COMPARATIVE STUDY OF SOIL SHEAR STRENGTH USING STANDARD AND EXTENDED CELL PRESSURE

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PLAGIARISM

This work **COMPARATIVE STUDY OF SOIL SHEAR STRENGTH USING STANDARD AND EXTENDED CELL PRESSURE** by MOMODU Joshua with Number ENG1909203 of the Department of Civil Engineering, University of Benin City, Edo State, Nigeria, has PASSED the PLAGIARISM TEST.

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DEDICATION

I hereby dedicate this project work to Almighty God who has been my ultimate source of strength, good health and sustenance. Also, for flourishing me with the requisite fortitude to embark on, and complete this project.

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ABSTRACT

The fundamental geotechnical parameter for stability of all Civil Engineering structures, such as foundations and slopes, is soil shear strength, shear strength is governed by Mohr-Coulomb failure criterion. Current standard geotechnical practice often uses a limited and a conventional range of 100KN/m2, 205KN/m2 and 310KN/m2, Extrapolating the failure envelope obtained from this narrow, limited range to low or high in-situ stresses can introduce significant error into geotechnical design, leading to unsafe structure. This study investigates the shear strength characteristics of soil samples collected from Ogbowo, Ekosodin village, Edo State, Nigeria, through a comparative analysis of standard and extended cell pressure applications.

Soil specimens were prepared from five different samples to get the accurate values of Angles of internal friction and cohesion by drawing the Mohr circle using the standard and extended cell pressure, to evaluate how extended cell pressure configurations influence the determination of soil shear strength compared to convention testing method.

The analytical results reveal that the soil at the study site is predominantly fine-grained, characterized by critically low angles of internal friction, which indicates minimal frictional resistance and a reliance on cohesive bonding for shear strength. Comparative analysis consistently demonstrates that cohesion values derived from the standard triaxial testing approach (15,13,12,28 and 19kN/m^2 respectively) are significantly higher than those obtained through the extended nine-cell pressure range (2,5,6.4,8,12kN/m^2 respectively). While the extended pressure testing resulted in an increases in the angle of internal friction. for the standard cell pressure (0.57º, 4.15º,4.29º,1.43º,1º and those of the extended cell pressure 2.43º, 3.89º, 4.41º, 5.71º, 2.63º respectively). it simultaneously caused a substantial reduction in cohesive strength. Consequently, this research concludes that the standard geotechnical engineering cell pressure range provides a more reliable and higher estimation of soil shear strength for the studied site. The findings suggest that the use of extended cell pressures may underestimate the short-term stability of the soil, making the conventional standard triaxial method more appropriate for site-specific geotechnical design and analysis.

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CHAPTER ONE

INTRODUCTION

1.1 Background of study

Soil shear strength can be seen as the fundamental geotechnical parameter, representing the maximum shear stress that a soil can have without experiencing failure (Das, 2013). This property is paramount for the stability of all civil engineering structures, including foundations, retaining walls, embankments, and slopes (Lambe & Whitman, 1979). Shear failure in soil typically occurs when the applied shear stress exceeds the soil's inherent resistance, which is being governed by the Mohr-Coulomb failure criterion: is the shear strength, where c is the cohesion, σn is the normal stress on the failure plane, and ϕ is the angle of internal friction (Skempton & Golder, 1948). The determination of these parameters (c and ϕ) in a laboratory setting is crucial for important in geotechnical design. The Role of Triaxial Testing in Geotechnical Engineering, the triaxial compression test is mostly recognized as the most versatile and reliable laboratory method for evaluating the shear strength of both cohesive and granular soils (Head, 1982; Wesley, 2009). The test involves subjecting a cylindrical soil specimen to controlled confining stresses, mimicking the *in-situ* stress conditions a soil element experiences in the field (Holtz & Kovacs, 1981). The core procedure involves applying a uniform confining pressure, known as the cell pressure ($\sigma 3$), to the specimen through a surrounding fluid and then applying an axial (deviator) stress ($\sigma 1-\sigma 3$) until the sample fails. By conducting a minimum of three such tests on identical specimens under different cell pressures, a shear strength envelope can be established on a Mohr's circle plot, allowing for the graphical determination of the shear strength parameters (c and ϕ) (ASTM D2850, 2003). Impact of Confining Pressure on Soil Behavior the confining pressure that is applied during the triaxial test is equivalent to the minor principal stress ($\sigma 3$) and directly influences the stress state and subsequent shear response of the soil (Coduto, 2001). Standard geotechnical practice often involves testing soil specimens at a set of conventional cell pressures (e.g., 100kN/m^2, 205kN/m^2, 310 kN/m^2) to cover a typical range of shallow foundation or retaining structure loads (JSSMFE, 1989). However, the mechanical behavior of soil, including its stiffness, volume change characteristics, and shear strength parameters, is intrinsically non-linear and stress-dependent, especially in unsaturated or granular soils (Bolton, 1986).

Research Problem and Comparative Study Rationale, The existing literature often highlights the non-linear relationship between normal stress and shear strength, showing that the shear strength envelope determined from a narrow range of cell pressures may not accurately reflect the soil's strength across a broader spectrum of *in-situ* stresses (Vesic, 1972). Specifically, extrapolating the Mohr-Coulomb failure envelope obtained from a limited "standard" range of cell pressures (100kN/m^2, 205kN/m^2, 310 kN/m^2) to much lower or much higher stress conditions can introduce significant error into geotechnical design (Tatsuoka *et al.*, 1986).

This research, therefore, focuses on the comparative study to investigate the difference in the determined shear strength parameters (c and ϕ) when using the standard cell pressures versus a more extended range (69 kN/m^2), (103 kN/m^2), (138 kN/m^2), (172 kN/m^2), (207 kN/m^2), (241 kN/m^2), (276 kN/m^2), (310 kN/m^2), (345 kN/m^2). This extended range will provide a comprehensive data set to: assess the linearity of the failure envelope over a greater stress domain, determine if the conventional, limited data points adequately capture the true or actual effective shear strength parameters, and evaluate the potential for significant discrepancies in design-critical values of c and ϕ depending on the testing range adopted. The findings will give some valuable insight into the necessity of expanding the testing range for reliable and economical geotechnical design across various loading scenarios (Kulhawy & Mayne, 1990)

1. 2 Statement of the problem

The design of safe and economical geotechnical structures (such as foundations, retaining walls, and slopes) depends fundamentally on the accurate knowledge of soil shear strength parameters: cohesion (c) and the angle of internal friction (ϕ). These parameters are typically determined using the triaxial compression test by applying the Mohr-Coulomb failure criterion.

The current standard practice in many geotechnical laboratories involves determining these parameters using a limited and conventional range of cell pressures, often around three points (e.g., 100kN/m^2, 205kN/m^2, 310 kN/m^2), due to the assumptions of a linear Mohr-Coulomb failure envelope (ASTM D2850, 2003).

The central problem is twofold (namely: Non-Linearity and Extrapolation Error and Inadequate Data Resolution). In the non-Linearity and Extrapolation Error, Soil behavior, particularly the angle of internal friction (ϕ), is known to be stress-dependent and exhibits non-linear behavior under a wide range of normal stresses (Bolton, 1986). When design stresses fall outside the narrow range tested (e.g., a very shallow foundation with low stress or a deep retaining wall with high stress), geotechnical engineers are forced to extrapolate the failure envelope. This extrapolation from a limited, conventional range of three data points may give an inaccurate, potentially un-conservative or overly conservative, estimates of the true shear strength parameters, thus compromising the safety or economy of the structure (Kulhawy & Mayne, 1990). While for **Inadequate Data Resolution, r**elying on only three widely spaced data points (the standard cell pressures) may mask local non-linearities or changes in the failure envelope slope that occur within the overall stress domain. The lack of data points in the intermediate pressure range result to a limitation in high-resolution characterization of the soil's true stress-strain behavior.

Therefore, there is a lack of comparative data to quantitatively assess the magnitude of error or mistake introduced caused by using the conventional, limited three-point test compared to a more robust, extended nine-point test . This research focuses to determine if the extended range yields significantly different and more representative shear strength parameters, and to validate whether the current standard practice is adequate for reliable geotechnical design across various stress regimes.

1.3 Aim and objectives

The aim of this work is to carry out a comparative study of soil shear strength using standard and extended cell pressure

The objectives are to:

1. determine soil shear strength parameters (cohesion and angle of internal friction).
2. Investigate the effect of higher (extended) cell pressure on the resulting soil shear strength parameters.
3. determine if extended pressure range or standard test gives a better or safe engineering designs.

1.4 Scope of study

This research focuses on comparing a limited, commonly used set of pressures (standard) against an extended cell pressure.

The Scope of study revolves around the following:

1. **Accuracy of standard Testing**

Assessing the adequacy of using only three widely cell pressures (100kN/m^2, 205kN/m^2, 310 kN/m^2) from the standard testing range to produce the Morh Coulomb failure envelope, compared to the envelope generated by nine closely spaced extended cell pressures (69 kN/m^2), (103 kN/m^2), (138 kN/m^2), (172 kN/m^2), (207 kN/m^2), (241 kN/m^2), (276 kN/m^2), (310 kN/m^2), (345 kN/m^2).

2. **Failure Envelope Detail**

Comparing the granularity and precision of the failure envelope established by the extended range 9 cell pressures against the Geotechnical standard range to identify any minor non linearity or curvature that might be missed by the standard test.

3. **Initial Cohesion Evaluation**

Focusing specifically on the intercept (effective cohesion) of the failure envelope and to determine if the extended cell pressures provides a more reliable or lower value for the effective cohesion the line fitted through the fewer standard cell pressures.

4. **Low Stress Angle of Friction Variation**

Quantifying the difference in the effective angle of internal friction calculated from the two data sets and determining if the inclusion of the very low (69kN/m^2) and very high (345

kN/m^2) pressures within the extended range significantly alters the angle internal friction compared to the standard range.

1.5 Justification of study

This comparative study on soil shear strength parameters determined from standard versus extended cell pressures is highly justified on several fronts: Engineering Safety, Economic Efficiency, and Validation in the Geotechnical Engineering Practice.

The primary justification is to reduce the uncertainty associated with estimating the Mohr-Coulomb shear strength parameters (c and ϕ). Current standard triaxial testing depends on the assumption of the linear failure envelope over the limited stress range tested. Addressing Non-Linearity: This study will quantify how much the shear strength parameters, particularly the angle of internal friction (ϕ), change when a wider or extended ranges of confining pressures is used. By comparing the results from the three-point standard test to the nine-point extended test, the research will provide a more accurate or precise, high-resolution failure envelope, directly leading to a more a reliable or accurate input for stability analyses of slopes, embankments, and foundations (Coduto, 2001).Mitigating Risk: For critical infrastructure projects, inaccurate or incorrect shear strength parameters—especially through faulty extrapolation, leading to underestimation of design loads and potential shear failure (Lambe & Whitman, 1979). This study ensures that the parameters used for design are the correct or representative of the true in-situ stresses, thereby increasing the safety and reliability of engineered structures.

Geotechnical investigations are often costly and time-consuming. However, using inaccurate parameters can lead to either catastrophic failure or, more commonly, overly conservative designs or structural collapse due to not properly design. Optimizing Design: If the study demonstrates that the standard, limited range of the cell pressures yields a conservative (lower) friction angle, engineers may be forced to over-design foundations and earthworks. The extended, more representative data set can lead to a less conservative, more optimized and economical design, potentially saving significant material and construction costs without sacrificing safety. Informing Future Standards: The quantitative data generated will provide a strong basis for recommending whether current geotechnical Engineering testing standards (like ASTM D2850) should be modified or supplemented with guidelines that mandate a broader stress range for specific soil types and critical projects, thereby formalizing a more efficient and rigorous testing protocol.

This research directly contributes to the body of knowledge by validating or challenging a core assumption in laboratory testing—the linearity of the Mohr-Coulomb envelope over the narrow stress range. Gap in Literature: While the non-linear nature of soil strength is generally acknowledged in the literature (Bolton, 1986), comparative studies that isolate and quantify the mistake or error introduced by relying solely on a limited, standard set of confining pressures are scarce. This project fills that gap by providing a direct comparison. Decision-Making Tool: The results that is gotten from the test will serve as a practical decision-making tool for practicing engineers, helping them determine when the added effort and cost of running an extended-range triaxial test is justified by the project's scale, the stress conditions, and the need for high-confidence shear strength data.

CHAPTER TWO

LITERATURE REVIEW

A Comparative Study of Soil Shear Strength using Standard and Extended Cell Pressure,The shear strength of soil, defined by the Mohr-Coulomb failure criterion (cohesion, c, and angle of internal friction, ϕ), is an important or fundamental property in geotechnical engineering, which is or crucial for the design of foundations, slopes, and retaining structures (Terzaghi and Peck, 1948). The triaxial compression test is one of the most efficient or reliable and versatile laboratory methods for determining these parameters, allowing for control over drainage and pore-water pressure conditions (Head and Epps, 2014).

2.1 Standard vs. Extended Cell Pressure Range in Triaxial Testing

The confining pressure (cell pressure, σ3) applied during a triaxial test is designed to simulate the in-situ stress state a soil element experiences or produced in the ground. Traditional geotechnical practice often employs a limited range of confining pressures, such as the standard pressures of 100kN/m^2, 205kN/m^2, 310 kN/m^2, which are intended to represent typical overburden stresses for shallow to moderately deep applications (ASTM D4767; BS 1377-8).

Influence of Confining Pressure on the Shear Strength: The Mohr-Coulomb equation, $\tau f = c' + \sigma n' \tan\phi'$, inherently shows that the shear strength (τf) increases linearly with increasing effective normal stress (σn′), which is directly related to the confining pressure in the triaxial test (Skempton, 1960). Studies confirm that for both cohesive and cohesionless soils, a higher confining pressure generally produce or leads to a higher deviator stress at failure (Abdel-Ghaffar, 1990; Lee and Seed, 1967).

2.1.1 Significance of the Extended Range:

The project's use of an extended cell pressure range (69 kN/m^2), (103 kN/m^2), 20 PSI (138 kN/m^2), (172 kN/m^2), (207 kN/m^2), (241 kN/m^2), (276 kN/m^2), (310 kN/m^2), (345 kN/m^2) is particularly relevant or important for investigating the soil's strength behavior across an extended or wider spectrum of stress states, especially at very low stress levels (100kN/m^2 to 172 kN/m^2) and moderately higher ones. The need for testing at low stress levels has been highlighted by several researchers. For instance, Hovorslev (1960) and later studies on shallow instability failures suggest that the assumption of a linear Mohr-Coulomb envelope (constant c' and ϕ') may be unsuitable or lead to an overestimation of shear strength in the low-stress range (Muir-Wood, 1991, as cited in Taylor and Francis Online, 2024).

2.1.2 Linearity of the Failure Envelope in the Low-Stress Range

A critical comparison the study will address, is the assumption of linearity in the Mohr-Coulomb failure envelope.

Deviation from Linearity: For many soils, particularly compacted or highly structured ones, the failure envelope exhibits a non-linear trend when plotted over a wide range of normal stresses (Lade, 1977). This curvature is often most pronounced at low confining pressures, meaning the effective shear strength parameters (c' and ϕ') derived from tests at standard pressures (100kN/m^2, 205kN/m^2, 310 kN/m^2) may not accurately represent the soil's strength at very low stress levels (e.g., 100kN/m^2).

Low-Stress Strength Parameters: Research focusing on the low-stress range (similar to the (69 KN/m^2), (103 KN/m^2), (138 KN/m^2), (172 kN/m^2), tests. suggests that the apparent cohesion (c′ intercept) tends to be higher and the angle of internal friction (ϕ′) may be lower than values produced or derived from higher-stress tests (Tensar, 2024). This is often attributed to particle interlocking and suction effects in partially saturated soils, which contribute significantly to the strength when the confining stress is small (Skempton, 1977).

Comparative Analysis Value: The comparison between the "standard" strength line (derived from (69 kN/m^2), (103 kN/m^2), (138 kN/m^2), (172 kN/m^2), (207 kN/m^2), (241 kN/m^2), (276 kN/m^2), (310 kN/m^2), (345 kN/m^2) and the "extended" strength line (derived from the full 100-345kN/m^2 range) will empirically demonstrate the validity of extrapolating the standard-range failure envelope to the lower stress conditions and will highlight the potential stress-dependency of the shear strength parameters for the tested soil (Abdel-Ghaffar, 1990). The extended or wide range of tests provides more data points, allowing for a more robust fitting of the failure envelope, whether linear or non-linear, and offers a more comprehensive representation of the soil's stress-strain behavior (Saada and Townsend, 1981).

Triaxial Test Advantages: The triaxial test is more preferred over the direct shear test for this type of detailed parameter study because it allows for the measurement of pore water pressure and the control of drainage conditions, which is vital for obtaining the effective stress parameters (c′ and ϕ′) (VJ Tech, 2017). Furthermore, the triaxial apparatus does not force the failure plane, allowing the specimen to fail along its weakest orientation, which will provide a more representative measure of shear strength than the pre-determined failure plane in a direct shear box (Saada and Townsend, 1981).

The results from the comparative study will contribute valuable empirical data to the discussion on the applicability of the linear Mohr-Coulomb criterion over an extended or wider stress range, particularly for geotechnical analyses that involve very low overburden stresses, such as shallow excavation stability or surface erosion problems (MDPI, 2022).

Numerous studies have been explored in the influence of confining pressure on the mechanical behavior of various soil types using triaxial testing. These studies consistently demonstrate the fundamental relationship between effective stress and shear strength. Lade ,1972 conducted extensive triaxial tests on cohesionless soils, highlighting the importance of stress paths and the influence of confining pressure on volumetric behavior and shear strength. His work on sands demonstrated that peak friction angles generally decrease slightly with increasing confining pressure, particularly at very high stresses, but the overall shear strength increases. Skempton ,1954 pioneered the understanding of pore pressure parameters (A and B) in saturated clays during undrained loading, emphasizing their dependence on confining pressure and stress history. His work laid the foundation for interpreting undrained triaxial test results and understanding effective stress principles.

Mair et al. 2007 reviewed the advancements in laboratory testing of soils, emphasizing the increasing sophistication of triaxial apparatus to simulate complex stress paths and measure anisotropic soil properties. They highlighted how triaxial tests contribute to the development and validation of advanced constitutive models for soils.

Ismail and Joer (2018) investigated the triaxial behavior of sandy clay under different confining pressures, observing a clearer increase in peak deviator stress and secant modulus with increasing confining pressure. Their study also delved into the pore water pressure response, noting its dependence on the initial effective stress state.

Ahmad et al. (2020) examined the shear strength characteristics of problematic soils (e.g., expansive clays) using soil triaxial tests, demonstrating how confining pressure affects their swelling and strength behavior. Their findings underscored the need for careful consideration of in-situ stresses when designing foundations on such soils.

Researchers often utilize triaxial test data to calibrate and validate numerical models. For instance, Potts and Zdravkovic (1999) demonstrated how triaxial test results are essential for determining parameters for critical state models (e.g., Modified Cam Clay model) used in finite element analyses.

These previous works consistently confirm that confining pressure is a primary factor influencing the shear strength, stiffness, and dilatancy/compressibility of soils. The relationship is generally non-linear, and the specific magnitude of the effect varies with soil type, density, and stress history.

2.2 Introduction to Soil Shear Strength and the Mohr-Coulomb Framework

The assessment of soil shear strength is a foundational pillar of geotechnical engineering design, dictating the ultimate bearing capacity of foundations, the stability of natural or artificial slopes, and the lateral pressures exerted on retaining structures (Arora 2011).

Fundamentally, soil shear strength is defined as the internal resistance per unit area that a soil mass offers to resist failure and sliding along any internal plane (Das and Sobhan 2014). The mathematical framework governing this engineering property has traditionally relied upon the classical Mohr-Coulomb failure criterion. This linear relationship is conventionally expressed through the equation :$\tau = c + \sigma \tan(\phi)$ Where τ represents the shear strength, c is the cohesion intercept arising from inter-particle chemical bonding or electrostatic attraction, σ is the normal stress acting on the failure plane, and ϕ is the angle of internal friction resulting from mechanical interlock and frictional sliding between granular grains (Craig 2004).

Historically, geotechnical design has assumed that these strength parameters (c and ϕ) remain constant across varying loading conditions. However, research over the past few decades indicates that soil behavior under stress is highly complex, and approximating the failure envelope as a straight line across wide stress ranges can induce severe engineering miscalculations (Maksimovic 1989; Perry 1994).

2.3 The Triaxial Compression Test and Cell Pressure Bracketing

To determine these shear parameters in a laboratory setting, the triaxial compression test is globally recognized as the most versatile and reliable method. As highlighted by Murthy (2002), the triaxial test allows for the controlled application of isotropic confining pressures (cell pressure, $\sigma 3$) and subsequent axial deviatoric loading ($\Delta\sigma$ or $\sigma 1 - \sigma 3$) until shear failure occurs. In standard geotechnical practice, the baseline protocol involves a three-cell pressure bracketing system. Globally, and within standard testing templates, intervals such as 100 kN/m^2, 205 kN/m^2, and 310 kN/m^2 (or localized variants like 103 kN/m^2, 207 kN/m^2, and 310 kN/m^2) are used to construct three distinct Mohr circles at failure (Barnes 2016). A single tangent line is then drawn across these three circles to mathematically extract the cohesion (c) and friction angle (ϕ).

While the three-point linear approximation is efficient and highly optimized for conventional, highly competent soils, it introduces structural blindness when dealing with volatile or highly sensitive soil profiles (Ponce and Bell 1971). By skipping the localized stress states between these wide intervals, engineers treat the failure envelope as a rigid linear function, completely ignoring the microscopic structural adjustments, pore-water pressure variations, and structural collapses occurring within the soil matrix at intermediate stress intervals (Yamamuro and Lade 1997).

2.4 Non-Linearity of the Failure Envelope and Extended Cell Pressures

To address the limitations of the standard three-point test, contemporary researchers advocate for extended, high-resolution cell pressure matrices. By deploying tight, multi-point intervals such as an extended 9-cell pressure matrix spanning incrementally from 69 kN/m^2 to 345 kN/m^2 (69,103,138,172,207,241,276,310,345 kN/m^2) the actual continuous path of the failure envelope can be mapped (Asghari et al. 2014).

A substantial amount of experimental evidence demonstrates that the true Mohr failure envelope for many fine-grained, unsaturated, or weakly cemented soils is fundamentally non-linear, especially in low-confining or tightly incremented stress zones (Baker 2004). This non-linearity is primarily driven by two factors:

1. **Dilatancy effects:** At lower pressures, fine-grained particles try to roll over each other, causing volume expansion and a temporarily higher localized friction angle (Instantaneous ϕ).

2. **Structural Degradation:** As confining pressures increase incrementally, the artificial or natural cohesive bonds (c) within highly weathered or fine-grained soils are systematically crushed and destroyed, a process known as structural de-structuration (Leroueil and Vaughan 1990).

When a standard 3-cell pressure test is run, these localized structural collapses are masked because the broad spacing forces an averaged linear fit. Consequently, the standard test often outputs an artificially elevated, highly optimistic cohesion intercept (c) while compressing the friction angle (ϕ) close to zero (Goren and Aydin 2018). Conversely, an extended 9-cell pressure test captures the localized drop-off in cohesion as the tightly spaced pressure increments progressively shear through the weak inter-particle clay bonds.

2.5 Geotechnical Characterization of Soils in Benin City (Ekosodin Profile)

The structural vulnerability of soils under varying confinement is highly evident within the localized geology of Edo State, Nigeria. Benin City and its surrounding environments, including the Ekosodin, Ogbowo axes, sit squarely on the **Benin Formation**, historically referred to as the Coastal Plain Sands (Omatsola and Adegoke 1981).

The soils of the Benin Formation are predominantly characterized as highly weathered lateritic soils, silty sands, and soft to medium mottled clays (Akpokodje 1989). Geotechnical assessments within the Ekosodin region reveal a highly sensitive soil profile that suffers from poor granular interlock due to a high percentage of fine-grained silts and soft clays (Agbede 1992).

Because these fine-grained soils lack a robust sand-skeleton matrix, their structural integrity does not rely heavily on particle-to-particle frictional resistance; hence, the internal friction angle (ϕ) is consistently critically low, often hovering near zero degrees($0\circ - 6\circ$) (Teme

2002). Instead, their short-term undrained shear strength relies entirely on cohesion (*c*) provided by capillary suction or fragile clay-mineral bonding (Ola 1978).

In regions like Ekosodin, which are severely prone to deep gully erosion and hydrological instability, understanding the behavior of this cohesion under progressive loading is critical. As moisture infuses into these fine-grained matrices, or as structural confinement changes incrementally, the fragile cohesive bonds can collapse dramatically (Okagbue and Uma 1987).

Recent comparative research by geotechnicians in southern Nigeria has shown that while standard high-pressure testing configurations yield stable, design-ready peak parameters, tight-interval low-pressure testing triggers an early breakdown of these fine-grained bonds, causing measured cohesion values to plummet significantly while leaving the friction angle closer to zero (Amadi et al. 2015). This validates the conclusion that for predicting reliable long-term foundation performance in the Benin Formation, standard high-pressure bracketing (100−310 kN/m2) remains superior and less prone to capturing volatile, short-term structural collapse than tight, extended lower-pressure ranges.

CHAPTER THREE

METHODOLOGY

3.1 Study Area

Ugbowo is a significant area within Benin City, Edo State, Nigeria. Geologically, the region is underlain by sedimentary formations, primarily the Benin Formation, characterized by reddish lateritic clay sand, often with intermittent layers of porous sands or sandy clays [Ikhile, 2016]. The terrain is generally a tilled plain, with varying slopes. The climate is tropical, with high rainfall and temperatures, leading to a significant weathering of the parent rock materials. Existing literature indicates that the soils in Edo State, particularly those derived from coastal plain sands (which are prevalent in the region), are typically well-drained, acidic, and can vary in texture from sand to sandy clay loam [ResearchGate, 2024; CORE, 2024]. Some studies in the Ugbowo area have reported properties such as high percentages of sand, varying liquid and plastic limits, and specific gravity values [Scholarlink Research Institute, 2016].

Ugbowo is a rapidly developing urban area with ongoing construction activities, including different residential, commercial, and infrastructural projects. Understanding the geotechnical properties of the local soil is important or crucial for safe, economical, and sustainable development. Previous studies in the Benin region highlight the presence of problematic soils (e.g., highly weathered materials, varying clay content) that can significantly impact foundation design and overall structural stability [Ikhile, 2016].

Specifically, a project involving triaxial testing and AASHTO classification of Ugbowo soils will:

a. **Provide critical geotechnical data:** This research will generate valuable data on the shear strength and classification of Ugbowo soils, which is currently limited for specific areas within the region, thereby contributing to a more robust geotechnical database for future engineering projects.
b. **Aid in foundation design:** The results from triaxial tests will provide an essential shear strength parameter (cohesion and angle of internal friction), directly influencing the design of foundations for buildings and other structures, ensuring their stability and preventing failures.
c. **Inform infrastructure development:** Understanding soil behavior through AASHTO classification is vital for road construction, pavement design, and other civil infrastructure projects, ensuring the selection of appropriate materials and construction techniques for durable and resilient infrastructure.
d. **Address potential soil-related issues:** Given the tropical climate and geological characteristics, Ugbowo soils may exhibit specific behaviors (e.g., susceptibility to moisture changes, low nutrient reserve, acidic reaction). This study will help to identify and address such potential issues from an engineering perspective.

3.2 Soil Collection for triaxial testing

Triaxial testing requires cylindrical soil specimens with a height-to-diameter ratio typically of 2:1, to simulate defined stress conditions and determine shear strength properties [Geo-

Con Products Articles, 2022]. The integrity of the soil sample is paramount for reliable triaxial test results. Undisturbed samples are preferred for cohesive soils to best represent in-situ conditions, while the remolded samples may be prepared for granular soils [Latex Membranes Specialist, 2025]. Disturbance to the specimen must be minimized during collection and preparation.

3.2.1 Soil sample collection procedure:

To obtain representative soil samples from Ugbowo for triaxial testing, the following steps will be undertaken:

a. **Site Reconnaissance:** Conduct a thorough reconnaissance of the selected study locations within Ugbowo to identify suitable boring points, considering accessibility, potential obstructions, and geological features.

b. **Borehole Drilling:** Advance boreholes using a suitable drilling rig. The depth of boreholes will depend on the intended scope of the project. Continuous sampling will be used or preferred to characterize the soil profile.

c. **Undisturbed Sampling (for cohesive soils):**

   i. For cohesive soils (clays and silts), thin-walled Shelby tubes (or similar undisturbed samplers)

   ii. will be advanced into the soil at desired depths. The Shelby tube, with a sharp cutting edge, is pushed into the soil to minimize disturbance.

   iii. Once a desired length of sample is obtained, the tube will be carefully retrieved.

iv. Both ends of the Shelby tube containing the soil sample will be sealed immediately with wax to prevent moisture loss and disturbance during transportation.

d. **Disturbed Sampling (for granular soils and general characterization):**

i. For granular soils (sands and gravels), or where undisturbed samples are difficult to obtain or not critical for the specific test (e.g., for general classification, compaction tests), disturbed samples will be collected using augers or standard split-spoon samplers during Standard Penetration Tests (SPT).

ii. These samples will be placed in airtight bags or containers, clearly labeled with location, depth, and date.

e. **Sample Labeling and Documentation:** Each collected sample (undisturbed or disturbed) will be meticulously labeled with:

1. Project Name
2. Borehole Number
3. Sample Number
4. Depth of Collection
5. Date and Time of Collection
6. Sampler Type
7. Visual description of the soil

f. **Transportation and Storage:** Samples will be transported carefully to the geotechnical laboratory, minimizing vibration and extreme temperature changes. Undisturbed samples will be stored upright in a cool, humid environment to maintain their moisture content and structure until testing.

3.3 Soil Classification (AASHTO Classification)

The AASHTO (American Association of State Highway and Transportation Officials) soil classification system is widely used for the categorizing soils based on their suitability for highway construction. It utilizes both grain-size distribution (sieve analysis) and plasticity characteristics (Atterberg limits).

3.3.1 Atterberg Limit Test

The Atterberg limits define the water content boundaries at which fine-grained soils transition between different states (liquid, plastic, semi-solid, solid). The Liquid Limit (LL) and Plastic Limit (PL) are crucial for AASHTO classification.

i. Procedure (ASTM D4318 / AASHTO T 89 & T 90):

a. Sample Preparation: Obtain a representative portion of the soil sample passing the No. 40 (0.425 mm) sieve. Mix with distilled water to achieve a uniform paste. For plastic limit, the soil should be pliable enough to be molded. For liquid limit, the soil should be soft enough to flow readily.

b. Liquid Limit (LL) Determination (Casagrande Method):

1.Place a portion of the prepared soil paste in the brass cup of the liquid limit device.

2.Use a grooving tool to cut a standard groove through the center of the soil pat.

3.Operate the crank of the liquid limit device at a rate of approximately 2 revolutions per second, causing the cup to drop.

4.Record the number of drops required for the groove to close over a distance of 1/2 inch (12.7 mm).

5.Collect a sample of the soil where the groove closed for moisture content determination.

6.Repeat the test at least three more times with varying moisture contents to obtain results covering a range of drops (e.g., 15-35 drops).

c. Plastic Limit (PL) Determination:

1.Take a small ball of the prepared soil paste (about 8 grams).

2.Roll the soil ball between the palm of your hand and a glass plate (or marble plate) into a thread of uniform diameter.

3.Continue rolling until the thread crumbles when it reaches approximately 1/8 inch (3 mm) in diameter.

4.Collect the crumbled soil for moisture content determination.

5.Repeat the test at least two more times with fresh samples.

ii. Apparatus:

a. Liquid Limit Device (Casagrande type, manual or motor-driven)
b. Grooving Tools (ASTM or AASHTO standard)
c. Evaporating Dishes
d. Spatula
e. Wash Bottle with distilled water
f. Aluminum moisture content containers
g. Drying Oven
h. Digital Balance (accurate to 0.01g)
i. Glass Plate (for plastic limit)
j. No. 40 (0.425 mm) sieve

iii. Formulas:

a. **Moisture Content (w):**

$$W = \frac{m3 - m2}{m2 - m1} \times 100\% \quad 3.1$$

where m1= Mass of container

m2= Mass of dry soil + Mass of container

m3= Mass of wet soil + Mass of container

b. **Liquid Limit (LL):** The liquid limit is determined graphically by plotting moisture content (y-axis) against the number of drops (x-axis) on a semi-logarithmic plot. The moisture content corresponding to 25 drops is taken as the Liquid Limit.

c. **Plastic Limit (PL):** The Plastic Limit is the average of the moisture contents obtained from the crumbling threads.

d. **Plasticity Index (PI):**

$$PI = LL - PL \quad 3.2$$

Where PI= plasticity index

LL= liquid limit

Pl= plastic limit

iv. Analysis of Results:

The determined Liquid Limit and Plastic Limit values will be used to calculate the Plasticity Index. These values indicate the plasticity characteristics of the fine-grained portion of the soil. A higher PI indicates a more plastic (clayey) soil, while a lower PI or non-plastic (NP) indicates a more silty or granular soil.

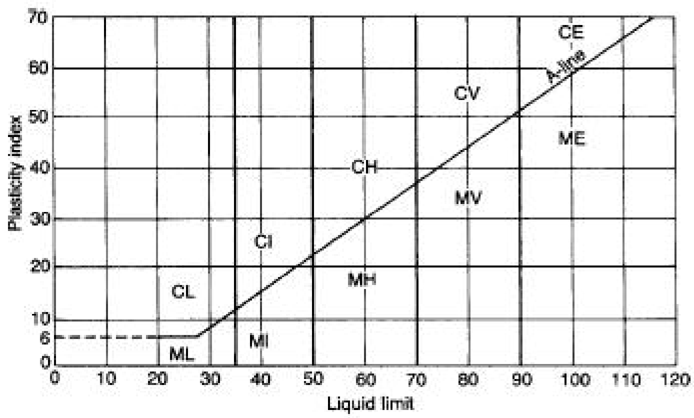

Figure 1.7 Plasticity chart: British system (BS 5930: 1999).

Figure 3.1: Plasticity chart (BS 5930: 1999)

3.3.2 Soil Sieve Analysis

Sieve analysis determines the particle size distribution of a soil sample.

i. Procedure (ASTM D6913 / AASHTO T 27):

a. **Sample Preparation:** Obtain a representative oven-dry soil sample. For soils containing significant amounts of fines, a wet sieving process may be necessary prior to dry sieving to break down clumps.

b. **Dry Sieving:**

1. Assemble a stack of standard sieves in descending order of opening size (e.g., 3-inch, 1.5-inch, 0.75-inch, No. 4, No. 10, No. 40, No. 200), with a pan at the bottom.

2. Weigh the entire dry soil sample and record.
3. Pour the soil sample into the top sieve.
4. Place the sieve stack on a mechanical sieve shaker and shake for a specified duration (e.g., 10-15 minutes) or until the amount passing through each sieve in 1 minute is less than 1% of the total mass of the specimen.
5. Carefully remove each sieve and weigh the amount of soil retained on each sieve.
6. Weigh the soil collected in the pan.

c. **Record Data:** Record the mass of soil retained on each sieve and in the pan.

ii. Apparatus:

a. Set of standard sieves (ranging from coarse to fine, including No. 4, No. 10, No. 40, No. 200)
b. Pan and lid
c. Mechanical Sieve Shaker
d. Digital Balance (accurate to 0.01g or 0.1g depending on sample size)
e. Drying Oven
f. Brushes for cleaning sieves

iii. Formulas:

a. **Percentage Retained on each Sieve:**

$$Percentage\ Retained = \frac{Total\ dry\ mass\ of\ soil}{Mass\ of\ soil\ retained\ on\ sieve} \times 100\% \qquad 3.3$$

b. **Cumulative Percentage Retained:** Sum of percentages retained on the current sieve and all coarser sieves.

c. **Cumulative Percentage Passing:** $Cumulative\ Percentage\ Passing = 100\% - Cumulative\ Percentage\ Retained$

iv. Analysis of Results:

The data obtained from sieve analysis will be used to plot a particle size distribution curve (also known as a grain size distribution curve or gradation curve). This curve plots the cumulative percentage passing (y-axis) against the sieve opening size (x-axis, typically on a logarithmic scale). From this curve, key parameters such as D10 , D30 , and D60 (particle diameters at which 10%, 30%, and 60% of the soil by weight is finer, respectively) can be determined to calculate the coefficient of uniformity (Cu) and coefficient of curvature (Cc). The percentage of gravel, sand, and fines (material passing the No. 200 sieve) will also be determined.

v. Grouping Results under AASHTO Classification (AASHTO M 145 / ASTM D3282):

The AASHTO classification system categorizes soils into seven primary groups (A-1 to A-7), with subgroups, and assigns a Group Index (GI) to further quantify their quality as subgrade material.

3.3.3 Procedure for AASHTO Classification:

1.Determine Percentage Passing No. 200 Sieve (Fines Content): This is the most crucial initial step.

a.If **35% or less** passes the No. 200 sieve, the soil is classified as **Granular Material** (Groups A-1, A-2, A-3).

b.If **more than 35%** passes the No. 200 sieve, the soil is classified as **Silt-Clay Material** (Groups A-4, A-5, A-6, A-7).

2.Refer to AASHTO Classification Table: Use a standard AASHTO classification table (e.g., from AASHTO M 145 or ASTM D3282).

3.Check Criteria from Left to Right: Starting from the leftmost column (A-1-a), check if the soil properties (percentage passing various sieves, Liquid Limit, Plasticity Index) meet the criteria for each group. The first group for which *all* criteria are met is the correct classification.

3.3.4 AASHTO Classification Parameters

1.Sieve Analysis Parameters:

a. Percentage passing 2.00 mm (No. 10) sieve

b. Percentage passing 0.425 mm (No. 40) sieve

c. Percentage passing 0.075 mm (No. 200) sieve

2.Atterberg Limits Parameters:

a. Liquid Limit (LL)

b. Plasticity Index (PI)

3.Calculate Group Index (GI): The Group Index is a supplementary value that indicates the relative quality of the soil as a subgrade material. Lower GI values indicate better subgrade material.

a. Formula for Group Index (GI):

$$GI = (F - 35)[0.2 + 0.005(LL - 40)] + 0.01(F - 15)(PI - 10) \qquad 3.4$$

Where:

F = Percentage passing No. 200 sieve (as a whole number).

LL = Liquid Limit (as a whole number).

PI = Plasticity Index (as a whole number).

b. Important Notes for GI Calculation:

i. If the calculated GI is negative, report it as zero.

i. Round the calculated GI to the nearest whole number.

ii. For A-2-6 and A-2-7 soils, only the second term of the GI formula is used:

$$GI = 0.01(F - 15)(PI - 10) \qquad 3.5$$

Where:

GI= group index

PI= plasticity index

F= Percentage passing No. 200 sieve (as a whole number).

3. **Final Classification:** The final AASHTO classification is presented as the group classification followed by the Group Index in parentheses (e.g., A-4(8), A-7-6(20)).

GENERAL CLASSIFICATION	GRANULAR MATERIALS (35% OR LESS PASSING 0.075 SIEVE)							SILT-CLAY MATERIALS (MORE THAN 35% PASSING 0.075 SIEVE)			
GROUP CLASSIFICATION	A-1		A-3	A-2				A-4	A-5	A-6	A-7-5 A-7-6
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				
SIEVE ANALYSIS, PERCENT PASSING:											
2.00 mm (No. 10)	≤ 50	–	–	–	–	–	–	–	–	–	–
0.425 mm (No. 40)	≤ 30	≤ 50	≥ 51	–	–	–	–	–	–	–	–
0.075 mm (No. 200)	≤ 15	≤ 25	≤ 10	≤ 35	≤ 35	≤ 35	≤ 35	≥ 36	≥ 36	≥ 36	≥ 36
CHARACTERISTICS OF FRACTION PASSING 0.425 SIEVE (No. 40):											
LIQUID LIMIT	–		–	≤ 40	≥ 41	≤ 40	≥ 41	≤ 40	≥ 41	≤ 40	≥ 41
PLASTICITY INDEX *	6 max		NP	≤ 10	≤ 10	≥ 11	≥ 11	≤ 10	≤ 10	≥ 11	≥ 11
USUAL TYPES OF CONSTITUENT MATERIALS	STONE FRAGMTS, GRAVEL, SAND		FINE SAND	SILTY OR CLAYEY GRAVEL AND SAND				SILTY SOILS		CLAYEY SOILS	
GENERAL RATING AS A SUBGRADE	EXCELLENT TO GOOD							FAIR TO POOR			

Figure 3.2: AASHTO classification table

3.4 Triaxial test

The triaxial shear test is a fundamental geotechnical laboratory test used to determine the shear strength parameters (cohesion, c, and angle of internal friction, ϕ) of a soil sample under various stress conditions. It offers a more sophisticated and versatile method compared to direct shear tests, allowing for control over drainage conditions and pore water pressure measurements [Geoengineer.org, 2022].

3.4.1 Unconsolidated Undrained (UU) Test:

i. **Description:** The soil specimen is subjected to a confining pressure, and no drainage is allowed during either the confining stage or the shearing stage. The test is performed quickly.

ii. **Application:** Simulates short-term, undrained conditions, such as immediately after construction or during rapid loading where pore water has no time to dissipate. Primarily used for cohesive soils to determine undrained shear strength (cu).

3.4.2 Triaxial test procedure

The Unconsolidated Undrained (UU) triaxial compression test is a quick and common method used to determine the undrained shear strength parameters (cu and ϕu) of cohesive soils. It's particularly relevant for situations where loads are applied rapidly, and there's insufficient time for pore water pressure to dissipate or for consolidation to occur.

I. Apparatus Required:

a. **Triaxial Load Frame:** Capable of applying a constant rate of axial deformation.

b. **Triaxial Cell:** A transparent cell (usually Perspex or acrylic) to enclose the soil specimen, allowing for visual observation. It has connections for applying confining pressure and a low-friction piston for axial loading.

c. **Pressure System:** To apply and control confining pressure (cell pressure, $\sigma 3$). This typically involves an air/water interface, pressure regulators, and sometimes automated pressure controllers.

d. **Proving Ring or Load Cell:** To measure the axial load applied to the specimen.

e. **Deformation Dial Gauge or LVDT:** To measure the vertical compression (axial strain) of the specimen.

f. **Rubber Membrane:** To encase the soil specimen and prevent water from entering/leaving the specimen during the test.

g. **O-rings:** To seal the rubber membrane to the top cap and bottom pedestal.

h. **Membrane Stretcher:** To aid in placing the rubber membrane over the specimen.

i. **Top Cap and Bottom Pedestal:** Used to hold the specimen within the triaxial cell. These should have the same diameter as the specimen.

j. **Specimen Preparation Equipment:**

i. Wire saw, trimming knife for undisturbed samples.

ii. Split mold, rammer, and cylindrical block for remolded samples.

iii. Measuring devices (calipers, weighing balance) for determining initial dimensions and weight.

k. **De-aired water:** For filling the triaxial cell.

II. Specimen Preparation:

1. **Obtain Representative Samples:** For UU tests, undisturbed soil samples are preferred as they represent in-situ conditions. If undisturbed samples are not feasible or show cracks, reconstituted samples can be prepared at their in-situ density and moisture content.
2. **Trim the Specimen:** Carefully trim the soil sample to the desired cylindrical shape (typically with a height-to-diameter ratio of 2:1, e.g., 38 mm diameter and 76 mm height). Ensure the ends are flat and perpendicular to the specimen's axis.
3. **Measure Initial Dimensions and Weight:** Accurately measure the initial diameter (at top, middle, and bottom) and height of the specimen. Determine its initial weight for calculating density and water content.
4. **Encase the Specimen:** Place the specimen on the bottom pedestal of the triaxial cell. Carefully stretch the rubber membrane over the specimen using a membrane stretcher, ensuring it's free of wrinkles and punctures. Seal the membrane to the bottom pedestal and the top cap using two O-rings at each end.

III. Test Procedure (for one confining pressure):

The UU test is essentially a two-stage test, but in practice, for saturated soils, there's no consolidation stage.

1. **Assemble the Triaxial Cell:**

a. Place the bottom pedestal with the prepared specimen onto the base of the triaxial cell.

b. Place the top cap on the specimen.

c. Lower the triaxial cell body over the specimen and secure it to the base plate, ensuring a watertight seal.

d. Carefully bring the axial load piston (ram) into contact with the top cap of the specimen, without applying significant deviator stress (axial load). This ensures proper seating and alignment.

2. **Fill the Triaxial Cell with Water:**

a. Open the bleed valve of the triaxial cell.

b. Fill the cell with de-aired water, ensuring no air bubbles are trapped inside. Once filled, close the bleed valve tightly.

3. **Apply Confining Pressure (σ3):**

a. Connect the cell pressure line to the triaxial cell.

b. Gradually apply the desired confining pressure (σ3) to the water in the triaxial cell. In a UU test, **drainage valves are kept closed throughout the entire test (both during confining pressure application and shearing)**. This ensures undrained conditions. For a truly saturated soil, the application of confining pressure in undrained conditions will not cause consolidation.

4. **Initiate Shearing (Axial Loading):**

a. Once the confining pressure is stable, begin applying axial load to the specimen using the loading frame. The axial load is applied at a constant rate of deformation (strain-controlled). The rate is typically chosen such that failure occurs within 10-20 minutes (e.g., 0.4 mm/min for fine-grained soils).

b. Crucially, during this shearing stage, the drainage valve remains closed, and pore water pressure is typically NOT measured in a standard UU test.

5. Record Data:

a. As the axial load is applied, record readings from the proving ring (or load cell) and the deformation dial gauge at regular intervals of axial deformation (e.g., every 0.5 mm or 1% axial strain).

b. Continue applying the load until the specimen fails (indicated by a peak in deviator stress, or significant bulging and no further increase in load, or a predetermined axial strain, often 20%).

IV. Performing Tests at Different Confining Pressures:

To obtain the undrained shear strength parameters (c_u and ϕ_u), you need to perform a series of at least three UU tests on identical soil specimens, each at a *different confining pressure*.

1. **Test 1:** Perform the procedure outlined above with an initial confining pressure (e.g., $\sigma_{3,1}$).
2. **Test 2:** Prepare a new, identical specimen. Repeat the procedure with a second, higher confining pressure (e.g., $\sigma_{3,2}$).
3. **Test 3:** Prepare another identical specimen. Repeat the procedure with several, higher confining pressure.

V. Data Analysis and Interpretation:

For each test:

1. **Calculate Axial Strain (ϵ_a):** $\epsilon_a = \frac{\Delta L}{L_0}$ 3.6

where:

a. ΔL = Change in length (vertical deformation)

b. $L0$= Initial length of the specimen

2. **Calculate Current Cross-sectional Area (Ac):** Since the volume of the specimen remains constant in undrained conditions (assuming full saturation and neglecting membrane penetration), the cross-sectional area changes as the specimen deforms. $Ac = Lc \quad V0 = L0 - \Delta LA0 \quad L0 = 1 - \epsilon aA0$

 3.7

where:

$V0$ = Initial volume of the specimen

Lc = Current length of the specimen ($L0-\Delta L$)

$A0$ = Initial cross-sectional area of the specimen

3. **Calculate Axial Stress ($\sigma 1$):** $\sigma 1=Ac$ Axial Load

4. **Calculate Deviator Stress ($\Delta\sigma$ or q):** $\Delta\sigma=\sigma 1-\sigma 3$

5. **Plot Stress-Strain Curves:** For each test, plot a graph of deviator stress ($\Delta\sigma$) (y-axis) versus axial strain (ϵa) (x-axis).

6. **Determine Peak Deviator Stress:** From each stress-strain curve, identify the peak deviator stress at failure ($\Delta\sigma f$). If no clear peak is observed (e.g., for highly plastic clays), use the deviator stress at a defined axial strain (e.g., 15% or 20%).

7. **Construct Mohr Circles:**

a. For each test, at failure, the major principal stress is $\sigma 1f=\sigma 3+\Delta\sigma f$, and the minor principal stress is $\sigma 3$.

b. Plot a Mohr circle for each test on a τ (shear stress) vs. σ (normal stress) diagram. Each circle will have a center at $(2\sigma 1f+\sigma 3,0)$ and a radius of $2\Delta\sigma f$.

8. **Determine Shear Strength Parameters (cu and ϕu):**

a. For saturated cohesive soils tested under UU conditions, the undrained shear strength (su or cu) is generally considered constant independent or regardless of the confining pressure. Therefore, the Mohr circles for all tests should ideally be of approximately the same diameter.

b. Draw a common tangent line to these Mohr circles. This line represents the failure envelope in terms of total stresses.

c. For saturated fine-grained soils, the failure envelope for UU tests is typically a horizontal line. The intercept of this line with the shear stress axis (τ) is the undrained cohesion (cu), and the undrained angle of internal friction (ϕu) is approximately 0.

CHAPTER FOUR

PRESENTATION AND DISCUSSION OF RESULTS

4.1 Atterberg limit Test Results

The summary of the liquid limit (LL), Plastic limit (PL) and Plasticity index (PI) results obtained from the atterberg limit test conducted in the laboratory are presented in Table 4.1 and the full laboratory results are in the Appendix.

TABLE 4.1: Atterberg Limit Test Results for the Sampling Points

S/N	SAMPLE ID	LL (%)	PL (%)	PI (%)
1	BH1	29.92	22.37	7.55
2	BH2	26.52	16.02	10.50
3	BH3	29.83	16.17	13.66
4	BH4	45.69	23.46	22.23
5	BH5	44.18	18.12	26.06

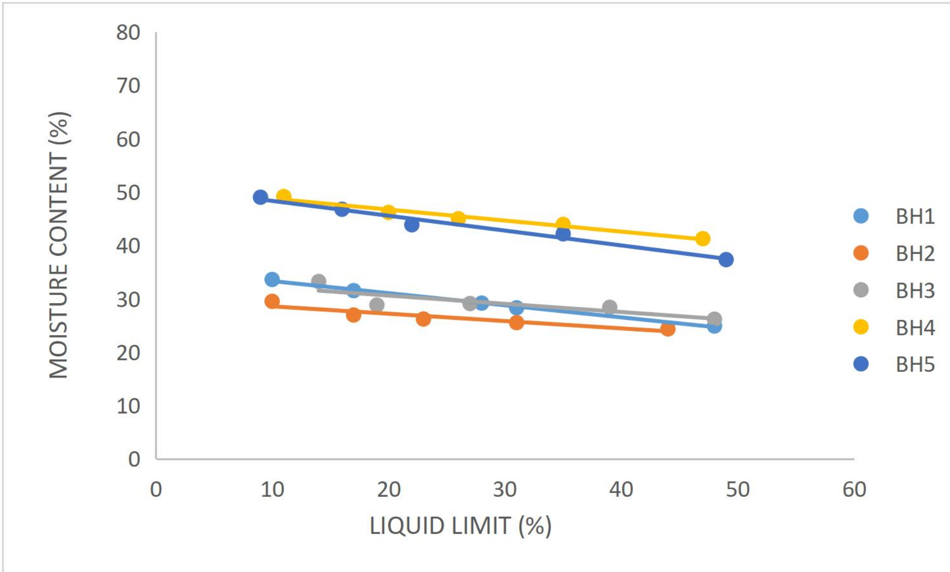

Figure 4.1 Graph of Liquid Limit for the Sampling Points

It can be observed from the sample identity number for BH1, BH2 and BH3 with range of LL=29.922, 26.52 and 29.83 respectively, indicates that the soil is a CL soil, since the PI for BH1, BH2 and BH3 cuts across the A-line, while for BH4 and BH5 the PI also cuts across the A-line which indicate that the soil is a CI soil.

4.2 Soil sieve Analysis Test

The selected sieve sizes based on AASHTO soil classification system are shown for the sample collected in table 4.2 and the full laboratory results are in the Appendix.

TABLE 4.2: Soil sieve Analysis Test Results for the Sampling Points

S/N	SAMPLE ID	2.00mm	0.425mm	0.075mmm
1	BH1	99.86	77.26	21.42
2	BH2	99.81	68.23	16.45
3	BH3	99.94	75.1	14.77
4	BH4	99.88	69.29	17.26
5	BH5	100	75.87	19.06

It can be seen from the sample identity BH1 that the soil is a sand with Fraction Percentage (%) Coarse Sand=20.00%, Medium Sand=40.00%, Fine Sand=20.00, while for BH2 the soil is a sand with (%) Coarse Sand=22.00%, Medium Sand=48.00%, Fine Sand=11.00,

while for BH3 the soil is a sand with (%) Coarse Sand=22.00%, Medium Sand=45.00%, Fine Sand=18.00, while for BH4 the soil is a sand with (%) Coarse Sand=28.00%, Medium Sand=42.00%, Fine Sand=12.00, while for BH5 the soil is a sand with (%) Coarse Sand=25.00%, Medium Sand=40.00%, Fine Sand=16.00.

4.3 General Soil Classification Based on AASHTO Classification System

The General soil classification based on AASHTO classification system can be gotten from the combination from the Atterberg limit Test Results and the selected sieve sizes based on AASHTO soil classification system are shown for the sample collected in table 4.2 and the full laboratory results are in the Appendix.

TABLE 4.3: General soil classification based on AASHTO classification system for the Sampling Points

S/N	SAMPLE ID	LL	PI	% PASSING 0.075mm	AASHTO CLASS
1	BH1	29.922	7.55	21.42	A-2-4
2	BH2	26.52	10.50	16.45	A-2-4
3	BH3	29.83	13.66	14.77	A-2-6
4	BH4	45.69	22.23	17.26	A-2-7
5	BH5	44.18	26.06	19.06	A-2-7

It can be observed from the soil sample identity BH1, the general soil classification can be gotten using both Plasticity chart in Figure 3.1 and AASHTO classification table in Figure 3.2.

Since less than 35% passes through the 0.075mm sieve the soil group is between A-1, A-3, and A-2, since LL=29.922, the group is between A-2-4 and A-2-6, since PI=7.55 the group classification is A-2-4, which is usually silty or clayey gravel and sand. Using the plasticity chart since PI is above the A-line the soil is CL.

It can be observed from the soil sample identity BH2, the general soil classification can be gotten using both Plasticity chart in Figure 3.1 and AASHTO classification table in Figure 3.2.

Since less than 35% passes through the 0.075mm sieve the soil group is between A-1, A-3, and A-2, since LL=26.52, the group is between A-2-4 and A-2-6, since PI=10.50 the group classification is A-2-4, which is usually silty or clayey gravel and sand. Using the plasticity chart since PI is above the A-line the soil is CL.

It can be observed from the soil sample identity BH3, the general soil classification can be gotten using both Plasticity chart in Figure 3.1 and AASHTO classification table in Figure 3.2.

Since less than 35% passes through the 0.075mm sieve the soil group is between A-1, A-3, and A-2, since LL=29.83, the group is between A-2-4 and A-2-6, since PI=13.66 the group classification is A-2-6, which is usually silty or clayey gravel and sand. Using the plasticity chart since PI is above the A-line the soil is CL.

It can be observed from the soil sample identity BH4, the general soil classification can be gotten using both Plasticity chart in Figure 3.1 and AASHTO classification table in Figure 3.2.

Since less than 35% passes through the 0.075mm sieve the soil group is between A-1, A-3, and A-2, since LL=45.69, the group is between A-2-5 and A-2-7, since PI=22.23 the group classification is A-2-7, which is usually silty or clayey gravel and sand. Using the plasticity chart since PI is above the A-line the soil is CI.

It can be observed from the soil sample identity BH5, the general soil classification can be gotten using both Plasticity chart in Figure 3.1 and AASHTO classification table in Figure 3.2.

Since less than 35% passes through the 0.075mm sieve the soil group is between A-1, A-3, and A-2, since LL=44.18, the group is between A-2-5 and A-2-7, since PI=26.06 the group classification is A-2-7, which is usually silty or clayey gravel and sand. Using the plasticity chart since PI is above the A-line the soil is CI.

4.4 Shear Strength Results from the Varied Cell Pressures

The summary of the variation of the standard cell pressure and extended cell pressure obtained from the unconcolidated undrained (UU) Triaxial test results are shown in **TABLE 4.4** and the full laboratory sheets are represented in the Appendix.

TABLE 4.4: Variation of standard cell pressure and extended cell pressure for the Sampling Points

S/N	BOREHOLE SAMPLE NUMER	CELL PRESSURE VARIATION (kN/m²)	C(kN/m²)	(Φº)
1	BH 1	103,207,310	15.00	0.57
		69,103,138,172,207,241,276, 310,345	2.00	2.43
2	BH 2	103,207,310	13.00	4.15
		69,103,138,172,207,241,276, 310,345	5.00	3.89
3	BH 3	103,207,310	12.00	4.29
		69,103,138,172,207,241,276, 310,345	6.40	4.41
4	BH 4	103,207,310	28.00	1.43
		69,103,138,172,207,241,276, 310,345	8.00	5.71
5	BH 5	103,207,310	19.00	1.00
		69,103,138,172,207,241,276, 310,345	12.00	2.63

Considering the obtained data from the UU triaxial test conduted in the laboratory that are presents in Table 4.4, there is a compelling comparison between the standard and extended cell pressure testing across the five distinct borehole samples. By examining the cohesion (C) and the internal angle of friction (Φ∘), we can observe distinct behavioral shifts when the testing resolution is expanded from three standard cell pressures to nine extended cell pressures. In a standard triaxial test, using only three cell pressures (103, 207, and 310 kN/m2) provides a high-level, generalized view of soil behavior. However, introducing the extended range stretching from 69 kN/m^2 to 345 kN/m^2 captures a much more complex look at the soil matrix under varying confinement conditions. Looking closely at the results, an interesting trend emerges across almost all the boreholes: while expanding the cell pressures, it generally leads to a notable reduction in the measured cohesion (C) alongside an increase in the internal angle of friction (Φ◦). For instance, in BH 1, the standard test yields a cohesion of 15.0 kN/m2 and a very low friction angle of 0.57◦. Once the extended range is applied, the cohesion drops sharply to 2.0 kN/m2, while the friction angle increases to 2.43◦. This same pattern repeats in BH 2, 3, 4, and 5. In BH 4, the standard cohesion is quite high at 28 kN/m2 with a friction angle of 1.43°, but the extended test redistributes these values to 8 kN/m2 and 5.71°, respectively. This consistent shift happens because the standard, limited data points can artificially skew the Mohr-Coulomb failure envelope. When you only plot three points at higher pressures, the linear fit might overestimate the intercept on the shear stress axis (cohesion) while underestimating the slope (friction angle).

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 CONCLUSION

From the analysis carried out shows that the Triaxial Soil Test (Ekosodin Site) sample 1, this soil sample has a very low internal friction angle, indicating that the soil particles do not rely much on frictional resistance for shear strength. Instead the cohesion value is relatively high and become the dominant contributor to the soil's shear strength. Triaxial Soil Test (Ekosodin Site) sample 2 this soil sample shows low internal friction and moderately high cohesion. The shear strength is primarily governed by cohesion, with minimal contribution from internal friction Triaxial Soil Test (Ekosodin Site) sample 3 this soil sample exhibits low internal friction and moderate cohesion which implies that its shear strength is largely controlled by cohesion and it provides the primary shear strength.

Triaxial Soil Test (Ekosodin Site) sample 4 this soil sample has a low angle of internal friction indicating very limited frictional resistance between soil particles. The cohesion is relatively high meaning the entire shear strength of the soil depends heavily on cohesion.

Triaxial Soil Test (Ekosodin Site) sample 5, this soil sample shows the angle of internal friction is extremely low which implies that the soil has a minimal frictional resistance between particles, this indicates poor granular interlock, typical of fine grained soils like soft clays or silts and the cohesion is moderately high showing that the shear strength relies heavily on cohesion, not particle friction. Triaxial Soil Test at (Ekosodin Site) sample 1, with Nine Cell Pressure

Shows that the sample angle of internal friction is low indicating a minimal frictional resistance between soil particles. The cohesion value indicate is also very low indicate that the soil has very little cohesion strength, which implies that the soil is very weak with limited resistance to shear stress from both cohesion and internal friction. Triaxial Soil Test at (Ekosodin Site) sample 2, with Nine Cell Pressure Shows that the sample angle of internal friction is very low, indicating the soil has minimal resistance due to inter-particle friction. The cohesion value is also very low, which shows that it has limited shear strength Triaxial Soil Test at (Ekosodin Site) sample 3, with Nine Cell Pressure shows that the sample angle of internal friction is very low which provides minimal resistance to shearing from particle friction and the cohesion value is moderate which gives some strength from clay bonding or slight cementation, but is not enough for the application high load bond .Triaxial Soil Test at (Ekosodin Site) sample 4, with Nine Cell Pressure shows that the sample angle of internal friction is very low, indicating that the soil has limited resistance to shear due to particle interlock and the cohesion value a moderate level, showing that the soil has some bonding strength. Triaxial Soil Test at (Ekosodin Site) sample 5, with Nine Cell Pressure shows that the sample angle of internal resistance is very low, indicating a minimal grain frictional resistance and the cohesion values is moderately high which suggests the soil's strength is largely due to inter-particle.

Comparing the angle of internal friction and cohesion of (BH1) sample 1 using the Triaxial test for the three cell pressure and nine cell pressure which are 0.57 º and 2.43 º respectively show that both angles of internal friction are closer to zero, but the Triaxial test with three cell pressure with angle of internal friction 0.57º is negligible, offering no particle frictional resistance to sliding. While the cohesion of 13kN/m^2 in the three cell pressure is dominated by a small cohesive bond whereas the cohesion in the nine cell pressure which is 2kN/m^2 in sample 1 is almost purely frictionless and non-cohesive. It also shows that the internal angle of friction using the three cell pressure of 0.57º increase to 2.43º

In the nine cell pressure, while the cohesion in the three cell pressure of 15kN/m^2 reduced to 2kN/m^2 in the nine cell pressure.

Comparing the angle of internal friction and cohesion of (BH2) sample 2 using the Triaxial test for the three cell pressure and nine cell pressure which are 4.15º and 3.89º respectively shows that both are closer to zero. Negligible strength gain with depth or load. While the cohesion value in three cell pressure of 13kN/m^2 has a measurable initial strength, while the cohesion value of 5kN/m^2 is almost frictionless and non-cohesive. And it can be seen from the result from the three cell pressure, the angle of internal friction and cohesion is higher than the angle of internal friction and cohesion in the nine cell pressure.it also shows that the internal friction in the three cell pressure of 4.15 º reduced to 3.89º.while the cohesion in the three cell pressure of 13kN/m^2 reduced to 5kN/m^2 in the nine cell pressure.

Comparing the angle of internal friction and cohesion of (BH3) sample 3 using the Triaxial test for the three cell pressure and nine cell pressure which are 4.29 º and 4.41 º respectively show that both angles of internal friction are practically the same and close to zero, offering negligible resistance under load, while the cohesion value of 12kN/m^2 for the three cell pressure is significantly higher than the cohesion value of 6.4kN/m^2 in the nine cell pressure.

From the test it also show that angle of internal friction in the three cell pressure of 4.29 increase to 4.41 in the nine cell pressure and the cohesion value of 12kN/m^2 in the in the three cell pressure decrease to 6.4kN/m^2.

Comparing the angle of internal friction and cohesion of (BH4) sample 4 using the Triaxial test for the three cell pressure and nine cell pressure which are 1.43º and 5.71º respectively show that both angles of internal friction are closer to zero, meaning negligible strength gain with load or depth, and the angle of internal friction in the nine cell pressure of 5.71º is slightly higher but still weak, frictional component, while the cohesion value of $28kN/m^2$ for three cell pressure is significantly higher, while the cohesion value of $8kN/m^2$ for the nine cell pressure is less, having little strength.

From the test it also show that angle of internal friction in the three cell pressure of 1.43 increase to 5.71 in the nine cell pressure and the cohesion value of $28kN/m^2$ in the in the three cell pressure decrease to $8kN/m^2$.

Comparing the angle of internal friction and cohesion of (BH5) sample 5 using the Triaxial test for the three cell pressure and nine cell pressure which are 1º and 2º respectively show that both angles of internal friction are critically low, but values are closer to zero meaning the offer no frictional strength, but the cohesion value of $19kN/m^2$ in the three cell pressure and the cohesion value of $12kN/m^2$ in the nine cell pressure are both low, but the cohesion value of $19kN/m^2$ in the three cell pressure is significantly higher, providing a measurable strength component.

From the test it also show that angle of internal friction in the three cell pressure of 1º increase to 2º in the nine cell pressure and the cohesion value of $19kN/m^2$ in the in the three cell pressure decrease to $12kN/m^2$.

It can be seen from the results gotten using the three cell pressure from BH1, BH2, BH3, BH4 and BH5 which are $15kN/m^2$, $13kN/m^2$, $12kN/m^2$, $28kN/m^2$ and $19kN/m^2$ respectively, they all have a higher cohesion value than using the nine cell pressure with cohesion values $2kN/m^2$, $5kN/m^2$, $6.4kN/m^2$, $8kN/m^2$ and $12kN/m^2$ from BH1, BH2, BH3, BH4 and BH5 as the cohesion provides the vital short term stability, due to angle of internal friction closer to zero, Therefore using the standard Geotechnical Engineering cell pressure of $100kN/m^2$, $205kN/m^2$, $310kN/m^2$ is better than using extended cell pressures of $69kN/m^2$, $103kN/m^2$, $138kN/m^2$, $172kN/m^2$, $207kN/m^2$, $241kN/m^2$, $276kN/m^2$, $310kN/m^2$, $345kN/m^2$ to determine the soil shear strength.

RECOMMENDATIONS

1. Adopt Standard High-Pressure Bracketing for Final Engineering Designs

Based on the conclusion, standard geotechnical cell pressures ($100kN/m^2$, $205kN/m^2$, and $310kN/m^2$) should remain the baseline for determining official design parameters in this region. The extended low-pressure intervals artificially compress the soil fabric step-by-step, inducing premature shearing and yielding uncharacteristically low, over-conservative cohesion values (e.g., $2kN/m^2$) that do not reflect true peak in-situ performance under standard load configurations.

2. Mandate Unconsolidated Undrained (UU) Testing for Immediate Stability Analysis

Because the friction angle (ϕ) is practically zero ($<6^\circ$ across all boreholes), the soil behaves as a purely cohesive medium ($\phi=0$ concept). Laboratory testing protocols for structural works in Ekosodin must strictly rely on rapid Unconsolidated Undrained (UU) triaxial tests to capture the critical short-term undrained shear strength ($Su=c$), as the soil offers negligible long-term frictional resistance under load.

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3. Prohibit the Use of Shallow Foundations for High-Load Structures

With friction angles nearing zero and cohesion values dropping drastically under progressive confinement, the ultimate bearing capacity (qu) for shallow foundations will be critically low. Shallow strip or pad footings must be strictly prohibited for multi-story or high-load engineering projects in the Ekosodin area; they will face immediate shear failure or catastrophic differential settlement.

4. Implement Deep Foundation Systems (Piling) into Stable Strata

For heavy structural loads, foundations must bypass these weak, low-friction, moisture-sensitive upper layers entirely. Design recommendations should specify deep pile foundations (end-bearing piles) driven past these soft clay/silt zones down to the deeper, more competent granular layers of the Benin Formation where true frictional resistance can be established.

5. Utilize Raft (Mat) Foundations for Lightweight/Medium Structures

Where deep piling is economically unfeasible for low-to-medium-weight residential buildings, a rigid **raft foundation** must be enforced. Because the soil's strength relies entirely on weak, volatile cohesive bonds, a mat foundation is necessary to distribute structural loads uniformly over the entire building footprint, minimizing the risk of localized punching shear failure.2) as a worst-case scenario, mandating immediate bio-engineering (vegetation pinning) and concrete dynamic terracing to prevent massive landslides.

6. Compulsory Chemical Stabilization to Artificially Introduce Frictional Interlock

To safely develop infrastructure on this soil, it must undergo chemical modification. It is recommended to stabilize the subgrade using cement or lime injection. Chemical additives will induce flocculation and cementation, artificially creating artificial granular interlock to increase the internal friction angle well above the critically weak $0.57^\circ - 5.71^\circ$range.

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Type of Test	**LL**		**LL**		**LL**		**LL**		**LL**	
No. of Blows/shrinkage %	48.00		31.00		28.00		17.00		10.00	
Container No.	EN		VS		GJ		MZ		IZ	
Wt of wet soil & container (g)	35.30		35.80		46.89		44.40		48.98	
Wt of dried soil & container (g)	31.65		31.65		40.53		37.76		41.22	
Wt of container (g)	16.99		16.99		18.74		16.70		18.14	
Wt of dry soil (Wd) (g)	14.66		14.66		21.79		21.06		23.08	
Wt of moisture (Wm) (g)	3.65		4.15		6.36		6.64		7.76	
Moisture contain 100 (Wm/Wd)	24.90		28.31		29.19		31.53		33.62	
Type of Test		**PL**		**PL**		**PL**		**PL**		**PL**
No. of Blows/shrinkage %										
Container No.		ACP		JA		RE				
Wt of wet soil & container (g)		19.17		22.43		20.60				
Wt of dried soil & container (g)		18.52		21.61		18.74				
Wt of container (g)		13.63		16.86		13.65				
Wt of dry soil (Wd) (g)		4.89		4.75		5.09				
Wt of moisture (Wm) (g)		0.65		0.82		1.86				
Moisture contain 100 (Wm/Wd)		13.29		17.26		36.54				

APPENDIX

Table A-1 LABORATORY REPORT OF LIQUID LIMITS AND PLASTIC LIMITS TEST BH1

Table A-2 LABORATORY REPORT OF LIQUID LIMITS AND PLASTIC LIMITS

Type of Test	**LL**		**LL**		**LL**		**LL**		**LL**	
No. of Blows/shrinkage %	44.00		31.00		23.00		17.00		10.00	
Container No.	VP		JA		IZ		PAY		TA	
Wt of wet soil & container (g)	45.90		38.66		48.55		48.56		50.93	
Wt of dried soil & container (g)	40.22		34.33		42.10		41.95		43.27	
Wt of container (g)	16.90		17.37		17.49		17.43		17.33	
Wt of dry soil (Wd) (g)	23.32		16.96		24.61		24.52		25.94	
Wt of moisture (Wm) (g)	5.68		4.33		6.45		6.61		7.66	
Moisture contain 100 (Wm/Wd)	24.36		25.53		26.21		26.96		29.53	
Type of Test		**PL**		**PL**		**PL**		**PL**		**PL**
No. of Blows/shrinkage %										
Container No.		JA		VI		NIC				
Wt of wet soil & container (g)		25.15		22.99		19.94				
Wt of dried soil & container (g)		24.04		22.17		19.09				
Wt of container (g)		17.18		17.00		13.78				
Wt of dry soil (Wd) (g)		6.86		5.17		5.31				
Wt of moisture (Wm) (g)		1.11		0.82		0.85				
Moisture contain 100 (Wm/Wd)		16.18		15.86		16.01				

TEST BH2

Table A-3 LABORATORY REPORT OF LIQUID LIMITS AND PLASTIC LIMITS TEST BH3

Type of Test		**LL**		**LL**		**LL**		**LL**		**LL**	
No. of Blows/shrinkage %		48.00		39.00		27.00		19.00		14.00	
Container No.		B3		HOH		PLU		RTI		EC	
Wt of wet soil & container (g)		48.33		40.80		46.50		46.68		40.96	
Wt of dried soil & container (g)		41.83		35.74		39.97		40.11		35.20	
Wt of container (g)		17.00		17.93		17.54		17.32		17.87	
Wt of dry soil (Wd) (g)		24.83		17.81		22.43		22.79		17.33	
Wt of moisture (Wm) (g)		6.50		5.06		6.53		6.57		5.76	
Moisture contain 100 (Wm/Wd)		26.18		28.41		29.11		28.83		33.24	
Type of Test			**PL**		**PL**		**PL**		**PL**		**PL**
No. of Blows/shrinkage %											
Container No.			PRI		AV		RT				
Wt of wet soil & container (g)			30.77		24.95		26.46				
Wt of dried soil & container (g)			29.11		23.65		25.39				
Wt of container (g)			17.48		17.41		17.40				
Wt of dry soil (Wd) (g)			11.63		6.24		7.99				
Wt of moisture (Wm) (g)			1.66		1.30		1.07				
Moisture contain 100 (Wm/Wd)			14.29		20.83		13.39				

Table A-4 LABORATORY REPORT OF LIQUID LIMITS AND PLASTIC LIMITS TEST BH4

Type of Test	**LL**		**LL**		**LL**		**LL**		**LL**	
No. of Blows/shrinkage %	47.00		35.00		26.00		20.00		11.00	
Container No.	PA		OZ		OZ		RT		VIR	
Wt of wet soil & container (g)	43.60		41.21		44.26		40.32		40.00	
Wt of dried soil & container (g)	35.85		33.96		35.93		33.11		32.39	
Wt of container (g)	17.07		17.46		17.42		17.50		16.91	
Wt of dry soil (Wd) (g)	18.78		16.50		18.51		15.61		15.48	
Wt of moisture (Wm) (g)	7.75		7.25		8.33		7.21		7.61	
Moisture contain 100 (Wm/Wd)	41.27		43.94		45.00		46.19		49.16	
Type of Test		**PL**		**PL**		**PL**		**PL**		**PL**
No. of Blows/shrinkage %										
Container No.		B		BAT		TA				
Wt of wet soil & container (g)		24.27		24.79		21.69				
Wt of dried soil & container (g)		23.17		23.23		20.83				
Wt of container (g)		18.45		17.01		16.92				
Wt of dry soil (Wd) (g)		4.72		6.22		3.91				
Wt of moisture (Wm) (g)		1.10		1.56		0.86				
Moisture contain 100 (Wm/Wd)		23.31		25.08		21.99				

Table A-5 LABORATORY REPORT OF LIQUID LIMITS AND PLASTIC LIMITS TEST BH5

Type of Test	**LL**		**LL**		**LL**		**LL**		**LL**	
No. of Blows/shrinkage %	49.00		35.00		22.00		16.00		9.00	
Container No.	VIO		ME		V7		STO		RT	
Wt of wet soil & container (g)	38.25		43.50		50.09		50.60		41.14	
Wt of dried soil & container (g)	32.38		35.82		40.00		40.00		33.24	
Wt of container (g)	16.66		17.61		17.00		17.34		17.13	
Wt of dry soil (Wd) (g)	15.72		18.21		23.00		22.66		16.11	
Wt of moisture (Wm) (g)	5.87		7.68		10.09		10.60		7.90	
Moisture contain 100 (Wm/Wd)	37.34		42.17		43.87		46.78		49.04	
Type of Test		**PL**		**PL**		**PL**		**PL**		**PL**
No. of Blows/shrinkage %										
Container No.		PLU		PA		RT				
Wt of wet soil & container (g)		24.72		23.93		23.84				
Wt of dried soil & container (g)		23.58		22.95		22.84				
Wt of container (g)		17.59		17.20		17.37				
Wt of dry soil (Wd) (g)		5.99		5.75		5.47				

Wt of moisture (Wm) (g)		1.14		0.98		1.00				
Moisture contain 100 (Wm/Wd)		19.03		17.04		18.28				

Table A-6 LABORATORY REPORT OF SIEVE ANALYSIS TEST BH1

SIEVE NO.				
APPROX IMPERIAL EQUIV (inches)	BRITISH STANDARD SIEVE SIZES (mm)	RETAINED IN gm	PASSING IN gm	PASSING IN (%)
3	75			
2 ½				
2	50			
1 ½	37.5			
1	26.5			
¾	20			
½	14			
⅜	10			
¼	6.3			
3/16	5			
⅛	3.35		100	
7	2.36	0.06	99.94	99.94
10	2	0.08	99.86	99.86
14	1.18	1.92	97.94	97.94
25	0.6	17.26	80.68	80.68
36	0.425	3.42	77.26	77.26
52	0.3	11.88	65.38	65.38
72	0.212	26.25	39.13	39.13
100	0.15	6.58	32.55	32.55
200	0.075	11.13	21.42	21.42

Table A-7 LABORATORY REPORT OF SIEVE ANALYSIS TEST BH2

SIEVE NO.				
APPROX IMPERIAL EQUIV (inches)	BRITISH STANDARD SIEVE SIZES (mm)	RETAINED IN gm	PASSING IN gm	PASSING IN (%)
3	75			
2 ½				
2	50			
1 ½	37.5			
1	26.5			
¾	20			
½	14			
⅜	10			
¼	6.3			
3/16	5			
⅛	3.35		100	
7	2.36	0.19	99.81	99.81
10	2	0	99.81	99.81
14	1.18	2.19	97.62	97.62
25	0.6	20.87	76.75	76.75
36	0.425	8.52	68.23	68.23
52	0.3	10.24	57.99	57.99
72	0.212	26.73	31.26	31.26
100	0.15	4.62	26.64	26.64
200	0.075	10.19	16.45	16.45

Table A-8 LABORATORY REPORT OF SIEVE ANALYSIS TEST BH3

SIEVE NO.				
APPROX IMPERIAL EQUIV (inches)	BRITISH STANDARD SIEVE SIZES (mm)	RETAINED IN gm	PASSING IN gm	PASSING IN (%)
3	75			
2 ½				
2	50			
1 ½	37.5			
1	26.5			
¾	20			
½	14			
⅜	10			
¼	6.3			
3/16	5			
⅛	3.35		100	
7	2.36	0.06	99.94	99.94
10	2	0	99.94	99.94
14	1.18	1.67	98.27	98.27
25	0.6	17.36	80.91	80.91
36	0.425	5.81	75.1	75.1
52	0.3	13.17	61.93	61.93
72	0.212	27.69	34.24	34.24
100	0.15	6.86	27.38	27.38
200	0.075	12.61	14.77	14.77

Table A-9 LABORATORY REPORT OF SIEVE ANALYSIS TEST BH4

SIEVE NO.				
APPROX IMPERIAL EQUIV (inches)	BRITISH STANDARD SIEVE SIZES (mm)	RETAINED IN gm	PASSING IN gm	PASSING IN (%)
3	75			
2 ½				
2	50			
1 ½	37.5			
1	26.5			
¾	20			
½	14			
⅜	10			
¼	6.3			
3/16	5			
⅛	3.35		100	
7	2.36	0	100	100
10	2	0.12	99.88	99.88
14	1.18	2.5	97.38	97.38
25	0.6	21.5	75.88	75.88
36	0.425	6.59	69.29	69.29
52	0.3	12.43	56.86	56.86
72	0.212	24.36	32.5	32.5
100	0.15	6.04	26.46	26.46
200	0.075	9.2	17.26	17.26

Table A-10 LABORATORY REPORT OF SIEVE ANALYSIS TEST BH5

SIEVE NO.				
APPROX IMPERIAL EQUIV (inches)	BRITISH STANDARD SIEVE SIZES (mm)	RETAINED IN gm	PASSING IN gm	PASSING IN (%)
3	75			
2 ½				
2	50			
1 ½	37.5			
1	26.5			
¾	20			
½	14			
⅜	10			
¼	6.3			
3/16	5			
⅛	3.35		100	
7	2.36	0	100	100
10	2	0	100	100
14	1.18	1.72	98.28	98.28
25	0.6	17.15	81.13	81.13
36	0.425	5.26	75.87	75.87
52	0.3	12.2	63.67	63.67
72	0.212	27.78	35.89	35.89
100	0.15	6.52	29.37	29.37
200	0.075	10.31	19.06	19.06

Table A-11 LABORATORY REPORT OF TRIAXIAL COMPRESSION TEST BH1

Strain Dial	Stress Dial 1	SD2	SD3	SD4	SD5	SD6	SD7	SD8	SD9
0	0	0	0	0	0	0	0	0	0
15	2.5	1.8	3.9	2.5	7	4	8.4	3.1	8
30	2.7	2.5	4	3	7.2	4.1	8.9	4	10
45	2.7	2.6	4.1	3.1	7.3	4.2	9	4.5	10.7
60	2.7	2.7	4.1	3.3	7.3	4.5	9.1	4.9	11
75	2.7	2.8	3.9	3.4	7.3	4.7	9.1	5.1	7
90	2.7	2.8	3.9	3.6	7.3	4.8	9.1	5.5	7.5
105	2.7	2.9	3.9	3.7	7.3	5	9.1	6	8
120	2.7	2.9	3.9	3.8	7.3	5.1	9.2	6.1	8.1
135	2.7	2.9	3.9	3.9	7.3	5.1	9.2	6.7	8.5
150	2.7	2.9	3.9	3.9	7.3	5.2	9.2	7	9
165	2.7	2.9	3.9	4.0	7.3	5.3	9.2	7.1	9.1
180	2.7	2.9	3.9	4.1	7.3	5.4	9.5	7.3	9.7
210	2.7	3	3.9	4.2	7.3	5.7	9.7	8	10
240	2.7	3	3.9	4.2	7.3	5.9	9.8	8.4	10.6
270	2.7	3.1	3.9	4.2	7.3	6	9.8	8.7	11.2
300	2.7	3.1	3.9	4.2	7.3	6.1	9.8	8.8	11.9
330	2.7	3.1	3.9	4.2	7.3	6.4	9.8	8.8	12.2
360	2.7	3.1	3.9	4.7	7.3	6.7	9.8	8.8	12.9
390	2.7	3.2	3.9	4.8	7.3	7	9.8	8.8	13.8
420	2.7	3.2	3.9	4.9	7.3	7.1	9.8	8.8	14
450	2.7	3.5	3.9	4.9	7.3	7.2	9.8	9	14
480	2.7	3.5	3.9	5	7.3	7.2	9.8	9.1	14
510	2.7	3.5	3.9	5.1	7.3	7.4	9.8	9.4	14
540	2.7	3.5	3.9	5.2	7.3	7.8	9.8	9.7	14
570	2.7	3.5	3.9	5.3	7.3	8	9.8	10	14
600	2.7	3.5	3.9	5.4	7.3	8.1	9.8	10.2	14
630	2.7	3.5	3.9	5.5	7.3	8.2	9.8	10.5	14.1
660	2.7	3.5	3.9	5.6	7.3	8.3	9.8	10.7	14.1
690	2.7	3.5	3.9	5.9	7.3	8.5	9.8	10.9	14.2
720	2.7	3.5	3.9	6	7.3	8.7	9.8	10.9	14.7
750	2.7	3.5	3.9	6	7.3	9	9.8	10.9	15

Table A-12 LABORATORY REPORT OF TRIAXIAL COMPRESSION TEST BH2

Strain Dial	Stress Dial 1	SD2	SD3	SD4	SD5	SD6	SD7	SD8	SD9
0	0	0	0	0	0	0	0	0	0
15	2	1.9	2.2	9.5	4	10	14	14	20
30	2.3	2	2.8	9.6	5	10.6	14	17.3	25
45	2.6	2.1	3.1	9.6	5.5	11	14	21.2	31.4
60	2.9	2.2	3.2	9.6	6	11.1	14	22.2	32.5
75	2.9	2.3	3.2	9.6	6.1	11.2	14	23.2	34.7
90	2.9	2.5	3.5	9.6	6.2	11.3	14	24	34.9
105	2.9	2.6	3.8	9.6	6.3	11.4	14	24	36.2
120	2.9	2.7	3.9	9.6	6.7	11.8	14	25	36.7
135	2.9	2.8	4	9.6	7	12.2	14	25.5	36.8
150	2.9	3	4.1	9.6	7.1	12.3	14	26.1	37
165	2.9	3.1	4.2	9.6	7.4	12.5	14	27	37.3
180	2.9	3.2	4.3	9.6	7.8	12.6	14	27.2	37.5
210	2.9	3.4	4.8	9.6	8.3	12.9	14	28	37.9
240	2.9	3.4	5	9.6	8.7	12.9	14	29.1	38.5
270	2.9	3.5	5.1	9.6	9.1	13	14	30.1	39.1
300	2.9	3.5	5.2	9.6	9.7	13.3	14	30.5	39.5
330	2.9	3.5	5.4	9.6	10	13.5	14	31.1	39.6
360	2.9	3.5	5.6	9.6	11	13.9	14	32	39.9
390	2.9	3.5	6	9.6	11.3	14	14	33	40
420	2.9	3.5	6.1	9.6	11.5	14	14	33.5	40.1
450	2.9	3.5	5	9.6	12	14	14	34	40.6
480	2.9	3.5	6	9.6	12.1	14	14	34.5	41
510	2.9	3.5	6	9.6	13	14	14	35	41.1
540	2.9	3.5	6.4	9.6	13	14	14	35.5	41.1
570	2.9	3.5	6.8	9.6	13	14	14	36	41.3
600	2.9	3.5	7	9.6	13	14	14	36.7	41.5
630	2.9	3.5	7.4	9.6	13	14	14	37	41.9
660	2.9	3.5	7.5	9.6	13	14	14	37.4	41.9
690	2.9	3.5	7.5	9.6	13	14	14	37.9	42
720	2.9	3.5	7.6	9.6	13	14	14	38	42.2
750	2.9	3.5	8	9.6	13	14	14	38.9	42.5

Table A-13 LABORATORY REPORT OF TRIAXIAL COMPRESSION TEST BH3

Strain Dial	Stress Dial 1	SD2	SD3	SD4	SD5	SD6	SD7	SD8	SD9
0	0	0	0	0	0	0	0	0	0
15	7	3	5.2	4.8	4	9	10	9	12
30	8.5	3	5.8	5.6	4.6	10	12.2	10.8	14
45	9	4.5	6.2	6.5	5.2	10.5	14	12	15.2
60	9	4.9	6.8	7	5.5	10.8	15	13	16
75	9	5.1	7.2	7.6	6	10.8	16	13.5	16.5
90	9	5.3	7.5	8.2	6.5	11.2	16.5	14	16.8
105	9	5.4	7.8	8.8	6.5	11.5	16.8	14.8	17.5
120	9	5.7	8	9.2	6.5	11.8	17.2	15.2	17.8
135	9	5.9	8.5	9.4	6.5	12	17.5	16	18
150	9	6	8.8	9.8	6.8	12.3	17.8	16.8	18.2
165	9	6.4	9	10.2	7.6	12.8	18	17.5	18.5
180	9	6.5	9.5	10.6	8	13.2	18.5	17.8	18.8
210	9	7	10.2	11.1	9	13.6	19.5	19	19
240	9	7.2	11	11.5	10	14	20	20.5	19.8
270	9	8	11.5	12	10.8	14.5	20.5	21.5	22.5
300	9	8.1	12	12.2	11.2	15	21	22.5	23.8
330	9	8.1	12.2	12.6	11.5	15.2	21.2	23	24.5
360	9	8.3	12.5	12.8	11.8	15.5	21.5	23.2	25
390	9	8.7	12.6	12.8	11.8	16	21.5	23.5	25
420	9	8.8	12.6	12.8	12	16	21.5	23.5	25.2
450	9	8.9	12.6	12.8	12.1	16.3	21.5	23.5	25.2
480	9	9	12.6	12.8	12.1	16.8	21.5	23.5	25.2
510	9	9.2	12.6	12.8	12.2	16.8	21.5	23.5	25.2
540	9	9.4	12.6	12.8	12.4	16.8	21.5	23.5	25.2
570	9	9.5	12.6	12.8	12.8	16.8	21.5	23.5	25.2
600	9	10	12.6	12.8	13.2	16.8	21.5	23.5	25.2
630	9	10	12.6	12.8	13.5	16.8	21.5	23.5	25.2
660	9	10	12.6	12.8	13.8	16.8	21.5	23.5	25.2
690	9	10.1	12.6	12.8	14	16.8	21.5	23.5	25.2
720	9	10.5	12.6	12.8	14.4	16.8	21.5	23.5	25.2
750	9	10.7	12.6	12.8	14.8	16.8	21.5	23.5	25.2

Table A-14 LABORATORY REPORT OF TRIAXIAL COMPRESSION TEST BH4

Strain Dial	Stress Dial 1	SD2	SD3	SD4	SD5	SD6	SD7	SD8	SD9
0	0	0	0	0	0	0	0	0	0
15	0	0	0	0	0	0	0	0	0
30	3	9.2	13	4.4	14	9.1	6.6	17	16
45	4	10.1	15	4.5	15	10.3	7.4	19.7	18.3
60	5.1	11.1	15.5	6	15.8	11	8.1	20.9	19.4
75	6.8	12	16	6.7	16	12	8.9	21.2	19.9
90	7.1	12.5	16.3	7.3	16.5	12.2	9	21.5	20
105	8.5	13	16.7	8	16.8	13	10	21.9	20.1
120	8.7	13.1	17	8.6	17	13.4	10.1	22	20.6
135	8.9	13.1	17.6	9	17.9	14	11	22.5	21.2
150	9	13.1	18	9.6	18	14.9	11.2	22.9	21.8
165	9	13.1	18.1	10	18.1	15	12	23	22.2
180	9.0	13.3	18.2	10.6	18.2	15.2	12.5	23	22.9
210	9	13.5	18.3	11	18.4	16	13.3	23.5	23.5
240	9	13.9	18.9	12	18.5	16.5	14.8	24	24
270	9	14	19	13	18.6	17	15.9	24.8	25.1
300	9	14.1	19.3	14	19	17.9	16	25.1	26
330	9	14.6	19.7	15	19.3	18	18	25.5	26.5
360	9	14.6	20.3	16	19.8	18.8	18.8	26	27
390	9	14.6	20.4	16.8	20	19	18.9	26.3	27.5
420	9	14.6	20.6	17	20.1	19.7	19	26.3	28
450	9	14.6	20.8	17.9	20.5	20	19.4	26.3	28.7
480	9	14.6	21	18	20.1	20.1	19.9	26.3	29
510	9	14.6	21.1	18.6	20.4	20.6	20.1	26.3	29.1
540	9	14.6	21.1	19	20.5	21	21	26.3	29.5
570	9	14.6	21.1	19.3	20.5	21.3	21.3	26.3	29.6
600	9	14.6	21.1	20	20.6	21.8	21.9	26.3	29.8
630	9	14.6	21.1	20.1	20.9	22	22.1	26.3	30
660	9	14.6	21.1	20.4	21.1	22	22.3	26.3	30
690	9	14.6	21.1	20.8	21.5	22	22.5	26.3	30
720	9	14.6	21.1	21	21.6	22	22.7	26.3	30
750	9	14.6	21.1	21.3	21.7	22	22.8	26.3	30

Table A-15 LABORATORY REPORT OF TRIAXIAL COMPRESSION TEST BH5

Strain Dial	Stress Dial 1	SD2	SD3	SD4	SD5	SD6	SD7	SD8	SD9
0	0	0	0	0	0	0	0	0	0
15	3.7	3.5	11	10.5	5.9	10	8	7.2	6.9
30	4	4.4	12	11.3	7	10.9	9	8.8	7.8
45	4	5.3	11	12	7.6	11.1	9.9	9.2	8.3
60	4	6	11	12.6	8.6	11.5	10.4	10	9
75	4	6.1	11	12.8	9	11.8	10.9	10.5	9.1
90	4	6.2	11	12.8	9	12.2	11.2	11	9.2
105	4	6.6	11	12.8	9	12.5	11.5	11.5	9.9
120	4	7	11	12.8	9.4	12.9	12	12	10.1
135	4	7.5	11	12.8	9.8	13	12.2	12.1	10.5
150	4	8	11	12.8	10	13.1	12.5	12.7	10.9
165	4.1	8.3	11	12.8	10	13.5	12.7	13	11.2
180	4.2	8.8	11	12.8	10.1	13.7	13	13.4	11.9
210	4.2	9.6	11	12.8	10.3	13.8	13.5	13.8	12.7
240	4.3	10	11	12.8	10.5	14	13.7	14.2	13.2
270	4.6	10.2	11	12.8	10.7	14	14	14.7	14
300	4.7	10.2	11	12.8	11	14.1	14.2	14.8	14.5
330	4.9	10.2	11	12.8	11.1	14.1	14.5	15.2	15
360	5	10.2	11	12.8	11.3	14.3	15	15.3	15.7
390	5	10.2	11	12.8	11.6	14.3	15.2	16.1	16.3
420	5.1	10.2	11	12.8	12	14.8	15.2	16.5	17
450	5.2	10.2	11	12.8	12.2	14.9	15.2	17	17.3
480	5.8	10.2	11	12.8	12.5	15.1	15.2	17.5	18
510	6	10.2	11	12.8	12.5	15.1	15.2	17.5	18.6
540	6	10.2	11	12.8	12.9	15.1	15.2	17.5	18.6
570	6.1	10.2	11	12.8	12.9	15.1	15.2	17.5	18.6
600	6.2	10.2	11	12.8	13	15.1	15.2	17.5	18.6
630	6.2	10.2	11	12.8	13.3	15.1	15.2	17.5	18.6
660	6.2	10.2	11	12.8	13.6	15.1	15.2	17.5	18.6
690	6.5	10.2	11	12.8	14	15.1	15.2	17.5	18.6
720	6.7	10.2	11	12.8	14	15.1	15.2	17.5	18.6
750	7	10.2	11	12.8	14.2	15.1	15.2	17.5	18.6