STUDY OF STRESS-STRAIN BEHAVIOUR OF COHESIVE SOILS MIXED WITH GRANITE DUST UNDER TRIAXIAL TEST

BY

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CERTIFICATION

This is to certify that this research work **“STUDY OF STRESS STRAIN BEHAVIOUR OF COHESIVE SOIL MIXED WITH GRANITE DUST UNDER TRIAXIAL TEST ”** was carried out by **EZEAMALUKWUE CHIEMERIE GABRIEL** with the registration number **(NAU/2017224015)** and that all the information used as contained in the work is a reflection of my personal research. All other sources of information obtained from other literary publication have been duly acknowledged.

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APPROVAL PAGE

This research work "STUDY OF STRESS STRAIN BEHAVIOUR OF COHESIVE SOILS MIXED WITH GRANITE DUST UNDER TRIAXIAL TEST" has been assessed and approved by department of civil engineering, faculty of engineering Nnamdi Azikiwe University, Awka.

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DEDICATION

This project work is dedicated to Almighty God for his infinite mercy and kindness over my life. I equally dedicate this work to my lovely parent Mr. & Mrs. Gabriel Ezeamalukwue who serve as a real source of inspiration towards my academic pursuit and my siblings for their immense contribution.

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Special thanks go to Almighty God for giving me the strength to complete this work and also for His guidance and protection throughout my stay in Nnamdi Azikiwe University I will forever be grateful to my parents, Mr. & Mrs. Gabriel Ezeamalukwue for their financial and moral support throughout my stay in school and also i am appreciating my lovely sister Ifeoma Ezeamalukwue for her support and prayers.

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ABSTRACT

This research study explores the application of granite dust as a soil stabilizer. The study investigates the effectiveness of utilizing granite dust to improve the engineering properties of soil, particularly in terms of stability and strength.

The impact of granite dust on the behavior of the soil subjected to loading was critically considered in this study. Soils blended with several amounts of granite powder at percentages of 25%, 35%, 45%, and 55% of the total weight of the soil were examined. The index properties, compaction and triaxial tests were executed to evaluate the performance of both the treated and untreated samples.

The result from the specific gravity test shows that theres an increase in the specific gravity of the natural soil sample when granite dust is added to it at varying percentages. The result from the Atterberg limit test shows that there is a decrease in the liquid limit of the soil and a corresponding decrease in the plastic limit of the soil due to the increase in the granite dust proportion. The result of the compaction test carried out shows that the dry density of the soil increases with addition of granite dust at varying percentages. The triaxial test analysis shows that there is an improvement in the strength properties of the soil as a result of an increase in the cell pressure with an increase in the granite dust proportion.

The results emphasized that granite dust has the potential to be a practical and sustainable choice for soil stabilization, demonstrating positive impacts on soil cohesion, compaction, and shear strength. This study offers valuable insights for construction professionals, indicating the feasibility of integrating granite dust as a soil stabilizer in diverse civil engineering endeavours.

TABLE OF CONTENTS

CERTIFICATION i
APPROVAL PAGE ii
DEDICATION iii
ACKNOWLEDGEMENT iv
ABSTRACT v
TABLE OF CONTENTS vi
LIST OF TABLES viii
LIST OF FIGURES ix
LIST OF PLATES xi
LIST OF SYMBOLS AND ABBREVIATIONS xii
CHAPTER ONE 1
1.0 INTRODUCTION 1
1.1 BACKGROUND OF STUDY 1
1.2 STATEMENT OF PROBLEM 3
1.3 AIMS AND OBJECTIVES 3
1.4 SCOPE OF STUDY 4
1.5 SIGNIFICANCE OF STUDY 4
CHAPTER TWO 5
2.0 LITERATURE REVIEW 5
2.1 PREMABLE 5
2.2 SOIL IMPROVEMENT 6
2.3 GRANITE DUST 7
2.3.1 USES OF GRANITE DUST. 7
2.4 COHESIVE AND COHENSIONLESS SOIL. 8
2.5 CLAY 10
2.5.1 CLAY MINERALOGY 11
KAOLINITE 12
MONTMORILLONITE 13
ILLITE 14
2.5.2 PROPERTIES OF CLAY 15

2.5.3 USES OF CLAY 16
2.6. STRESS-STRAIN BEHAVIOR OF SOILS UNDER TRIAXIAL TEST....17
2.7. SUMMARY OF REVIEWED LITERATURE 19
CHAPTER THREE 24
3.0 MATERIALS AND METHODOLOGY....24
3.1 METHOD OF SAMPLE COLLECTION AND PREPARATION OF SAMPLE COLLECTION....24
3.1.1. CLAY SOIL 24
3.1.2. GRANITE DUST....26
3.2. METHOD OF TESTING....26
3.3 LABORATORY TESTS....26
3.3.1. PARTICLE SIZE DISTRIBUTION (BS 1377: PART2, 1990). 27
3.3.2. SPECIFIC GRAVITY (BS1377: PART 2, 1990)....28
3.3.3 COMPACTION TEST (BS 1377:PART4,1990)....30
3.3.4. Atterberg/consistency limits test (BS 1377: PART 2, 1990). 31
3.3.5. Triaxial Test (BS EN 17892-8-2018)....35
CHAPTER FOUR....39
4.0. RESULT AND ANALYSIS....39
4.2. SPECIFIC GRAVITY TEST RESULT....43
4.3 COMPACTION TEST RESULT....45
4.4 ATTERBERG LIMITS RESULT....48
4.5 TRIAXIAL TEST RESULT 51
4.6 SUMMARY OF MOHR CIRCLE RESULT....61
CHAPTER FIVE 63
5.0. CONCLUSION AND RECOMMENDATIONS 63
5.1. CONCLUSION....63
5.2. RECOMMENDATION 63
REFERENCES 64
APPENDIX....68

LIST OF TABLES

Table 4.1 Natural soil property of sample A	39
Table 4.1.1. Natural soil property of sample B	39
Table 4.1.2. Sieve analysis of sample A	40
Table 4.1.3 Sieve analysis of sample B	41
Table 4.2. Specific gravity test result for sample A	43
Table 4.2.1. Specific gravity test result for sample B	44
Table 4.4 Atterberg limit result for sample A	48
Table 4.4.1 Atterberg limit result for sample B	48
Table 4.5: Triaxial test result of 100% clay (sample A)	51
Table 4.5.1: Triaxial test result of 75% clay, 25%GD (sample A)	52
Table 4.5.2: Triaxial test result of 65% clay, 35%GD (sample A)	53
Table 4.5.3: Triaxial test result of 55% clay, 45%GD (sample A)	54
Table 4.5.4: Triaxial test result of 45% clay, 55%GD (sample A)	55
Table 4.5.5: Triaxial test result of 100% clay (sample B)	56
Table 4.5.6: Triaxial test result of 75%, clay 25%GD (sample B)	57
Table 4.5.7: Triaxial test result of 65%, clay 35%GD (sample B)	58
Table 4.5.8: Triaxial test result of 55%, clay 45%GD (sample B)	59
Table 4.5.9: Triaxial test result of 45%, clay 55%GD (sample B)	60
Table 4.6 Summary of mohr circle result (SAMPLE A)	61
Table 4.6.1Summary of mohr circle result (SAMPLE B)	62

LIST OF FIGURES

Fig 2.0: Single unit of tetrahedral mineral. 11

Fig 2.1: Single unit of octahedral mineral 11

Fig 2.2: Structure of Kaolinite crystal 12

Fig 2.3: Structure of Montmorillonite crystal 14

Fig 2.4: Structure of Iillite crystal 15

Fig 4.1 Sieve analysis for sample A 40

Fig 4.1.1. Sieve Analysis for Sample B 42

Fig 4.2 Specific gravity result sample A 43

Fig 4.2.1 Specific gravity result sample B 44

Fig 4.3: Compaction test result of Sample A 45

Fig 4.3.1: Compaction test result of Sample B 46

Fig 4.3.2: Variation in MDD with increase in granite dust dosage (sample A) 46

Fig 4.3.3: Variation in MDD with increase in granite dust dosage (sample B) 47

Fig 4.4 Liquid limit result for sample A 48

Fig 4.4.1 Liquid limit result for sample B 49

Fig 4.4.2: Variation in the liquid limit with each addition of granite dust. 49

Fig 4.4.3: Variation in the plastic limit with each addition of granite dust. 50

Fig 4.5: Morh circle for 100% clay (sample A) 51

FIG 4.5.1: Stress-strain curve 100% clay (sample A) 52

Fig 4.5.2: Morh circle for 75% clay, 25%GD (sample A) 52

FIG 4.5.3: Stress-strain curve 75% clay, 25%GD (sample A) 53

Fig 4.5.4: Morh circle for 65% clay, 35%GD (sample A) 53

Fig 4.5.5: Stress-strain curve 65% clay, 35%GD (sample A) 54

Fig 4.5.6: Morh circle for 55% clay, 45%GD (sample A) 54

Fig 4.5.7: Stress-strain curve 55% clay, 45%GD (sample A) 55

Fig 4.5.8: Morh circle for 45% clay, 55%GD (sample A) 55

Fig 4.5.9: Stress-strain curve 45% clay, 55%GD (sample A) 56

Fig 4.6: Morh circle for 100% (sample B) 56

Fig 4.6.1: Stress-strain curve 100% clay (sample B) 57

Fig 4.6.2: Morh circle for 75%clay, 25%GD (sample B) 57

Fig 4.6.3: Stress-strain curve 75% clay, 25%GD (sample B) 58

Fig 4.6.4: Morh circle for 65%clay, 35%GD (sample B) 58

Fig 4.6.5: Stress-strain curve 65% clay, 35%GD (sample B) 59

Fig 4.6.6: Morh circle for 55%clay, 45%GD (sample B) 59

Fig 4.6.7: Stress-strain curve 55% clay, 45%GD (sample B) 60

Fig 4.6.8: Morh circle for 45%clay, 55%GD (sample B) 60

Fig 4.6.9: Stress-strain curve 45% clay, 55%GD (sample B) 61

LIST OF PLATES

Plate 3.0: Map showing location of soil sample A 26

Plate 3.1: Map showing location of soil sample B 26

Plate 3.2: Apparatus for sieve analysis test 28

Plate 3.3: Apparatus for specific gravity test 30

Plate 3.4: Apparatus for compaction test 31

Plate 3.5: Apparatus for Atterbeg limits test. 34

Plate 3.6: Apparatus for triaxial test 38

LIST OF SYMBOLS AND ABBREVIATIONS

AASHTO - American association of state of highway and transportation officials

ASTM - American society for testing and materials

BS - British standard

C - Cohesion

CL - Clay soil

SC- Clayey sand

GD - Granite Dust

CU - Coefficient of uniformity

CC - Coefficient of curvature

DD - Dry density

GS - Specific gravity

LL - Liquid limit

MDD - Maximum dry density

OMC - Optimum moisture content

PL - Plastic limit

PI - Plasticity index

USCS - Unified soil classification system

σ - Sigma

Φ - Phi (angle of internal friction)

τ- tau (shear strength)

CHAPTER ONE

INTRODUCTION

1.1 BACKGROUND OF STUDY

Soil is a construction material in various civil engineering projects, and it supports structural foundations. It is very important to predict the stress and strain imposed to a given point in a soil mass due to certain loading conditions. This is necessary to estimate settlement and to conduct stability analysis and to determine the stress conditions on underground and earth retaining structures (Thu Htun, 2011).

Geotechnical engineering involves the use of soils as engineering materials. This enables engineers to identify suitable soil environments for building and construction purposes. A soil's ability to compact and maintain its consistency under pressure determines whether it will provide a suitable foundation for building. (Akolade and Olaniyan, 2014).

Cohesive soil is a type of soil that contains a significant amount of clay particles. These soils are also known as clayey soils and are typically found in areas where sedimentary rock has weathered over time. The cohesive nature of this type of soil comes from the strong bonds that form between the clay particles due to their small size and flat shape. (Chandra, et. al.2021).

One of the defining characteristics of cohesive soil is its ability to hold water for long periods of time. This can make it difficult to work with, especially in construction projects where the soil needs to be excavated or compacted (Neeraj 2021). Additionally, when cohesive soil becomes saturated with water, it can become very soft and lose its strength, making it prone to collapse or landslide.

The strength and deformation behavior of soil materials are often evaluated by using conventional triaxial compression tests, where a cylindrical soil specimen under axisymmetric loading condition is tested to simplify the field conditions. However, most of the field problems, such as strip footings, embankments, and slopes are three dimensional or close to a plane strain condition.(Akolade, et al.,2014). The stress-strain and strength behavior under plane strain condition may differ from those measured under triaxial condition. To evaluate the soil behavior in such cases, a variety of plane strain or true triaxial devices have been developed.

During the triaxial test, measurements are taken of the axial deformation of the soil sample, as well as the change in the volume of the sample. From the triaxial test data, it is possible to extract fundamental material parameters about the sample, including its angle of shearing resistance, apparent dilatancy angle. These parameters are then used in computer models to predict how the material will behave in a large-scale engineering application.(lekha, 2012). An example would be predict the stability of the soil on a slope, whether the slope will collapse or whether the soil will support the shear stresses of the slope and remain in place. Triaxial tests are used along with other tests to make such engineering predictions.

In this study, from the triaxial test data, the fundamental shear strength parameters C and Φ (Cohesion and angle of internal friction), will be determined and together with other parameters from index properties and compaction test. The strength properties of the soil will be evaluated. This will help in providing information on whether the soil will support the shear stress developed and still remain intact.

1.2 STATEMENT OF PROBLEM

Construction on Cohesive soils such as soft clay/high plasticity silty soils leads to stability problems during the construction and long term settlements during the service due to its very low shear strength and very high compressibility.

Granite dust is a by-product of the granite industry and can be used as a soil stabilizer to improve the strength and durability of cohesive soils. Cohesive soils, such as clay, tend to have low shear strength and high compressibility, making them difficult to use as construction materials. When granite dust is mixed with cohesive soils, it acts as a binding agent and increases the soil's shear strength, making it more suitable for construction applications. The granite dust particles fill the voids in the soil, reducing its porosity and improving its density. This, in turn, increases the soil's load-bearing capacity, making it more resistant to deformation and settlement.

Results obtained from this study will help to solve problems associated with construction with emphases on road construction on problematic cohesive soils and will also contribute towards sustainable waste reduction by channeling waste to wealth in the construction industry.

1.3 AIMS AND OBJECTIVES

The aim of this report is to study the strain stress behavior of cohesive soil mixed with granite dust.

The underlying objectives are;

1. To evaluate the index properties of the cohesive soil samples treated with granite dust

2. To determine the compaction and strength properties of soil sample treated with granite dust.

3. Determine the optimum improvement quantity and proportion needed for improving cohesive soil sample with granite dust.

1.4 SCOPE OF STUDY

The scope of this research is centered on evaluating the stress strain behavior of cohesive soil samples mixed with granite dust. How this additive affects the engineering behaviour of cohesive soil samples will also be evaluated.

Specific test to be done include specific gravity, index properties, compaction[BSL], Atterberg limits and triaxial test . All test shall be conducted in accordance with British standard code.

1.5 SIGNIFICANCE OF STUDY

The need for this study is to provide maximum improvement effects of granite dust on cohesive soils.

Understanding the engineering behaviour of these cohesive soil mixed with granite dust is needful so as to allow for practically application.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 PREMABLE

The stress-strain behavior of soils describes the way in which soil deforms and responds to external stresses. This behavior can vary depending on factors such as soil type, moisture content, and the rate of loading. Generally, the stress-strain behavior of soils can be divided into three phases:

1. Elastic behavior: At low stress levels, cohesive soil behaves elastically, meaning that it deforms proportionally to the applied stress but returns to its original shape when the stress is removed (Das, B.M 2010). This behavior is described by Hooke's law, which states that stress is proportional to strain.

2. Plastic behavior: As the stress increases, cohesive soil may undergo plastic deformation, which means that it deforms permanently without returning to its original shape when the stress is removed (Lambe, et al. 2012). This plastic deformation is due to the soil particles sliding past each other.

3. Failure: If the stress continues to increase beyond the yield point, the soil will reach its ultimate strength and will fail, which can lead to collapse or deformation of structures built on it (Bowles, J.E.2009).

The stress-strain behavior of soils is important in the design and construction of structures on or in them, such as foundations, retaining walls, and slopes. Understanding the strength, stiffness, and deformation characteristics of soils is critical to ensure the safety and stability of the structures. Soil mechanics and geotechnical engineering are the fields of study that deal with the stress-strain behavior of soils and their engineering applications.

2.2 SOIL IMPROVEMENT

Soil improvement refers to the process of modifying the properties of natural soils to make them suitable for specific engineering applications. The need for soil improvement arises when the natural soil at a site is unsuitable for a particular engineering project due to factors such as low strength, high compressibility, low bearing capacity, or poor drainage. Soil improvement can be achieved using various techniques, including:

1. Compaction: This involves the densification of soil by applying mechanical energy to reduce its volume and increase its density. Compaction improves the soil's bearing capacity, reduces settlement, and increases shear strength(Lambe, et al. 2012).

2. Grouting: This involves the injection of a fluid grout material into the soil to fill voids, cracks, or weak zones. Grouting improves the soil's strength, reduces permeability, and improves its load-carrying capacity (Bruce, 2017).

3. Stabilization: This involves adding a stabilizing agent, such as lime, cement, or fly ash, to the soil to improve its strength, stiffness, and durability (Mitchell, 2005).

4. Reinforcement: This involves the placement of reinforcing elements, such as geotextiles, geogrids, or steel bars, into the soil to increase its tensile strength and reduce deformation (Koerner, 2012).

5. Preloading: This involves applying a temporary load to the soil to consolidate it and reduce its settlement before construction.

6. Vibro-compaction: This involves the use of a vibrating probe to compact the soil by densifying and rearranging its particles, improving the soil's load-carrying capacity and reducing its compressibility.

The choice of soil improvement technique depends on the type of soil, the nature of the engineering project, and the site-specific conditions. Soil improvement is an important aspect of geotechnical engineering, and its successful implementation is critical for the safe and cost-effective construction of infrastructure projects such as buildings, bridges, and highways.

2.3 GRANITE DUST

Granite dust is an industrial by-product with an ever-increasing demand in the construction industry. It is an industry by-product originating from the primary crushing stage of aggregates. it is deposited in huge amounts at quarry sites and crushing industries. Granite dust is a non-plastic material that exhibits high shear strength with zero carbon emissions. The physical properties, chemical composition, and mineralogy of stone dust vary with the type of parent rock, but is consistent with the quarry at site.

2.3.1 USES OF GRANITE DUST.

Although granite dust is considered a waste product, it has several potential uses, including:

1. Soil Cement Blocks: Granite dust can be incorporated into soil-cement blocks to enhance their strength and durability. This application is particularly useful in low-cost housing construction (Nataraja. et al. 2019).

2. Stabilization of Soil: Adding granite dust to soils can enhance their engineering properties, such as improved strength, stability, and reduced permeability. It is commonly used in road construction and slope stabilization projects. (Indraratna, B., & Reddy, K. R. 2018).

3. Concrete Production: Granite dust can be used as a partial replacement for fine aggregate (sand) in concrete production. It can improve the workability, strength, and durability of concrete. (Raman, S. N., &Subraman 2012).

4. Manufacture of Bricks and Tiles: Granite dust can be utilized in the production of bricks and tiles. It acts as a filler material, improving the mechanical properties of the products. (Sathik, et al 2019).

2.4 COHESIVE AND COHENSIONLESS SOIL.

Soils get classified as cohesive or cohesionless. A cohesive soil has an attraction between particles of the same type, origin, and nature. Therefore, cohesive soils are a type of soil that stick to each other. Cohesive soils are the silts and clays, or fine-grained soils. A cohesionless soil (non-cohesive) soil are soils that do not adhere to each other and rely on friction. These soils are the sands and gravels, or coarse-grained soils. The cohesive soil type is particularly relevant when it comes to erosion, and stormwater runoff as they soils are less likely to or harder to erode. Hence, cohesive soil particles stick to each other. These soils can be a mix of grain sizes, but are usually primarily fine-grained. The cohesionless soils may be larger grained and erode easier as they do not stick to each other (Jackson, 2020).

Soils can behave quite differently depending on their geotechnical characteristics. In cohesionless soils, where the grains are larger than 75μm, the engineering behavior is influenced mainly by the relative proportions of the different sizes and the density of the packing. These soils are also known as granular soils. In cohesive soils, where the grains are smaller than 75 µm, the mineralogy of the grains and the water content will have greater influence than the grain size on the soil properties. The borderline between cohesionless and cohesive soils is 75 µm, which is the smallest grain size one can distinguish with the naked eye (Braja M. Das, 2011)

Cohesive soil include clay, silty clay, sandy clay, and clayey silt soils. Cohesive soil properties are the physical and chemical characteristics of these soils that make them cohesive. Some of the key properties of cohesive soils include:

1. Plasticity: Cohesive soils have a high degree of plasticity, which means they can be molded and shaped without cracking or breaking.

2. Consistency: The consistency of cohesive soils refers to their ability to maintain their shape and resist deformation under stress.

3. Compressibility: Cohesive soils are highly compressible and can experience significant settlement under loads.

4. Shear strength: Cohesive soils have a high shear strength, which means they can resist forces that try to cause sliding or failure.

5. Swelling and shrinkage: Cohesive soils are susceptible to swelling and shrinkage due to changes in moisture content.

6. Permeability: Cohesive soils have low permeability, which means they do not allow water to pass through easily.

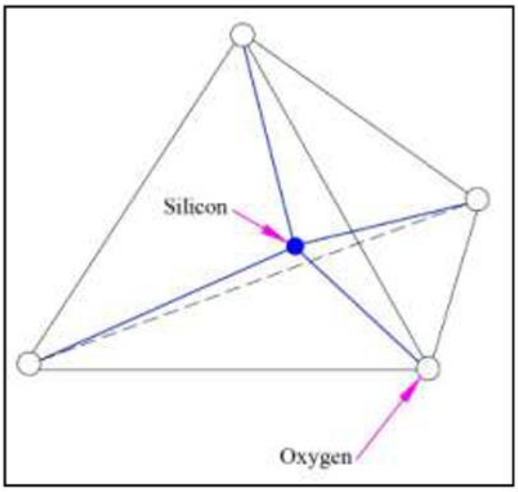
Understanding these properties is important for designing foundations, retaining walls, and other structures built on cohesive soils. Cohesive soil properties can also influence the stability of slopes and embankments, and the behavior of underground utilities and pipelines.

2.5 CLAY

Clay minerals are typically formed long periods of time by the gradual chemical weathering of rocks, usually silicate bearing by low concentrations of carbonic acid and other dilute solvents. These solvents, usually acidic, migrate through the weathering rock after leaching through upper weathered layers. In addition to the weathering process, some clay minerals are formed by hydrothermal activity. Clay may be formed in placed as residual deposits in soil, but thick deposits usually are formed as the result of a secondary deposition process after they have been eroded and transported from their original location of formation. Clay deposits are typically associated with very low energy depositional environment. Primary clays, also known as Kaolin, are located at the site of formation (Hansen, 2017). Secondary clay deposits have been moved by erosion and water from their primary location. They are naturally occurring materials composed of primarily grained materials. Clay deposits are mostly composed of clay minerals, a subtype of phyllosillitate minerals which import plasticity and harden when fired or dried they also any contain variable amount of water trapped in the mineral structure by polar attraction. Organic materials which do not impart plasticity may also be a part of clay deposits. Clay are distinguished from other grained soils by differences in size and mineralogy. However silt also exhibits clay properties. The distinction between silt and clay varies by discipline, Geologists and soil scientists usually consider the separation to occur at a particular size of 2µm clay being finer than silt. Geotechnical engineers distinguish clay and soil based on the plasticity properties of the soil as measured by the soil. Atterberg limits ISO. 14688 grades clay particles being smaller than 2µm and & silts bigger.

2.5.1 CLAY MINERALOGY

Clay minerals are crystalline sheet like structure, which consist of hydrous alumino-silicates and metallic ions. There are two fundamental crystal units of clay minerals, i.e. tetrahedral and octahedral. A tetrahedral unit belongs to four oxygen enclosing silicon, where as an octahedral unit composes of six oxygen or hydroxyls at corners surrounding aluminum, magnesium, iron or other ions. The schematic of basic tetrahedral and octahedral unit are presented in Figures 2.0 and 2.1 respectively.



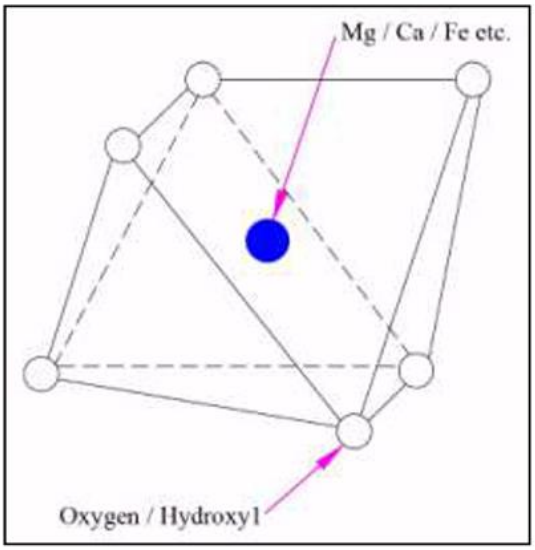


Fig 2.0: Single unit of tetrahedral mineral. **Fig 2.1: Single unit of octahedral mineral**

Based on the arrangement of stacks, bonding, isomorphous substitution, and presence of metallic ions, different clay minerals can be constituted. Some of the common clay minerals are kaolinite, montmorillonite, illite, nontronite, muscovite, etc. However, for engineering purpose kaolinite, montmorillonite and illite have particular importance in geotechnical engineering.

KAOLINITE

Kaolinite is known as 1:1 mineral because the inherent crystal structure consists of one tetrahedral and one octahedral sheet. Successive basic layers are bonded together by hydrogen bond between hydroxyls of the octahedral sheet and oxygen of the tetrahedral sheet. Due to this hydrogen bond, a large crystal of kaolinite is developed. The thickness of the basic crystal layer is 0.72 nm. A schematic of the crystal structure of kaolinite is presented in Figure 2.2 below.

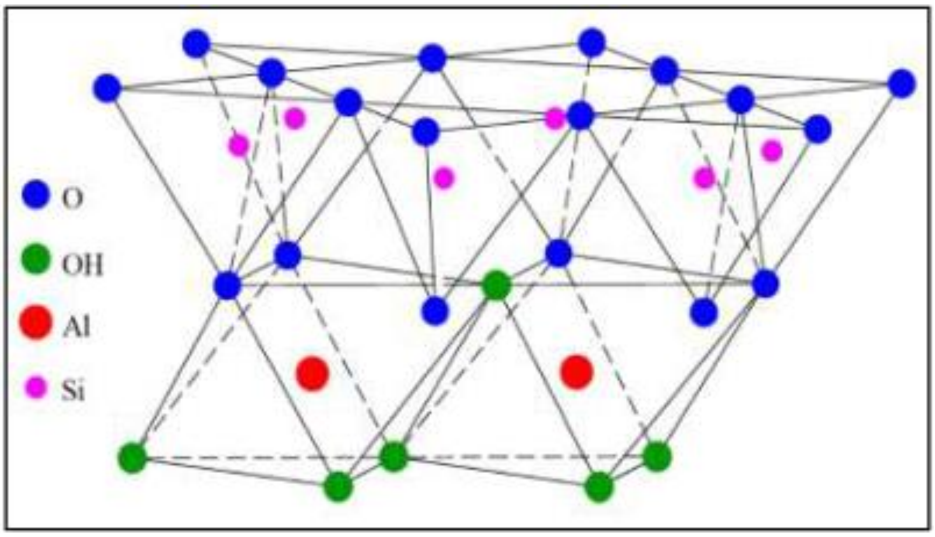


Fig 2.2: Structure of Kaolinite crystal

Typically, exposed hydroxyl can be replaced by exchangeable ions, and 3 Al can be substituted for 4 Si . Moreover, the presence of a divalent ion can cause a substitution of divalent ion for 3 Al . The ranges of cation exchange capacity in kaolinite are in between 3 to 15 meq/100g. The surface morphology of kaolinite mineral is characterized by six sided hexagonal plates. The lateral dimension and thickness of the plates are ranged from 0.1 to 0.4 m and 0.05 to 2μm, respectively because of the crystal structure and morphology, the typical specific surface area of kaolinite ranges between 10 and 20m/g.

MONTMORILLONITE

The basic unit of montmorillonite consisted of two silica sheets and one alumina sheet. This mineral is known as 2:1 mineral where the distance between the unit cells is approximately 0.96nm. The top of the silica sheets are bonded by van der Waals force, and there is a net negative charge deficiency in octahedral sheet. Therefore, water and exchangeable ions can center and break the layer. The structural unit of montmorillonite is presented in Figure 2.3.Because of the layer separation and hydration, montmorillonite mineral is characterized by swelling behavior. In addition, montmorillonite minerals show extensive isomorphous substitution for 4 Si and 3 Al by available cations. According to the literature, 3 Al can replace as much as 15% of 4 Si in the tetrahedral sheet. The overall charge deficiency resulting from the ion substitution ranges from 0.5 to 1.2 per unit cell. The typical ranges of cation exchange capacity of montmorillonite are between 80 and 150 meq/100g.

The surface morphology of montmorillonite mineral is characterized by equidimensional flakes, and may appear as thin films. Furthermore, directional strain may cause by large amount of substitution of 3 Fe and/or 2 Mg for 3 Al ,which may result needle shaped fabric structure in montmorillonite. Due to the inherent configuration and high surface activity, the specific surface area (exclusive of interlayer zone) of montmorillonite can vary from 50 to 120m/g.

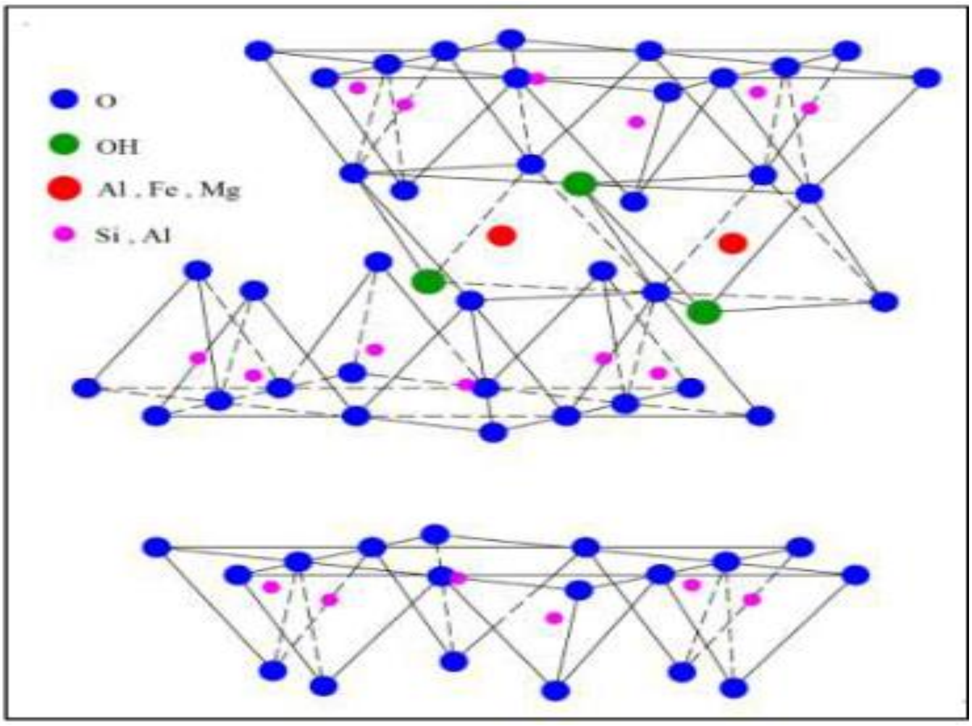


Fig 2.3: Structure of Montmorillonite crystal

ILLITE

Illite mineral is composed of two silica sheets and one alumina sheet, and known as 2:1 mineral. The basic unit configuration is similar to montmorillonite; however, the basic layers are bonded by potassium. The diameters of hexagonal aperture in silica sheet are exactly similar to the ionic radius of potassium (K). Therefore, the presence of potassium (K) makes the bond between the layers very strong. The schematic of the structure ofillite is presented in Figure 2.4. The overall charge deficiently is mostly in the silica sheets, and ranged between 1.3 to 1.5 unit per cell. The additional charge is balanced by non-exchangeable potassium (K) ions. The typical cation exchange capacity of montmorillonite ranges from 10 to 40 meq/100 g (Firoozi, 2016).

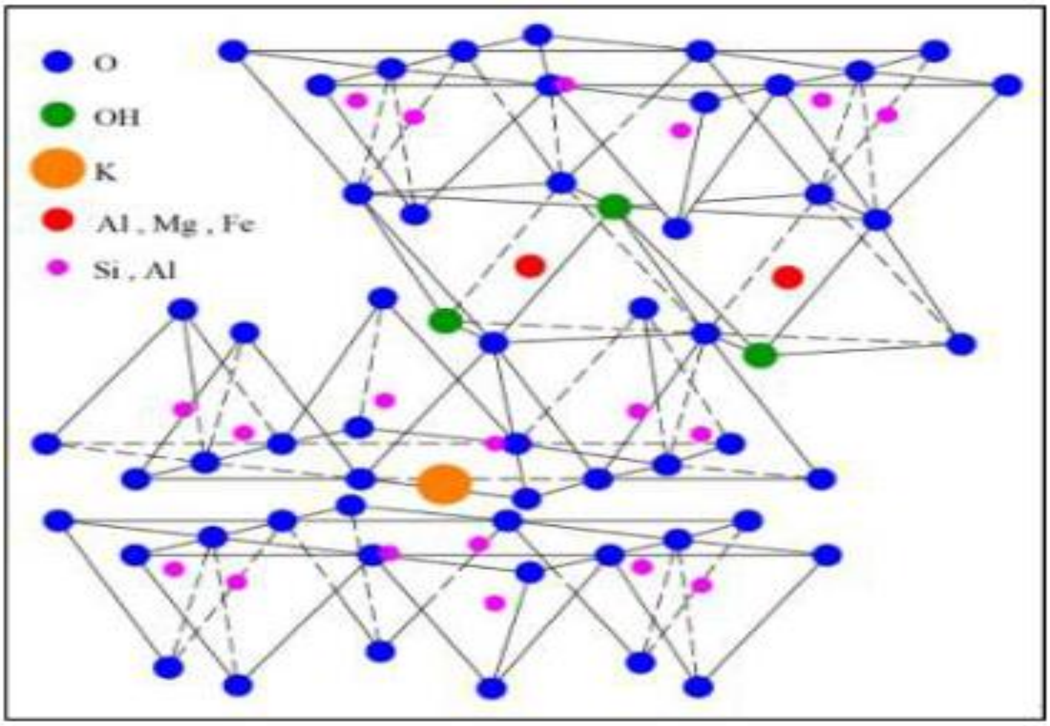


Fig 2.4: Structure of Iillite crystal

The fabric morphology of illite is characterized by hexagonal small flaky particles when well crystallized. According to Mitchel, 2006, the surface area of this mineral ranges from 65 to 100m/g

2.5.2 PROPERTIES OF CLAY

Some of the properties of clay include the following:

1. Plasticity: Clay soils exhibit high plasticity and can be molded when wet. The plasticity index (PI) is commonly used to quantify the plasticity of clay soils. (Das, B. M. (2019).

2. Shrink-Swell Behavior: Clay soils undergo volumetric changes with changes in moisture content. They shrink when drying and swell when wetting, leading to potential ground movements. (Mitchell, J. K., & Soga, K. (2005).

3. Cohesion: Clay soils have high cohesive strength due to the attractive forces between clay particles. Cohesion is an important parameter in stability analysis and foundation design. (Coduto, D. P. (2010).

4. Permeability: Clay soils typically exhibit low permeability, resulting in poor drainage characteristics. The hydraulic conductivity of clay soils is generally low. (Holtz, R. D., Kovacs, W. D., & Sheahan, T. C. (2011).

5. Compressibility: Clay soils are highly compressible and exhibit significant settlement under load. Consolidation tests are performed to evaluate the compressibility characteristics of clay soils. (Terzaghi, K., Peck, R. B., &Mesri, G. (2006).

6. Shear Strength: Clay soils possess shear strength due to the interlocking of particles and the presence of cohesive forces. Shear strength is crucial for stability analysis and design of slopes, foundations, and retaining structures. (Lambe, T. W., & Whitman, R. V. (2011).

2.5.3 USES OF CLAY

Clay has been used in engineering for centuries due to its unique properties. Here are some common engineering uses of clay:

1. Building materials: Clay is a primary ingredient in the production of bricks, roof tiles, and other building materials. These materials are durable and long-lasting and can be easily molded and shaped into various forms.

2. Landscaping and construction: Clay is often used in landscaping and construction projects to stabilize soil and prevent erosion. It can also be used as a filler material for embankments and roadways.

3. Geotechnical engineering: Clay is used in geotechnical engineering to determine the strength and stability of soil. It can also be used as a lining material for waste containment systems.

4. Environmental remediation: Clay is used in environmental remediation projects to remove contaminants from soil and water. It has the ability to absorb toxins and contaminants, making it an effective tool for cleaning up polluted areas.

5. Soil stabilization: Clay can be used for soil stabilization in construction projects. It can be added to soil to improve its properties and prevent erosion.

6. Ceramics: Clay is a key material in the production of ceramics, including pottery, porcelain, and other ceramic products. It is used for its plasticity, strength, and heat resistance.

7. Drilling mud: Clay is also used as a component of drilling mud in oil and gas exploration. It helps to lubricate and cool the drill bit, and also helps to maintain pressure in the borehole.

2.6. STRESS-STRAIN BEHAVIOR OF SOILS UNDER TRIAXIAL TEST

In soil mechanics, triaxial testing is a common method used to measure the stress-strain behavior of soils. The test involves confining a cylindrical soil specimen within a triaxial cell, and applying an axial load to the specimen while controlling the confining pressure and measuring the resulting deformation (ASTM International, 2018).

During the triaxial test, the stress-strain behavior of the soil specimen can be observed in three dimensions: axial stress (σ_1), radial stress (σ_2 and σ_3), and axial strain (ε_1) or lateral strain (ε_2 and ε_3). The relationship between these stresses and strains can be plotted on a stress-strain curve, which is a graphical representation of the soil's behavior under different loads.

The initial stage of the test involves applying a confining pressure to the soil specimen, which is gradually increased until the desired level is achieved. Then, an axial load is applied to the specimen, causing the axial stress to increase. As the axial stress increases, the soil undergoes deformation and the lateral strain increases. (lekha, 2012).

The stress-strain curve of the soil typically exhibits three distinct stages: elastic, yielding, and failure. During the elastic stage, the soil deforms elastically, and the strain is proportional to the applied stress. Once the soil reaches its yield point, the strain increases at a much faster rate than the applied stress. At this stage, the soil has undergone plastic deformation, and it will not return to its original shape once the load is removed. Finally, when the axial stress reaches its peak, the soil fails and experiences a rapid increase in strain. The peak strength of the soil is typically reached at a certain point in the curve, which is known as the peak strength point. After the soil has failed, the axial stress decreases rapidly, and the strain continues to increase until the test is complete.

The results of a triaxial test can be used to determine the shear strength of the soil, which is an important parameter in geotechnical engineering. Additionally, the test can provide valuable information about the soil's behavior under different loads, which can be used to design foundations and other structures that are supported by soil.

2.7. Summary of Reviewed Literature

Although, many researchers work mostly on the use of fly ash, lime, cement, wood ash for the stabilization of soil, but today granite dust is also considered as one of the materials for soil stabilization.

Eltwati, et. al, (2020), studied the potential of granite dust to improve the engineering properties of Soft soils for road construction. The findings revealed that the geotechnical properties of clayey soils are enhanced considerably by the addition of granite dust. The results have shown that mixing the clayey soil with the granite dust improved the maximum dry density. Adding content of 8% of the granite powder to the untreated soil attains the best results. Also, the study found that normal stress gets higher as the shear stress elevates. The shear strength of modified soils with granite dust is 3 times better than untreated soils.

Ribeiro and camapum (2003), studied the use of quarry dust, aiming their use as the main material for the base layer of flexible pavements. The material was characterized by laboratory test to evaluate its utilization potential. The result shows that the quarry dust has a reasonable potential for use as a construction material for the base layer of flexible pavements although the quarry waste evaluated in that study seemed more appropriate to low volume roads

Ahmed Eltwati, (2021), Studied the engineering properties of clay soils stabilized with waste granite dust. In his study, It was revealed that adding granite dust to soft soils improved the dry density of natural soil by 16%. Soils mixed with 9% of the granite dust yielded the highest value of MDD, which is 1.86 g/cm^3. The result proved also that adding granite dust to soils enhanced the performance of soil subject to CBR loading. The value of CBR of natural soil was enhanced from 3.65% to 16.5 %. Therefore, the soils have a much larger bearing ratio when exposed to moderate pressure. The results showed that as the amount of granite dust increases, the value of CBR improves. The soils blended with 9% granite dust yielded the highest value of the CBR.

Akolade, et. al, (2020), made a comprehensive investigation in some geotechnical properties of cohesive soil before and after addition of different percentages of granite dust. The result revealed that increasing percentages of granite dust has an increment in the maximum dry density with a successive reduction of optimum moisture content. Thus making it more suitable as a stabilizing agent. That is, the higher the granite dust added to the soil, the higher the value of the dry density and the lower the moisture content. As more percent of Granite Dust was added to the cohesive soil A, B and C, the shear strength of the soils increased. Thus, granite dust increases the shear strength of soil. From the Mohr Circle analysis, it is observed that there is an increase in the principal stress as a result of an increase in the cell pressure with increase in the granite dust proportion.

Gudla, et. al, (2021), studied the suitability of granite dust as a sustainable additive for geotechnical applications. The result revealed that there were significant improvement in the compaction characteristics of the soil, attributed to the increase in coarse fraction followed by decrease in water absorbtion capacity of the soil.

D.A. Ahmed et al.(2015), studied the effect of granite dust on geotechnical properties of soil. This study examined the impact of granite dust on the geotechnical properties of soil, including shear strength, compaction characteristics, and permeability. The researchers found that the addition of granite dust improved these properties, leading to better soil stability and reduced erosion potential.

J. Rakshit et al. (2015), made an in-depth investigation on the influence of granite dust on the geotechnical properties of soil. This study investigated the effect of granite dust on soil engineering properties, including compaction, California bearing ratio, and shear strength. The researchers found that the addition of granite dust improved these properties, making the soil more suitable for construction and infrastructure projects.

S. Kumar et al. (2016), studied the effect of granite dust on soil stabilization and soil strength. This study examined the impact of granite dust on soil stabilization and strength, using a combination of laboratory tests and field trials. The researchers found that the addition of granite dust improved soil strength and stability, making it more suitable for construction and infrastructure projects.

N.M. Abdulrazzaq et al. (2019), made a comprehensive and thorough investigation on the Geotechnical properties of soil stabilized with granite dust and cement. This study investigated the effect of combining granite dust and cement for soil stabilization, focusing on geotechnical properties such as compaction and California bearing ratio. The researchers found that the combination of granite dust and cement was an effective method for soil stabilization, leading to improved geotechnical properties and greater soil strength.

E. Ogundipe and I. Oluyemi (2009), studied the effect of granite dust on the shear strength characteristics of lateritic soil. This study investigates the effect of granite dust on the shear strength characteristics of lateritic soil. The authors found that the addition of granite dust increased the shear strength parameters of the soil.

V. Shankar and K. Prakash (2016), studied the effect of granite dust on the shear strength of soil stabilized with lime. This article discusses the effect of granite dust on the shear strength of soil stabilized with lime. The authors found that the addition of granite dust improved the shear strength of the soil-lime mixture.

S. Sridhar et. al (2013), studied the improvement of soil shear strength using granite dust and tire chips"This study investigates the use of a combination of granite dust and tire chips as additives to improve the shear strength of soil. The authors found that the addition of the two additives improved the shear strength of the soil.

R. Balachandran et.al (2017), studied the strength improvement of clayey soil with granite dust and cement This study investigates the use of granite dust and cement as additives to improve the strength of clayey soil. The authors found that the addition of granite dust and cement improved the shear strength of the soil.

Dey, A., et al. (2021) conducted a study on the stabilization of expansive soils using granite dust and fly ash. The research demonstrated that the combination of granite dust and fly ash significantly improved the engineering properties of expansive soils, including reduced swelling potential, increased strength, and improved durability.

Aliabdo, A. A., & Fathy, A. S. (2017) conducted a study on the utilization of granite dust as a filler in cement for soil stabilization. The research demonstrated that the addition of granite dust improved the strength and stability of soil, reducing its susceptibility to erosion and enhancing its engineering properties.

Raju, G. K., et al. (2020) examined the potential of granite dust for stabilizing soft marine clay. The study found that the addition of granite dust enhanced the geotechnical properties of marine clay, such as increased shear strength, reduced compressibility, and improved stability, making it suitable for construction purposes

.Barros, J. C., et al. (2020) studied the utilization of granite dust as a raw material for the production of sustainable manufactured artificial stones. The research showed that granite dust could be effectively utilized in the production of manufactured stones, such as pavers and countertops, providing aesthetic appeal, durability, and cost-effectiveness.

Samarakoon, S. M. S. M., &Abeygunawardane, A. P. (2018) investigated the use of granite dust as a stabilizing agent for expansive clay soils. The study demonstrated that the addition of granite dust reduced the plasticity and swell potential of expansive clay soils, improving their geotechnical properties.

Rahman, M. A., et al. (2019) explored the effect of granite dust on the engineering properties of clayey soils. The results showed that the addition of granite dust enhanced the stability, shear strength, and resistance to erosion of clayey soils, making them more suitable for geotechnical applications.

Uzoh, F. C., &Onyelowe, K. C. (2017) investigated the effect of granite dust on the strength characteristics of lateritic soil. The study found that the inclusion of granite dust improved the shear strength and California bearing ratio (CBR) of the soil, making it suitable for construction purposes.

Balakrishnan, M., et al. (2018) investigated the effect of granite dust on the stabilization of expansive clay soils. The study revealed that the inclusion of granite dust reduced the plasticity and swelling potential of expansive clay soils, resulting in improved stability and reduced susceptibility to volume changes.

Ogbonnaya, et. al (2011), investigated the potential effect of granite dust on the geotechnical properties of abakaliki clays. The research involved the addition of granite dust in the dossage of 10%, 15%, 20% by weight to the natural soil sample. The results of their study demonstrated a significant increase in the maximum dry density of the clay soil sample as well as a corresponding increase in the shear strength of the soil when granite dust is added to it at varying percentages.

CHAPTER THREE

3.0 MATERIALS AND METHODOLOGY

This chapter describes the details of the laboratory experiments carried out. A wide range of laboratory tests was carried out to investigate the effects of granite dust on the stress-strain behavior of cohesive soils under triaxial test.

3.1 METHOD OF SAMPLE COLLECTION AND PREPARATION OF SAMPLE COLLECTION

The materials used in this work are as follows

1. Clay Soil
2. Granite dust

3.1.1. CLAY SOIL

Two clay soil sample were used for this research, sample A was obtained from IfiteAwka, Anambra state. Its coordinate is latitude 6°15'40N and longtitude 7°8'06.44E. Sample B was collected from EFAB estate. EFAB estate is located at Amansea along Enugu-Onitsha express way Awka, Anambra state. Its coordinate is latitude 6°14′56.832N longitude 7°8′4.56E.

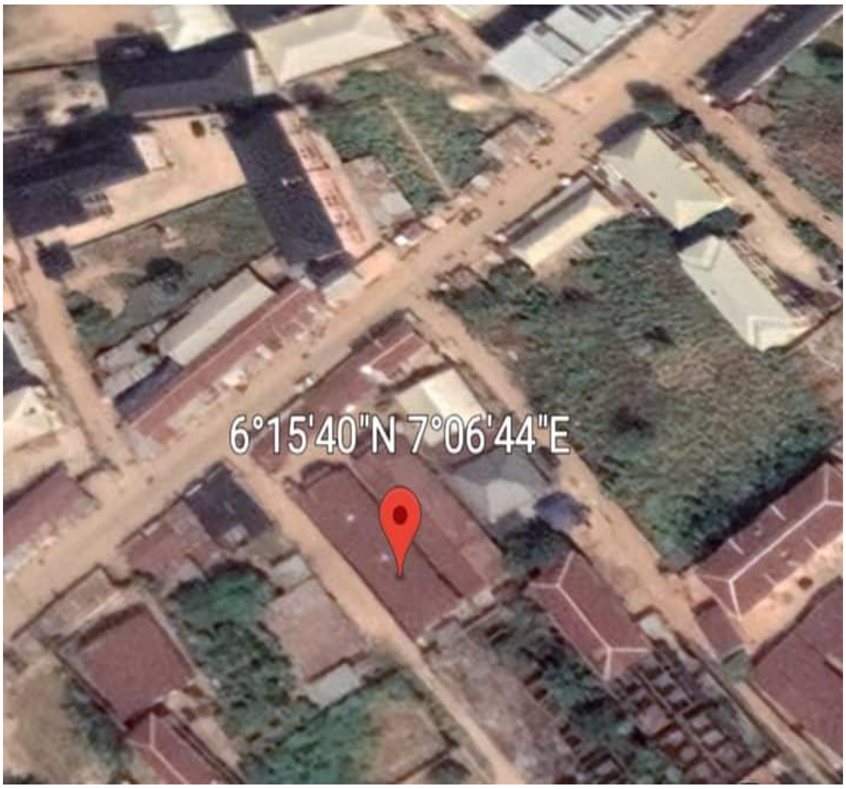


Plate 3.0: Map showing location of soil sample A

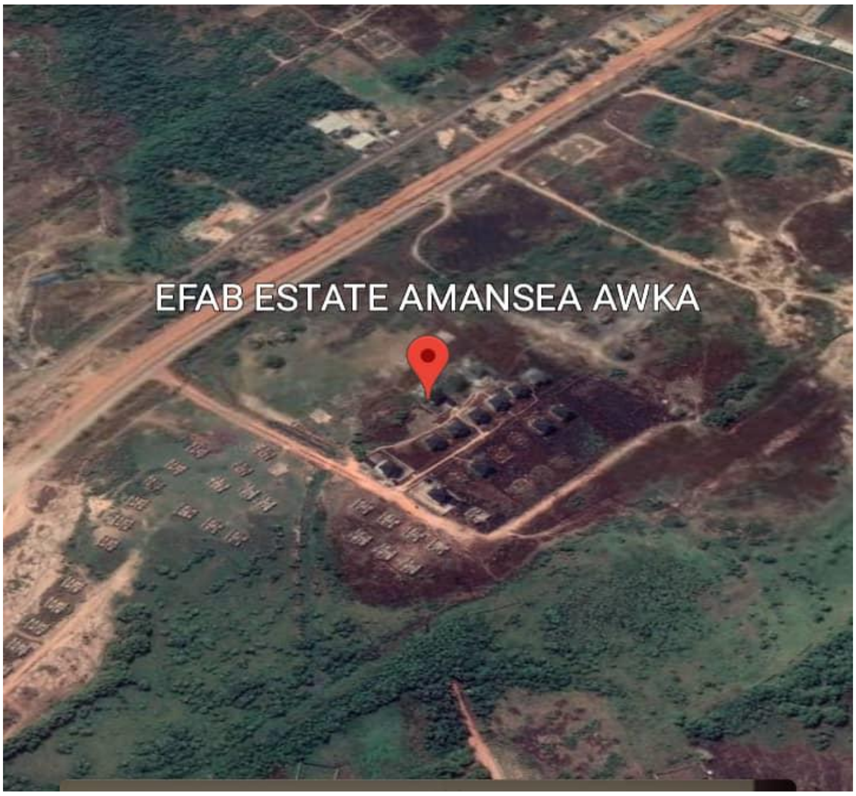


Plate 3.1: Map showing location of soil sample B

The clay sample obtained was collected with the aid of a digger and shovel at a depth of 600m. The sample passed all physical test that could classify them as clay soil which is brown in color, has smooth texture and exhibit plastic behavior when wet and .the was sample was prepared in the soil laboratory of the civil engineering department.

3.1.2. GRANITE DUST

The granite dust used for this research was obtained at a construction dump site along Enugu-Onitsha express way.

3.2. METHOD OF TESTING

The method of testing used in this work was done in accordance with BS 1377, 1990.

3.3 LABORATORY TESTS

This Section presents the experiment procedure and laboratory test that will be adopted for this research work. The test that will be conducted are as follows:

1. Particle Size Distribution.
2. Specific gravity
3. Compaction test
4. Atterberg limits
5. Triaxial test

3.3.1. PARTICLE SIZE DISTRIBUTION (BS 1377: PART2, 1990).

Sieve analysis of soil is a simple operation of dividing a sample of aggregate into fractions, each consisting of particles of the same size.

Aim.

The test is performed to determine the percentage of different grain sizes contained within a given soil.

Apparatus

Weighing balance, bowel, set of sieves, cleaning brush, sieve shaker (vibrator), timing device and a thermostatically controlled drying oven.

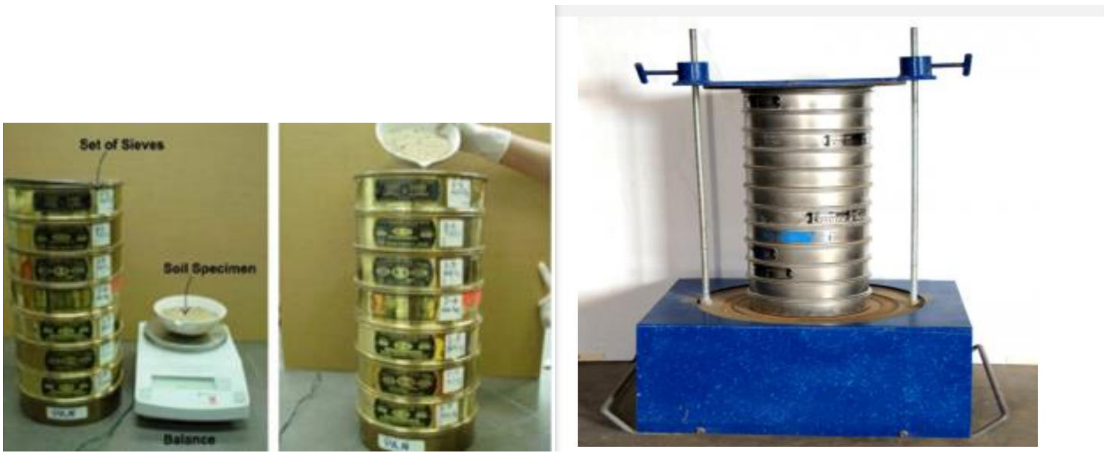


Plate 3.2: Apparatus for sieve analysis test

Procedure

Five hundred (500) gram of each of the soil samples were washed with sieve no. 200 (0.075mm) with tap water until the water was clear. The purpose was to wash out the fines (silt and clay). The washing was continued until the filtrate becomes clear. The residues were carefully poured back to a pan from sieve after washing without leaving any particle behind and then dried in the oven. The dried residues were taken out of the oven, allowed to cool in the desiccator and weighed. Their weights were recorded. The sieves used include: 2mm, 1.18mm, 0.85mm, 0.6mm, 0.425mm, 0.3mm, 0.15mm, 0.075mm, and the pan. The sieves were arranged in descending order from 2mm to 0.075mm to the pan in the sieve shaker. Plate 3.2 shows the way the sieves were arranged in descending order. The soil samples were poured into the topmost sieve (2mm), closed with appropriate cover and screwed tightly to the sieve shaker. Mechanical sieve shaker was used for the sieving. The sieve shaker was switched on and allowed to shake the soil for ten (10) minutes.

After this, it is switched off and the mass retained on each sieve were recorded in tabular form. The readings obtained are used to compute the percentage of soil sample passing each sieve. The percentages passing were plotted against the sieve sizes in a semi-logarithmic graph to produce the grading curve.

3.3.2. SPECIFIC GRAVITY (BS1377: PART 2, 1990).

Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at the same temperature. The specific gravity of a soil is used in the phase relationship of air, water, and solids in a given volume of density bottle or pycnometer, porcelain dish, weighing balance, funnel and spoon.

An oven dried soil sample passing through No.40 sieve size (425um) was left for about one (1) hour in the desiccator to cool. The empty density bottle was weighed and recorded as M1. The oven dried soil sample was poured into the density bottle up to 1/3 the volume and then weighed to obtain M2. Some distilled water was added to the soil in the bottle and stirred thoroughly and the vacuum pump was used to remove all air bubbles in the sand-water mixture. More distilled water was added up to the white mark on the bottle neck and this was left for eight (8) hours to saturate. The density bottle + soil + water were then weighed to obtain M3. Finally the density bottle was emptied, washed thoroughly with distilled water, dried in the oven, cooled in the desiccator and was filled with distilled water to the same mark. Again the vacuum pump was applied to remove air bubbles. The density bottle + water were weighed to obtain M4. Using Equation 3.6, the specific gravity of the soil was obtained.

Specific gravity, GS = $\frac{W2-W1}{(W4-W1)-(W3-W2)}$

Where:

W1=weight of empty density bottle

W2=weight of density bottle with dry soil

W3=weight of density bottle with soil and water

W4=weight of density bottle with water only

Plate 3.3: Apparatus for specific gravity test

3.3.3 COMPACTION TEST (BS 1377:PART4,1990)

This laboratory test is performed to determine the relationship between the moisture content and the dry density of soil for a specified compaction energy.

Aim

Compaction test is carried out with the aim of determining the moisture-density relationship of the soil. It is used to determine the maximum dry density at which the soil can be compacted given specified compaction energy. This test is used primarily to provide field control for earth works where typical specifications will require that the soil can be compacted to at least a certain percentage of maximum dry density obtained in the laboratory compaction test.

Apparatus

The following apparatus were used for the test: a cylindrical metal mould with effective height of 2450mm and volume 1000cm³, 2.5kg metal rammer, cylinder for measuring water, metal tray for mixing, tray for collecting weighted samples, electronic weighing machine, grease for oiling and cleaning rammer, hand trowels for mixing, two iron slab red-like scrappers for cutting surface of soil when compacted into mould, chisel, scoop, hammer, and a container for collecting samples to oven dry.

Plate 3.4: Apparatus for compaction test

Procedure

British Standard Light (BSL) codes of practice were adopted for the test. Three thousand gram (3000g) of each sample was collected with the aid of the weighing balance and then poured into a pan. It was then poured into a mixing tray and 4% of 3000g i.e., 120g was measured for water using the 1000ml cylinder and poured into the sample in the mixing tray. The sample was thoroughly mixed and put into the mould in three different layers and was compacted using the 2.5kg rammer at 27 blows per layer. After the third layer compaction, the compacted sample plus mould was weighed and recorded. Soil samples were collected from the top and bottom of the compacted soil respectively and put into moisture content cans for moisture content determination. The remaining soil in the mould was poured back into the mixing tray and remixed with an additional amount of water (4%) more for this and then compacted again. The process was repeated until the value of the weight of sample plus mould began to drop and was usually after five times (4%, 8%, 12%, 16%, and 20%).

The samples in the container were then oven dried for twenty-four (24) hours. They were brought out, cooled in the desiccator, reweighed and recorded as dry weight. The moisture contents, bulk density and dry density were calculated. A graph of dry density against moisture content was plotted. Maximum dry density (MDD) and optimum moisture content (OMC) was read off in the graph at the peak point. For BS light, the sample was thoroughly mixed and put into the mould in three different layers and was compacted using 2.5kg at 27 blows.

3.3.4. Atterberg/consistency limits test (BS 1377: PART 2, 1990).

The Atterberg limits test is a classification test used to determine the moisture content at which fine-grained clay and silt soils transition between the different phases.

Purpose:

The Atterberg/consistency limited test determines that moisture content at which the soil will flow under its own weight. It defines the boundaries of several states of consistency of plastic soil. It is used to determine the plasticity of soil through the determination of the following parameters: liquid limit, plastic limit, plasticity index etc.

Liquid Limit Test:

It is the moisture content at which the soil begins to behave like fluid under the influence of a standard blows.

Apparatus:

The materials and equipment used include; the liquid limit (Cassagrande) apparatus, mortar and pistil, a thermostatically controlled oven, a grooving tool, a flat glass plate of about 10mm thick (500mm long and 500mm wide) for mixing soil, two palette knives, a balance readable and accurate to 0.01g, moisture content cans, a bottle of distilled water, calculator etc.

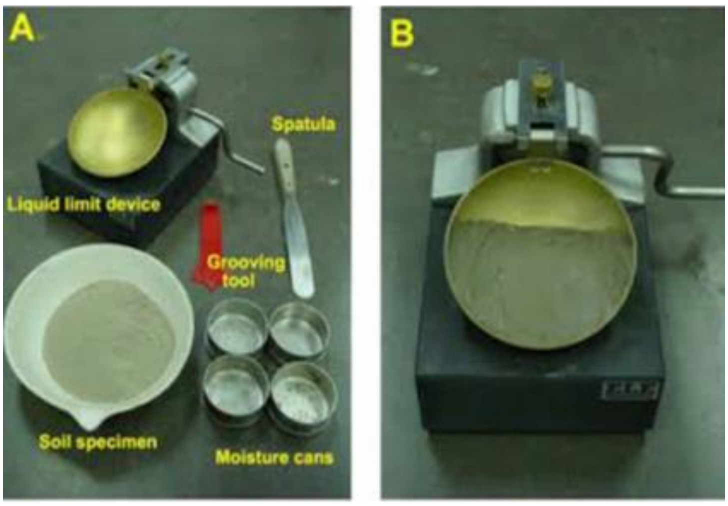


Plate 3.5: Apparatus for Atterbeg limits test.

Procedure:

About five hundred grams (500g) of soil passing through sieve no. 40 (0.425mm) was placed on a neat glass plate. A small quantity of distilled water was added to the soil from the water bottle. The soil was mixed thoroughly with the water until a uniform and consistent paste was produced. A portion of the paste was placed in the brass cup of the liquid limit apparatus using the spatula until it was almost half-filled. The top of the wet soil was levelled off symmetrically with the spatula such that it was parallel to the rubber base and the maximum depth of the soil was one (1) cm. A v-shaped groove, two (2) mm wide at the bottom, eleven (11) mm at the top and eight (8) mmdeep was made on the paste using a standard grooving tool with the grooving being applied at the center line of the cam follower and at an instantaneous tangent to the surface of the curved cup. After setting the counter of the apparatus to zero, the handle of the apparatus was turned at the rate of 2 revolutions per second. The number of blows required to close the groove were counted and recorded. Two representative moisture samples were taken from the closed groove section in moisture cans, weighed and recorded. The can with the wet soil were placed in the oven to dry. When this was done, more water was added to the soil mixture so as to use an even smaller number of blows to close the groove. The quantity of water added to the soil incrementally was conditioned to ensure that the number of blows required to close the groove fall within the given range :10-16, 17-21, 22-28, 29-41, 42-49. This procedure was repeated 4 more after the given drying period in the oven, they were put in the desiccator to cool, reweighed again to determine the mass of moisture cans plus dry contents. The moisture contents were computed using the formula for moisture content determination shown above in Equation (3.1). The results were presented in a lab reporting sheet and plotted on a semi-logarithmic graph paper (moisture content (%) against no of blows) and the best straight line drawn between the points. The moisture content at twenty-five blows defines the liquid limit.

Plastic Limit:

Plastic limit is the percentage moisture at which a soil can be rolled without breaking into threads three (3) mm in diameter. These parameters help to determine the plasticity and clay content of a soil sample.

Apparatus:

A flat glass plate for rolling plastic limit threads, spatula, distilled water, moisture content cans.

Procedure:

The soil paste at different moisture contents were rolled with the palm on a glass platen into threads until they begin to break at about 3mm. The threads were put into containers like those in the liquid limit test, weighed and recorded. They were then placed in the oven for twenty-four hours to dry. After drying, they are placed in the desiccator to cool and reweighed. The weights were used to determine their moisture contents. The average of this moisture content is the plastic limit. The results of the two tests were analyzed and the plasticity index (PI) was obtained as the numerical difference between the liquid limit (LL) and plastic limit (PL).

Plasticity Index, PI:

This is a measure of the plasticity of a soil. The plasticity index is the size of the range of water contents where the soil exhibits plastic properties. It is the difference between the liquid limit and the plastic limit as shown in Equation (3.2) below. Soils with a higher PI tend to be clayey while those with a lower PI tend to be silty.

$$PI = LL - PL \qquad \text{Eq.3.2}$$

Where

PI = Plasticity index

LL= liquid limit

PL =plastic limit

3.3.5. Triaxial Test (BS EN 17892-8-2018)

Triaxial shear strength test on soil measures the mechanical properties of the soil. In this test, soil sample is subjected to stress, such that the stress resulted in one direction will be different in perpendicular direction. The material properties of the soil like shear resistance, cohesion and the dilatancy stress is determined from this test.

Aim

The primary aim of the triaxial test is to determine the shear strength parameters of soil under different stress conditions. The test involves applying a constant axial load to a cylindrical soil sample while subjecting it to different confining pressures.

Apparatus

The triaxial test apparatus consists of several components, which are typically assembled in a compact and sturdy frame. The main components of a typical triaxial test apparatus include:

1. Triaxial cell: The triaxial cell is a cylindrical vessel that contains the soil sample. It has a top and bottom end cap that holds the sample in place and allows for drainage of fluids. The cell is usually made of stainless steel and has a number of ports for various functions, such as filling and draining the cell, applying confining pressure, and measuring pore water pressure.

2. Confining pressure system: The confining pressure system is used to apply a uniform pressure to the outside of the triaxial cell. The system typically consists of a pressure chamber, a pressure regulator, and a pump. The confining pressure can be increased or decreased as needed to simulate different stress conditions.

3. Axial loading system: The axial loading system is used to apply a vertical load to the soil sample. It typically consists of a load frame, which holds the triaxial cell in place, and a piston that applies the load to the top of the sample. The load can be applied at a constant rate or varied over time to simulate different loading conditions.

Other Apparatus used in Triaxial test includes:

Wise saw, Oven, Rubber membrane, Calipers, evaporating dishes and membrane stretcher.

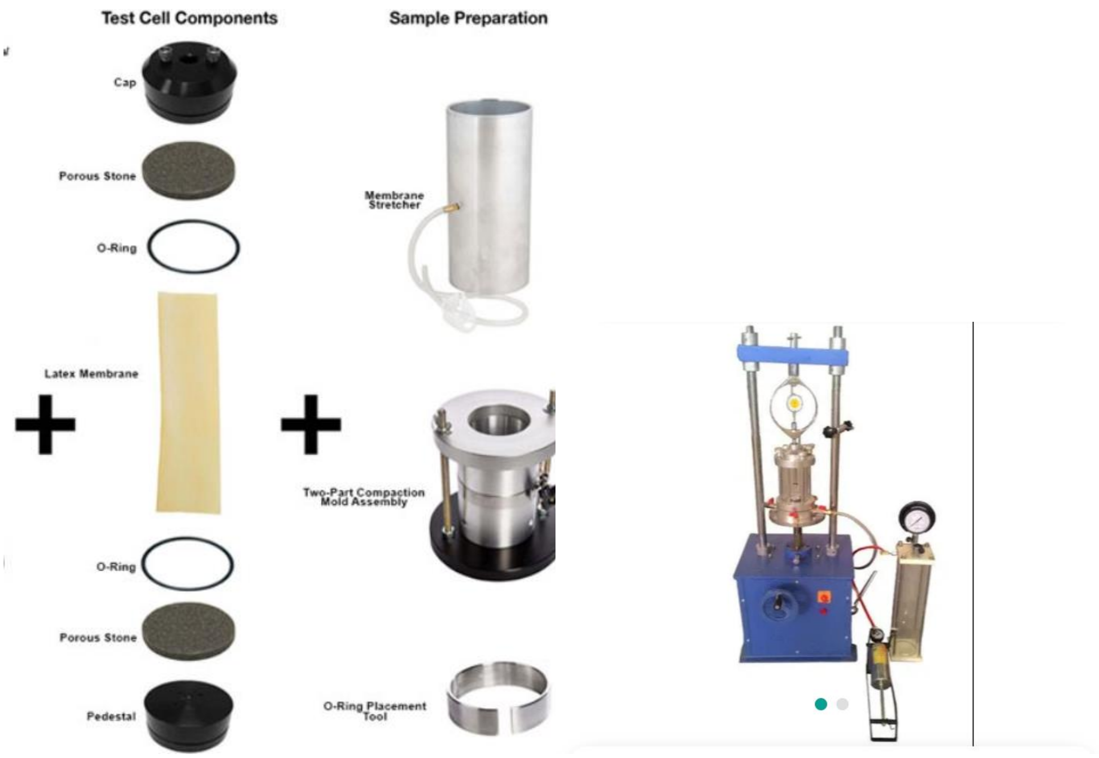


Plate 3.6: Apparatus for triaxial test

Procedure

Sample preparation: A representative soil sample is obtained and prepared according to the desired dimensions and specifications. The sample is then placed in the triaxial cell and saturated with water or other appropriate fluid.

The sample weighted and placed inside cell, over two circular pieces of plastic with the same diameter as the soil sample. A rubber membrane (sleeve), is fitted around the sample and the two pieces of plastic. This skin around the sample will avoid contact between the water and the soil. Two rubber rings are attached around the sleeve (at the top and at the bottom) for no contact with water. The cell was filled with water and the air inside removed. the pressure inside the cell was increased (Kn/m^2) by means of the foot pump. The metal tube was placed over the top of soil sample. This tube will apply vertical load to the sample. Once this tube is located over the sample. Set the readings to zero. The difference in height of the sample, before and after the sample fails, is measured (strain), keeping constant the lateral pressure. And the stress dial is recorded too (at the same time, in comparison with the strain).

Three tests are run, with different confined lateral stress (98.1, 196.1, and 294.2kN/m^2). What occurs when the sample fails is that, the readings of stress dial start to go back to the previous readings, while the vertical pressure is still acting. Once the sample fails, the test was stopped. The water inside the cell was removed and the sample was brought out to be studied.

CHAPTER FOUR

4.0. RESULT AND DISCUSSION

The results analysis was carried out in this section to relate the properties of the soil tested and its relation to the aim of the project work.

The results presented in this section include sieve analysis, specific gravity, Atterberg limits, compaction and triaxial tests.

Table 4.1 Natural soil property of sample A

Natural soil properties	Quantity
Liquid limit	40.12%
Plastic limit	23.79%
Plasticity index	16.3
Percentage passing sieve 0.075mm	70.18%
AASHTO Classification	A-7-5
Maximum dry density (kN/m^3)	18.97
Optimum moisture content (%)	11.18

Table 4.1.1. Natural soil property of sample B

Natural soil properties	Quantity
Liquid limit	43.30%
Plastic limit	25.82%
Plasticity index	17.78
Percentage passing sieve 0.075mm	81.18%
AASHTO Classification	A-7-5
Maximum dry density (KN/m^3)	18.10
Optimum moisture content (%)	11.08

4.1 SIEVE ANALYSIS TEST RESULT

The result of the particle size distribution analysis are summarized in table 4.1.2 and table 4.1.3 and plotted in fig. 4.1 and fig. 4.1.1 below.

Table 4.1.2. Sieve analysis of sample A

Sieve size (mm)	Mass Retained (g)	% mass retained (g)	Cumulative % mass retained (g)	Percentage finer (%)
2.0	0	0	0	100
1.18	1.31	1.05	1.05	98.95
0.85	7.22	5.77	6.82	93.19
0.6	18.20	14.54	21.36	78.64
0.425	22.19	17.74	39.10	60.9
0.3	27.33	21.85	60.95	39.06
0.15	26.44	21.14	82.08	17.92
0.075	16.60	13.27	95.32	4.68
Tray	5.81	4.64	100	0
Total	125.10			

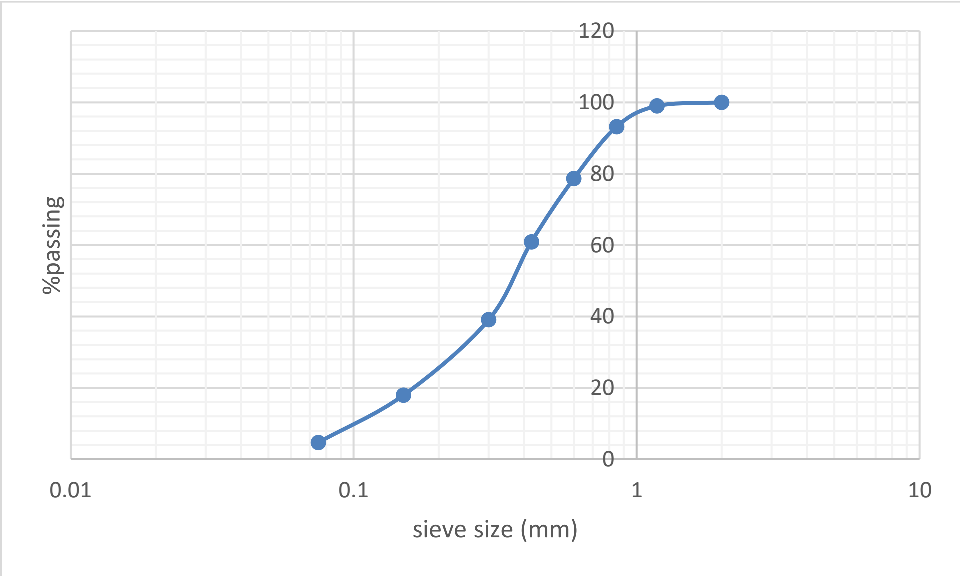


Fig 4.1 Sieve analysis for sample A

CU =$\frac{D60}{D10}$

CC = $\frac{(D30)2}{D60 \times D10}$

Where:

D10 = 0.12

D30 = 0.25

D60 = 0.43

CU =$\frac{0.43}{0.12}$ = 3.58

CC = $\frac{(0.25)2}{0.43 \times 0.12}$ = 1.21

Table 4.1.3 Sieve analysis of sample B

Sieve size (mm)	**Mass Retained (g)**	**% mass retained (g)**	**Cumulative % mass retained (g)**	**Percentage finer (%)**
2.0	0	0	0	100
1.18	0.2	0.2	0.2	99.8
0.85	4.6	4.6	4.8	95.2
0.6	12.7	12.8	17.6	82.4
0.425	17.5	17.64	35.24	64.76
0.3	19.0	19.15	54.48	45.52
0.15	27.6	27.82	82.28	17.72
0.075	12.5	12.60	94.88	5.12
Tray	5.1	5.14	100	0
Total	99.20			

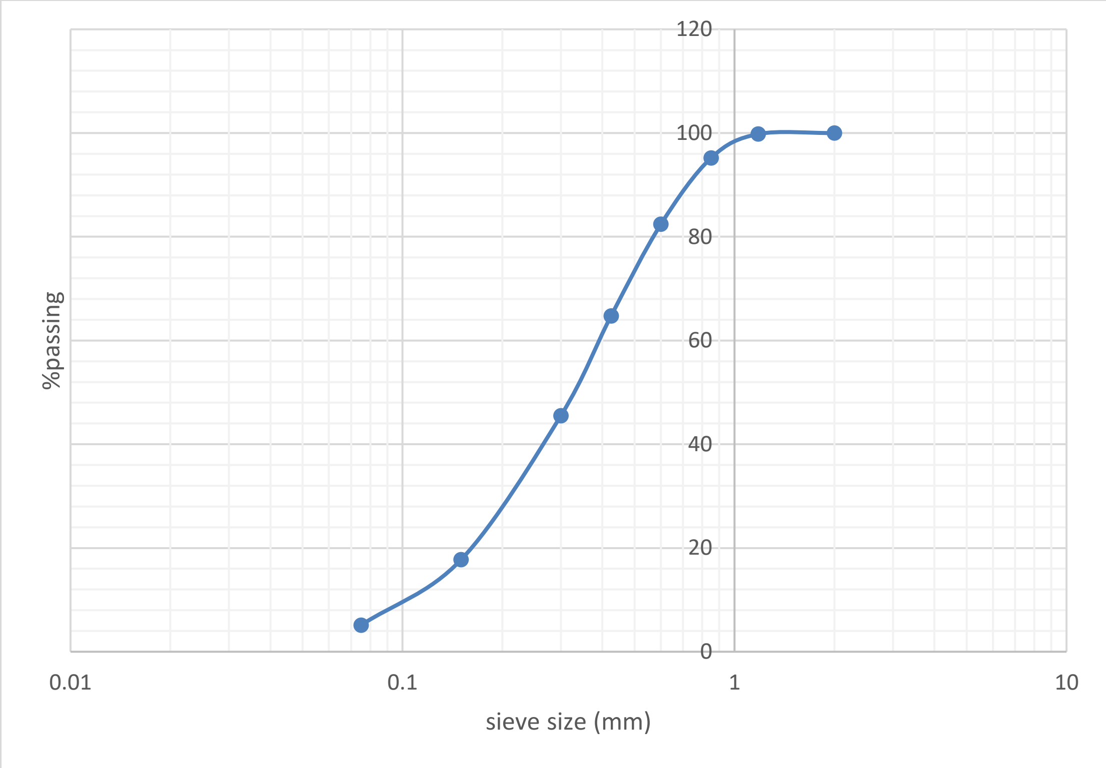


Fig 4.1.1. Sieve Analysis for Sample B

$$CU = \frac{D60}{D10}$$

$$CC = \frac{(D30)2}{D60 \times D10}$$

Where:

D10 = 0.11

D30 = 0.22

D60 = 0.3

$$CU = \frac{0.3}{0.11} = 2.72$$

$$CC = \frac{(0.22)2}{0.3 \times 0.11} = 1.47$$

The value of CU for the two soil samples indicates that soil is poorly graded clayey sand (SC) using the USCS soil classification system as they are both less than 4. When CU is less than 4, the soil is classified as poorly graded or uniformly graded soil.

4.2. SPECIFIC GRAVITY TEST RESULT

The specific gravity characteristics of the clay soil in its natural state and after treatment with varying percentages of granite dust are summarized in the table 4.2 and table 4.2.1 below.

Table 4.2. Specific gravity test result for sample A

Test	Specific gravity
100% clay	2.69
75% clay 25% Granite dust	2.71
65% clay, 35% Granite dust	2.72
55% clay, 45% Granite dust	2.74
45% clay, 55% Granite dust	2.75

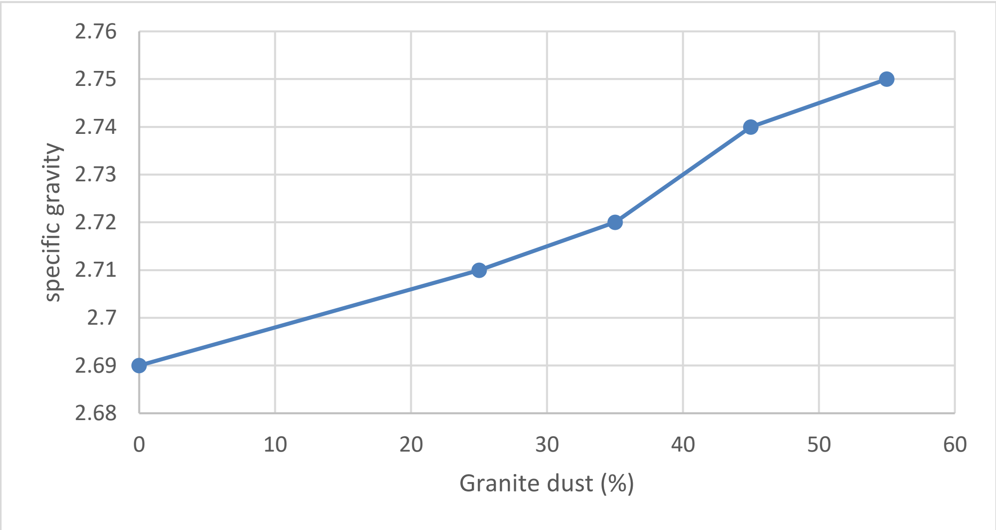


Fig 4.2 Specific gravity result sample A

Table 4.2.1. Specific gravity test result for sample B

Test	Specific gravity
100% clay	2.68
75% clay 25% Granite dust	2.70
65% clay, 35% Granite dust	2.71
55% clay, 45% Granite dust	2.72
45% clay, 55% Granite dust	2.74

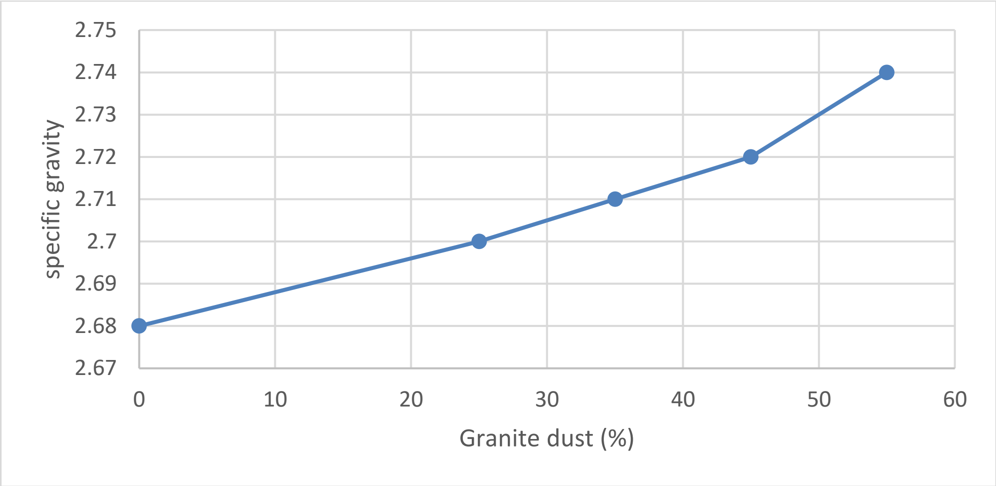


Fig 4.2.1 Specific gravity result of sample B

The result of specific gravity of the two sample as shown in figure 4.2 and figure 4.2.1 shows that there is an increase in the specific gravity of the soil sample with each addition of granite dust. The specific gravity of sample A was 2.69 but after the addition of 25%, 35%, 45, and 55% of granite dust, the specific gravity increased to 2.71, 2.72, 2.74 and 2.75 respectively. Similar trend was also observed in specimen B as the specific gravity of the soil was 2.68, but after the addition of 25%, 35%, 45%, and 55% of granite dust, the specific gravity increased to 2.70, 2.71, 2.72, and 2.74 respectively.

In a similar study conducted by Gudla et.al (2021), the suitability of granite dust as a sustainable material for geotechnical applications was investigated. The research involved the addition of granite dust in the dosage of 20%, 40%, 60%, and 80% by weight to the natural soil sample. The result of their study demonstrated a significant increase in the specific gravity of the of the soil from its initial specific gravity of 2.70. Following the addition of 20%, 40%, 60%, and 80% of granite dust, the specific gravity increased to 2.72, 2.74, 2.76, and 2.78 respectively. This study aligns with our own research which indicates that granite dust increases the specific gravity of cohesive soils when added to it.

4.3 COMPACTION TEST RESULT.

The compaction characteristics of the clay soil in its natural state and after treatment with varying percentages of granite dust are summarized and plotted in Fig. 4.3 and fig. 4.3.1 below.

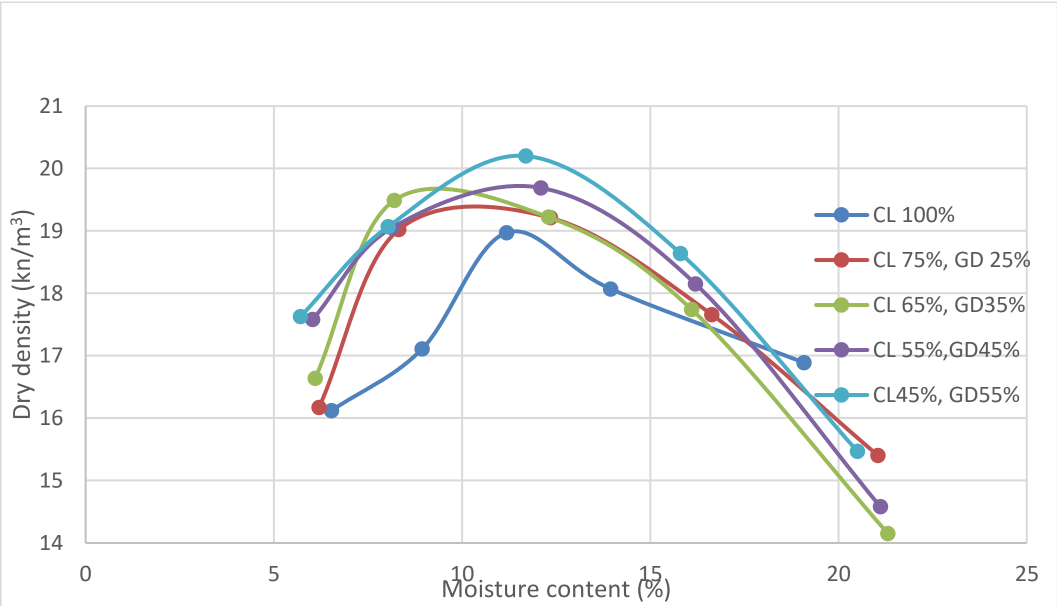


Fig 4.3: Compaction test result of Sample A

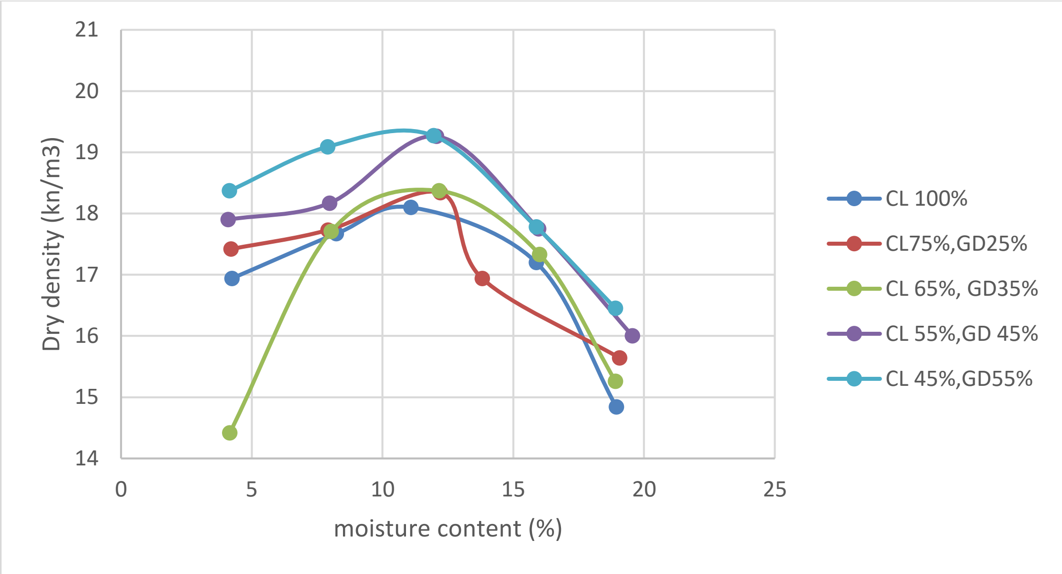


Fig 4.3.1: Compaction test result of Sample B

The MDD and OMC for the soil samples A and B without inclusion of granite dust are 18.97kn/m³, 11.18%; 18.10km/³, 11.08%; respectively. The MDD and OMC for the soil samples A and B with the inclusion of granite dust at various percentages of 25%, 35%, 45%, 55%, are: Soil Sample A 19.21kn/m³, 12.35%; 19.22kn/m³, 12.30%; 19.69kn/m³, 12.09%; 20.20kn/m³, 11.69%; Soil Sample B 18.34kn/m³, 12.21%; 18.37kn/m³, 12.16%; 19.26kn/m³, 12.06%; 19.27kn/m³, 11.95%, respectively.

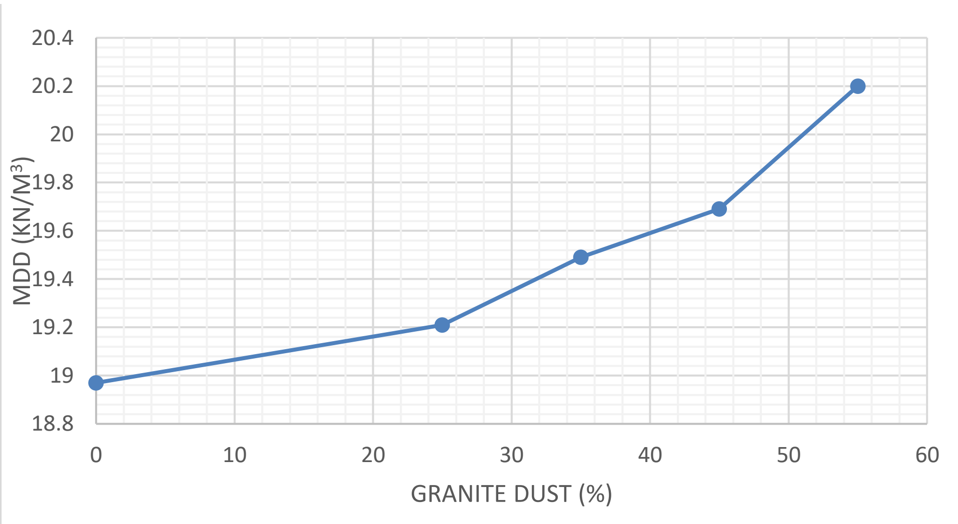


Fig 4.3.2: Variation in MDD with increase in granite dust dosage (sample A)

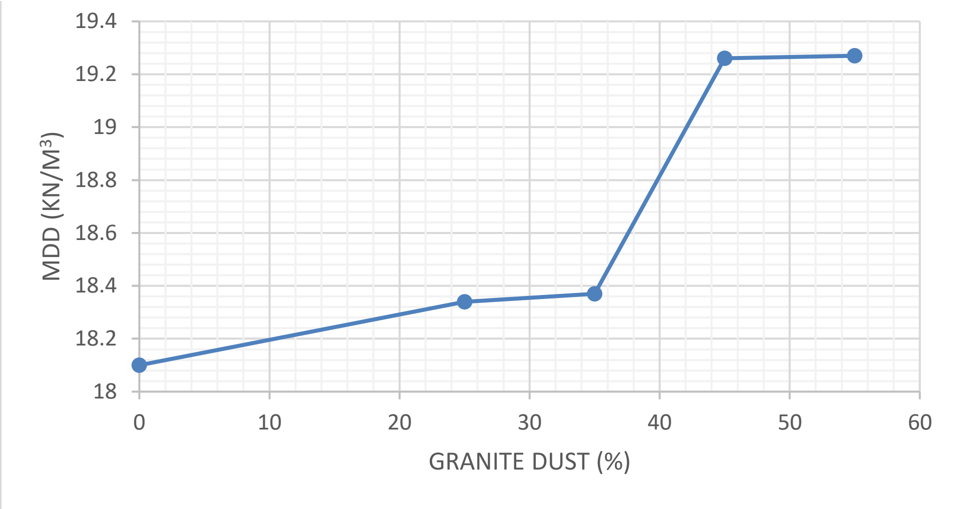


Fig 4.3.3: Variation in MDD with increase in granite dust dosage (sample B)

Figure 4.3.2 and figure 4.3.3 shows the variation of maximum dry density with granite dust content. The maximum dry density of the natural soil for sample A was 18.97kn/m³ and sample B was 18.10km/m³. The addition of 55% granite dust increased the maximum dry density the most with sample A having a dry density of 20.20kn/m³ and sample B with a maximum dry density of 19.27kn/m³

In a similar study conducted by Ogbonnaya et. al (2011), the potential effect of granite dust on the geotechnical properties of abakaliki clays was investigated. The research involved the addition of granite dust in the dosage of 10%, 15%, 20% by weight to the natural soil sample. The results of their study demonstrated a significant increase in the maximum dry density of the clay soil sample from its initial MDD of 1.68mg/m². following the addition of granite dust to the soil, the MDD increased to 1.76, 1.77, and 1.93mg/m² after the addition 10%, 15%, and 20% of granite dust respectively. This study aligns with our own research, which also indicates that granite dust improves the MDD of soils when added to it.

4.4 ATTERBERG LIMITS RESULT

The results of the liquid and plastic limit tests in the fractions passing the No.40 British Standard Sieve are summarized in the table 4.4 and table 4.4.1 and plotted in fig. 4.4 and fig, 4.4.1 below.

Table 4.4 Atterberg limit result for sample A

Sample	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)
100% CLAY	40.12	23.79	16.33
75%CL, 25% GD	35.2	18.93	16.27
65%CL, 35%GD	33.60	16.38	17.22
55%CL, 45%GD	31.2	14.92	16.28
45%CL, 55%GD	26.8	12.46	14.34

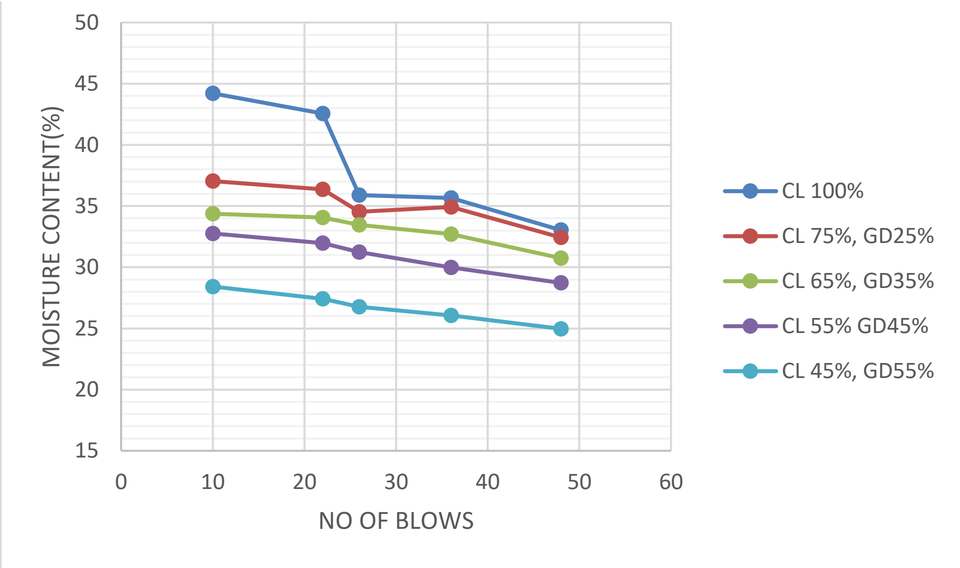


Fig 4.4 Liquid limit result for sample A

Table 4.4.1 Atterberg limit result for sample B

Sample	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)
100% CLAY	43.3	25.82	17.78
75%CL, 25% GD	41.2	22.34	18.86
65%CL, 35%GD	37.5	19.65	17.85
55%CL, 45%GD	36.20	17.51	18.7
45%CL, 55%GD	33.40	16.32	17.08

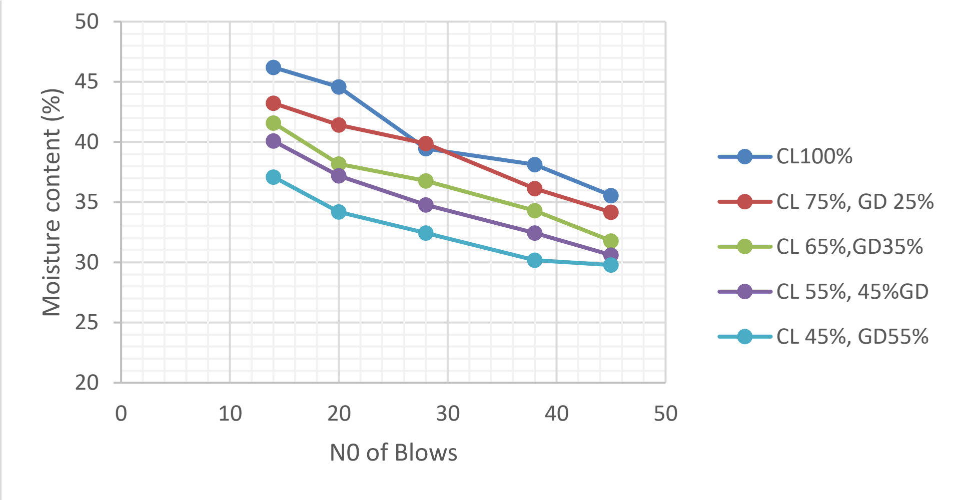


Fig 4.4.1 Liquid limit result for sample B

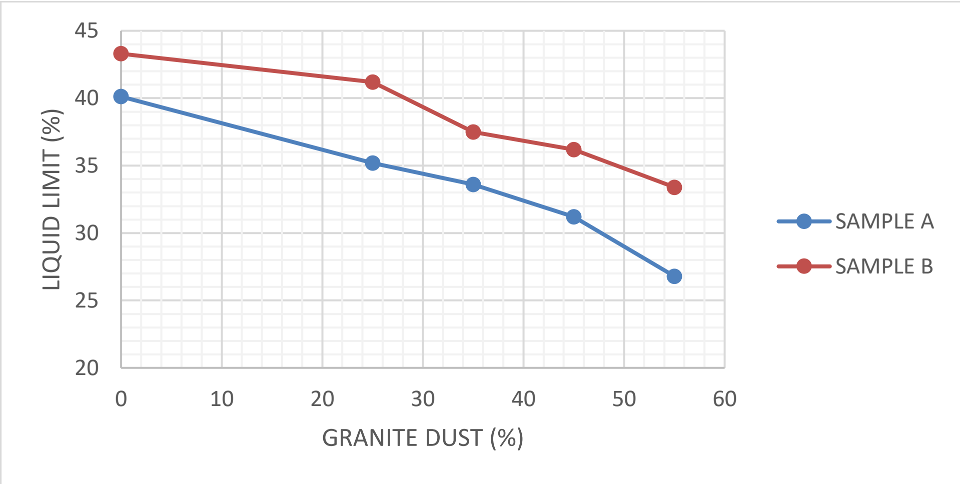


Fig 4.4.2: Variation in the liquid limit with each addition of granite dust.

Figure 4.4.2 shows the variation in the liquid limit of the natural soil with each percentage of granite dust. It is observed that the addition of granite dust decreases the liquid limit of the natural soil sample. A general trend of decrease in liquid limit of the soil sample A from its natural value of 40.12% to 26.8% at 55 % granite dust content was noticed. Similar trend was also observed in sample B as the liquid limit moved from its natural value of 43.3% to 33% at 55% addition of granite dust.

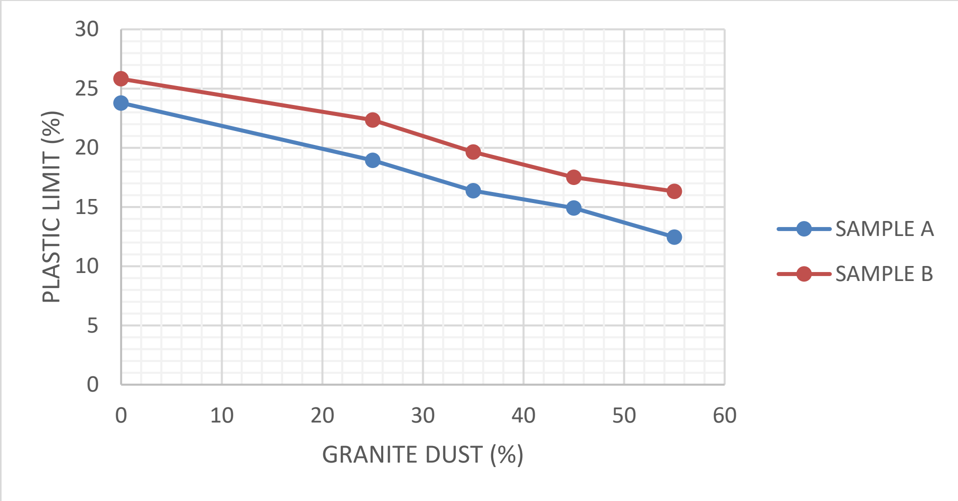


Fig 4.4.3: Variation in the plastic limit with each addition of granite dust.

Figure 4.4.3 shows the variation in the plastic limit of the natural soil with each percentage of granite dust. It is observed that the addition of granite dust decreases the plastic limit of the natural soil sample. A general trend of decrease in plastic limit of the soil sample A from its natural value of 23.79% to 12.46% at 55 % granite dust content was noticed. Similar trend was also observed in sample B as the plastic limit moved from its natural value of 25.82% to 16.32% at 55% addition of granite dust.

A similar study by Oyediran. I. A (2019), the performance analysis of some quarry dust treated soils were analysed. Three soil samples was used in the research, 2%, 4%, 6%, 8%, 10%, and 20% of quarry dust (by weight) was added to the soil and mixed thoroughly. In the liquid limit and plastic limit test, all the soil samples had the highest reduction and % change of both parameters upon the addition of 20% of quarry dust. This study aligns with our own research, which indicates that granite dust when added to cohesive soils has the tendency to increases the soils resistance to deformation.

4.5 TRIAXIAL TEST RESULT

The unconsolidated undrained tests were performed with the digital triaxial testing machine for the cell confining pressures of 98.1KN/M^2, 196.1KN/M^2 and 294.2KN/M^2. The Mohr's circles were drawn for the tests and the stress vs strain graph for the tests were also plotted. The shear strength of soil samples was evaluated using the formular below:

$$\tau_f = \{C + (\sigma \tan \Phi)\}$$

Where:

τ_f = shear strength

C = Cohesion

σ = normal stress

Φ = Angle of internal friction

The results obtained from the tests are given in the tables below:

Table 4.5: Triaxial test result of 100% clay(sample A)

TEST	CELL PRESSURE (KN/M^2) σ_3	DEVIATOR STRESS (KN/M^2)	σ_1 (KN/M^2)
1	98.1	87.24	185.34
2	196.1	118.12	314.22
3	294.2	134.08	428.28

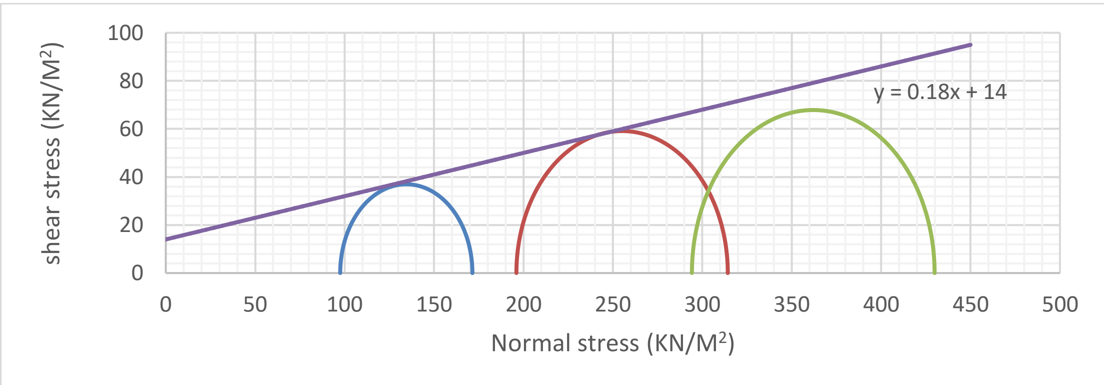


Fig 4.5: Morh circle for 100% clay (sample A)

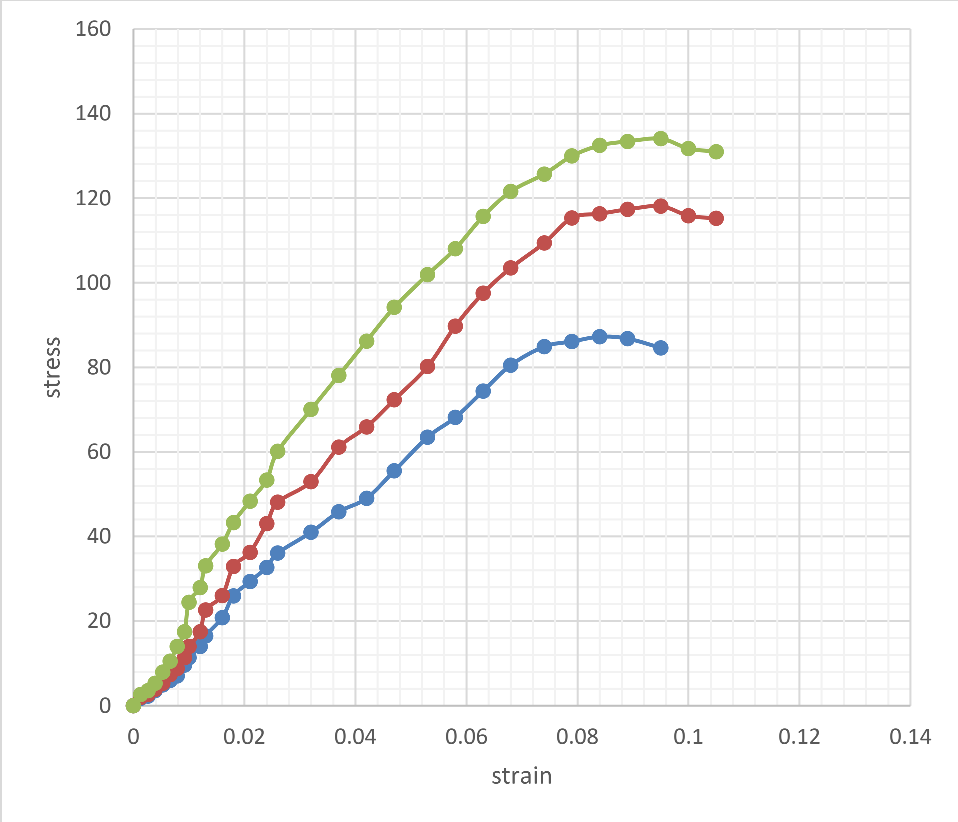


FIG 4.5.1: Stress-strain curve 100% clay (sample A)

Table 4.5.1: Triaxial test result of 75% clay, 25%GD(sample A)

TEST	CELL PRESSURE (KN/M^2) σ_3	DEVIATOR STRESS (KN/M^2)	σ_1 (KN/M^2)
1	98.1	98.55	196.65
2	196.1	120.12	3316.22
3	294.2	132.60	426.82

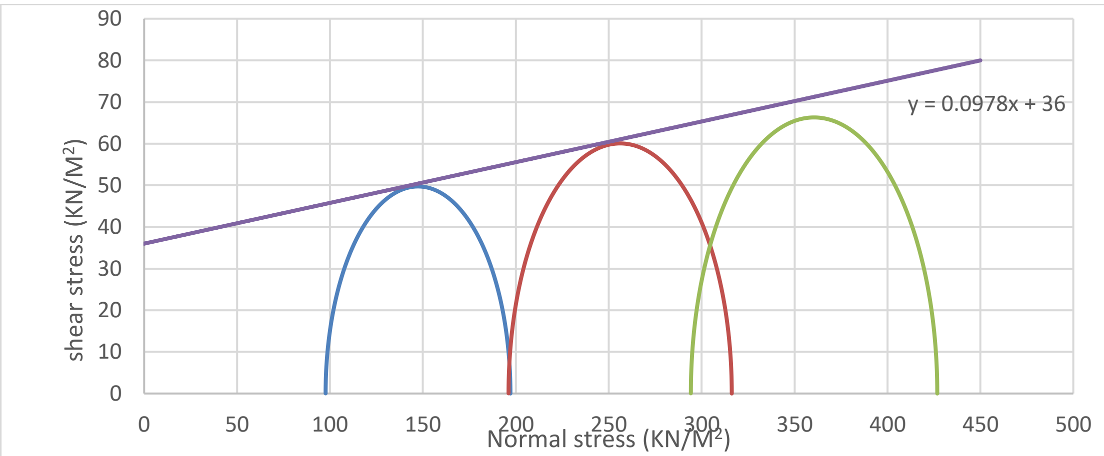


Fig 4.5.2: Morh circle for 75% clay, 25%GD (sample A)

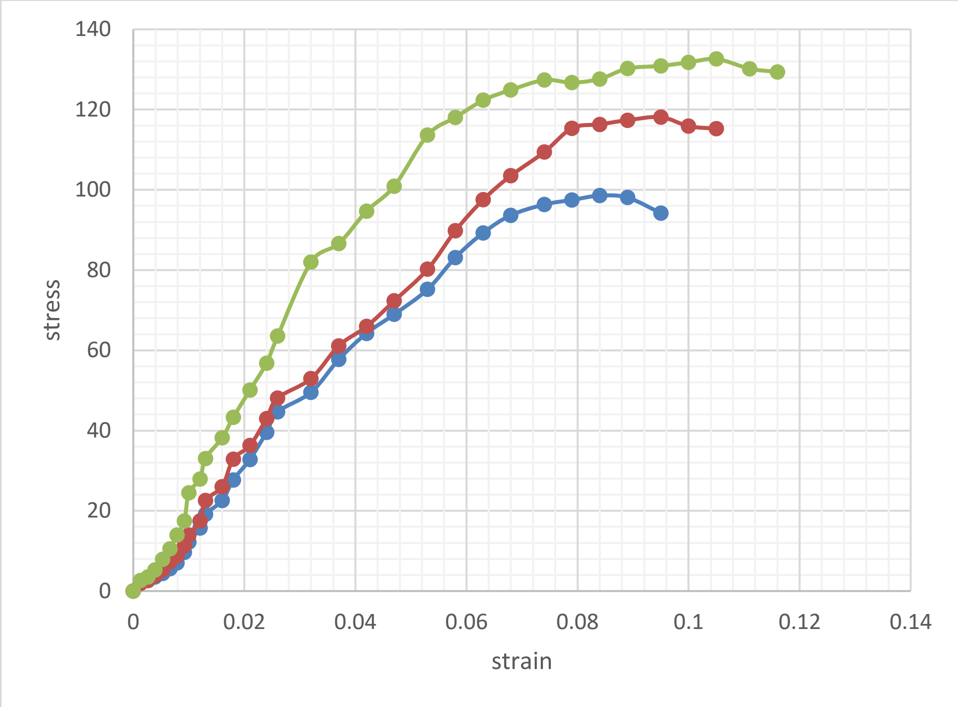


FIG 4.5.3: Stress-strain curve 75% clay, 25%GD (sample A)

Table 4.5.2: Triaxial test result of 65% clay, 35%GD (sample A)

TEST	CELL PRESSURE (KN/M^2) σ_3	DEVIATOR STRESS (KN/M^2)	σ_1 (KN/M^2)
1	98.1	111.47	209.57
2	196.1	131.83	327.93
3	294.2	138.93	433.13

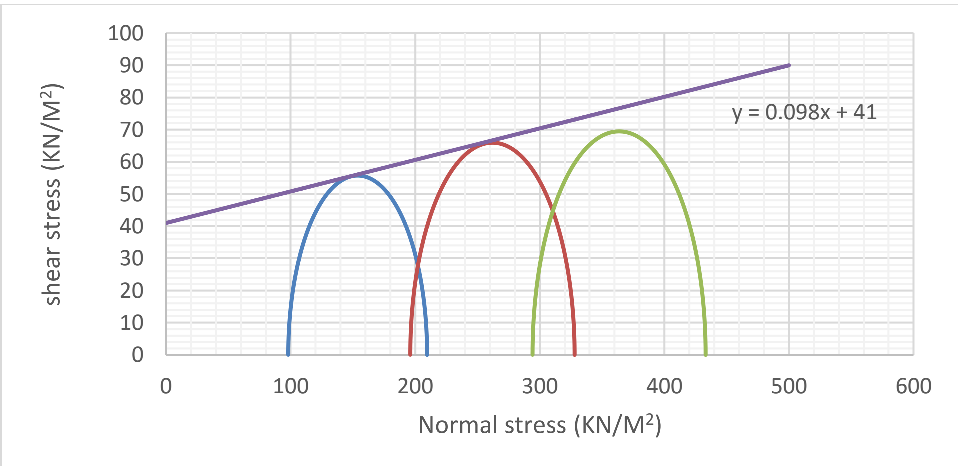


Fig 4.5.4: Morh circle for 65% clay, 35%GD (sample A)

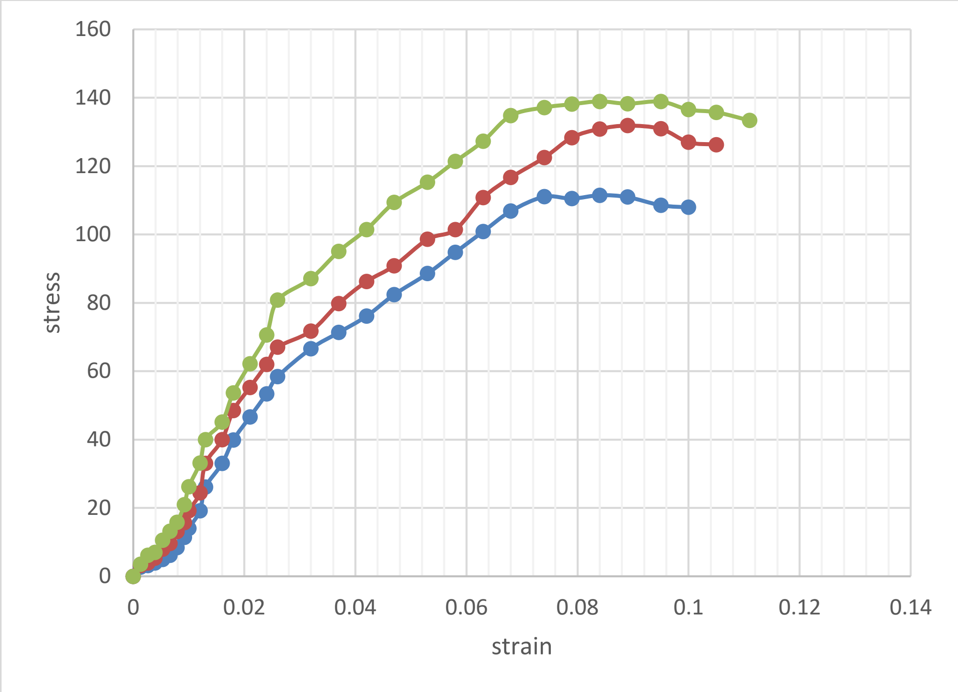


FIG 4.5.5: Stress-strain curve 65% clay, 35%GD (sample A)

Table 4.5.3: Triaxial test result of 55% clay, 45%GD (sample A)

TEST	CELL PRESSURE (KN/M^2) σ_3	DEVIATOR STRESS (KN/M^2)	σ_1 (KN/M^2)
1	98.1	113.35	211.45
2	196.1	148.63	344.73
3	294.2	169.84	464.04

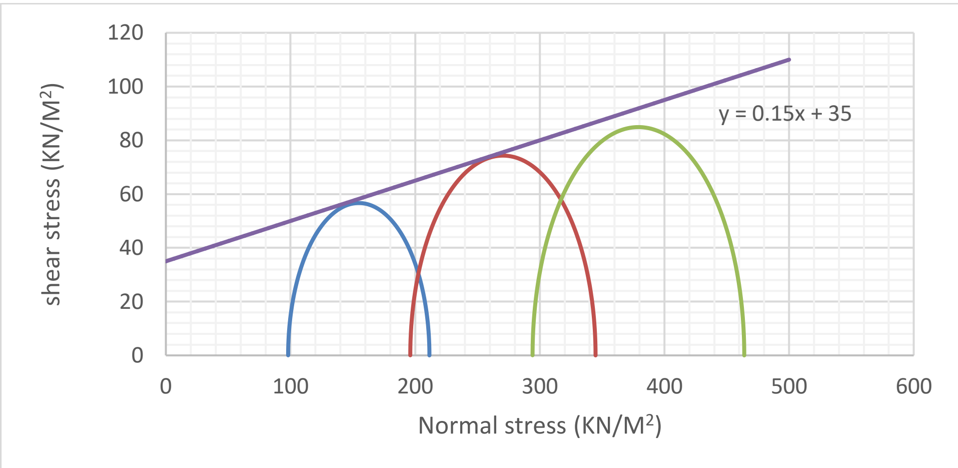


Fig 4.5.6: Morh circle for 55% clay, 45%GD (sample A)

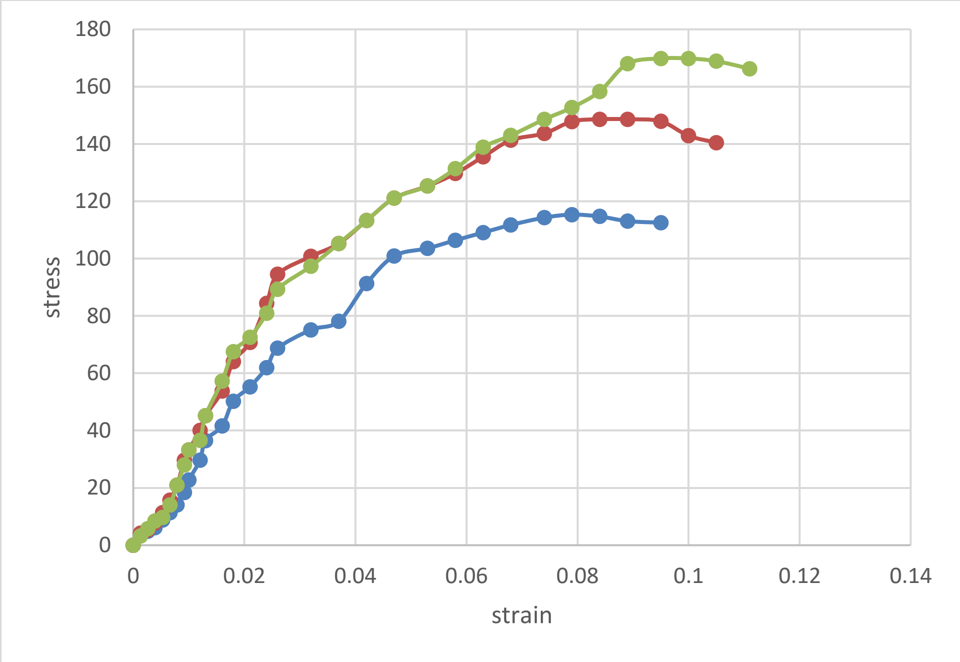


FIG 4.5.7: Stress-strain curve 55% clay, 45%GD (sample A)

Table 4.5.4: Triaxial test result of 45% clay, 55%GD (sample A)

TEST	CELL PRESSURE (KN/M²) σ_3	DEVIATOR STRESS (KN/M²)	σ_1 (KN/M²)
1	98.1	131.19	229.29
2	196.1	210.61	406.71
3	294.2	233.33	527.53

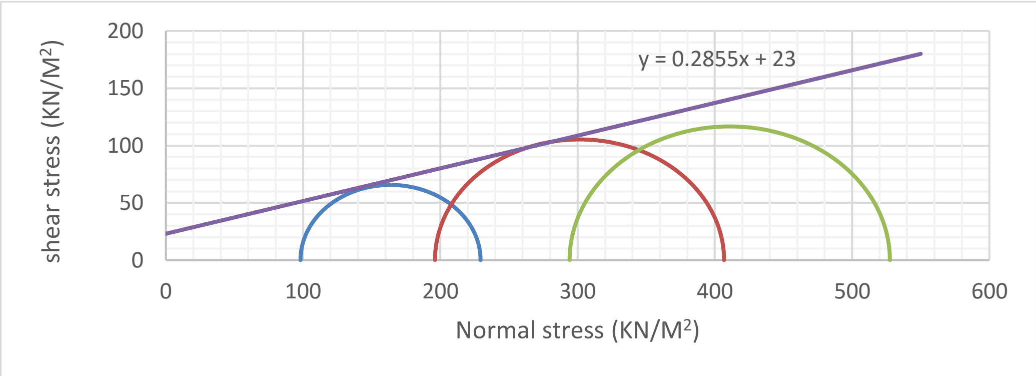


Fig 4.5.8: Morh circle for 45% clay, 55%GD (sample A)

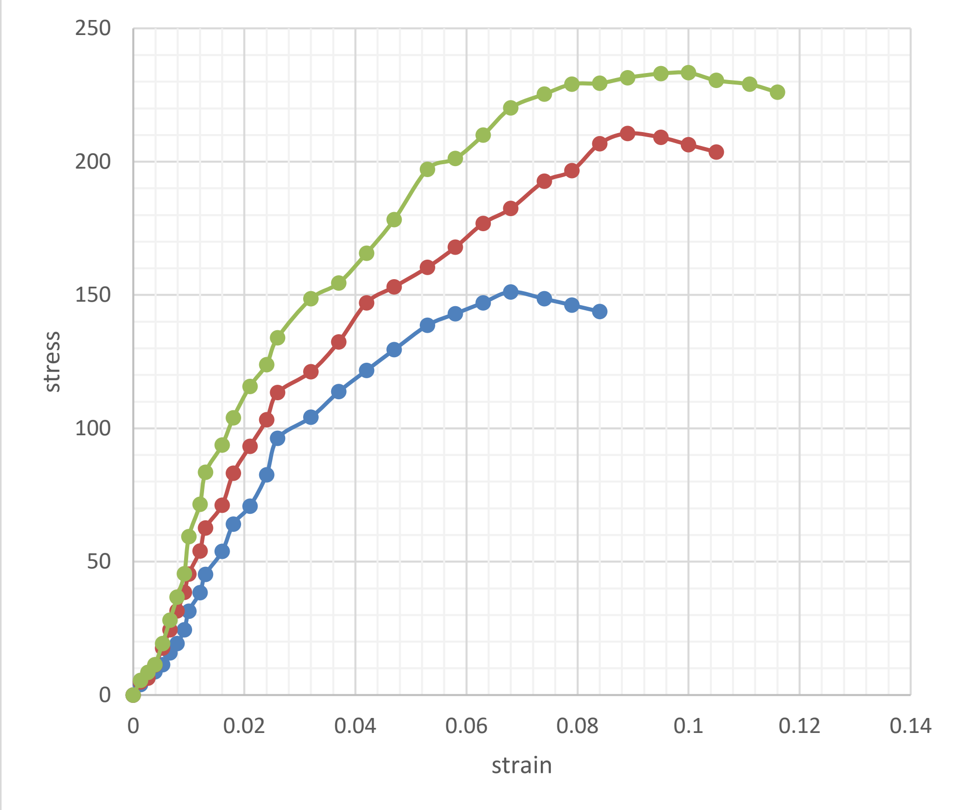


FIG 4.5.9: Stress-strain curve 45% clay, 55%GD (sample A)

Table 4.5.5: Triaxial test result of 100% clay (sample B)

TEST	CELL PRESSURE (KN/M^2) σ_3	DEVIATOR STRESS (KN/M^2)	σ_1 (KN/M^2)
1	98.1	72.79	170.89
2	196.1	118.12	314.22
3	294.2	135.70	429.9

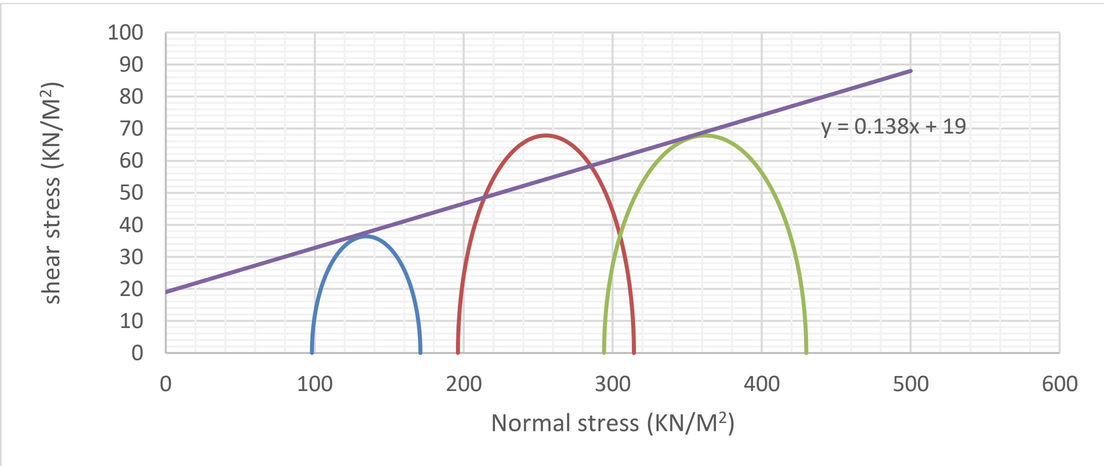


Fig 4.6: Morh circle for 100% (sample B)

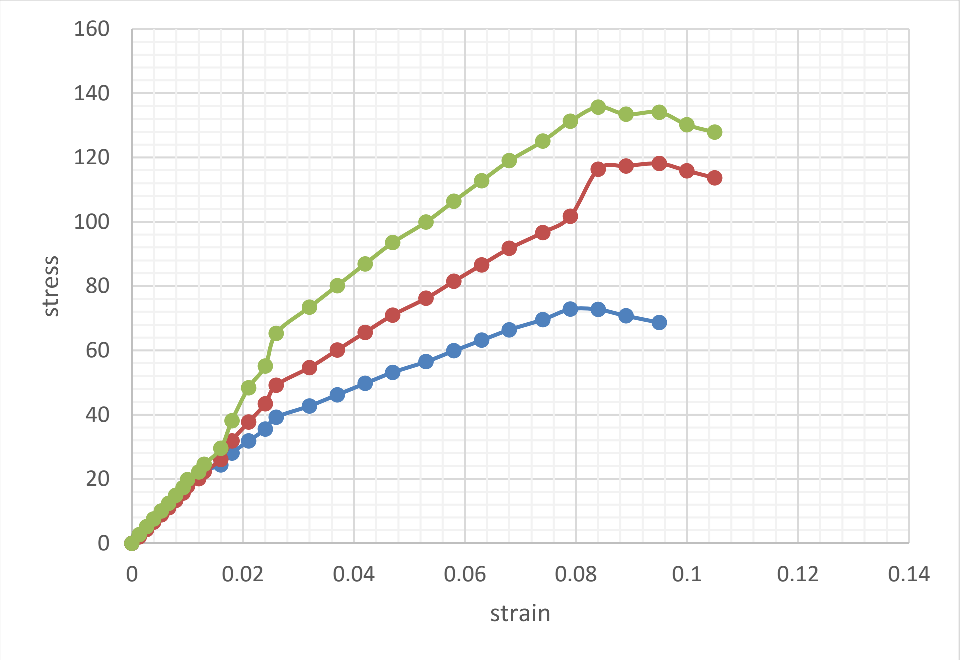


FIG 4.6.1: Stress-strain curve 100% clay (sample B)

Table 4.5.6: Triaxial test result of 75%,clay 25%GD (sample B)

TEST	CELL PRESSURE (KN/M^2) σ_3	DEVIATOR STRESS (KN/M^2)	σ_1 (KN/M^2)
1	98.1	82.17	180.27
2	196.1	120.12	316.22
3	294.2	137.32	431.52

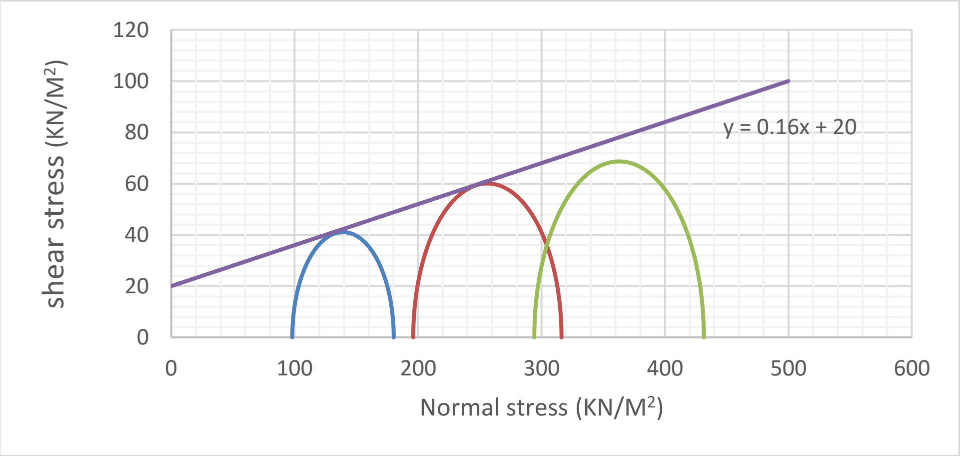


Fig 4.6.2: Morh circle for 75%clay, 25%GD (sample B)

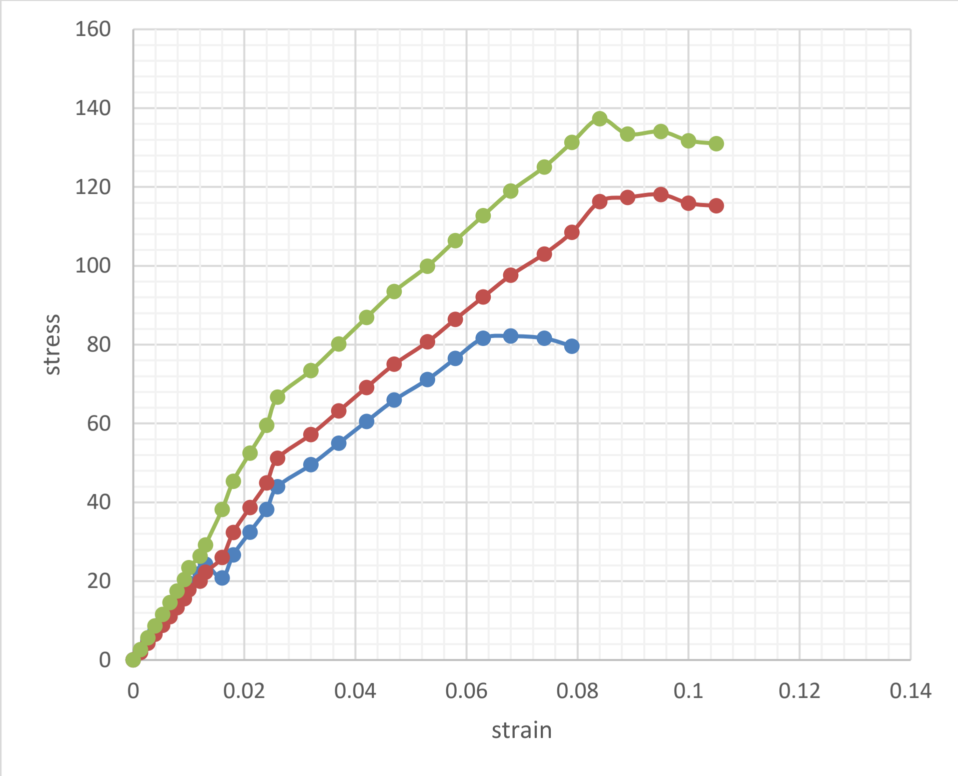


FIG 4.6.3: Stress-strain curve 75% clay, 25%GD (sample B)

Table 4.5.7: Triaxial test result of 65%,clay 35%GD (sample B)

TEST	CELL PRESSURE (KN/M²) σ_3	DEVIATOR STRESS (KN/M²)	σ_1 (KN/M²)
1	98.1	80.39	178.49
2	196.1	122.22	318.32
3	294.2	144.44	438.64

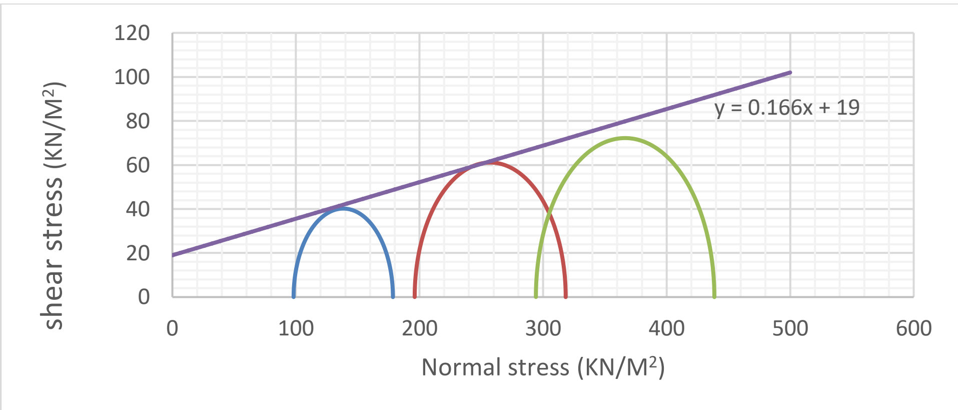


Fig 4.6.4: Morh circle for 65%clay, 35%GD (sample B)

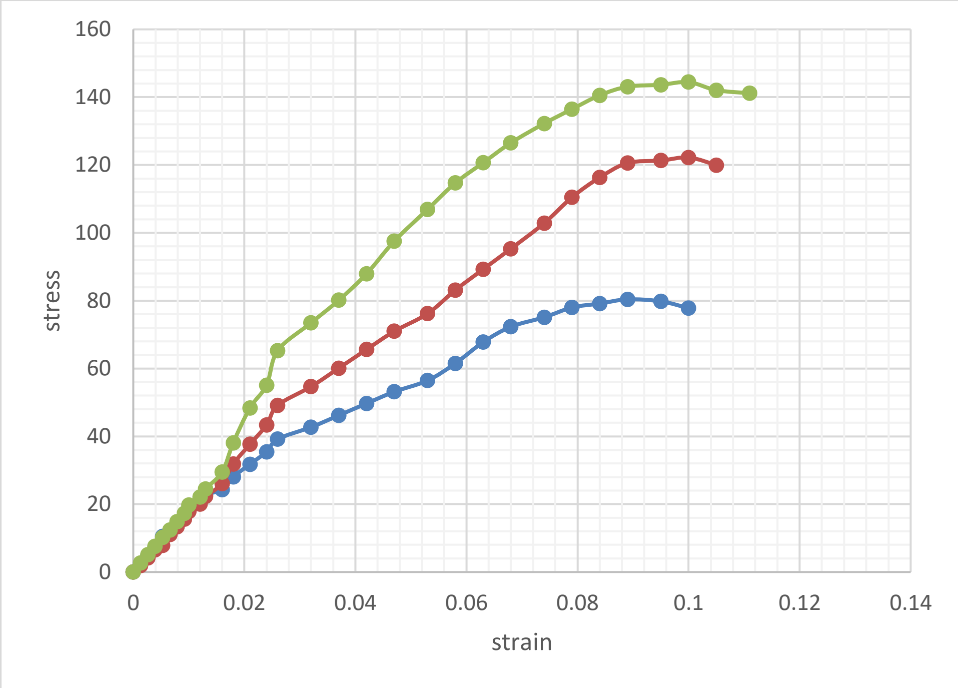


FIG 4.6.5: Stress-strain curve 65% clay, 35%GD (sample B)

Table 4.5.8: Triaxial test result of 55%,clay 45%GD (sample B)

TEST	CELL PRESSURE (KN/M^2) σ_3	DEVIATOR STRESS (KN/M^2)	σ_1 (KN/M^2)
1	98.1	93.06	191.16
2	196.1	132.60	328.70
3	294.2	169.84	464.04

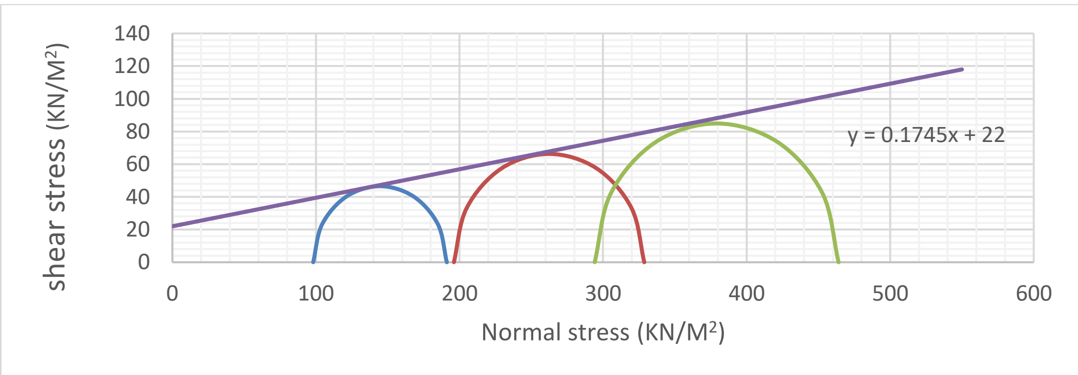


Fig 4.6.6: Morh circle for 55%clay, 45%GD (sample B)

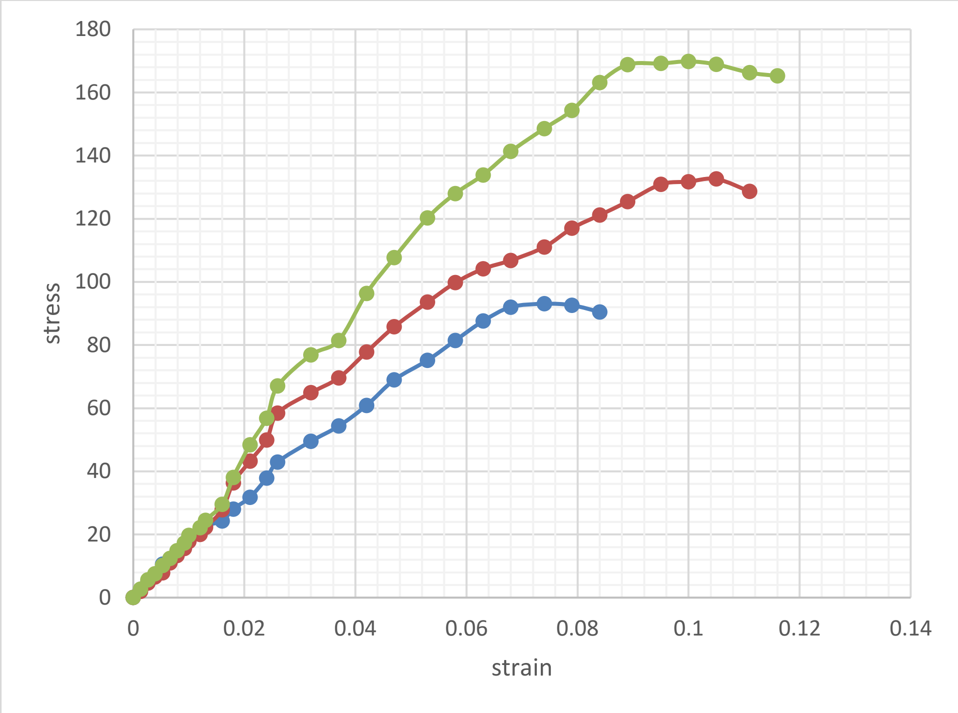


FIG 4.6.7: Stress-strain curve 55% clay, 45%GD (sample B)

Table 4.5.8: Triaxial test result of 45%,clay 55%GD (sample B)

TEST	CELL PRESSURE (KN/M^2) σ_3	DEVIATOR STRESS (KN/M^2)	σ_1 (KN/M^2)
1	98.1	109.39	207.49
2	196.1	147.62	343.72
3	294.2	178.46	472.66

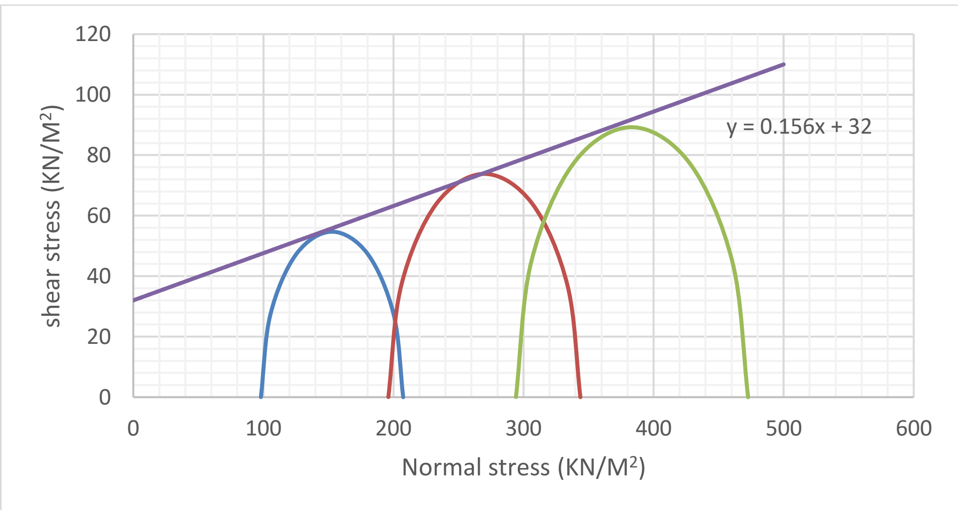


Fig 4.6.8: Morh circle for 45%clay, 55%GD (sample B)

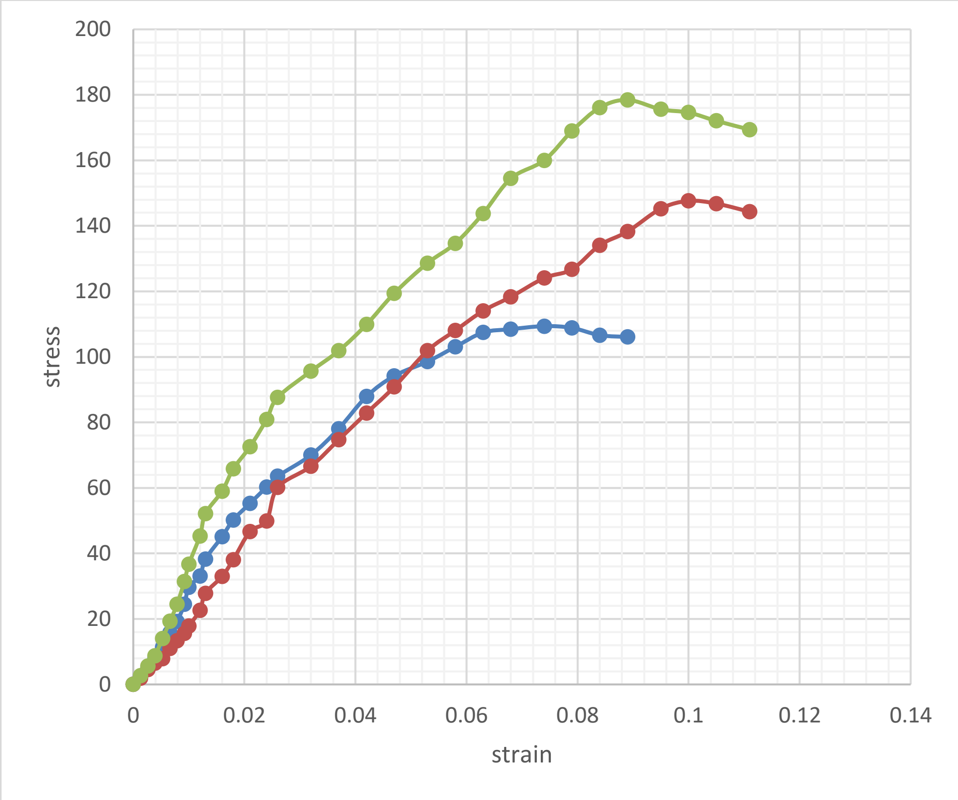


FIG 4.6.9: Stress-strain curve 45% clay, 55%GD (sample B)

4.6 SUMMARY OF MOHR CIRCLE RESULT

Table 4.6 summary of mohr circle result (SAMPLE A)

SAMPLE	C (kn/m^2)	Φ	Ʈf (kn/m^2)
100% CL	14	10.20^o	91.06
75%CL, 25%GD	36	5.59^o	77.77
65%CL, 35%GD	41	5.60^o	83.47
55%CL, 45%GD	35	8.53^o	104.60
45%CL,55%GD	23	15.93^o	173.41

Table 4.6.1 summary of mohr circle result (SAMPLE B)

SAMPLE	C(kn/m^2)	Φ	Ʈ (kn/m^2)
100% CL	19	7.86^o	78.34
75%CL, 25%GD	20	9.09^o	89.04
65%CL, 35%GD	19	9.43^o	91.85
55%CL, 45%GD	22	9.90^o	102.98
45%CL,55%GD	32	8.87^o	105.76

The result of the triaxial test, from the mohr circle analysis shows that the shear strength of the soil keep increasing as the percentage of granite dust added to the soil increased.

In a similar study by ogbonnaya et. al (2020), the potential effects of granite dust on the geotechnical properties of Abakaliki clays was investigated. The study involved the addition of granite dust in the dosages of 10%, 15%, and 20% by weight to the natural soil sample. The result of their triaxial test showed that cohesion decreases while angle of internal friction increased with addition of granite dust. The overall result nevertheless remained an increase in shear strength as the percentage of granite dust is increased. This study aligns with our own research, which indicates that granite dust when added to cohesive soils increases the shear strength of the soil.

CHAPTER FIVE

5.0. CONCLUSION AND RECOMMENDATIONS

5.1. CONCLUSION

The research used two cohesive soil samples, identified as a poorly graded clayey sand (SC) by the unified soil classification system (USCS) and classified as an A-7-5 soil by the AASHTO soil classification system. The specific gravity test revealed an increase in specific gravity as the amount of granite dust added to the soil increased. The compaction test showed an increase in the dry density of the soil with each addition of granite dust. The Atterberg limit test demonstrated a reduction in the liquid limit and plastic limit of the natural soil with increasing amounts of granite dust. Finally, the Mohr circle analysis indicated an increase in principal stress due to an increase in cell pressure with higher proportions of granite dust.

5.2. RECOMMENDATION

Based on the conclusions, the following recommendations can be made:

Firstly, the addition of granite dust can be recommended for soil stabilization, enhancing the soil's load-bearing capacity and compactness.

Secondly, engineers should consider the specific increment in specific gravity when designing material mixes, aiming to optimize the composition of soil and granite dust.

Thirdly, the decrease in plasticity suggests caution in applications requiring specific plasticity properties.

Lastly, the observed increase in principal stress highlights the need to consider the soil's behaviour under load in structural analysis and design.

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APPENDIX

TABLE 6.1. Compaction test, 100% clay (sample A).

Test (%)	Volume of mould (m^3)	Mass of wet soil (kg)	Moisture content (%)	Bulk density (kg/m^3)	Dry density (kg/m^3)	Dry density (kn/m^3)
4	0.001	1.75	6.53	1750	1642.73	16.12
8	0.001	1.90	8.94	1900	1744.08	17.11
12	0.001	2.15	11.18	2150	1933.80	18.97
16	0.001	2.10	13.95	2100	1842.90	18.07
20	0.001	2.05	19.08	2050	1721.5	16.89

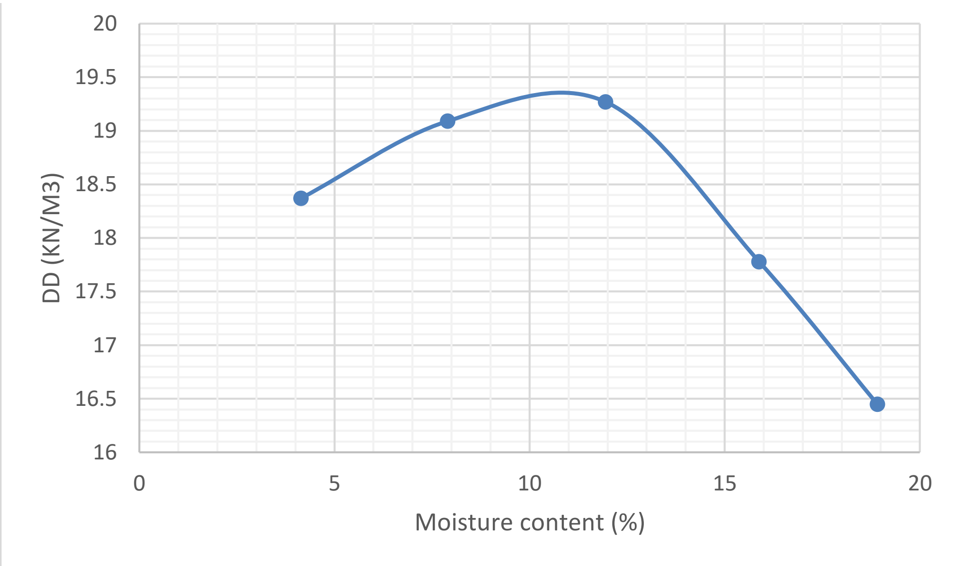

FIG 6.1 Compaction curve 100% clay (sample A)

TABLE 6.1.1. Compaction test 75% Clay, 25%GD. (Sample A)

Test (%)	Volume of mould (m^3)	Mass of wet soil (kg)	Moisture content (%)	Bulk density (kg/m^3)	Dry density (kg/m^3)	Dry density (kn/m^3)
4	0.001	1.75	6.20	1750	1647.83	16.17
8	0.001	2.10	8.32	2100	1938.70	19.02
12	0.001	2.20	12.35	2200	1958.16	19.21
16	0.001	2.10	16.63	2100	1800.56	17.66
20	0.001	1.90	21.05	1900	1569.60	15.40

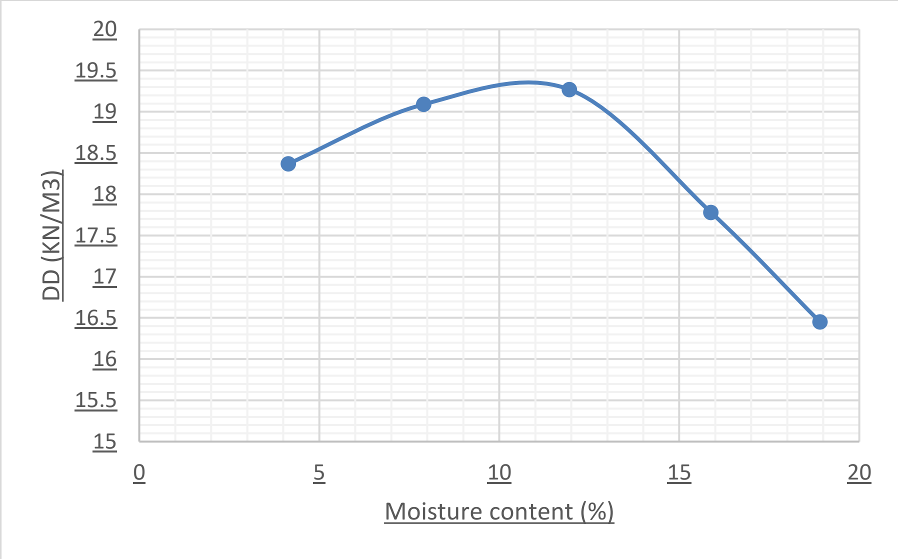

FIG 6.1.1 Compaction curve 75% clay, 25%GD (sample A)

TABLE 6.1.2. Compaction test. 65% Clay, 35%GD. (sample A)

Test (%)	Volume of mould (m^3)	Mass of wet soil (kg)	Moisture content (%)	Bulk density (kg/m^3)	Dry density (kg/m^3)	Dry density (kn/m^3)
4	0.001	1.80	6.10	1800	1696.51	16.64
8	0.001	2.15	8.20	2150	1987.06	19.49
12	0.001	2.20	12.30	2200	1959.04	19.22
16	0.001	2.10	16.10	2100	1808.79	17.74
20	0.001	1.75	21.31	1750	1442.59	14.15

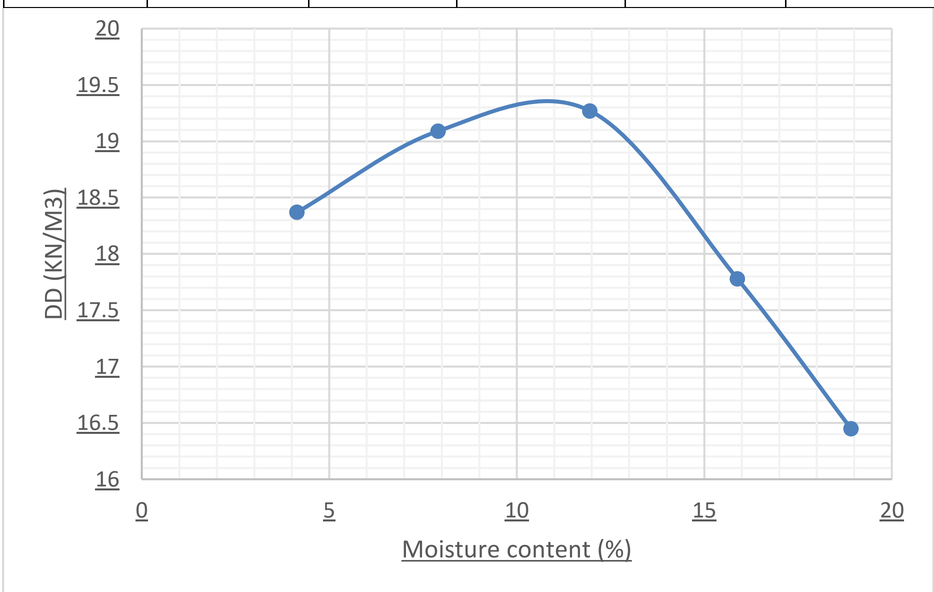

FIG 6.1.2 Compaction curve 65% clay, 35%GD (sample A)

TABLE 6.1.3. Compaction test. 55% Clay, 45%GD. (sample A)

Test (%)	Volume of mould (m^3)	Mass of wet soil (kg)	Moisture content (%)	Bulk density (kg/m^3)	Dry density (kg/m^3)	Dry density (kn/m^3)
4	0.001	1.90	6.03	1900	1791.95	17.58
8	0.001	2.10	8.14	2100	1941.93	19.05
12	0.001	2.25	12.09	2250	2007.32	19.69
16	0.001	2.15	16.20	2150	1850.26	18.15
20	0.001	1.80	21.11	1800	1486.37	14.58

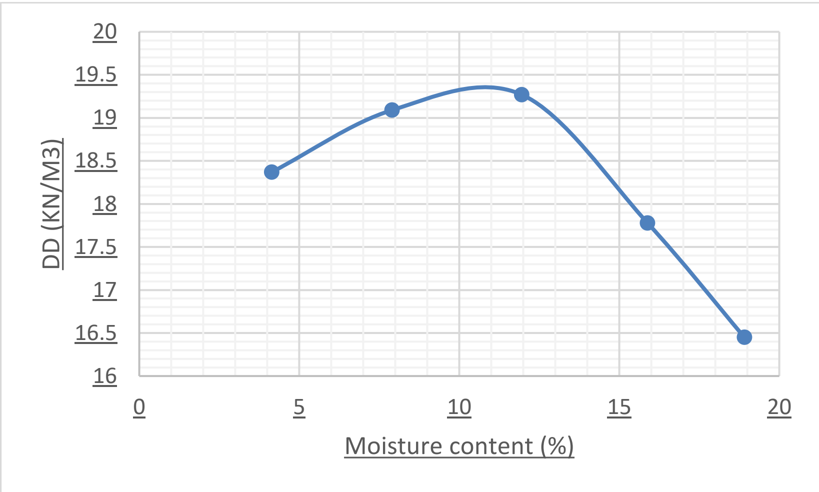

FIG 6.1.3 Compaction curve 55% clay, 45%GD (sample A)

TABLE 6.1.4. Compaction test. 45% Clay, 55%GD. (sample A)

Test (%)	Volume of mould (m^3)	Mass of wet soil (kg)	Moisture content (%)	Bulk density (kg/m^3)	Dry density (kg/m^3)	Dry density (kn/m^3)
4	0.001	1.90	5.71	1900	1797.37	17.63
8	0.001	2.10	8.04	2100	1943.72	19.07
12	0.001	2.30	11.69	2300	2059.27	20.20
16	0.001	2.20	15.80	2200	1899.83	18.64
20	0.001	1.90	20.50	1900	1576.76	15.47

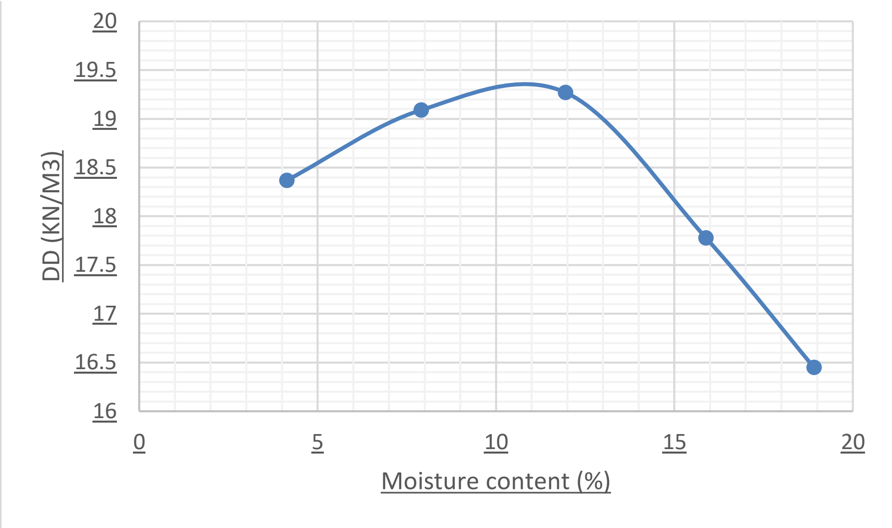

FIG 6.1.4 Compaction curve 45% clay, 55%GD (sample A)

TABLE 6.1.5. Compaction test 100%clay (sample B)

Test (%)	Volume of mould (m^3)	Mass of wet soil (kg)	Moisture content (%)	Bulk density (kg/m^3)	Dry density (kg/m^3)	Dry density (kn/m^3)
4	0.001	1.80	4.24	1800	1726.78	16.94
8	0.001	1.95	8.23	1950	1801.72	17.67
12	0.001	2.05	11.08	2050	1845.52	18.10
16	0.001	2.00	15.88	2000	1725.92	17.20
20	0.001	1.80	18.95	1800	1512.85	14.84

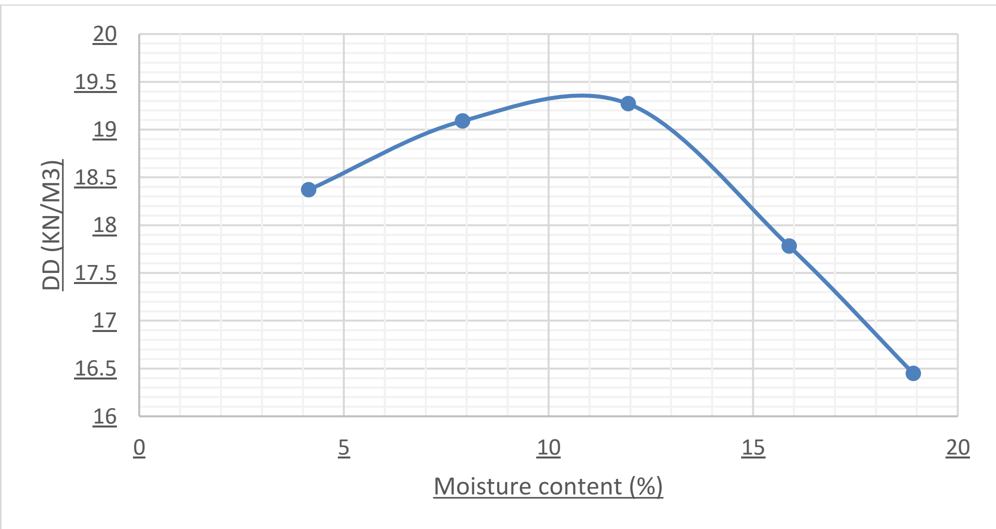

FIG 6.1.5 Compaction curve 100% clay (sample B)

TABLE 6.1.6. Compaction test 75%, 25% GD(sample B)

Test (%)	Volume of mould (m^3)	Mass of wet soil (kg)	Moisture content (%)	Bulk density (kg/m^3)	Dry density (kg/m^3)	Dry density (kn/m^3)
4	0.001	1.85	4.21	1850	1775.26	17.42
8	0.001	1.95	7.91	1950	1807.06	17.73
12	0.001	2.10	12.21	2100	1871.49	18.34
16	0.001	2.00	15.81	2000	1726.97	16.94
20	0.001	1.90	19.07	1900	1595.70	15.65

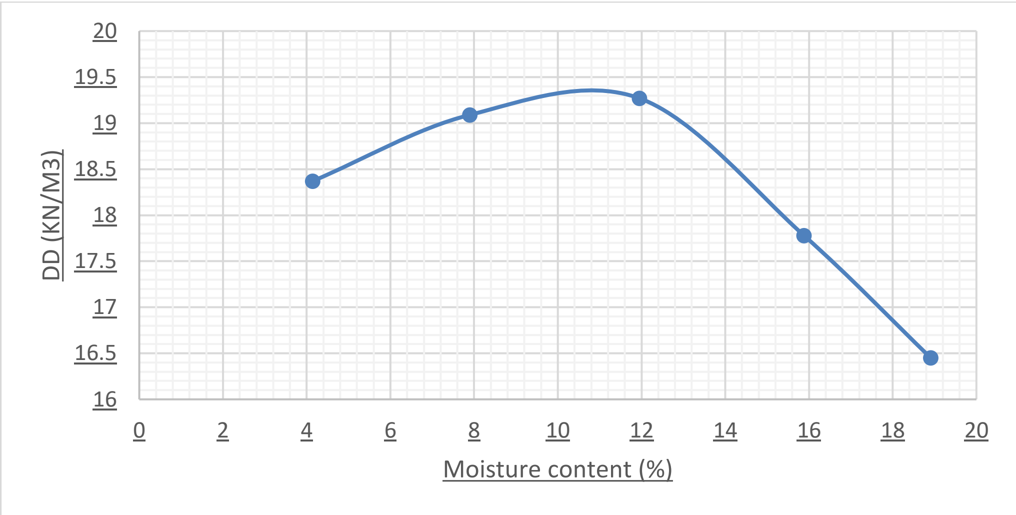

FIG 6.1.6 Compaction curve 75% clay, 25%GD (sample B)

TABLE 6.1.7. Compaction test 65%, 35% GD(sample B)

Test (%)	Volume of mould (m^3)	Mass of wet soil (kg)	Moisture content (%)	Bulk density (kg/m^3)	Dry density (kg/m^3)	Dry density (kn/m^3)
4	0.001	1.85	4.16	1850	1776.11	14.42
8	0.001	1.95	8.04	1950	1804.89	17.71
12	0.001	2.10	12.16	2100	1872.33	18.37
16	0.001	2.05	16.01	2050	1767.09	17.33
20	0.001	1.85	18.96	1850	1555.14	15.26

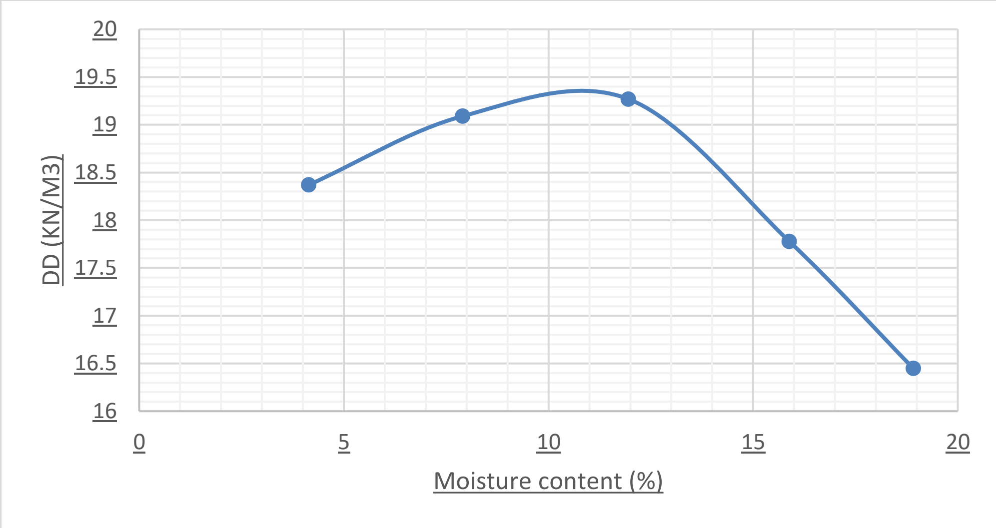

FIG 6.1.7 Compaction curve 65% clay, 35%GD (sample B)

TABLE 6.1.8. Compaction test 55%, 45% GD(sample B)

Test (%)	Volume of mould (m^3)	Mass of wet soil (kg)	Moisture content (%)	Bulk density (kg/m^3)	Dry density (kg/m^3)	Dry density (kn/m^3)
4	0.001	1.90	4.09	1900	1825.34	17.90
8	0.001	2.00	7.97	2000	1852.37	18.17
12	0.001	2.20	12.06	2200	1963.23	19.26
16	0.001	2.10	15.97	2100	1810.81	17.75
20	0.001	1.95	19.56	1950	1630.98	16.00

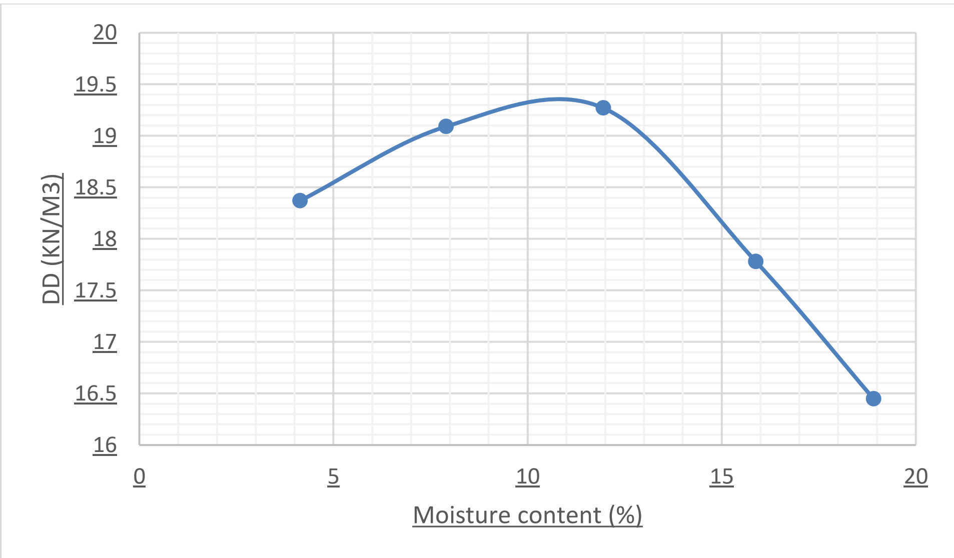

FIG 6.1.8: Compaction curve 55% clay, 45%GD (sample B)

TABLE 6.1.9. Compaction test 45%, 55% GD(sample B)

Test (%)	Volume of mould (m^3)	Mass of wet soil (kg)	Moisture content (%)	Bulk density (kg/m^3)	Dry density (kg/m^3)	Dry density (kn/m^3)
4	0.001	1.95	4.14	1950	1872.48	18.37
8	0.001	2.10	7.90	2100	1946.24	19.09
12	0.001	2.20	11.95	2200	1965.16	19.27
16	0.001	2.10	15.88	2100	1812.22	17.78
20	0.001	2.00	18.91	2000	1681.94	16.45

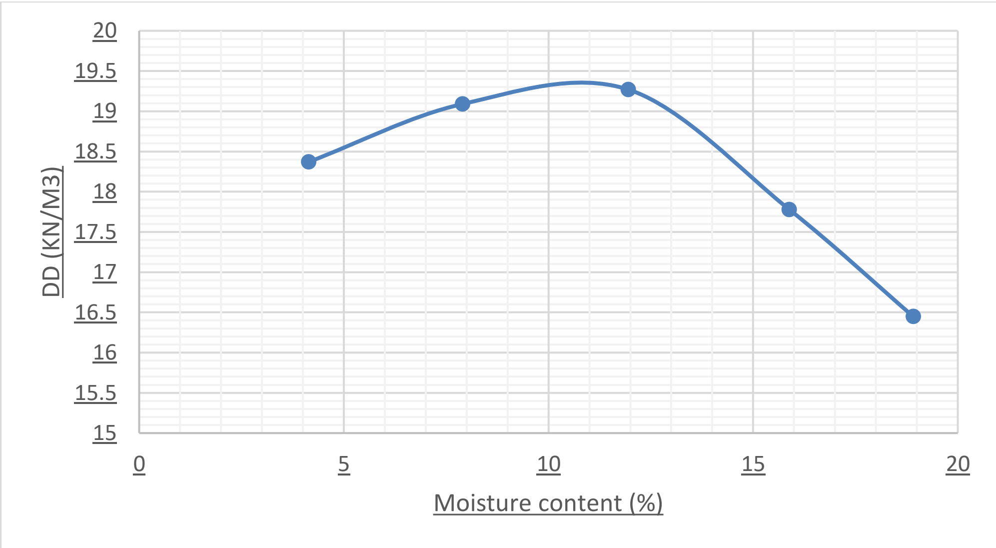

FIG 6.1.9: Compaction curve 45% clay, 55%GD (sample B)

APPENDIX B

TABLE 6.2. Triaxial test 100% clay (sample A)

Deformation on dial guage (Div)	Proving ring reading (98.1kn/m^2)	Proving ring reading (196.1kn/m^2)	Proving ring reading (294.2kn/m^2)	Sample deformation (mm)	Strain	Corrected area (m^2)	Stress (98.1kn/m^2) (kn/m^2)	Stress (196.1kn/m^2) (kn/m^2)	Stress (294.2kn/m^2) (kn/m^2)
0	0	0	0	0	0	0.001134	0.00	0.00	0.00
10	1	1.1	1.5	0.1	0.0013	0.001135	1.76	1.94	2.64
20	1.3	1.5	2	0.2	0.0026	0.001137	2.29	2.64	3.52
30	2	2.2	3	0.3	0.0039	0.001138	3.51	3.87	5.27
40	2.8	3	4.5	0.4	0.00526	0.001139	4.92	5.27	7.90
50	3.4	4.2	6	0.5	0.0066	0.001142	5.95	7.36	10.51
60	4	5	8	0.6	0.0079	0.001143	7.00	8.75	14.00
70	5.5	6.5	10	0.7	0.0092	0.001144	9.62	11.36	17.48
80	6.5	8	14	0.8	0.01	0.001145	11.35	13.97	24.45
90	8	10	16	0.9	0.012	0.001147	13.95	17.44	27.90
100	9.5	13	19	1	0.013	0.00115	16.52	22.61	33.04
120	12	15	22	1.2	0.016	0.001152	20.83	26.04	38.19
140	15	19	25	1.4	0.018	0.001155	25.97	32.90	43.29
160	17	21	28	1.6	0.021	0.001158	29.36	36.27	48.36
180	19	25	31	1.8	0.024	0.001162	32.70	43.03	53.36
200	21	28	35	2	0.026	0.001164	36.08	48.11	60.14
240	24	31	41	2.4	0.032	0.001171	40.99	52.95	70.03
280	27	36	46	2.8	0.037	0.001178	45.84	61.12	78.10
320	29	39	51	3.2	0.042	0.001183	49.03	65.93	86.22
360	33	43	56	3.6	0.047	0.001189	55.51	72.33	94.20
400	38	48	61	4	0.053	0.001197	63.49	80.20	101.92
440	41	54	65	4.4	0.058	0.001203	68.16	89.78	108.06
480	45	59	70	4.8	0.063	0.00121	74.38	97.52	115.70
520	49	63	74	5.2	0.068	0.001217	80.53	103.53	121.61
560	52	67	77	5.6	0.074	0.001225	84.90	109.39	125.71
600	53	71	80	6	0.079	0.001231	86.11	115.35	129.98
640	54	72	82	6.4	0.084	0.001238	87.24	116.32	132.47
680	54	73	83	6.8	0.089	0.001244	86.81	117.36	133.44
720	53	74	84	7.2	0.095	0.001253	84.60	118.12	134.08
760	0	73	83	7.6	0.1	0.00126		115.87	131.75
800	0	73	83	8	0.105	0.001267		115.23	131.02
840	0	0	0	8.4	0.111	0.001275			
880	0	0	0	8.8	0.116	0.001283			

TABLE 6.2.1: Triaxial test 75% clay, 25%GD (sample A)

Deformation on dial guage (Div)	Proving ring reading (98.1kn/m²)	Proving ring reading (196.1kn/m²)	Proving ring reading (294.2kn/m²	Sample deformation (mm)	Strain	Corrected area (m²)	Stress (98.1kn/m²) (kn/m²)	Stress (196.1kn/m²) (kn/m²)	Stress (294.2kn/m²) (kn/m²)
0	0	0	0	0	0	0.001134	0.00	0.00	0.00
10	1.2	1.1	1.5	0.1	0.0013	0.001135	2.11	1.94	2.64
20	1.5	1.5	2	0.2	0.0026	0.001137	2.64	2.64	3.52
30	2	2.2	3	0.3	0.0039	0.001138	3.51	3.87	5.27
40	2.5	3	4.5	0.4	0.00526	0.001139	4.39	5.27	7.90
50	3.2	4.2	6	0.5	0.0066	0.001142	5.60	7.36	10.51
60	4	5	8	0.6	0.0079	0.001143	7.00	8.75	14.00
70	5.5	6.5	10	0.7	0.0092	0.001144	9.62	11.36	17.48
80	7	8	14	0.8	0.01	0.001145	12.23	13.97	24.45
90	9	10	16	0.9	0.012	0.001147	15.69	17.44	27.90
100	11	13	19	1	0.013	0.00115	19.13	22.61	33.04
120	13	15	22	1.2	0.016	0.001152	22.57	26.04	38.19
140	16	19	25	1.4	0.018	0.001155	27.71	32.90	43.29
160	19	21	29	1.6	0.021	0.001158	32.82	36.27	50.09
180	23	25	33	1.8	0.024	0.001162	39.59	43.03	56.80
200	26	28	37	2	0.026	0.001164	44.67	48.11	63.57
240	29	31	48	2.4	0.032	0.001171	49.53	52.95	81.98
280	34	36	51	2.8	0.037	0.001178	57.72	61.12	86.59
320	38	39	56	3.2	0.042	0.001183	64.24	65.93	94.67
360	41	43	60	3.6	0.047	0.001189	68.97	72.33	100.93
400	45	48	68	4	0.053	0.001197	75.19	80.20	113.62
440	50	54	71	4.4	0.058	0.001203	83.13	89.78	118.04
480	54	59	74	4.8	0.063	0.00121	89.26	97.52	122.31
520	57	63	76	5.2	0.068	0.001217	93.67	103.53	124.90
560	59	67	78	5.6	0.074	0.001225	96.33	109.39	127.35
600	60	71	78	6	0.079	0.001231	97.48	115.35	126.73
640	61	72	79	6.4	0.084	0.001238	98.55	116.32	127.63
680	61	73	81	6.8	0.089	0.001244	98.07074	117.36	130.23
720	59	74	82	7.2	0.095	0.001253	94.17398	118.12	130.89
760	0	73	83	7.6	0.1	0.00126		115.87	131.75
800	0	73	84	8	0.105	0.001267		115.23	132.60
840	0	0	83	8.4	0.111	0.001275			130.20
880	0	0	83	8.8	0.116	0.001283			129.38

TABLE 6.2.2: Triaxial test 65% clay, 35%GD (sample A)

Deformation on dial guage (Div)	Proving ring reading (98.1kn/m^2)	Proving ring reading (196.1kn/m^2)	Proving ring reading (294.2kn/m^2)	Sample deformation (mm)	Strain	Corrected area (m^2)	Stress (98.1kn/m^2) (kn/m^2)	Stress (196.1kn/m^2) (kn/m^2)	Stress (294.2kn/m^2) (kn/m^2)
0	0	0	0	0	0	0.001134	0.00	0.00	0.00
10	1.5	1.8	2	0.1	0.0013	0.001135	2.64	3.17	3.52
20	1.8	2.2	3.5	0.2	0.0026	0.001137	3.17	3.87	6.16
30	2.2	3	4	0.3	0.0039	0.001138	3.87	5.27	7.03
40	2.8	4.5	6	0.4	0.00526	0.001139	4.92	7.90	10.54
50	3.5	5.5	7.5	0.5	0.0066	0.001142	6.13	9.63	13.13
60	4.8	7.5	9	0.6	0.0079	0.001143	8.40	13.12	15.75
70	6.5	9	12	0.7	0.0092	0.001144	11.36	15.73	20.98
80	8	11	15	0.8	0.01	0.001145	13.97	19.21	26.20
90	11	14	19	0.9	0.012	0.001147	19.18	24.41	33.13
100	15	19	23	1	0.013	0.00115	26.09	33.04	40.00
120	19	23	26	1.2	0.016	0.001152	32.99	39.93	45.14
140	23	28	31	1.4	0.018	0.001155	39.83	48.48	53.68
160	27	32	36	1.6	0.021	0.001158	46.63	55.27	62.18
180	31	36	41	1.8	0.024	0.001162	53.36	61.96	70.57
200	34	39	47	2	0.026	0.001164	58.42	67.01	80.76
240	39	42	51	2.4	0.032	0.001171	66.61	71.73	87.11
280	42	47	56	2.8	0.037	0.001178	71.31	79.80	95.08
320	45	51	60	3.2	0.042	0.001183	76.08	86.22	101.44
360	49	54	65	3.6	0.047	0.001189	82.42	90.83	109.34
400	53	59	69	4	0.053	0.001197	88.55	98.58	115.29
440	57	61	73	4.4	0.058	0.001203	94.76	101.41	121.36
480	61	67	77	4.8	0.063	0.00121	100.83	110.74	127.27
520	65	71	82	5.2	0.068	0.001217	106.82	116.68	134.76
560	68	75	84	5.6	0.074	0.001225	111.02	122.45	137.14
600	68	79	85	6	0.079	0.001231	110.48	128.35	138.10
640	69	81	86	6.4	0.084	0.001238	111.47	130.86	138.93
680	69	82	86	6.8	0.089	0.001244	110.9325	131.83	138.26
720	68	82	87	7.2	0.095	0.001253	108.5395	130.89	138.87
760	68	80	86	7.6	0.1	0.00126	107.9365	126.98	136.51
800	0	80	86	8	0.105	0.001267		126.28	135.75
840	0	0	85	8.4	0.111	0.001275			133.33
880	0	0	0	8.8	0.116	0.001283			

TABLE 6.2.3: Triaxial test 55% clay, 45%GD (sample A)

Deformation on dial guage (Div)	Proving ring reading (98.1kn/m²)	Proving ring reading (196.1kn/m²)	Proving ring reading (294.2kn/m²)	Sample deformation (mm)	Strain	Corrected area (m²)	Stress (98.1kn/m²) (kn/m²)	Stress (196.1kn/m²) (kn/m²)	Stress (294.2kn/m²) (kn/m²)
0	0	0	0	0	0	0.001134	0.00	0.00	0.00
10	2.1	2.4	1.8	0.1	0.0013	0.001135	3.70	4.23	3.17
20	2.8	3	3.3	0.2	0.0026	0.001137	4.93	5.28	5.80
30	3.5	4.4	4.8	0.3	0.0039	0.001138	6.15	7.73	8.44
40	5	6.5	5.5	0.4	0.00526	0.001139	8.78	11.41	9.66
50	6.5	9	8	0.5	0.0066	0.001142	11.38	15.76	14.01
60	8	12	12	0.6	0.0079	0.001143	14.00	21.00	21.00
70	10.5	17	16	0.7	0.0092	0.001144	18.36	29.72	27.97
80	13	19	19	0.8	0.01	0.001145	22.71	33.19	33.19
90	17	23	21	0.9	0.012	0.001147	29.64	40.10	36.62
100	21	26	26	1	0.013	0.00115	36.52	45.22	45.22
120	24	31	33	1.2	0.016	0.001152	41.67	53.82	57.29
140	29	37	39	1.4	0.018	0.001155	50.22	64.07	67.53
160	32	41	42	1.6	0.021	0.001158	55.27	70.81	72.54
180	36	49	47	1.8	0.024	0.001162	61.96	84.34	80.90
200	40	55	52	2	0.026	0.001164	68.73	94.50	89.35
240	44	59	57	2.4	0.032	0.001171	75.15	100.77	97.35
280	46	62	62	2.8	0.037	0.001178	78.10	105.26	105.26
320	54	67	67	3.2	0.042	0.001183	91.29	113.27	113.27
360	60	72	72	3.6	0.047	0.001189	100.93	121.11	121.11
400	62	75	75	4	0.053	0.001197	103.59	125.31	125.31
440	64	78	79	4.4	0.058	0.001203	106.40	129.68	131.34
480	66	82	84	4.8	0.063	0.00121	109.09	135.54	138.84
520	68	86	87	5.2	0.068	0.001217	111.75	141.33	142.97
560	70	88	91	5.6	0.074	0.001225	114.29	143.67	148.57
600	71	91	94	6	0.079	0.001231	115.35	147.85	152.72
640	71	92	98	6.4	0.084	0.001238	114.70	148.63	158.32
680	70	92	104	6.8	0.089	0.001238	113.0856	148.63	168.01
720	70	91	106	7.2	0.095	0.001244	112.54	147.91	169.84
760	0	90	107	7.6	0.1	0.00126		142.86	169.84
800	0	89	107	8	0.105	0.001267		140.49	168.90
840	0	0	106	8.4	0.111	0.001275			166.27
880	0	0	0	8.8	0.116	0.001283			

TABLE 6.2.4: Triaxial test 45% clay, 55%GD (sample A)

Deformation on dial guage (Div)	Proving ring reading (98.1kn/m²)	Proving ring reading (196.1kn/m²)	Proving ring reading (294.2kn/m²)	Sample deformation (mm)	Strain	Corrected area (m²)	Stress (98.1kn/m²) (kn/m²)	Stress (196.1kn/m²) (kn/m²)	Stress (294.2kn/m²) (kn/m²)
0	0	0	0	0	0	0.001134	0.00	0.00	0.00
10	2.2	2.8	3.1	0.1	0.0013	0.001135	3.88	4.93	5.46
20	3.7	3.6	4.8	0.2	0.0026	0.001137	6.51	6.33	8.44
30	5	6.4	6.5	0.3	0.0039	0.001138	8.79	11.25	11.42
40	6.5	10	11	0.4	0.00526	0.001139	11.41	17.56	19.32
50	9	14	16	0.5	0.0066	0.001142	15.76	24.52	28.02
60	11	18	21	0.6	0.0079	0.001143	19.25	31.50	36.75
70	14	22	26	0.7	0.0092	0.001144	24.48	38.46	45.45
80	18	26	34	0.8	0.01	0.001145	31.44	45.41	59.39
90	22	31	41	0.9	0.012	0.001147	38.36	54.05	71.49
100	26	36	48	1	0.013	0.00115	45.22	62.61	83.48
120	31	41	54	1.2	0.016	0.001152	53.82	71.18	93.75
140	37	48	60	1.4	0.018	0.001155	64.07	83.12	103.90
160	41	54	67	1.6	0.021	0.001158	70.81	93.26	115.72
180	48	60	72	1.8	0.024	0.001162	82.62	103.27	123.92
200	56	66	78	2	0.026	0.001164	96.22	113.40	134.02
240	61	71	87	2.4	0.032	0.001171	104.18	121.26	148.59
280	67	78	91	2.8	0.037	0.001178	113.75	132.43	154.50
320	72	87	98	3.2	0.042	0.001183	121.72	147.08	165.68
360	77	91	106	3.6	0.047	0.001189	129.52	153.07	178.30
400	83	96	118	4	0.053	0.001197	138.68	160.40	197.16
440	86	101	121	4.4	0.058	0.001203	142.98	167.91	201.16
480	89	107	127	4.8	0.063	0.00121	147.11	176.86	209.92
520	92	111	134	5.2	0.068	0.001217	151.19	182.42	220.21
560	91	118	138	5.6	0.074	0.001225	148.57	192.65	225.31
600	90	121	141	6	0.079	0.001231	146.22	196.59	229.08
640	89	128	142	6.4	0.084	0.001238	143.78	206.79	229.40
680	0	131	144	6.8	0.089	0.001244		210.61	231.51
720	0	131	146	7.2	0.095	0.001253		209.10	233.04
760	0	130	147	7.6	0.1	0.00126		206.35	233.33
800	0	129	146	8	0.105	0.001267		203.63	230.47
840	0	0	146	8.4	0.111	0.001275			229.02
880	0	0	145	8.8	0.116	0.001283			226.03

TABLE 6.2.5: Triaxial test 100% clay (sample B)

Deformation on dial guage (Div)	Proving ring reading (98.1kn/m^2)	Proving ring reading (196.1kn/m^2)	Proving ring reading (294.2kn/m^2)	Sample deformation (mm)	Strain	Corrected area (m^2)	Stress (98.1kn/m^2) (kn/m^2)	Stress (196.1kn/m^2) (kn/m^2)	Stress (294.2kn/m^2) (kn/m^2)
0	0	0	0	0	0	0.001134	0.00	0.00	0.00
10	1	1.1	2	0.1	0.0013	0.001135	2.47	1.94	2.64
20	3	2.4	3	0.2	0.0026	0.001137	4.75	4.22	5.10
30	4	3.7	4	0.3	0.0039	0.001138	7.03	6.50	7.56
40	5	5	6	0.4	0.00526	0.001139	9.31	8.78	10.01
50	7	6.3	7	0.5	0.0066	0.001142	11.56	11.03	12.43
60	8	7.6	9	0.6	0.0079	0.001143	13.82	13.30	14.87
70	9	8.9	10	0.7	0.0092	0.001144	16.08	15.56	17.31
80	11	10.2	11	0.8	0.01	0.001145	18.34	17.82	19.74
90	12	11.5	13	0.9	0.012	0.001147	20.58	20.05	22.14
100	13	12.8	14	1	0.013	0.00115	22.78	22.26	24.52
120	14	15	17	1.2	0.016	0.001152	24.31	26.04	29.51
140	16	18	22	1.4	0.018	0.001155	28.05	31.86	38.10
160	18	22	28	1.6	0.021	0.001158	31.78	37.65	48.36
180	21	25	32	1.8	0.024	0.001162	35.46	43.37	55.08
200	23	29	38	2	0.026	0.001164	39.18	49.14	65.29
240	25	32	43	2.4	0.032	0.001171	42.70	54.65	73.44
280	27	35	47	2.8	0.037	0.001178	46.18	60.10	80.14
320	29	39	51	3.2	0.042	0.001183	49.70	65.60	86.90
360	32	42	56	3.6	0.047	0.001189	53.15	70.98	93.52
400	34	46	60	4	0.053	0.001197	56.47	76.19	99.92
440	36	49	64	4.4	0.058	0.001203	59.85	81.46	106.40
480	38	52	68	4.8	0.063	0.00121	63.14	86.61	112.73
520	40	56	72	5.2	0.068	0.001217	66.39	91.70	118.98
560	43	59	77	5.6	0.074	0.001225	69.55	96.65	125.06
600	45	63	81	6	0.079	0.001231	72.79	101.71	131.28
640	45	72	84	6.4	0.084	0.001238	72.70	116.32	135.70
680	44	73	83	6.8	0.089	0.001244	70.74	117.36	133.44
720	43	74	84	7.2	0.095	0.001253	68.64	118.12	134.08
760	0	73	82	7.6	0.1	0.00126		115.87	130.16
800	0	72	81	8	0.105	0.001267		113.65	127.86
840	0	0	0	8.4	0.111	0.001275			
880	0	0	0	8.8	0.116	0.001283			

TABLE 6.2.6: Triaxial test 75% clay, 25%GD (sample B)

Deformation on dial guage (Div)	Proving ring reading (98.1kn/m^2)	Proving ring reading (196.1kn/m^2)	Proving ring reading (294.2kn/m^2)	Sample deformation (mm)	Strain	Corrected area (m^2)	Stress (98.1kn/m^2) (kn/m^2)	Stress (196.1kn/m^2) (kn/m^2)	Stress (294.2kn/m^2) (kn/m^2)
0	0	0	0	0	0	0.001134	0.00	0.00	0.00
10	1	1.1	2	0.1	0.0013	0.001135	2.47	1.94	2.64
20	3	2.4	3	0.2	0.0026	0.001137	4.93	4.22	5.63
30	4	3.7	5	0.3	0.0039	0.001138	7.38	6.50	8.61
40	6	5	7	0.4	0.00526	0.001139	9.83	8.78	11.59
50	7	6.3	8	0.5	0.0066	0.001142	12.26	11.03	14.54
60	8	7.6	10	0.6	0.0079	0.001143	14.70	13.30	17.50
70	10	8.9	12	0.7	0.0092	0.001144	17.13	15.56	20.45
80	11	10.2	13	0.8	0.01	0.001145	19.56	17.82	23.41
90	13	11.5	15	0.9	0.012	0.001147	21.97	20.05	26.33
100	14	12.8	17	1	0.013	0.00115	24.35	22.26	29.22
120	12	15	22	1.2	0.016	0.001152	20.83	26.04	38.19
140	15	19	26	1.4	0.018	0.001155	26.67	32.38	45.37
160	19	22	30	1.6	0.021	0.001158	32.47	38.69	52.50
180	22	26	35	1.8	0.024	0.001162	38.21	44.92	59.55
200	26	30	39	2	0.026	0.001164	43.99	51.20	66.67
240	29	34	43	2.4	0.032	0.001171	49.53	57.22	73.44
280	32	37	47	2.8	0.037	0.001178	55.01	63.16	80.14
320	36	41	51	3.2	0.042	0.001183	60.52	69.15	86.90
360	39	45	56	3.6	0.047	0.001189	65.94	75.02	93.52
400	43	48	60	4	0.053	0.001197	71.18	80.70	99.92
440	46	52	64	4.4	0.058	0.001203	76.48	86.45	106.40
480	49	56	68	4.8	0.063	0.00121	81.65	92.07	112.73
520	50	59	72	5.2	0.068	0.001217	82.17	97.62	118.98
560	50	63	77	5.6	0.074	0.001225	81.63	103.02	125.06
600	49	67	81	6	0.079	0.001231	79.61	108.53	131.28
640	0	72	85	6.4	0.084	0.001238		116.32	137.32
680	0	73	83	6.8	0.089	0.001244		117.36	133.44
720	0	74	84	7.2	0.095	0.001253		118.12	134.08
760	0	73	83	7.6	0.1	0.00126		115.87	131.75
800	0	73	83	8	0.105	0.001267		115.23	131.02
840	0	0	0	8.4	0.111	0.001275			
880	0	0	0	8.8	0.116	0.001283			

TABLE 6.2.7: Triaxial test 65% clay, 35%GD (sample B)

Deformation on dial guage (Div)	Proving ring reading (98.1kn/m^2)	Proving ring reading (196.1kn/m^2)	Proving ring reading (294.2kn/m^2)	Sample deformation (mm)	Strain	Corrected area (m^2)	Stress (98.1kn/m^2) (kn/m^2)	Stress (196.1kn/m^2) (kn/m^2)	Stress (294.2kn/m^2) (kn/m^2)
0	0	0	0	0	0	0.001134	0.00	0.00	0.00
10	1	1.1	2	0.1	0.0013	0.001135	2.47	1.94	2.64
20	3	2.4	3	0.2	0.0026	0.001137	4.75	4.22	5.10
30	4	3.7	4	0.3	0.0039	0.001138	7.03	6.50	7.56
40	6	4.5	6	0.4	0.00526	0.001139	10.54	7.90	10.18
50	7	6.3	7	0.5	0.0066	0.001142	11.56	11.03	12.43
60	8	7.6	9	0.6	0.0079	0.001143	13.82	13.30	14.87
70	9	8.9	10	0.7	0.0092	0.001144	16.08	15.56	17.31
80	11	10.2	11	0.8	0.01	0.001145	18.34	17.82	19.74
90	12	11.5	13	0.9	0.012	0.001147	20.58	20.05	22.14
100	13	12.8	14	1	0.013	0.00115	22.78	22.26	24.52
120	14	15	17	1.2	0.016	0.001152	24.31	26.04	29.51
140	16	18	22	1.4	0.018	0.001155	28.05	31.86	38.10
160	18	22	28	1.6	0.021	0.001158	31.78	37.65	48.36
180	21	25	32	1.8	0.024	0.001162	35.46	43.37	55.08
200	23	29	38	2	0.026	0.001164	39.18	49.14	65.29
240	25	32	43	2.4	0.032	0.001171	42.70	54.65	73.44
280	27	35	47	2.8	0.037	0.001178	46.18	60.10	80.14
320	29	39	52	3.2	0.042	0.001183	49.70	65.60	87.91
360	32	42	58	3.6	0.047	0.001189	53.15	70.98	97.56
400	34	46	64	4	0.053	0.001197	56.47	76.19	106.93
440	37	50	69	4.4	0.058	0.001203	61.51	83.13	114.71
480	41	54	73	4.8	0.063	0.00121	67.77	89.26	120.66
520	44	58	77	5.2	0.068	0.001217	72.31	95.32	126.54
560	46	63	81	5.6	0.074	0.001225	75.10	102.86	132.24
600	48	68	84	6	0.079	0.001231	77.99	110.48	136.47
640	49	72	87	6.4	0.084	0.001238	79.16	116.32	140.55
680	50	75	89	6.8	0.089	0.001244	80.39	120.58	143.09
720	50	76	90	7.2	0.095	0.001253	79.81	121.31	143.66
760	49	77	91	7.6	0.1	0.00126	77.78	122.22	144.44
800	0	76	90	8	0.105	0.001267		119.97	142.07
840	0	0	90	8.4	0.111	0.001275			141.18
880	0	0	0	8.8	0.116	0.001283			

TABLE 6.2.8: Triaxial test 55% clay, 45%GD (sample B)

Deformation on dial guage (Div)	Proving ring reading (98.1kn/m²)	Proving ring reading (196.1kn/m²)	Proving ring reading (294.2kn/m²)	Sample deformation (mm)	Strain	Corrected area (m²)	Stress (98.1kn/m²) (kn/m²)	Stress (196.1kn/m²) (kn/m²)	Stress (294.2kn/m²) (kn/m²)
0	0	0	0	0	0	0.001134	0.00	0.00	0.00
10	1	1.1	2	0.1	0.0013	0.001135	2.47	1.94	2.64
20	3	2.6	3	0.2	0.0026	0.001137	4.75	4.57	5.63
30	4	3.7	4	0.3	0.0039	0.001138	7.03	6.50	7.56
40	6	4.5	6	0.4	0.00526	0.001139	10.54	7.90	10.18
50	7	6.3	7	0.5	0.0066	0.001142	11.56	11.03	12.43
60	8	7.6	9	0.6	0.0079	0.001143	13.82	13.30	14.87
70	9	8.9	10	0.7	0.0092	0.001144	16.08	15.56	17.31
80	11	10.2	11	0.8	0.01	0.001145	18.34	17.82	19.74
90	12	11.5	13	0.9	0.012	0.001147	20.58	20.05	22.14
100	13	12.8	14	1	0.013	0.00115	22.78	22.26	24.52
120	14	16	17	1.2	0.016	0.001152	24.31	27.78	29.51
140	16	21	22	1.4	0.018	0.001155	28.05	36.36	38.10
160	18	25	28	1.6	0.021	0.001158	31.78	43.18	48.36
180	22	29	33	1.8	0.024	0.001162	37.87	49.91	56.80
200	25	34	39	2	0.026	0.001164	42.96	58.42	67.01
240	29	38	45	2.4	0.032	0.001171	49.53	64.90	76.86
280	32	41	48	2.8	0.037	0.001178	54.33	69.61	81.49
320	36	46	57	3.2	0.042	0.001183	60.86	77.77	96.37
360	41	51	64	3.6	0.047	0.001189	68.97	85.79	107.65
400	45	56	72	4	0.053	0.001197	75.19	93.57	120.30
440	49	60	77	4.4	0.058	0.001203	81.46	99.75	128.01
480	53	63	81	4.8	0.063	0.00121	87.60	104.13	133.88
520	56	65	86	5.2	0.068	0.001217	92.03	106.82	141.33
560	57	68	91	5.6	0.074	0.001225	93.06	111.02	148.57
600	57	72	95	6	0.079	0.001231	92.61	116.98	154.35
640	56	75	101	6.4	0.084	0.001238	90.47	121.16	163.17
680	0	78	105	6.8	0.089	0.001244		125.40	168.81
720	0	82	106	7.2	0.095	0.001253		130.89	169.19
760	0	83	107	7.6	0.1	0.00126		131.75	169.84
800	0	84	107	8	0.105	0.001267		132.60	168.90
840	0	82	106	8.4	0.111	0.001275		128.63	166.27
880	0	0	106	8.8	0.116	0.001283			165.24

TABLE 6.2.8: Triaxial test 45% clay, 55%GD (sample B)

Deformation on dial guage (Div)	Proving ring reading (98.1kn/m²)	Proving ring reading (196.1kn/m²)	Proving ring reading (294.2kn/m²)	Sample deformation (mm)	Strain	Corrected area (m²)	Stress (98.1kn/m²) (kn/m²)	Stress (196.1kn/m²) (kn/m²)	Stress (294.2kn/m²) (kn/m²)
0	0	0	0	0	0	0.001134	0.00	0.00	0.00
10	1	1.1	2	0.1	0.0013	0.001135	2.47	1.94	2.64
20	3	2.6	3	0.2	0.0026	0.001137	4.75	4.57	5.63
30	5	3.7	5	0.3	0.0039	0.001138	7.91	6.50	8.79
40	7	4.5	8	0.4	0.00526	0.001139	11.41	7.90	14.05
50	9	6.3	11	0.5	0.0066	0.001142	15.76	11.03	19.26
60	11	7.6	14	0.6	0.0079	0.001143	19.25	13.30	24.50
70	14	8.9	18	0.7	0.0092	0.001144	24.48	15.56	31.47
80	17	10.2	21	0.8	0.01	0.001145	29.69	17.82	36.68
90	19	13	26	0.9	0.012	0.001147	33.13	22.67	45.34
100	22	16	30	1	0.013	0.00115	38.26	27.83	52.17
120	26	19	34	1.2	0.016	0.001152	45.14	32.99	59.03
140	29	22	38	1.4	0.018	0.001155	50.22	38.10	65.80
160	32	27	42	1.6	0.021	0.001158	55.27	46.63	72.54
180	35	29	47	1.8	0.024	0.001162	60.24	49.91	80.90
200	37	35	51	2	0.026	0.001164	63.57	60.14	87.63
240	41	39	56	2.4	0.032	0.001171	70.03	66.61	95.64
280	46	44	60	2.8	0.037	0.001178	78.10	74.70	101.87
320	52	49	65	3.2	0.042	0.001183	87.91	82.84	109.89
360	56	54	71	3.6	0.047	0.001189	94.20	90.83	119.43
400	59	61	77	4	0.053	0.001197	98.58	101.92	128.65
440	62	65	81	4.4	0.058	0.001203	103.08	108.06	134.66
480	65	69	87	4.8	0.063	0.00121	107.44	114.05	143.80
520	66	72	94	5.2	0.068	0.001217	108.46	118.32	154.48
560	67	76	98	5.6	0.074	0.001225	109.39	124.08	160.00
600	67	78	104	6	0.079	0.001231	108.85	126.73	168.97
640	66	83	109	6.4	0.084	0.001238	106.62	134.09	176.09
680	66	86	111	6.8	0.089	0.001244	106.11	138.26	178.46
720	0	91	110	7.2	0.095	0.001253		145.25	175.58
760	0	93	110	7.6	0.1	0.00126		147.62	174.60
800	0	93	109	8	0.105	0.001267		146.80	172.06
840	0	92	108	8.4	0.111	0.001275		144.31	169.41
880	0	0	0	8.8	0.116	0.001283			