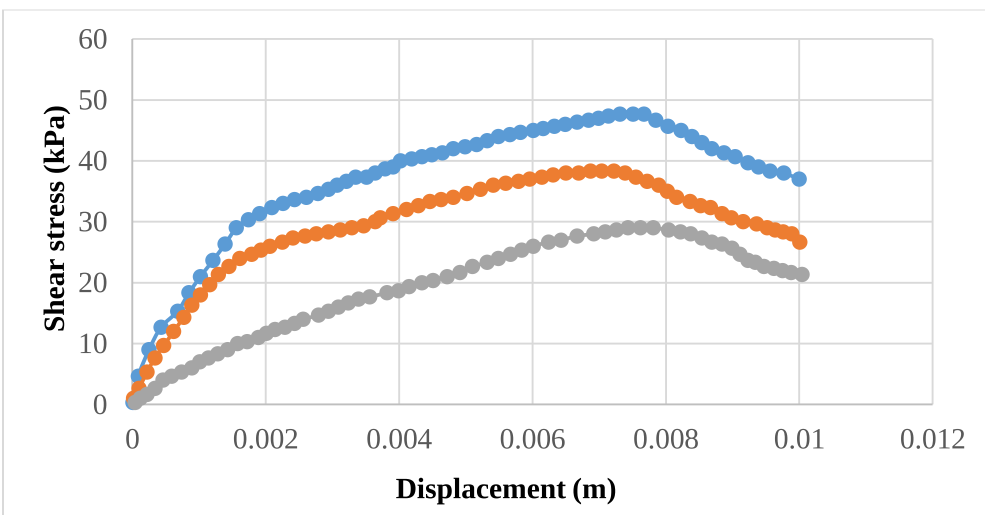


Figure 4.13. Direct shear test with 1% GGBS and 4% BA

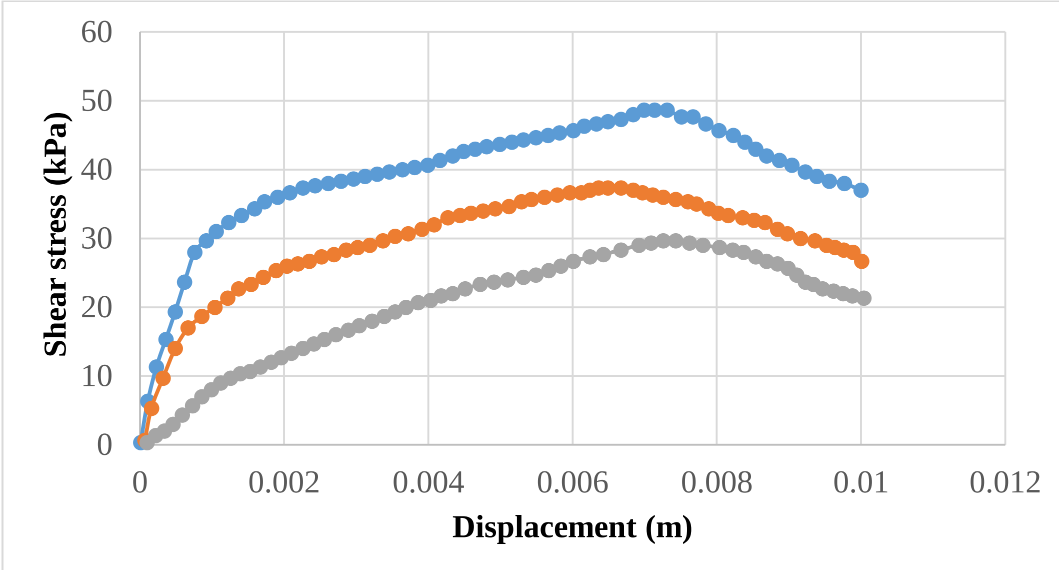


Figure 4.14. Direct shear test with 2% GGBS and 4% BA

It is also clear from Figures 4.13 – 4.18, that with the increase in the percentage of additives, the cohesion decreases, and the angle of internal friction increases, correspondingly, until this trend continues up to a certain threshold. The maximum change in cohesion and angle of internal friction is noted at 3% GGBS and 12% BA. After this point, the shear strength parameters bounce back, showing comparatively less shear stress. Generally, the cohesion of the soil varies from 24.5 kPa to 18 % and angle of internal friction varies from 5.42 to 11.41 degree for 3% of GGBS and 12% of BA. This increase in angle of internal friction is due to the interlocking of the soil particles, which occurs due to the cementitious effects of the additive in the combination of the expansive soil **(Sinalkar et al., 2020)** and as in Figure 4.9, the dry unit weight decreases with an increase in the %age of the additives, which may result in the reduction of the angle of internal friction, but it is the cementitious effect of additives with soil, which enables it to show an increase in the angle of internal friction.

As in Figure 4.3, the PI also decreases with an increase in the %ages of the additives, which alternately reduces the clay contents, transforming the smaller grains into the bigger ones, and it is natural that the internal friction is more for larger particles as compared to smaller ones, which alternately enhances the angle of internal friction. Furthermore, a decrease in the cohesion is quite natural, as with an increase in the %ages of additives, the moisture content increases (Figure 4.10), which may alternately loosen the bonds between particles. Another reason is that the addition of additives results in the reduction of the clay contents in the soil, and the decrease in % clay content is direct indication of this reduction in cohesion. It can also be seen from these Figures that the soil sample for 3% GGBS and 12% BA relatively fails at larger displacement (0.008 m), as compared to others, which also enables this specimen to provide higher shear stress, which alternately provides more favorable shear strength parameters.

The comparative evaluation of Figures 4.13 and 4.16 shows that the samples fail at almost same displacements, both for natural soil, and treated soil (3% GGBS, 12% BA), but the higher shear is again due to the cementitious effects of these binders with the soil. Considering the failure displacements in both cases, i.e., natural soil and treated soil, the shear stress is liable to be same, but it is not the case. Similarly, the specimens for lower %age of additives show different behavior than higher %age of additives after the failure, showing significant changes in residual strength, too. Similar findings were reported by **(Fattah et al., 2010)**, **(Sabat (2012)** and **(Wang et al., 2018)** in which the studies showed that the cohesion decreased and angle of internal friction increased for changes in the concentration of ceramic dust steel fibers. However, the the studied materials showed somewhat different composition to these studies, which was due to the changes in additives and soil types.

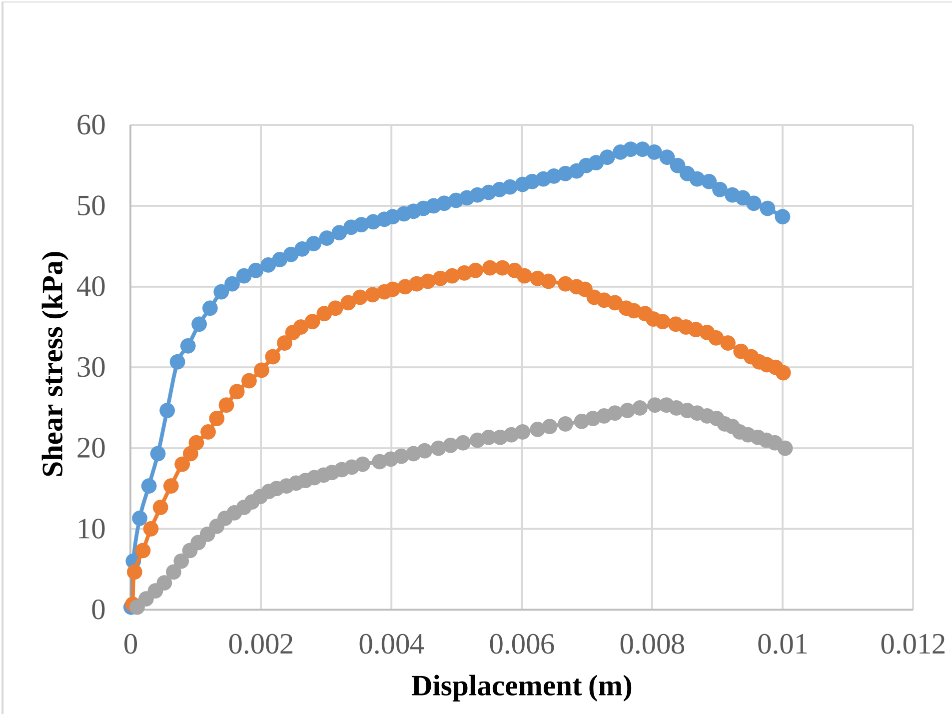


Figure 4.15. Direct shear test with 3% GGBS and 12% BA

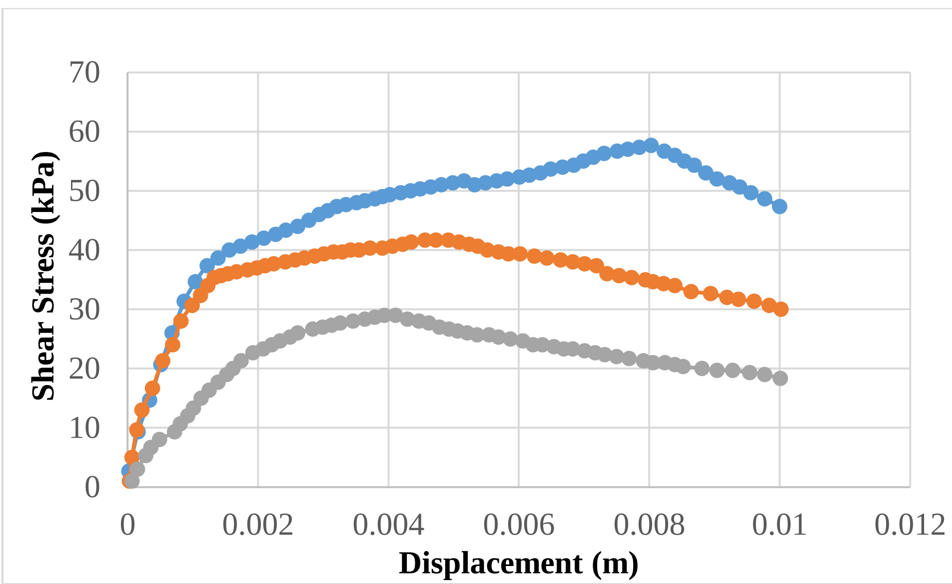


Figure 4.16. Direct shear test with 3% GGBS and 16% BA

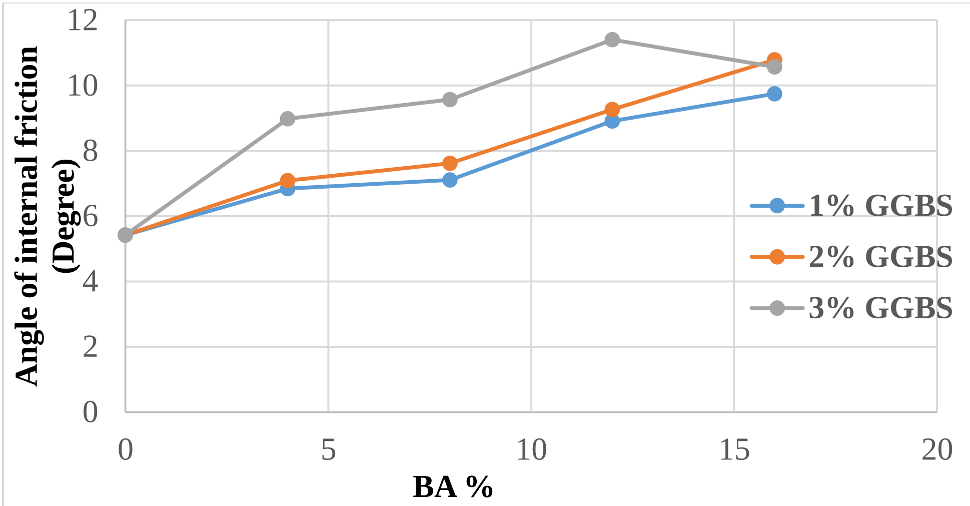


Figure 4.17. Change in angle of internal friction with reinforcement

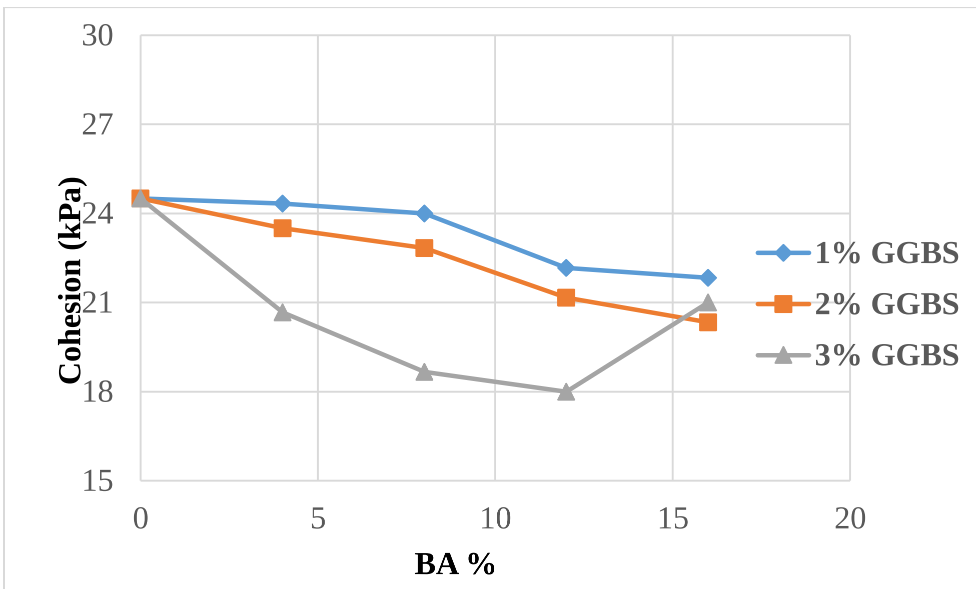


Figure 4.18 Change in cohesion with reinforcement

4.2.3.2 Direct shear test (Soaked)

The cohesion decreases and angle of internal friction increases with an increase in the %ages of additives, showing similar behavior to that of unsoaked conditions. The natural soil provides cohesion and angle of internal friction of 17.5 kPa and 7.24 degrees (soaked conditions); Figures 4.48 and 4.49 to that of 24.5 kPa and 5.42 degrees (unsoaked conditions); Figures 4.20

and 4.21. It can be seen from this test data that the cohesion decreases, and angle of internal fiction increases in soaked condition, which is due to the fact that the soaked soil provides low cohesion than unsoaked condition, tested under similar conditions, and the decrease in cohesion is due to the loss of adhesion between soil particles due to the soaking **(Salman, 2011)**. Similarly, the higher angle of internal friction in soaked condition is due to the fact that the sample shows more consolidation, which alternately packs the particles more, and hence shows higher angle of internal friction due to interlocking of the particles.

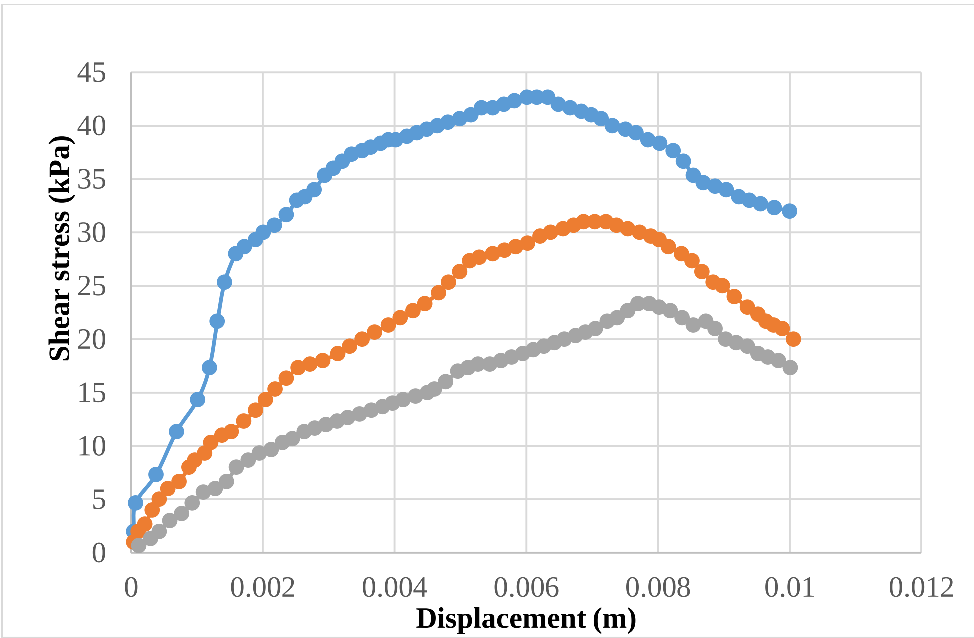


Figure 4.19. Shear stress vs displacement of natural soil in soaked condition

Furthermore, similar to that of unsoaked condition, the soaked treated soil provides a decrease in the cohesion and increase in the angle of internal friction (Figure 4.22 – 4.49). Generally, for the best alternative, i.e., 3% GGBS and 12%, the cohesion and angle of internal friction changes from 13 to 12 kPa, and from 13.08 to 14.98 degrees, respectively, and as discussed in the preceding section (unsoaked condition), the cohesion of the soil varies from 24.5 kPa to 18 % and angle of internal friction varies from 5.42 to 11.41 degree for 3% of GGBS and 12% of BA.

The test data shows that the unsoaked specimens provide more cohesion than soaked soil, which is quite natural, and less differences are noted in angle of internal friction for both conditions. Similarly, the shear stress profiles for 3% GGBS and 12% BA shows more smooth and consistent shear stress profiles after failure as compared to other alternatives, in which a sudden failure is noted, and for which, the soaked specimens fail at less displacement of 0.007 m as compared to 0.008 m in unsoaked condition. The shear stress profiles as shown in Figure 4.44 depict that the soaked specimens provide consistent and regular shear stress deformation profiles than unsoaked specimens (Figure 4.18), which provide sudden and abrupt failure, and it is quite interesting that the saturated specimen provides slightly higher shear stress as compared to unsoaked specimen for this alternative, and the strength increases from 43 kPa to 65 kPa (soaked conditions), and from 43 to 57 kPa (unsoaked conditions) which is due to the fact that cementitious effect is at its peak for these %ages of additives in combination to expansive soil in soaked conditions.

It can be seen from the test data, the shear stress of unsoaked specimens is always more than the soaked condition rather than this alternative. Generally, both soaked and unsoaked specimens for 3% GGBS and 12% BA provides higher strength as compared to all other options, which is the best alternative so far, in the study. Generally, the specimens strengthened with additives provide higher strength as compared to the natural soil. It is also quite interesting that the natural soil for both soaked and unsoaked conditions provide same shear stress for 3% GGBS and 12% BA, however, the differences are quite significant for lower treatments, and in which the unsoaked specimens provide higher shear stress as compared to soaked specimens. It can also be seen from Figures 4.20 – 4.21 and Figure 4.48 – 4.49 that the shear strength parameters profiles gradually widen up with an increase in the %ages of the additives, but after a certain threshold, the profiles initiate to merge, showing less changes in shear stress at very low and high %ages of treatments. The shear strength parameters overlap to each other, showing almost same shear stress for both soaked and unsoaked conditions, for 16% BA and all %ages of GGBS, i.e., 1%, 2% and 3%. Comparatively, the shear stress is less for 16% BA less than 12% BA, which means that there is no efficacy to increase the %ages of additives beyond a certain threshold.

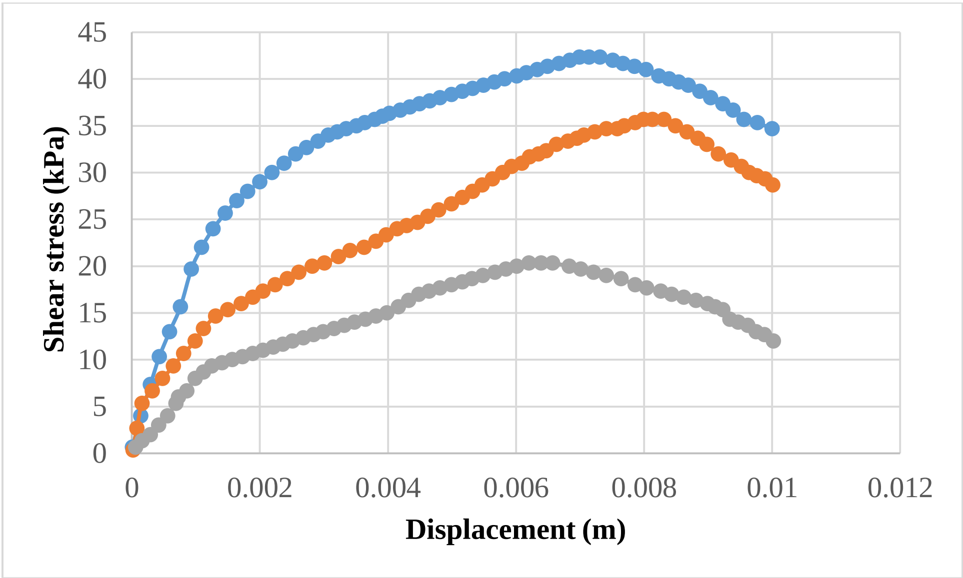


Figure 4.20. Shear stress vs displacement at 1% GGBS and 4% BA

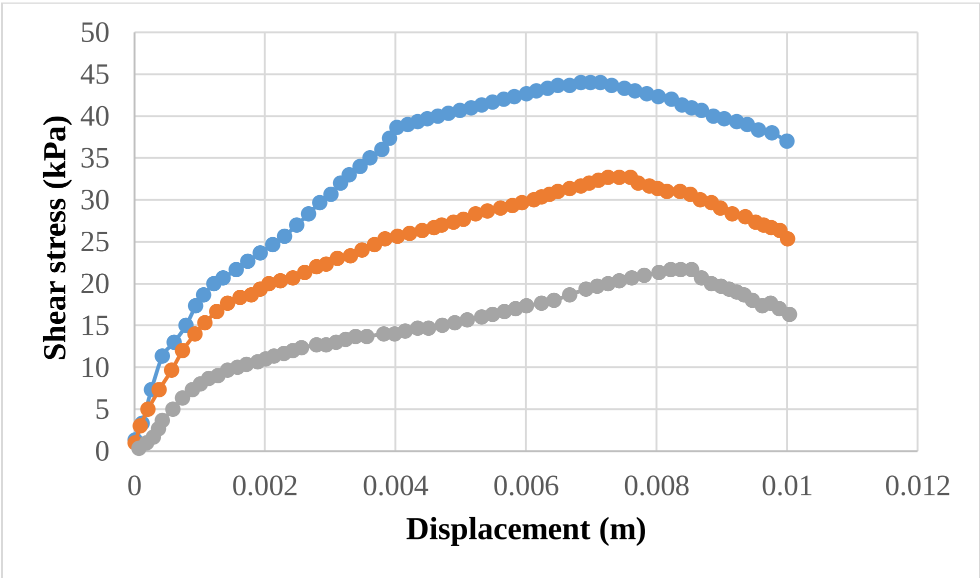


Figure 4.21. Shear stress vs displacement at 1% GGBS and 8 % BA (soaked)

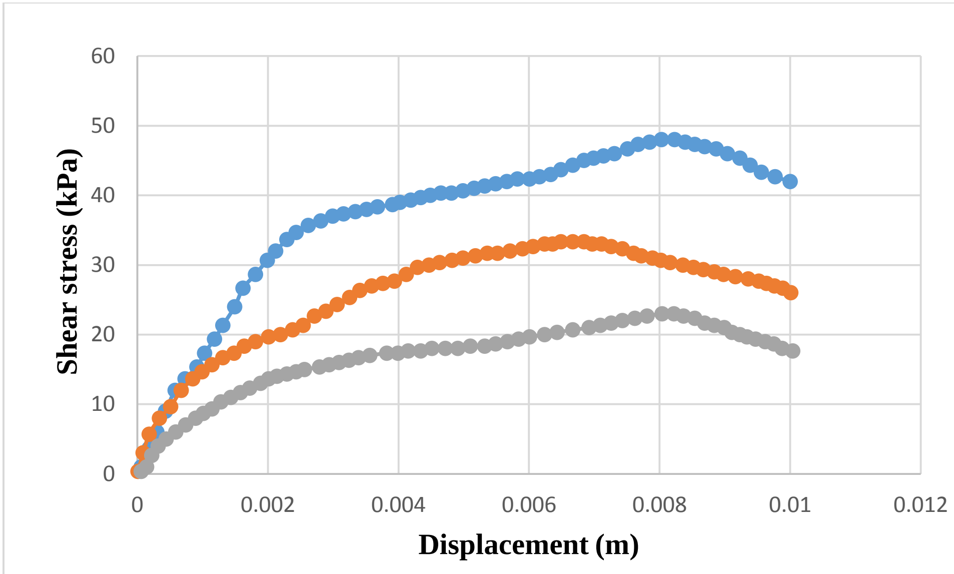


Figure 4.22. Shear stress vs displacement at 1% GGBS and 12% BA (soaked)

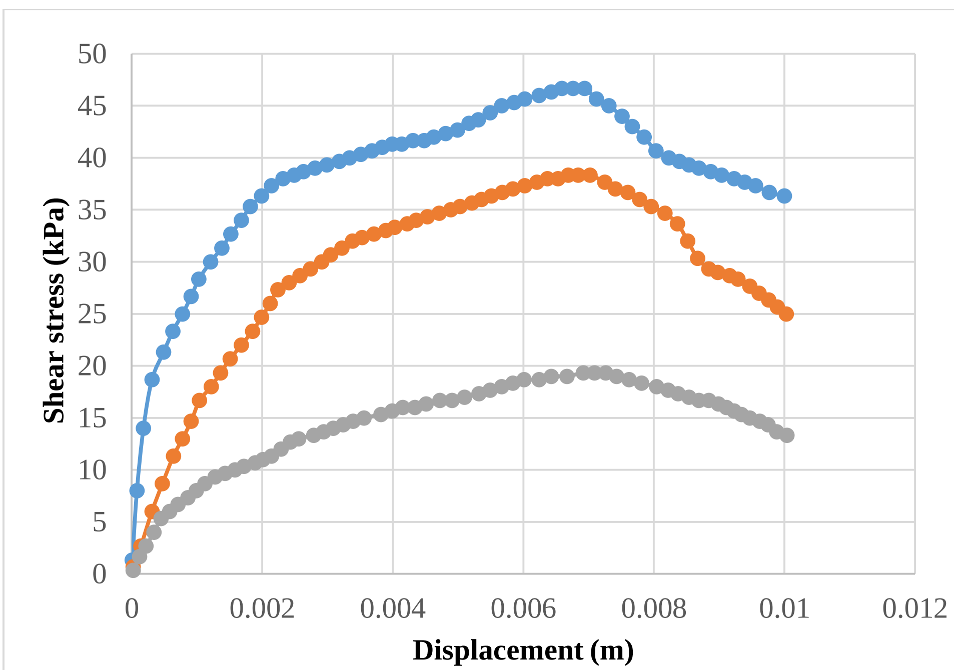


Figure 4. 23. Shear stress vs displacement at 1% GGBS and 16% BA (soaked)

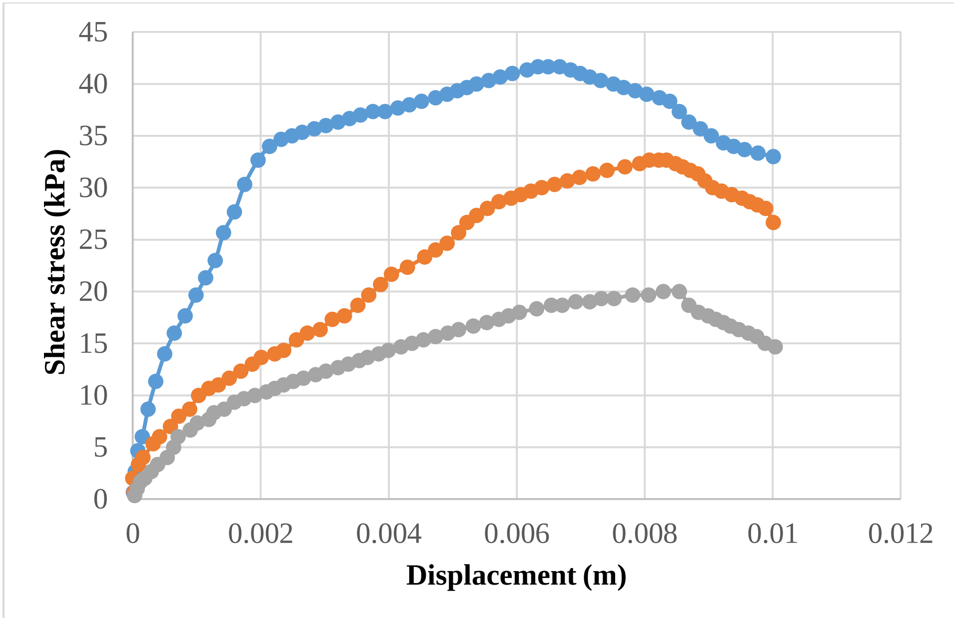


Figure 4.24. Shear stress vs displacement at 2% GGBS and 4% BA (soaked)

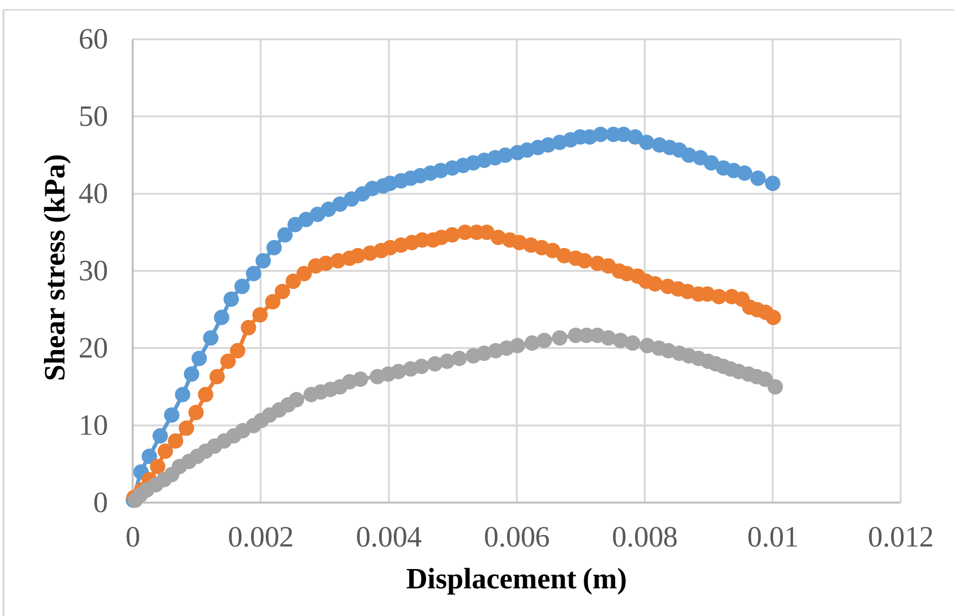


Figure 4. 25. Shear stress vs displacement at 2% GGBS and 8 % BA (soaked)

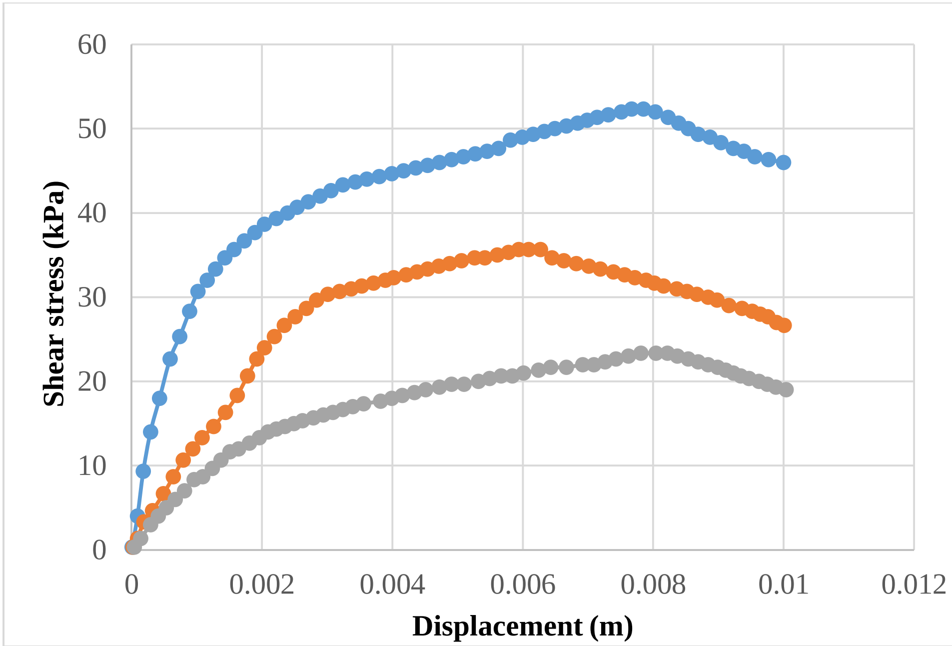


Figure 4.26. Shear stress vs displacement at 2% GGBS and 12 % BA (soaked)

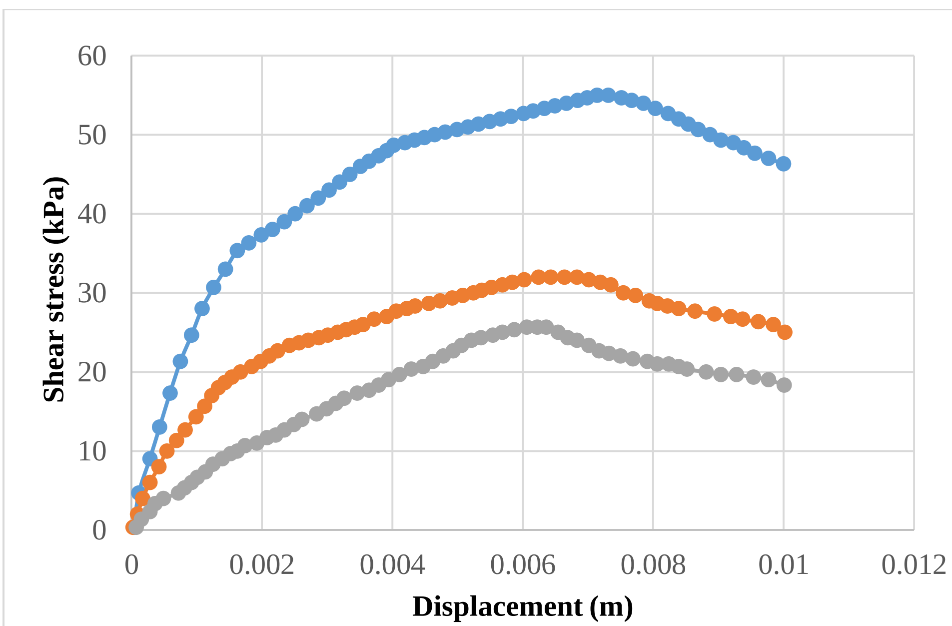


Figure 4.27. Shear stress vs displacement at 2% GGBS and 16 % BA (soaked)

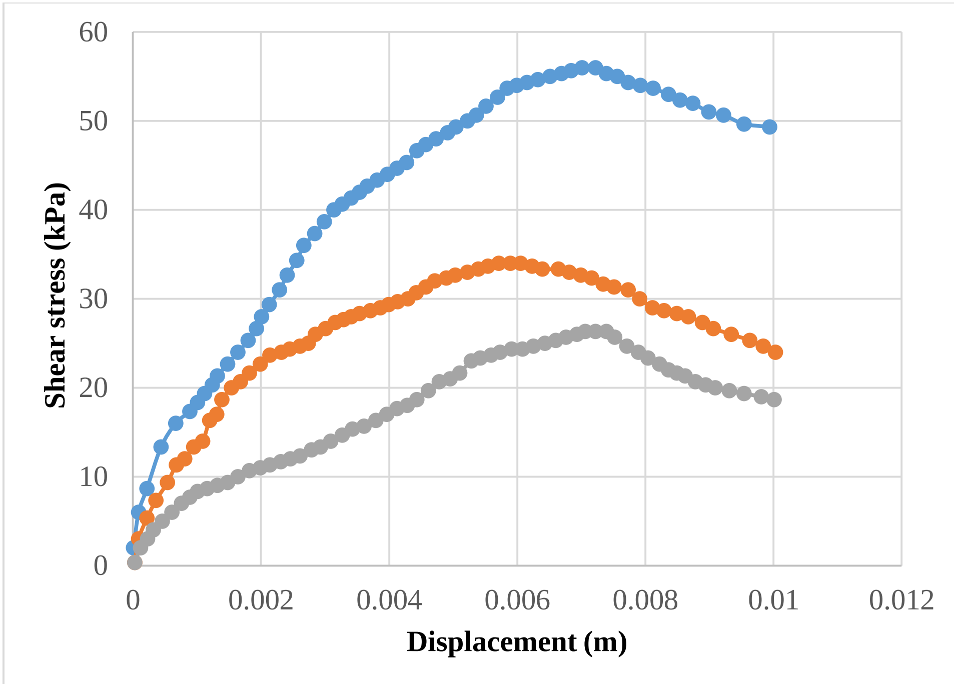


Figure 4.28. Shear stress vs displacement at 3% GGBS and 4 % BA (soaked)

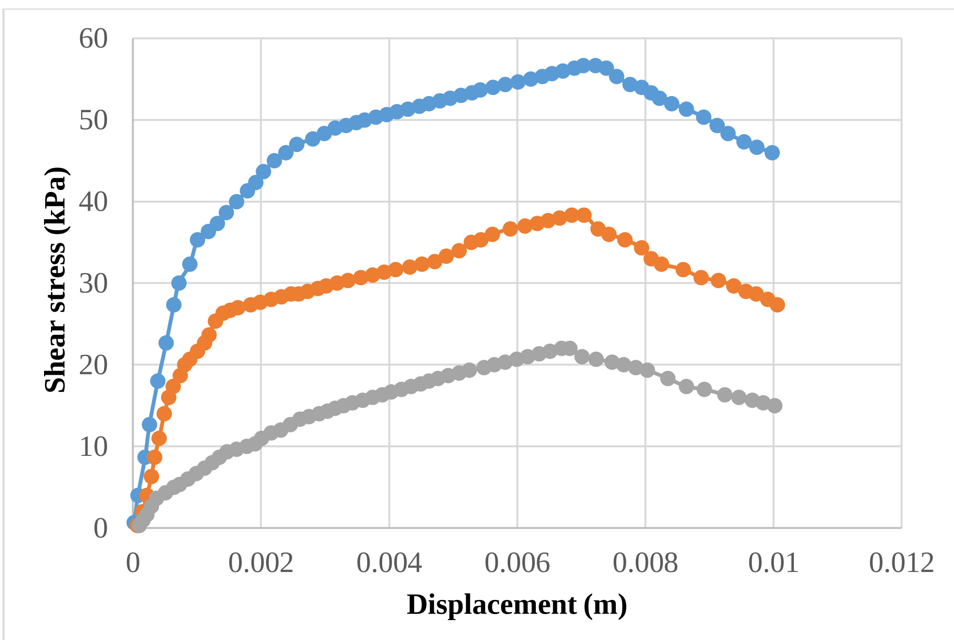


Figure 4.29. Shear stress vs displacement at 3% GGBS and 8 % BA (soaked)

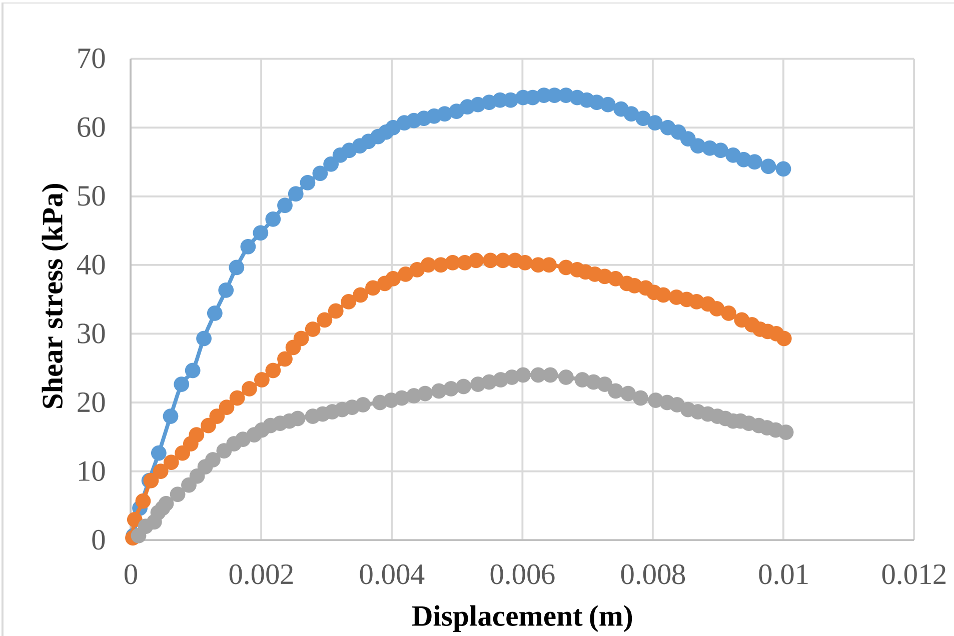


Figure 4.30. Shear stress vs displacement at 3% GGBS and 12 % BA (soaked)

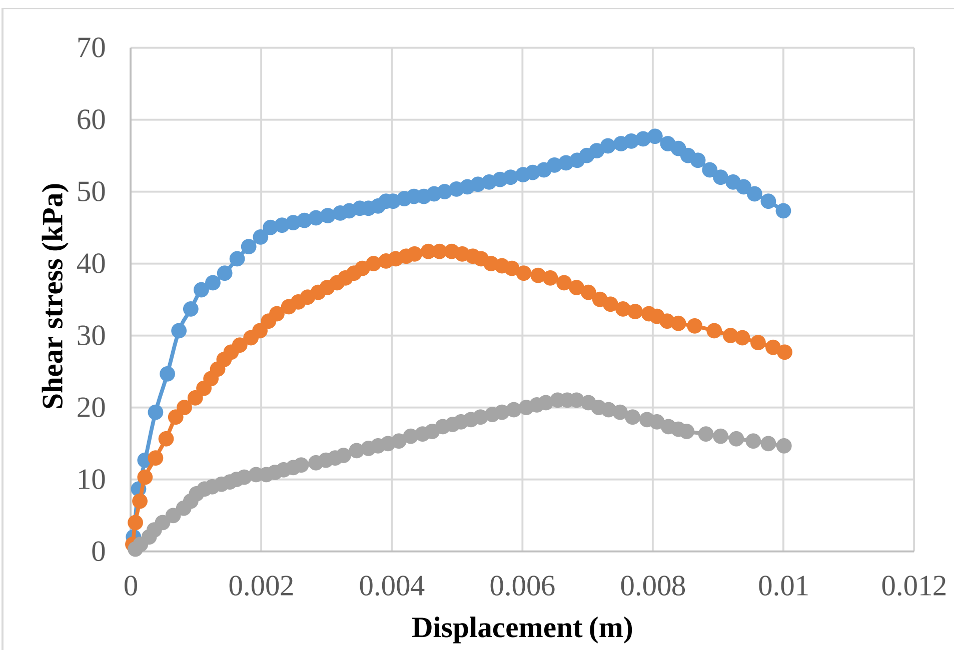


Figure 4.31. Shear stress vs displacement at 3% GGBS and 16% BA (soaked)

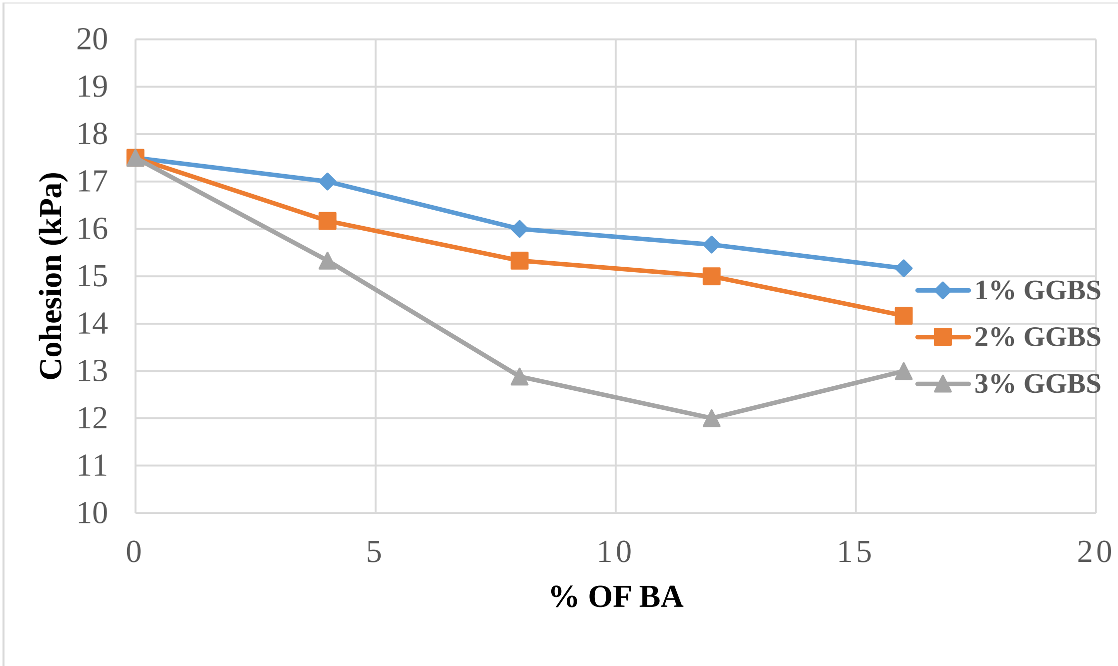


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Figure 4.32.Variation in cohesion with different percentage of reinforcement in soaked condition

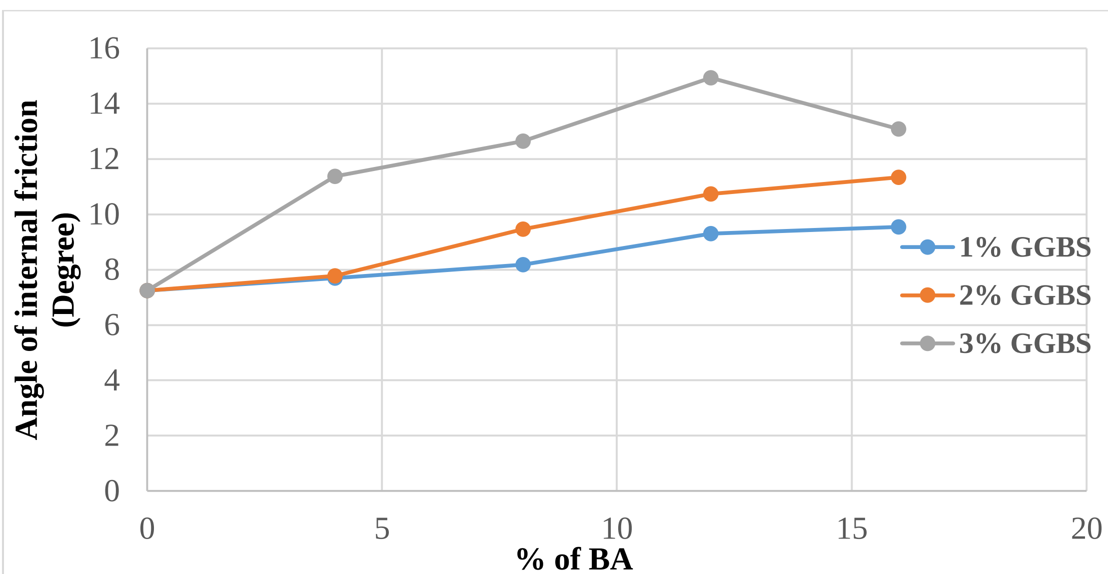

Figure 4.33. Angle of internal friction with different percentage of reinforcement in soaked condition

4.2.4 Unconfined compressive strength (UCS)

Figures 4.34 – 4.37 shows the unconfined compressive strength of both natural and treated soils, which shows that the natural soil provides an unconfined compressive strength of 98.4 kPa, which gradually increases with an increase in the %ages of BA and GGBS and is noted maximum at 12% BA for various %ages of GGBS, but after this point, the strength initiates to decrease, i.e., the change point. Generally, the unconfined compressive strength of the studied materials varies from 98.4 kPa (natural soil) to 246 kPa for 3% GGBS and 12% BA, which is almost 151% higher than the natural soil. It can be seen from Figures 4.34 – 4.37 that the increase in strength is minimal for 1% and 2% GGBS for various %ages of BA. This increase in strength is due to cementitious behavior of GGBS and pozzolanic effect of BA with the soil **(Khan, 2019)**. This work is similar to the findings as in **(Khan, 2019), (Osinubi et al., 2009), (Agarwal & Kaur 2014), (Shamshad et al., 2019)** and **(Sabat, 2012)**, in which the unconfined compressive strength of the soil increased with an increase in the %ages of reinforcement, when used different treatments such as, bagasse ash, gypsum, and bio enzyme in their studies. The test data of the studied material shows that for higher %ages of additives, UCS profiles appear overlapping to each other, showing less changes in the strength, and for 16 % BA, UCS profile shows lower peak than 12% BA, which is the best alternative, similar to that of direct shear test data.

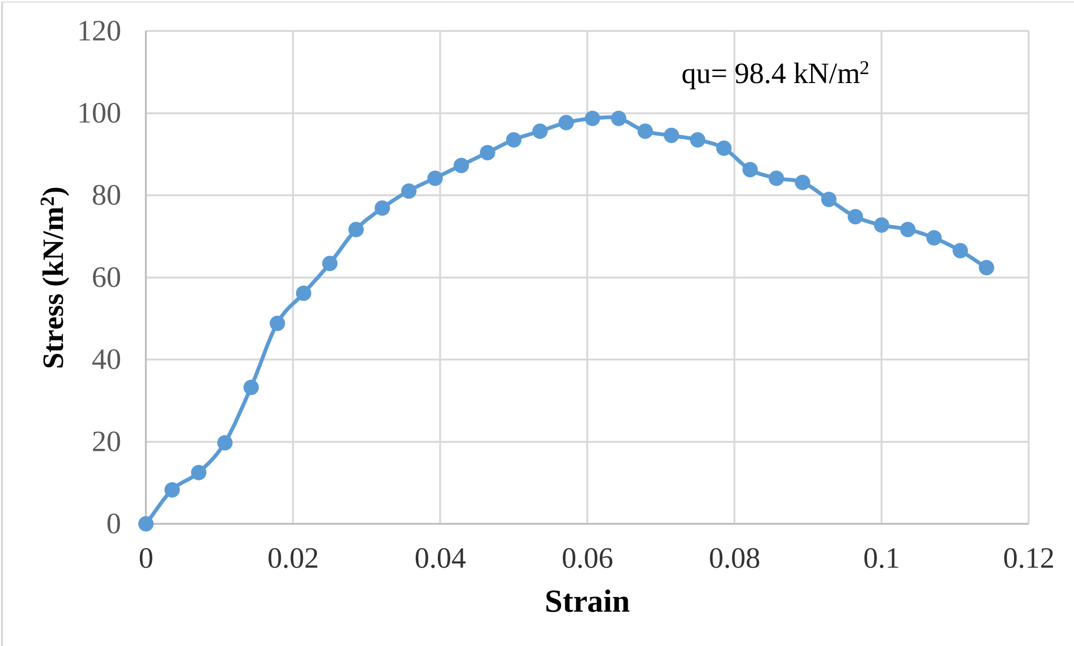

Figure 4.34. Unconfined compressive strength of natural soil

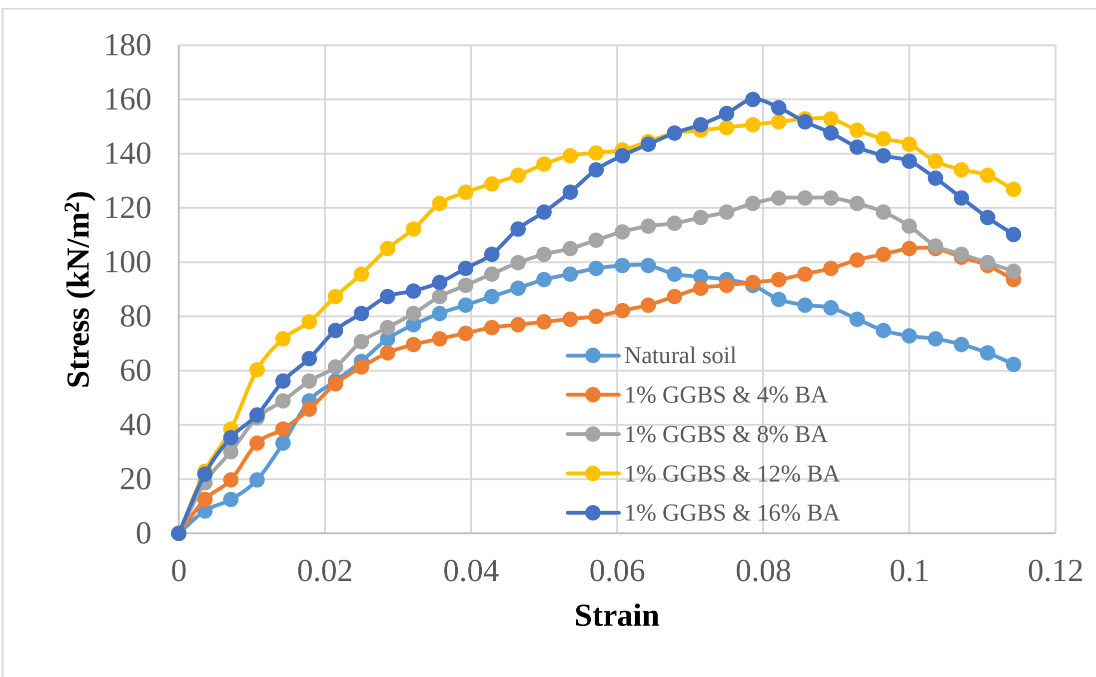

Figure 4.35. Unconfined compressive strength with change in percentage of BA and 1% GGBS

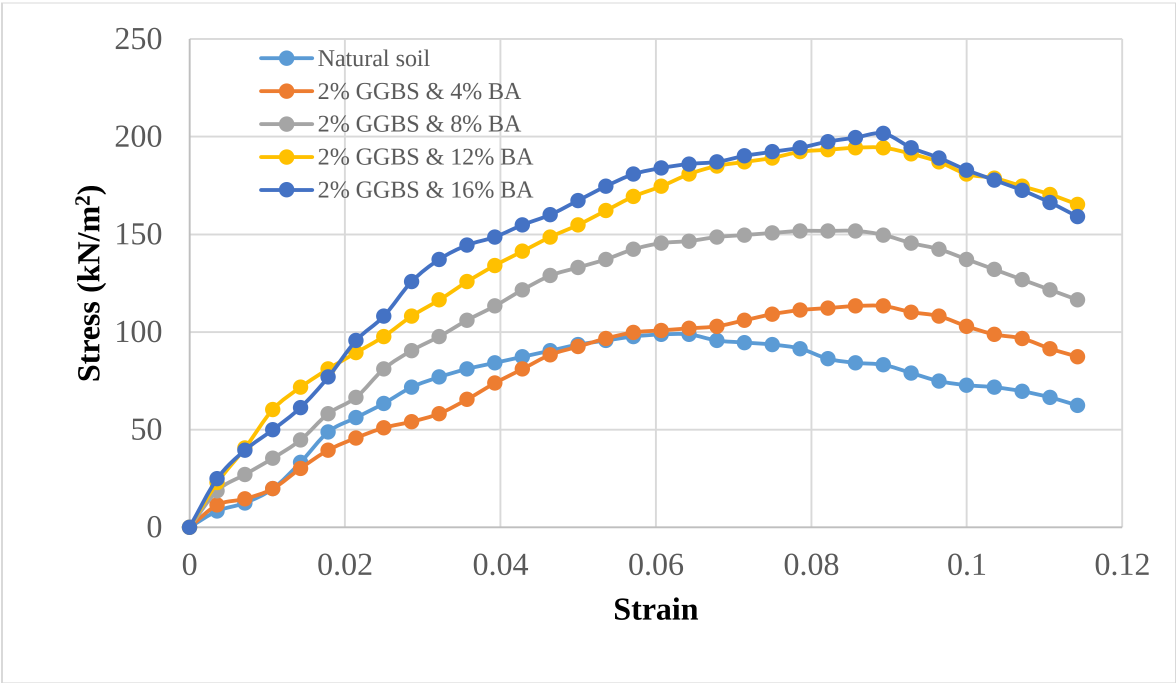

Figure 4.36. Unconfined compressive strength with change in percentage of BA and 2% GGBS

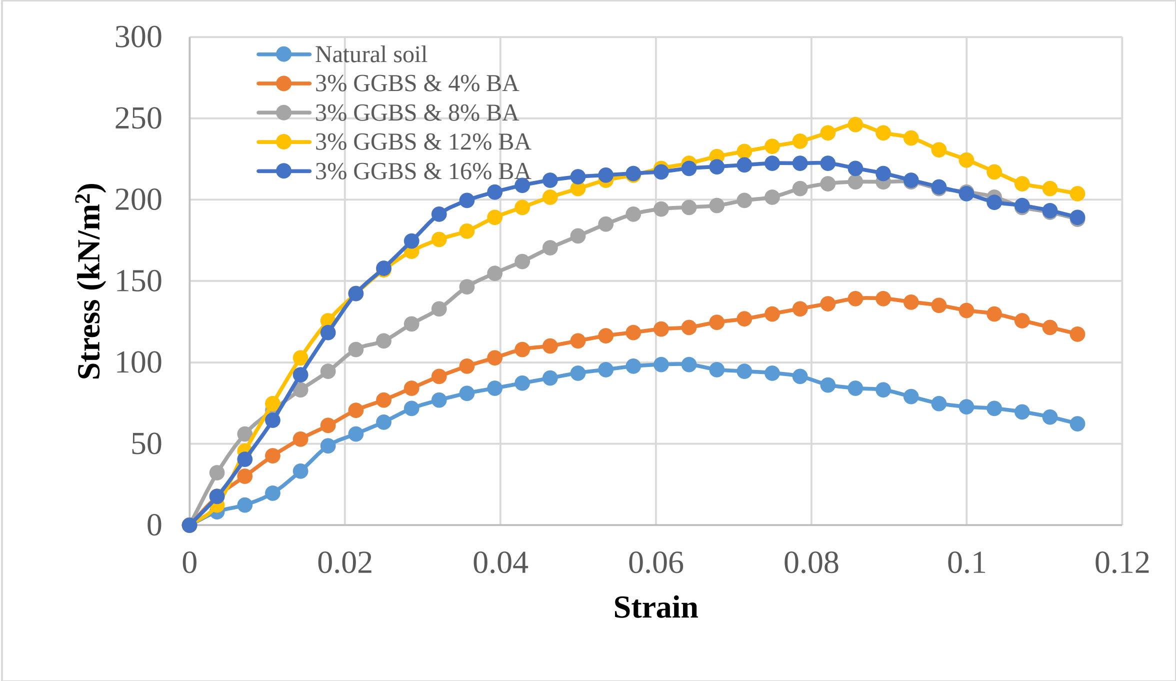

Figure 4.37. Unconfined compressive strength with change in percentage of BA and 3% GGBS

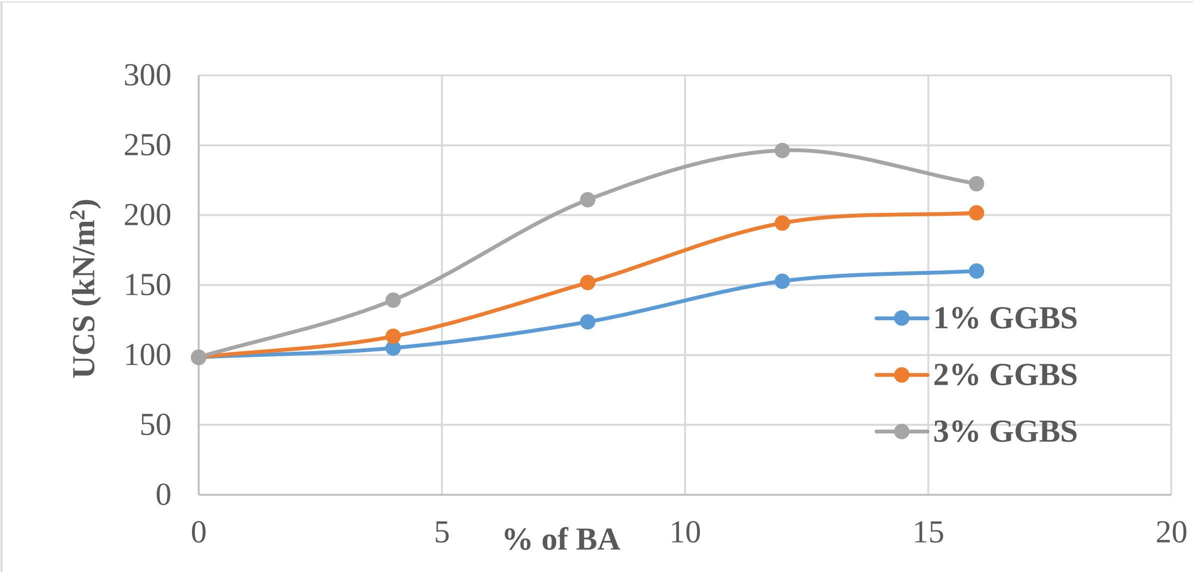

Figure 4.38. Variation in unconfined Compressive strength with different percentage of reinforcement

4.2.5 Swelling Potential

Figure 4.39 represents the swelling potential for both natural and treated soils which depicts that the natural soil shows a swelling potential of 9.75%, which gradually decreases with an increase in the %ages of the additives. The maximum decrease in swelling potential is noted for 3% GGBS and 12% BA, which is due to less plasticity index for this alternative as depicted in Figure 4.3. Generally, the swelling potential of the soil changes from 9.75 to 3.75 %, changing the soil behavior from high swelling potential to medium swelling potential, following the criteria as in (**Seed et al., 1962),** and which is also presented in Table 4**.2** The studies such as (**Khan et al., 2018), (Ahmed et al., 2015), (Gandhi 2012),** and **(Khan 2019),** reported that the swelling potential of soil decreased with an increase in the %ages of the additives. **(Gandhi 2012)** used bagasse ash to decrease the swelling potential of the expansive soil. Swelling potential of the soil decreased from 21% to 15% with 10% of the ash. **(Khan 2019)** used bagasse ash and gypsum in which swelling potential of high plastic clay decreased from 9.75 to 0.16 with 6% bagasse ash and 15% gypsum.

Table 4.2. Nature of soil w.r.t swelling potential

Swelling Potential	Nature of soil sensitivity
>25	Very high
5-25	High
1.5-5	Medium
<1.5	Low

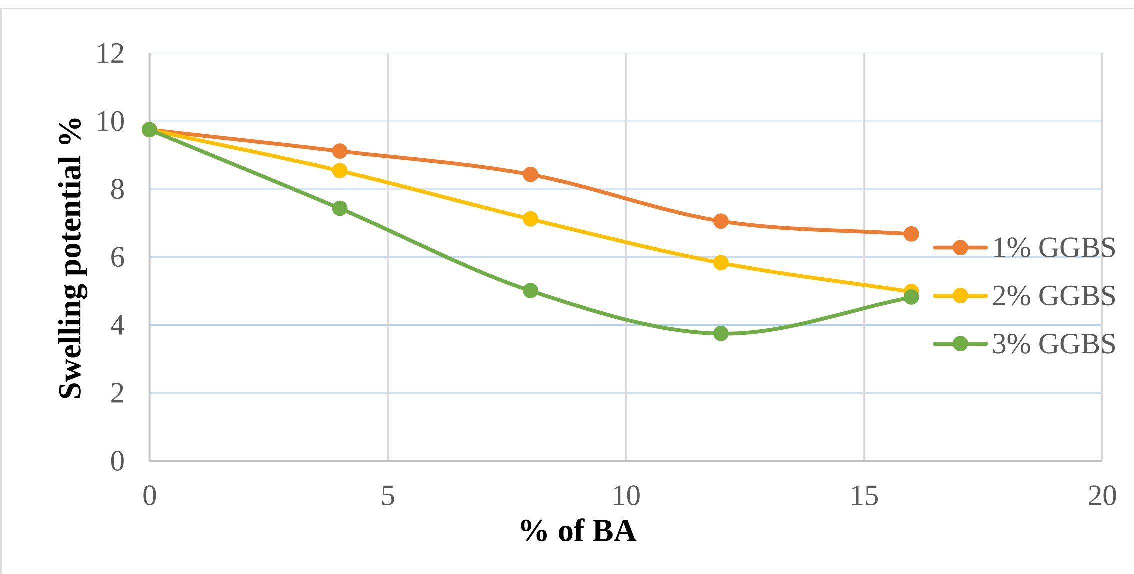

Figure 4.39. Swelling potential variation with change in reinforcement.

4.2.6 Consolidation

The natural soil shows a pre-consolidation pressure, compression index and swelling index of 90 kPa, 0.262 and 0.044, respectively, as shown in Figures 4.40 – 4.42. The pre-consolidation pressure, which reflects the past maximum overburden pressure on a soil specimen, gradually increases with an increase in the %ages of GGBS up to 12% BA, but after this point, it decreases, and which is maximum for 3% GGBS and 12% BA as shown in Figure 4.43. On the contrary, as the additives are increased, the swelling index and compression index gradually decreases, with the maximum decrease occurring at 3% GGBS and 12% BA, similar to that of void ratio profiles in Figure 4.45-4.46, and at this point, the soil provides the minimum swelling potential and compressibility of the expansive soil. which alternately shows a maximum pre-consolidation pressure of 480 kPa, with an increase of 433%. The decrease in the compression index and swelling index is maximum at the peak pre-consolidation pressure, which is due to the fact that prior to consolidation test, the samples have attained the maximum consolidation capability due to the past maximum effective overburden pressure in these specimens. The natural soil shows an initial and final void ratios of 0.9013. and 0.432, as shown in Figure 4.45 and 4.46. As in Figures 4.45 and 4.46, the final void ratio for 3% GGBS and 12% BA provides a quite divergent profile as compared to other profiles, despite that it has a higher pre-consolidation pressure, which alternately results in a higher shear stress. The studies such as (**Mohanty, 2016)** and **(Hussein & Ali, 2019)** indicated that as the percentage of the binder increased, the compression index and swelling index dropped. With the addition of up to 30% fly ash, the soil's compression index reduced from 0.298 to 0.136 **(Mohanty, 2016)**. The study further added that Cc and Cs decreased from 0.15 to 0.117 and 0.022 to 0.017 for changes in the concentration of polypropylene fiber up to 2%.

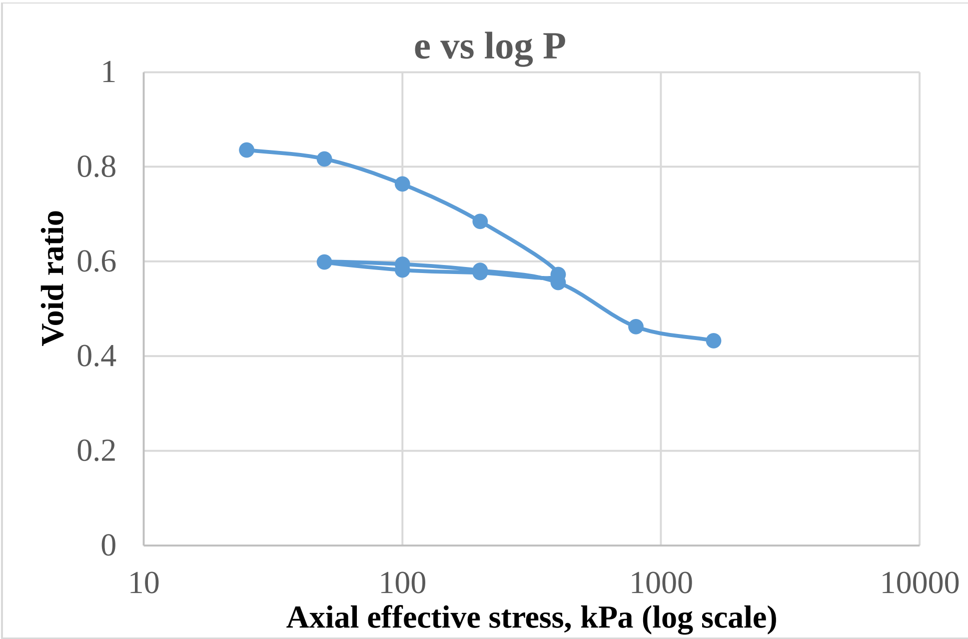

Figure 4.40. Void ratio vs axial effective stress graph of natural soil

Table 4.3. Cc and Cs calculations of natural soil

e_1	0.763462	P1	100
e_2	0.684557	P2	200
Cc	0.262114132		
e1	0.589492	P1	100
e2	0.576183	P2	200
Cs	0.044212	Cr	0.044212

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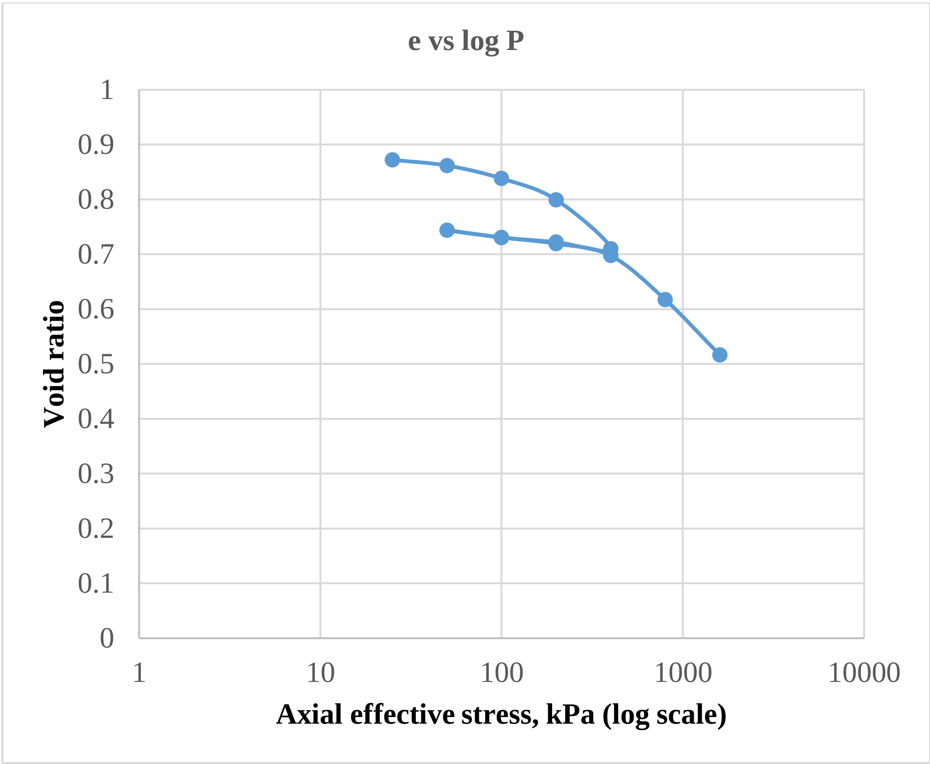

Figure 4. 41. Void ratio vs axial effective stress graph at 3% GGBS and 12% BA

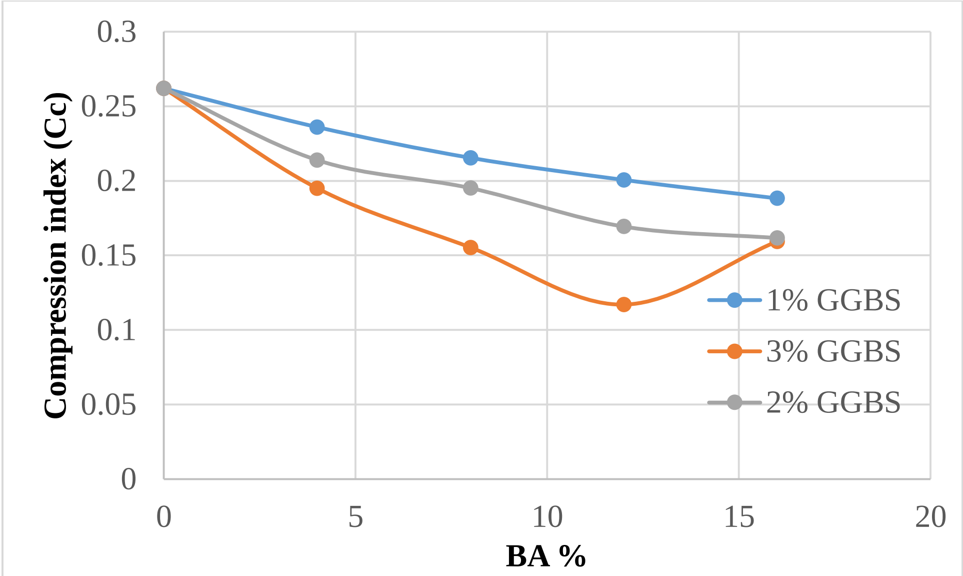

Figure 4.42. Variation in Cc w.r.t % of BA and GGBS

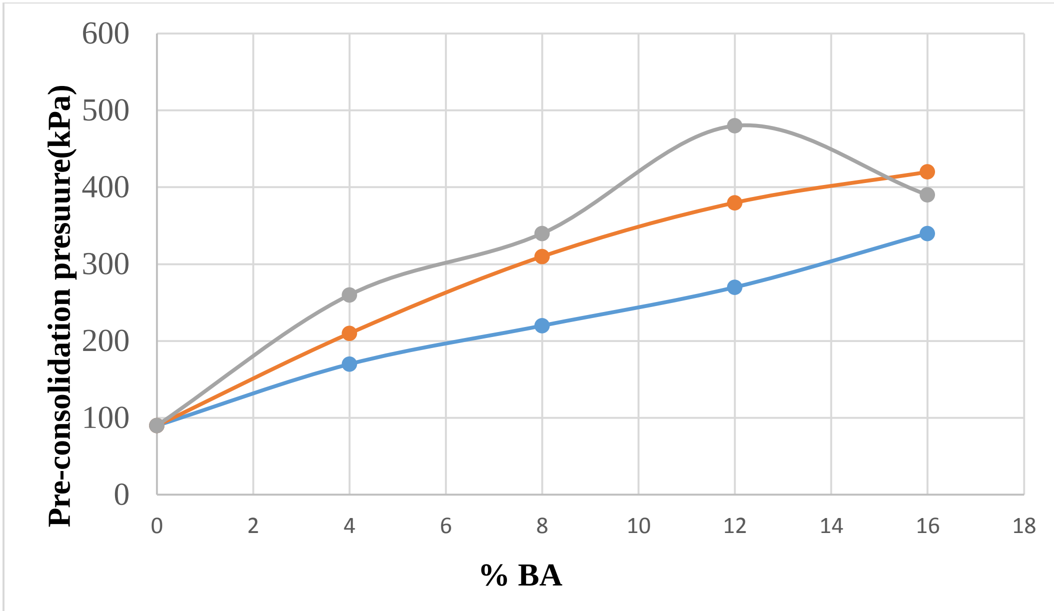

Figure 4.43. Variation in pre-consolidation pressure w.r.t percentage of GGBS and BA

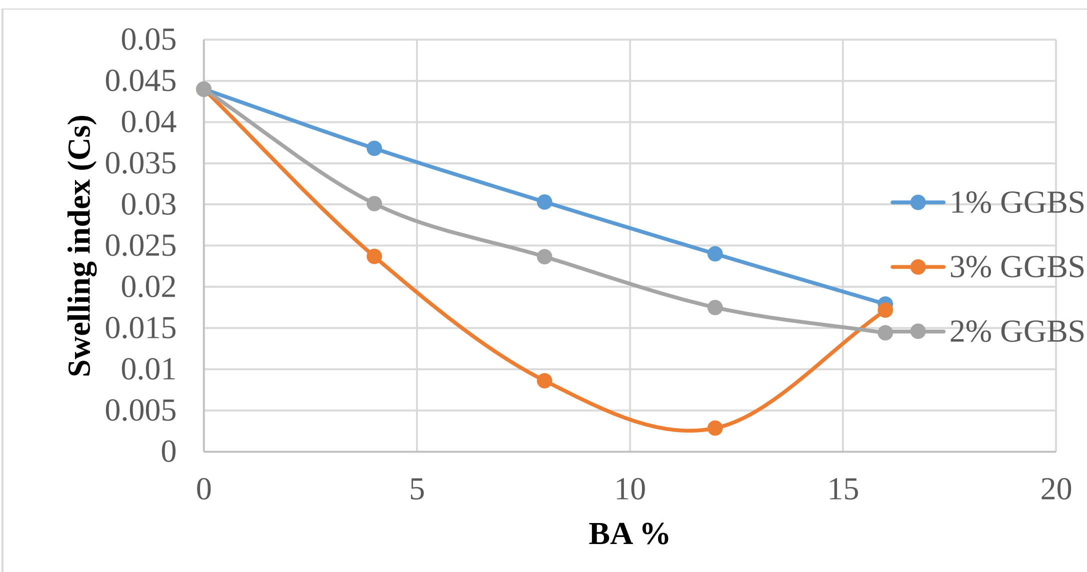

Figure 4.44. Variation in Cs w.r.t % of BA and GGBS

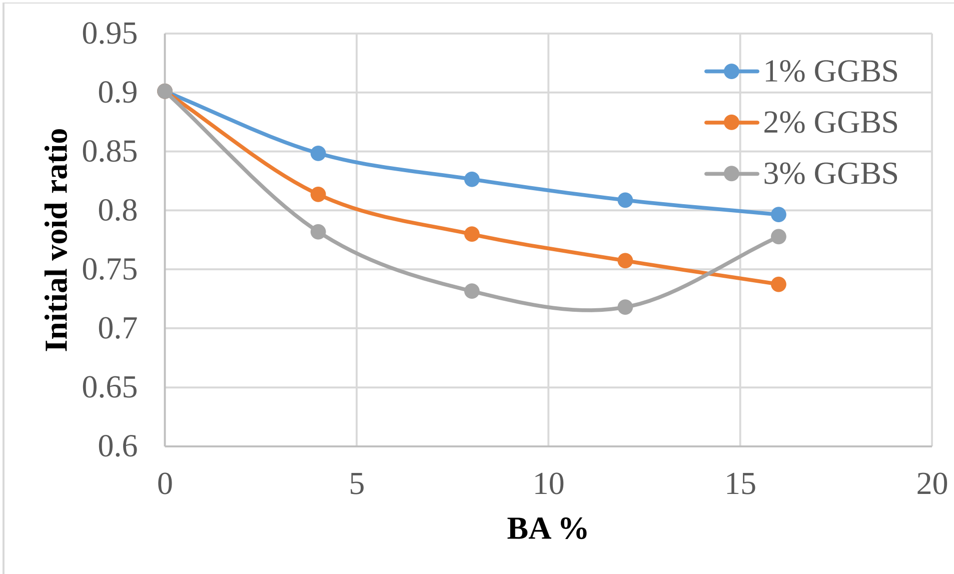

Figure 4.45. Variation in initial void ration with different percentage of additives

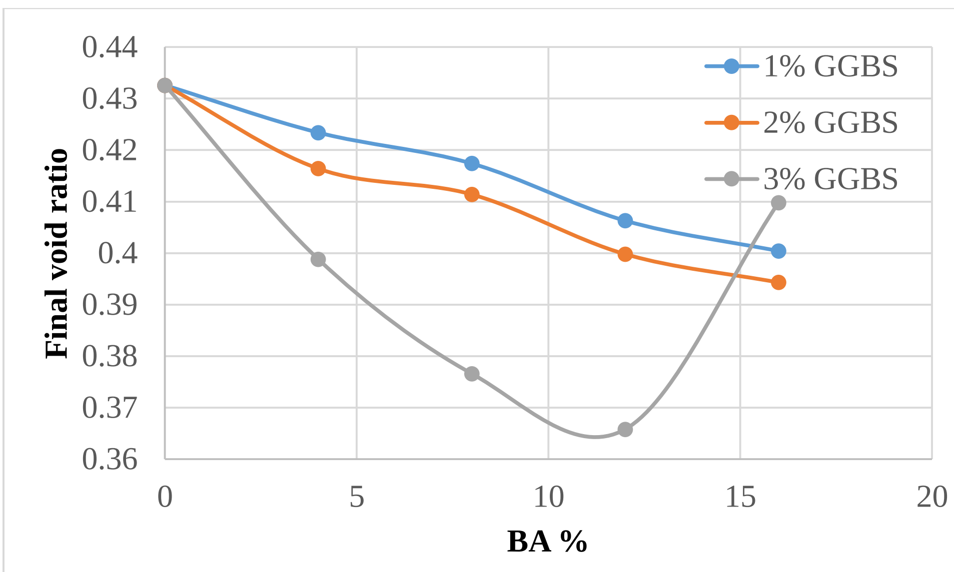

Figure 4.46. Variation in final void ration with different percentage of additives

Chapter 5

Conclusions and Recommendations

5.1 Conclusions

In this study, GGBS and BA were added as an addition to strengthen the expansive soils, and the mechanical behavior of the reinforced expansive soil was assessed. The impacts of additives on the performance parameters of the expansive soil were investigated in the laboratory using standard Proctor, unconfined compressive strength, oedometer, and direct shear tests. Based on the experimental test data, the study draws the following major conclusions:

- As the percentage of GGBS and BA increased, so did the soil's liquid and plastic limits. The soil was also changed from high plastic clay to low plastic clay with the addition of reinforcements. As the proportion of additions increased, the plasticity index of the soil reduced, suggesting that the percentage of clay in the soil reduced, lowering the soil's swelling potential.
- The maximum dry unit weight of the soil decreased as the percentage of GGBS and BA increased. The BA has a lower density and specific gravity than natural soil, resulting in a lower dry unit weight. The optimum moisture content increased from 23.2 to 26.78 percent because GGBS and BA particles are finer than soil particles. This was caused to an increase in the surface area of the soil particles, which was attributed to an increase in the surface area of the soil particles. It signifies that a greater amount of water was needed to wet soil particles with a larger surface area.
- The cohesion of the soil reduced as the percentage of the GGBS and BA increased. In unsoaked condition, the cohesion decreased from 24.5 kPa to 18 kPa, and which was

further reduced to 12 kPa in soaked condition. The decrease in cohesion was due the breaking of bonds between the soil particles. The angle of internal friction increased with an increase in %age of GGBS and BA, and which was due to the change in particles sizes, and as well as with an increase in the interlocking of the particles due to higher pre-consolidation pressure.

- The unconfined compressive strength of the soil increased from 98.4 kN/m^2 to 246.34 kN/m^2 with an increase in the percentage of the GGBS and BA The cementitious effects of pozzolanic activities of soil, bagasse ash and ground granulated blast furnace slag increased the unconfined compressive strength, and which was noted maximum at 12% BA and 3% GGBS, almost three times more than the natural soil, similar to that of shear strength parameters.
- The soil transformed from high swelling potential to low swelling potential as the amount of GGBS and BA increased. Furthermore, the re-consolidation pressure of the soil increased with an increase in the percentage of BA and GGBS, which alternately increased the %age reduction in compression index and swelling index.
- The alternative 3% GGBS and 12% BA provided more favorable results as compared to other alternatives.

5.2 Recommendations

- The impacts of GGBS with other pozzolanic materials such as, rice husk ash can also be tested to determine their suitability on the performance properties of expansive soils.
- The suitability of gypsum and bagasse ash to improve the performance properties of expansive soil can also be tested.

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