EFFECT OF WATER TABLE VARIATIONS ON SOIL STRENGTH PARAMETERS.

BY

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PLAGIARISM

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DEDICATION

This project is dedicated to God Almighty, who has provided me with the knowledge and wisdom I need to excel in every aspect of my academic career and has graciously supported my life.

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ABSTRACT

This research investigates the geotechnical implications of water table fluctuations on the stability of near-surface soil strata by evaluating the relationship between fundamental soil index properties and measured shear strength parameters. Laboratory testing on soil samples recovered from 1.0 meter and 2.0 meter depths from warri, encompassing Specific Gravity (Gs), Compaction, Atterberg Limits, and Triaxial Compression tests.

The analysis revealed a critically poor soil profile characterized by low Specific Gravity (Gs < 2.54) and negligible cohesive strength (C as 0kPa}) across both strata, strongly indicating the presence of organic or highly compressible lightweight solids. The study's primary finding is the exceptionally low angle of internal friction (19.4 at 1.0 m and 16.7 at 2.0 m). This deficiency means that the soil's shear strength is entirely frictional and thus 100% dependent on effective stress.

The results demonstrate that water table rise poses an acute risk by causing a severe reduction in τ, leading to an immediate and significant loss of shear strength and bearing capacity, confirming the extreme moisture sensitivity of the subgrade. Consequently, the soil is classified as unsuitable for foundation support without extensive ground improvement. Mitigation recommendations include removal and replacement or deep foundations to bypass the weak, high-risk zone, providing essential data for informed foundation design and geotechnical risk assessment.

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CHAPTER ONE

INTRODUCTION

1.1 Background of the study

The soil water table is the upper surface of the saturated zone of the soil, below which all pores and fractures are filled with water. It represents the point at which soil pore water pressure equals atmospheric pressure. Above this level lies the unsaturated (vadose) zone, where the soil contains both air and water (Todd and Mays, 2005).

The water table is not static. It rises during rainfall or irrigation and falls during dry periods or excessive groundwater pumping. Water table fluctuations are governed by recharge (from precipitation or irrigation), discharge (evaporation or pumping), and lateral subsurface flows (Freeze and Cherry, 1979).

Several factors affect the water table level in soil: They include Rainfall which directly impacts infiltration and recharge, Fine-grained soils (like clay) which impede water flow, affecting recharge rates and water table depth (Hillel, 1998), Plants can also draw water through root uptake and transpiration, lowering the water table. In valleys, the water table may be closer to the surface, while in hills it is usually deeper. Also, groundwater abstraction and land development affects the local and regional water table (Shiklomanov, 1993).

In foundation design and basement construction, Shallow water tables may increase hydrostatic pressure and require waterproofing or dewatering techniques and influence excavation depth and safety, further compromising slope stability and necessitating robust groundwater control techniques during excavation" (Das, 2010).

Soil strength refers to the ability of soil to resist deformation and failure under applied loads. It governs the performance and safety of geotechnical structures such as foundations, slopes, embankments, and retaining walls. The strength of soil is controlled by both its internal frictional resistance and cohesive forces, which together determine the shear resistance. Soil strength is the maximum resistance of soil to deformation and failure, often described in terms of its shear strength" (Craig, 2004).

Below are factors which may affect soil strength;

i. **Soil Type**: Granular soils (e.g., sand) derive strength mostly from friction, while clays have both cohesion and friction.

ii. **Water Content**: Increased water content can reduce strength, particularly in fine-grained soils, due to elevated pore water pressures.

iii. **Density and Compaction**: Denser soils generally exhibit higher shear strength.

iv. **Stress History**: Over-consolidated soils often display greater shear strength than normally consolidated soils.

v. **Soil Structure and Fabric**: Natural bonding or cementation can significantly increase strength.

Moisture variations and stress history are critical determinants of soil strength, especially in fine-grained soils (Terzaghi et al., 1996). Soil strength is a complex but vital property in geotechnical engineering. It depends on multiple factors including soil type, stress condition, water content, and structure. Understanding and accurately determining soil strength parameters ensures the safety and stability of engineering projects.

1.2 Statement of the problem

Changing water table levels can significantly affect soil strength due to alterations in pore water pressure, effective stress, and soil saturation. These variations pose critical challenges in geotechnical engineering, especially for slope stability, foundation performance, and earthworks. When the water table rises, pore water pressure increases, reducing effective stress and thereby decreasing the shear strength of the soil. In fine-grained soils like clay, increasing water content due to a higher water table can lead to softening of the soil, lowering its undrained shear strength (Reddy & Manjunatha, 2020; Mohamed & Farouk, 2023). This is especially problematic in embankments and excavations. Fluctuating water levels may result in long-term consolidation or collapse settlements, endangering foundation integrity. Also, over time, it can promote internal erosion or piping, particularly in silty soils. The movement of water through soil pores dislodges particles, weakening the soil matrix. Geotechnical designs must consider these effects to ensure long-term performance and safety of engineering works.

1.4 Aim and Objectives

To investigate and understand the influence of changes in the soil water table on the shear strength and bearing capacity of laterite and clay soil , with the goal of enhancing the safety, stability, and performance of geotechnical structures in the selected study area.

The objectives are;

i. To evaluate the relationship between water table fluctuations and soil strength parameters.

ii. To assess the effects of water table variation on the shear strength and bearing capacity of soil samples.

iii. To suggest mitigating techniques in case of poor soil strength parameters.

iv. To provide useful information from experimental findings to guide engineers during site characterization, foundation design and risk assessments.

1.3 Scope of the study

This experimental and analytical study aims at investigating the effects of changing water table level on soil shear strength and bearing capacity in both laterite and clay soil types. The investigation would combine results from field monitoring and laboratory testing to support safe and efficient geotechnical design in moisture-vulnerable environments. Steps in carrying out this study would involve;

i. Soil types to be investigated which are sensitive to moisture changes.

ii. Geotechnical investigations would involve determining the, grain size distribution, shear strength and bearing capacity.

iii. Shallow Water Table Conditions (e .g -100mm, -.200mm, and -300mm from the surface) would be considered. Seasonal water table fluctuations, artificially altered water table due to irrigation or drainage systems, the impact of both short-term (undrained) and long-term (drained) water table changes would be neglected in thi stsudy in the absence of numerical modelling.

iv. Parameters to be investigated are the shear strengths and allowable bearing capacity of the soil samples.

v. Laboratory tests to be performed are Triaxial Compression Tests (UU, CU, CD) and Consolidation test.

vi. Output from analysis would involve assessing the relationships between water table levels and soil strength/bearing capacity in the study area.

1.5 Justification of the study

In geotechnical engineering, the strength of soil and its bearing capacity are essential parameters that determine the ability of the ground to support structures safely. In waterlogged areas, where the soil is frequently or persistently saturated, the behavior of the soil changes significantly due to elevated pore water pressures, resulting in reduced effective stress (Das & Sobhan, 2022).

A high or fluctuating water table reduces the shear strength of soil by increasing pore water pressure and decreasing the intergranular stress, leading to; Soil softening, particularly in clays and silts, loss of cohesion and frictional resistance, bearing capacity failure due to insufficient resistance to vertical loads (Al-Saadi et al., 2021). Assessing the effects of varying soil water table levels on soil strength and bearing capacity in waterlogged areas is justified by the need to; Ensure structural safety, Prevent geotechnical failures, Enable cost-effective and resilient design, Respond to climate and environmental pressures, Support sustainable urban and rural development. This assessment forms a critical foundation for risk-informed design, construction planning, and infrastructure sustainability in water-sensitive environments.

CHAPTER TWO

LITERATURE REVIEW

2.1 Soil water table

The soil water table—also known as the groundwater table—is the upper surface of the phreatic (saturated) zone in the subsurface, at which the pore-water pressure equals atmospheric pressure. Above this, in the vadose (unsaturated) zone, pores contain both air and water; below it, all voids are completely water-filled. The water table thus separates two fundamentally different hydrologic regimes and controls the distribution of soil moisture and groundwater flow (Todd & Mays, 2005; Fetter, 2001).

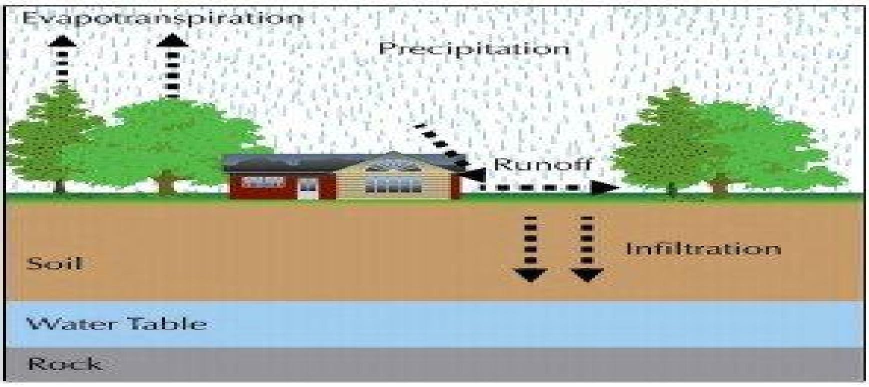


Fig 2.1 Soil water table (Benshof, 2014)

2.1.1 Vadose Zone and Capillary Fringe

Immediately above the water table lies the capillary fringe, where water is drawn upward by capillary forces into the pores against gravity. The thickness of this fringe is inversely related to pore size: in fine-textured soils (clays, silts) capillary rise can extend several meters, whereas in coarse sands it is often limited to a few centimeters (Hillel, 1998). Above the capillary fringe, matric suction decreases with elevation, defining the broader vadose zone where soil moisture varies continuously (Freeze & Cherry, 1979).

2.2 Fluctuations of Soil Water Table

The depth of the water table is highly dynamic, responding to:

i. Recharge: Infiltration of rainfall and snowmelt raises the table.

ii. Discharge: Evapotranspiration, baseflow to streams, and pumping lower it.

iii. Seasonality: Wet seasons typically produce high stands, while dry seasons induce drawdown (Todd & Mays, 2005).

iv. Anthropogenic Influences: Irrigation, drainage, and artificial recharge (e.g., injection wells) can cause rapid, localized shifts (Fetter, 2001).

2.3 Factors Influencing Water Table Position

i. Climate and Precipitation Patterns

   Intense or prolonged rainfall events increase recharge, elevating the water table, whereas drought conditions deplete it (Todd & Mays, 2005).

ii. Vegetation and Land Use

   Deep-rooted plants lower the water table through transpiration, while impermeable surfaces (e.g., urbanization) reduce recharge (Hillel, 1998).

iii. Topography and Subsurface Flow

   Topographic lows concentrate groundwater, raising the water table, while upland areas often exhibit deeper tables (Freeze & Cherry, 1979).

iv. Geologic and Soil Properties

   Heterogeneities in permeability and porosity control both the rate of vertical recharge and lateral groundwater movement (Fetter, 2001).

v. Human Activities, Groundwater extraction for agriculture or municipal supply causes drawdown cones, whereas managed aquifer recharge raises local water tables (Todd & Mays, 2005).

2.4 Effective Stress and Soil Mechanics effects on water table

The concept of effective stress) underlies soil behavior. A rise in the water table increases effective stress σ′\sigma'σ′ and influencing shear strength, compressibility, and stability. Conversely, lowering the water table can induce negative pore pressures (suction) that temporarily enhance unsaturated soil strength but may also lead to settlement upon rewetting (Freeze & Cherry, 1979; Hillel, 1998).

2.5 Measurement and Monitoring of water table

i. Observation Wells: Perforated pipes allow groundwater to equilibrate to the water table, which can be read manually or with automated loggers (Fetter, 2001).

ii. Piezometers: Measure pressure head at specific depths, providing continuous hydraulic head profiles (Todd & Mays, 2005).

iii. Pressure Transducers: Submersible sensors record water-level time series at high frequency, essential for detecting rapid fluctuations (Fetter, 2001).

iv. Geophysical Surveys: Electrical resistivity and ground-penetrating radar map saturated zones over large areas by exploiting contrasts in electrical properties .(Freeze & Cherry, 1979).

2.6 Environmental and Engineering Implications of water table variation

Groundwater Flow and Resource Management

Water-table configuration controls recharge-discharge balances, aquifer yields, and baseflow to rivers, informing sustainable withdrawals and ecosystem protection (Todd & Mays, 2005).

Agricultural Productivity

Shallow water tables can cause waterlogging and salt accumulation in the root zone, while deep tables may necessitate extensive irrigation (Hillel, 1998).

Geotechnical Stability

High water tables reduce effective stress, leading to reduced shear strength, slope failures, and bearing capacity loss for foundations (Freeze & Cherry, 1979).

Construction and Dewatering

Excavations below the water table require dewatering systems (wellpoints, sumps) to control pore pressures and prevent base heave or collapse (Fetter, 2001).

2.7 Management and Control Strategies

i. Subsurface Drainage: Tile drains, French drains, and geo-composites lower local water tables to mitigate waterlogging (Hillel, 1998).

ii. Artificial Recharge: Infiltration basins and recharge wells raise water tables to replenish aquifers (Todd & Mays, 2005).

iii. Land-Use Planning: Zoning and green infrastructure can optimize recharge and protect sensitive aquifer areas.

iv. Vegetation Management: Strategic planting of phreatophytes controls shallow groundwater through transpiration.

2.8 Shear strength of soil

Soil shear strength is a measure of the soil's ability to resist shear stress. It is one of the most critical parameters in geotechnical engineering, as it determines the stability of foundations, slopes, retaining walls, embankments, and earthworks. Shear strength controls how a soil mass will respond under applied loads, and failure occurs when the applied shear stress exceeds the soil's resistance to shearing(Terzaghi, K., Peck, R. B., and Mesri, G. 1996).

Shear strength (τ\tauτ) of soil is typically expressed using the Mohr–Coulomb failure criterion:

$$\tau = C + \sigma' \tan\emptyset \qquad \text{Equation (2.1)}$$

Where τ represents the shear strength, C represents the soils angle of cohesion , σ represents the effective normal stress upon loading and $tan\emptyset$ represents the angle of internal friction between the soil particles. The equation defines shear strength as the sum of:

i. **Cohesion (c):** the part of shear strength independent of normal stress (significant in clays and cemented soils).

ii. **Frictional resistance ($\sigma'\tan\emptyset$)** depends on effective stress and is dominant in granular soils.

Soils can be sheared under different drainage and loading conditions:

2.8.1 Drained Shear Strength

Occurs when excess pore water pressures have time to dissipate during loading (e.g., sands or clays under long-term loading).

2.8.2 Undrained Shear Strength

Occurs when loading is too rapid for drainage (e.g., clays during earthquakes or rapid construction) (Craig, 2004).

Table 2.1: Typical shear strength parameters

Soil Type	**Cohesion, c (kPa)**	**Friction Angle, φ (°)**
Soft Clay	10–25	20–25
Stiff Clay	25–50	25–30
Loose Sand	~0	28–32
Dense Sand	~0	30–40

2.8.3 Laboratory Methods of Determining Shear Strength

i. Direct Shear Test

Measures shear stress along a predetermined failure plane , offering a simple and suitable for cohesionless soils.

ii. Triaxial Compression Test

More versatile , but allows control of drainage and stress path. Types are; UU (Unconsolidated Undrained), CU (Consolidated Undrained), and CD (Consolidated Drained).

iii. Unconfined Compression Test

Used for cohesive soils without lateral confinement, also, it gives undrained shear strength value (Head,1992).

2.8.2 Factors Affecting Shear Strength

2.8.2.1 Soil Type

Granular soils (sands, gravels) gains their shear strength primarily from internal friction, while Fine-grained soils (clays) get their strength via cohesion and friction**.**

2.8.2.2 Moisture Content and Pore Pressure

Increasing water content or pore pressure reduces effective stress and strength. Fully saturated clays under undrained conditions may fail rapidly(Mitchell et al., 2005)..

2.8.2.3 Soil Structure and Fabric

Flocculated clays may have higher apparent cohesion than dispersed ones.

2.8.2.4 Over-consolidation Ratio (OCR)

Over-consolidated soils typically exhibit higher shear strength than normally consolidated soils.

2.8.2.5 Stress History

Soils previously subjected to high pressures may gain shear resistance.

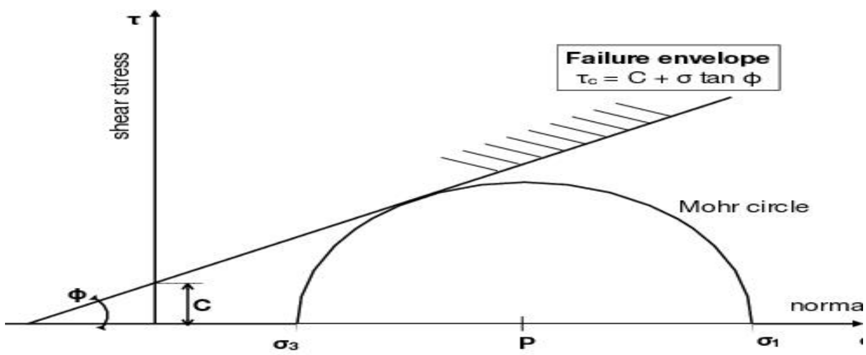


Fig 2.2 Mohr coulomb failure criterion **(**Diego, 2019)

The diagram you've provided is a Mohr-Coulomb failure criterion plot that combines Mohr's Circle with a shear strength failure envelope to describe the shear strength of soil or rock under applied stress conditions. It is used in soil and rock mechanics to determine the conditions under which materials fail due to shear. The point where the Mohr circle touches the failure envelope represents the failure condition of the material. It is commonly used in geotechnical engineering to evaluate stability of slopes, bearing capacity, and earth pressures.

2.8.3 Significance of shear strength in Engineering Applications

i. Shear strength controls slope failure potential; essential in landslide analysis.

ii. Determines bearing capacity and settlement behavior.

iii. Lateral earth pressure depends on soil shear strength.

iv. Shear resistance affects rutting and surface deformation.

(Das et al., 2018)

2.8.4 Effects of Groundwater and Water Table

Rising groundwater increases pore water pressure (uuu), decreasing effective stress and reducing shear strength which is critical in assessing slope stability during heavy rains or floods (Holtz et al., 2011).

2.9 California Bearing ratio of soil (S.B.C)

Soil bearing capacity refers to the ability of the ground to support the loads applied by structures without experiencing shear failure or excessive settlement. It is a critical parameter in geotechnical engineering, used to design safe and efficient foundations for buildings, bridges, towers, and other structures. Bearing capacity is the maximum contact pressure a soil can sustain without experiencing shear failure. (Terzaghi, 1943; Das, 2010).

There are different forms of bearing capacities explained below;

i. **Ultimate Bearing Capacity (qu):** A Theoretical maximum pressure that a soil can support before failure.

ii. **Net Ultimate Bearing Capacity (qnu):** Difference between ultimate bearing capacity and overburden pressure at foundation level.

iii. **Net Safe Bearing Capacity (qns):** Net ultimate divided by a safety factor.

iv. **Allowable Bearing Capacity (qa):** Maximum safe pressure considering both shear failure and settlement limits (Bowles, 1996).

2.9.1 Terzaghi's Bearing Capacity Theory

Karl Terzaghi (1943) provided the first analytical method for calculating bearing capacity for shallow strip foundations resting on a homogeneous soil. His equation is:

$$q_{u = cNc} + yD_fNq + 0.5yBNy \qquad \text{Equation (2.2)}$$

Where ;Cc = Cohesion of soil, γ = Unit weight of soil, **D_f** = Depth of foundation B=Width of foundation,Nc, Nq, Nγ Nc, Nq, N_\gammaNc ,Nq ,Nγ : Bearing capacity factors (functions of internal friction angle, φ).

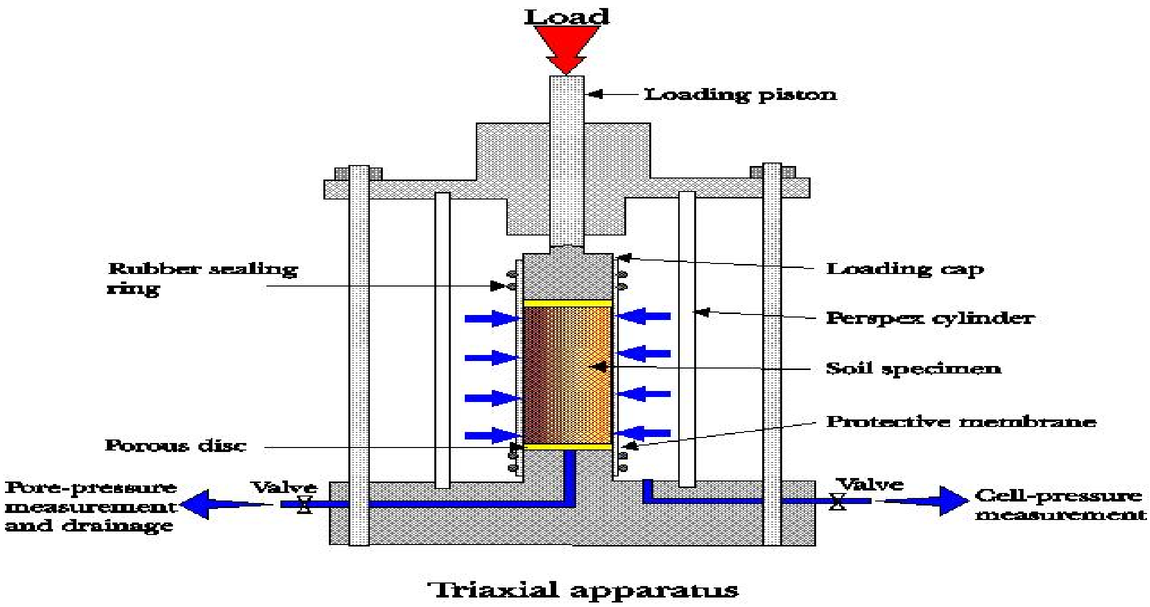


Fig 2.3 Triaxial testing apparatus(Geodata, 2018)

Table 2.2: Typical Allowable Bearing Capacities for values soil types

Soil Type	**Allowable Bearing Capacity (kN/m²)**
Soft clay	50–100
Medium clay	100–150
Dense sand	200–300
Gravel	300–600
Weathered rock	600–1200
Sound rock	> 4000

These are general values, as Site-specific testing is required (Peck et al., 1974)

2.9.2 Factors Influencing Bearing Capacity

i. **Soil Type and Properties:** Cohesion, internal friction, and stratification influence capacity.

ii. **Water Table Location:** Affects effective stress and shear strength.

iii. **Foundation Size and Shape:** Larger footings distribute load more efficiently.

iv. **Load Inclination and Eccentricity:** Reduce bearing capacity.

v. **Depth of Foundation (Df):** Deeper footings generally provide higher capacity (Das, 2010).

2.9.3 Modes of Bearing Capacity Failure

a. General Shear Failure

Sudden and catastrophic leading to well-defined failure surface and are common in dense or stiff soils.

b. Local Shear Failure

They show as progressive failures with less defined boundaries, Occurin in medium-dense soils.

c. Punching Shear Failure

In this failure mode, there is no lateral soil movement, usually common in loose or soft soils under small footings. Designs typically use a F.O.S between 2.5 and 3.0, (BS 8004, 2015)

2.9.4 Ground Improvement Techniques

When the bearing capacity is inadequate, the following methods are employed:

i. Soil Stabilization using lime, cement, or chemicals.

ii. Compaction (dynamic, vibro, or static methods).

iii. Installation of stone columns or geotextiles.

iv. Replacement of weak soil with stronger materials.

v. Use of deep foundations like piles or piers

Bearing capacity governs the type and depth of foundation required for a given structure. Overestimating can lead to foundation failure, while underestimating results in uneconomical design (Hausmann, 1990).

2.10 Review of past works

(Ahanaf et al., 2021) Since the stability of any foundation depends on the soil's bearing capacity, it is an important topic in geotechnical engineering, particularly in foundation engineering. Even a very well-designed building could collapse if it is placed on unsuitable soil with little bearing ability. The ultimate carrying capacity of soil rises with increasing water table depth. A greater depth has a greater impact on safe bearing capacity since it increases the weight of the surcharge. As much as feasible, the soil's strength decreases with increasing water table.

(Mohammad and Tahir, 2021) Around the world, shallow foundations are extremely prevalent. Therefore, when developing such footings, it becomes crucial to assess the parameters as precisely as feasible. Extensive and in-depth subsurface investigation is needed and desired. The soil's carrying ability is one of such factors. Numerous elements, including the application of eccentric and inclined loads, footing dimensions, soil relative density, and soil unit weight, all have an impact on bearing capacity. Scientists have repeatedly conducted experiments and put out a number of hypotheses to quantitatively assess the bearing capacity of foundations, particularly in the event of variations in the depth of the water table. Some of such theories/studies, along with the methods used and recommended, are attempted to be presented in this article.

(Chen et al., 2023) The carrying capacity of sandy shallow foundations is significantly impacted by variations in groundwater. The different water distributions in the soil mass would be caused by capillary water and groundwater in the shallow foundation. Consequently, the shallow foundation has three different kinds of water conditions. They are dry soil, capillary water impact zone, and fully saturated soil. To examine the impact of groundwater variation on the deformation behavior under various loading situations, a physical mode experiment was created for this investigation. A new configuration with a water-pressure control system was used to investigate the impact of water level and fluctuation times. Ten group model tests in all were conducted. The findings showed that there is a negative correlation between foundation carrying capability and water level height.

(Savo,2019)The soil's shear strength, which is roughly half of its compressive strength, is det ermined by its capacity to withstand shearing stresses and support load.Landslides, mining ca veins, rock fall, mudflow, and soil creep are all results of shear failure, which mostly occurs i n soil and rock.This project's goal is to examine how water affects shear strength and shear str ess characteristics.Moisture content has a negative effect on shear strength; the higher the moi sture content, the lower the shear strength.The foundation and structures will crumble, crack, and possibly slide as a result of the soil's declining shear strength, which is a very dangerous i mpact.

(Abdullah, 2022) The impact of GWT variation on pile foundation behavior has been examined in this study. The geotechnical program GEO5 has been used to do numerical analysis. The analysis has taken into consideration a set of six piles with a length of 10 m and a pile cap thickness of 1 m. For the pile, grade M20 concrete with Fe240 longitudinal and transverse reinforcement has been used. Pile foundation has been specifically examined in two distinct examples involving clayey and silty sand. To replicate the foundation in completely dry to fully saturated soil conditions, GWT has been adjusted. It has been observed how the pile behaves under axial and lateral loads. There have been reports of a noticeable shift in the pile group's settlement behavior.

(Lizhu et al., 2019) When the soil is saturated with water, the sandy soil foundation's bearing capacity is reduced by an average of 26.25% compared to when the soil is unsaturated. When a 100 kPa load is placed on the foundation beforehand, the bearing capacity of the foundation diminishes as the groundwater level rises, leading to more foundation settling. The cumulative settlement is 0.565 mm when the sea level rises by 4.3 m. As a result, people need to be aware of it and take appropriate action to limit groundwater level increase and guarantee project safety.

(Reddy and Manjunatha, 2020) Therefore, the goal of this study was to use the characteristics technique to examine how the water table affected the surrounding strip footings' ultimate bearing capacity. It was discovered that the water table has a significant impact on the stability lines' shape, but it has little bearing on where the maximum pressure occurs for both isolated and interfering footings. The findings indicate that Toyoura sand's ultimate bearing capacity can be lowered by 38% as a result of the water table's influence. The study also showed that, at a given water table depth, anisotropy in the angle of internal friction for Toyoura sand had a significant impact on the final bearing capacity of nearby footings.

(Shahriar et al ., 2013) To take into consideration the extra settlement brought on by water table fluctuations, several scholars suggested correction factors. Nevertheless, there hasn't been any literature on thorough settlement testing and its numerical modeling that takes ground water level influence into consideration. Through laboratory testing across a broad range of footing shapes, soil densities, water table depths, and stress levels, the aim of this paper is to evaluate the impact of water table rise on settling. In a settling tank, the tests were conducted. Water table rise was applied to the footings under operating stress, and the additional settlements were measured. FLAC was used to model the experimental setting, and the outcomes were contrasted with those of the laboratory testing.

(Nazile and Gergin, 2020) This study models and analyzes a structure's foundation systems on various soil profiles and groundwater levels. The basic design and application phases present a number of challenges. Many issues, including emergence, are specifically caused by the high groundwater level, the strain on the soil below its load-bearing capability, settlement, and liquefaction. The Plaxis 2D application was used to model foundation systems for raft and stacked raft foundations based on six distinct soil profiles with high groundwater levels. The investigation revealed that the soil-foundation interaction in both static and seismic scenarios is significantly influenced by the groundwater level, soil properties, and environmental factors.

CHAPTER THREE

METHODOLOGY

3.1 Study Areas

The selected region to be investigated in this study is Effurun Warri . as soil samples would be obtained from this region for shear strength and bearing capacity investigations. This area is however located in Warri Delta State. Below are the brief geology and Climatology of the study areas.

3.1.1 Geology and Climatology of Delta State

Delta State lies in the south-south region of Nigeria, between latitudes 5°00′–6°45′ N and longitudes 5°00′–6°30′ E. It occupies part of the Niger Delta Basin, a region characterized by extensive river networks, swamps, and low-lying coastal plains (Niger Delta Budget Monitoring Group, 2024).

Delta State experiences a humid tropical climate with two distinct seasons: a rainy season from April to October and a dry season from November to March. Mean annual rainfall ranges between 2,000 mm and 2,700 mm, while average temperatures vary from 27 °C to 30 °C, increasing northward (Okoduwa & Amaechi, 2024). Vegetation varies from mangrove swamps in the south to rainforest and derived savannah in the north. Recent studies show a gradual increase in temperature and a slight decline in rainfall, trends linked to climate change (Okoduwa & Amaechi, 2024).

Geology–Climate Interaction

The interplay of soft deltaic sediments and heavy rainfall contributes to flooding, erosion, and groundwater rise in many parts of the state. These conditions directly influence civil engineering design — especially foundations, drainage, and road construction (Oni & Olatunji, 2017).

3.1.2 Geology and Climatology of Study area

Geology and Climatology of Effurun, Delta State

Effurun is a major urban settlement in Uvie Local Government Area of Delta State, located between latitudes 5°30′–5°36′ N and longitudes 5°45′–5°50′ E, within the Niger Delta region of southern Nigeria. The area lies on a low-lying coastal plain that forms part of the Niger Delta sedimentary basin, characterized by extensive river networks and swampy terrain (Niger Delta Budget Monitoring Group),Subsurface investigations around Warri–Effurun show alternating sand and clay layers typical of fluvial and coastal depositional environments (Oni & Olatunji, 2017). The soil type is generally hydromorphic, with poor natural drainage and high water tables that influence building foundation design and road construction.

Effurun experiences a humid tropical climate, influenced by the tropical maritime air mass from the Atlantic Ocean. There are two main seasons — the rainy season (April–October) and the dry season (November–March) (Okoduwa & Amaechi, 2024). The average annual rainfall is about 2,500 mm, while the mean temperature ranges from 27 °C to 30 °C, with relative humidity exceeding 80% most of the year (Niger Delta Budget Monitoring Group, 2024).

Recent climatological studies indicate a gradual rise in temperature and a slight decline in rainfall totals, suggesting early signs of climate variability in the area (Okoduwa & Amaechi, 2024).The combination of loose, waterlogged sediments and heavy rainfall makes Effurun susceptible to seasonal flooding, erosion, and poor drainage. These environmental conditions directly affect civil engineering works, requiring well-designed drainage systems, soil stabilization, and elevated foundations to ensure structural stability (Earthdoc, 2024).

Fig 3.1 Plan view of the Effurun Delta State (Google Earth, 2023)

3.2 Materials

The below listed materials and equipment will be used in carrying out the experimental investigation. Clayey, silty, and sandy soils with a variety of plasticity and permeabilities will be tested in a lab using undisturbed soil samples. Triaxial test rigs that are consolidated-undrained and consolidated-drained (for p′-q′ behavior and pore-pressure response at different saturation levels), to assess compressibility and measure settlement under different water-table regimes, oedometer (one-dimensional consolidation) cells are used. Field plate-load test apparatus: loading frame, proving ring, hydraulic jack, settlement gauges, and rigid plates to measure in-situ stiffness and bearing capacity.

For Direct-Shear Testing, the Direct-shear box and shear carriage (standard 60 × 60 mm or 100 × 100 mm boxes), while for Normal-load frame (dead weights or hydraulic jack) , and Displacement gauge (dial gauge or LVDT) .Data logger or manual readout would be used to obtain result data.

Triaxial Testing would implement the Triaxial cell(s) with triaxial pressure intensifier (up to 2 MPa or more), Pore-pressure transducers and back-pressure saturation system, The Axial load frame (servo-controlled or manually-operated), Vertical deformation measurement (LVDTs or displacement transducers) , Membrane expander, pedestal, and confining fluid reservoir.

Unconfined Compression & Vane Shear would utilize theUnconfined compression machine (axial jack, proving ring, and platens) and Field vane shear apparatus (portable torque head, calibrated torque gauge).

Consolidation & Permeability test will require (for coupled strength–stiffness insights), An Oedometer (one-dimensional consolidation cell, loading yoke) , Falling-head and/or constant-head permeameter attachments.

3.2.1 Sample Collection and preparation

Dry sack bags will be used in collecting undisturbed laterite and clay soil specimens. The samples if saturated, will be heated in an oven to bring to dryness (A total of 2 samples each from the study areas for water depths of -100mm, -200mm respectively) will be collected and investigated in this study.

3.3 Methodology

Assessing the water-table effects must begin with Terzaghi's effective stress principle:

3.3.1 Triaxial Shear Tests (UU, CU, CD)

Equipment:

i. Triaxial Cell: A pressure chamber that holds a cylindrical soil specimen and allows the application of confining pressure.

ii. Axial Load Frame: A mechanical or hydraulic system to apply vertical loads on the specimen.

iii. Pore Pressure Transducers: To measure the internal pore water pressure during the test.

iv. Confining Pressure System: Typically utilizes water or oil to provide uniform lateral pressure.

v. Data Acquisition System: Records load, displacement, and pore water pressure data continuously during the test(BS 1377- 7:1990).

vi. Specimen Preparation Tools: Devices such as a Shelby tube to obtain and trim cylindrical specimens with standard dimensions (often a 2:1 height-to-diameter ratio).

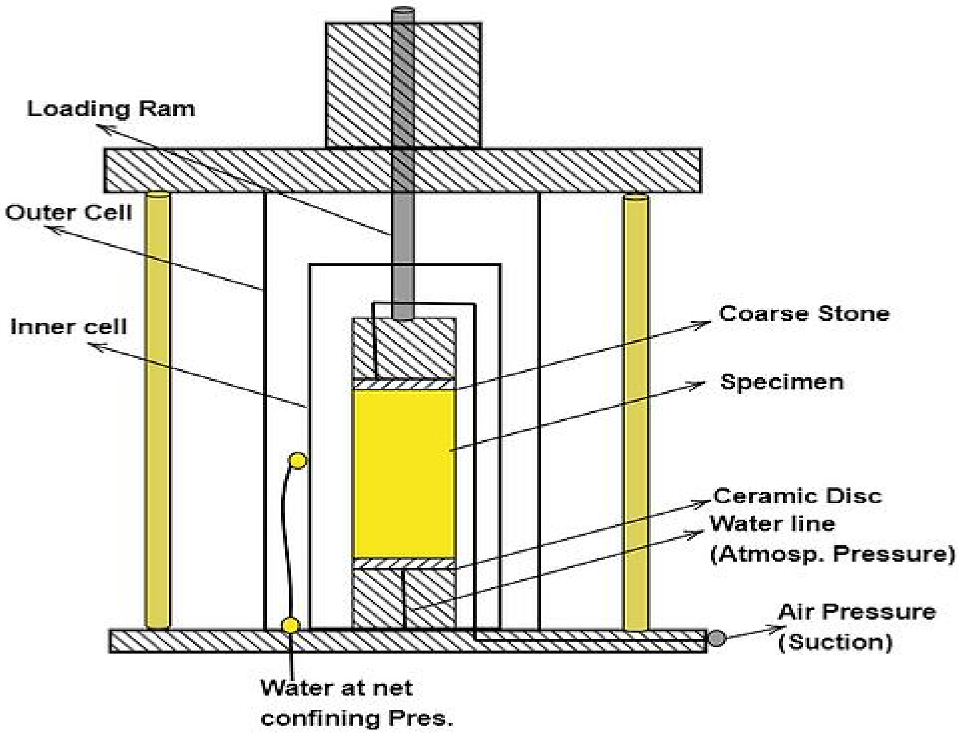


Fig 3.2 The tri-axial test set up (Testbook, 2018)

Procedures:

i. Specimen Preparation: A cylindrical specimen is carefully extracted (using a Shelby tube for undisturbed sampling) and trimmed to dimensions prescribed by the test standard(BS 1377- 7:1990).

ii. Saturation (for Consolidated Undrained (CU) tests): The specimen may be saturated by applying a back-pressure until the B-value (indicating saturation) is sufficiently high (generally above 0.95).

iii. Application of Confining Stress: The specimen is placed in the triaxial cell, and an initial isotropic confining pressure (σ3\sigma_3) is applied.

iv. Axial Loading: An axial load is applied incrementally (in strain-controlled or stress-controlled mode) while maintaining the confining pressure. The changes in axial load, specimen deformation, and pore pressure (if measured) are recorded continuously.

v. Failure and Data Recording: The test is run until the specimen reaches failure (peak deviator stress) or until a designated strain level is achieved. The recorded data are then used to construct stress–strain and Mohr circles.

The procedures and calculations for the triaxial test adhere to standard practices documented in (BS 1377- 7:, 1990) and are detailed in many geotechnical engineering laboratory manuals .

3.3.2 Direct Shear Tests under Variable Saturation

In a direct shear test conducted under variable saturation, a thin-walled soil specimen is first trimmed to fit snugly within a two–part shear box and is bounded at its top and bottom by porous stones. Once the specimen has been carefully placed and the initial normal load applied, water is introduced through the porous stones or by submerging the entire box to achieve a prescribed degree of saturation; by adjusting the depth of immersion or the back-pressure, one can simulate water tables at different heights relative to the shear plane. During saturation, the test water is ideally matched in chemistry to the soil's pore fluid to avoid osmotic effects and is allowed to infiltrate until the desired volumetric water content is attained.

As shearing begins, one half of the shear box moves laterally at a controlled, slow rate to ensure that pore pressures dissipate in drained conditions, or more rapidly when undrained behavior is sought; the rate of displacement is critical, because too rapid a movement in a "drained" test will develop excess pore pressures, whereas too slow a rate in an "undrained" test may permit unintended drainage. Throughout the test, normal and shear forces—and, in partially or fully saturated specimens, the volume of water entering or leaving the sample—are recorded continuously (BS 1377-7/9, 1990). The peak shear stress reached on the imposed plane is then plotted against the corresponding normal stress to define a shear-strength envelope. By repeating this procedure at increasing saturation levels, one observes systematically that as the water content rises toward full saturation the cohesion intercept and friction angle both diminish, reflecting the loss of matric suction and reduction in effective stress that accompanies groundwater rise

Outcomes: Variation of cohesion (c) and friction angle (ϕ) with saturation, directly showing strength loss as water table approaches the shear plane.

at which 50 percent consolidation is reached under each increment allows calculation of the coefficient of consolidation (cv) based on Terzaghi's one-dimensional consolidation theory, following the procedures detailed in (BS 1377-7/9, 1990)

Outcomes: Changes in compressibility and coefficient of consolidation, which indirectly affect bearing capacity through settlement estimates.

3.3.3 Compaction Test

Equipment:

i. Compaction Mold: A standard Proctor mold with a known volume (e.g., 1/30 cubic foot for Standard Proctor or 1/5 cubic foot for Modified Proctor tests).

ii. Compaction Hammer: A hammer of specified weight and drop height (typically 2.5 kg dropped from 305 mm for the Standard Proctor; higher energy parameters are used for the Modified Proctor) (Douglas,1983).

iii. Mixing Tools: Spatulas or mixers to blend soil with water.

iv. Oven and Precision Balance: For moisture content determination.

v. Measuring Devices: To confirm the exact dimensions of the mold and thus the volume(BS 1377-7/9, 1990).

Procedures:

i. Sample Preparation: The soil is first dried and sieved to remove particles larger than the specified size. The soil is then conditioned by mixing with water at various contents to create a range of moisture levels.

ii. Layered Compaction: The conditioned soil is placed in the mold in increments—usually three layers are used for the Standard Proctor. Each layer receives a fixed number of hammer blows (for example, 25 blows per layer) ensuring a uniform compaction.

iii. Measurement: After compaction, the sample is extracted carefully from the mold, and its weight is measured. A portion of the sample is then oven-dried to determine the moisture content.

iv. Repetition: The procedure is repeated for several moisture contents to develop a compaction curve (Douglas,1983).

Calculations:

i. Moisture Content (w):

ii. Dry Density:

iii. Compaction Curve: By graphing dry density versus moisture content, the optimum moisture content (the moisture at which the maximum dry density is achieved) is determined. This optimum value is critical for field compaction specifications(BS 1377- 4, 1990).

3.3.4 Empirical Correlations

Bearing Capacity Factors: Adjustments to standard factors (Nc,Nq,NγN_c, N_q, N_γNc ,Nq ,Nγ) via charts or formulae based on relative depth of water table to foundation depth.

SPT/CPT Correlations: Empirical equations relate corrected N-values or qc to shear strength and bearing capacity, implicitly incorporating water-table influence.

3.3.5 Soil Classification Test

Equipment:

i. Sieve Set: A stack of standard sieves (ranging from about 4.75 mm down to about 75 μm).

ii. Mechanical Sieve Shaker: To standardize the sieving process.

iii. Hydrometer: For sedimentation analysis of particles finer than 75 μm.

iv. Sedimentation Cylinders: Typically, of 1 L or 2 L capacity.

v. Drying Oven and Precision Balance: To determine moisture content and accurately weigh soil fractions.

vi. Pycnometer or Specific Gravity Bottle: For specific gravity measurements.

Procedures:

i. Sieve Analysis: The soil sample is oven-dried and then passed through a stack of sieves arranged in descending order of aperture size. The assembly is placed on a mechanical shaker for a predetermined time (commonly 10 minutes) (BS 1377- 2:2022).

ii. . Each fraction retained on the respective sieve is weighed, and the percentage by mass is calculated.

iii. Hydrometer Test: Fine-grained soil fractions that pass through the smallest sieve (usually <75 μm) are analyzed using a hydrometer. The soil is dispersed in a water solution with a deflocculant, and the hydrometer reading, taken at specified time intervals, indicates the concentration of particles in suspension based on Stokes' Law.

iv. Specific Gravity Test: In a separate test, the specific gravity of the soil particles is determined using a pycnometer or similar device. This value is important for calculating void ratios and porosity in further analyses.

Calculations:

Grain Size Distribution: The percentage passing for each sieve is computed by mass balance. Cumulative distribution curves are plotted on a semi-logarithmic graph (BS 1377- 2:2022).

CHAPTER FOUR

ANALYSIS AND DISCUSSION OF RESULTS

This chapter presents a detailed discussion and interpretation of the laboratory test results, aligning the index properties (Sieve Analysis,Specific Gravity, Compaction, Atterberg Limits, and Triaxial test) with the shear strength parameters (Cohesion and Angle of Internal Friction) for the 1.0-meter and 2.0-meter soil strata. The primary focus is to evaluate the derived engineering characteristics against the project objectives: quantifying the effects of water table variation on soil strength and ultimate bearing capacity, and developing data-driven mitigation techniques for site characterization and foundation design. The ensuing discussion transitions the analysis from empirical observation to actionable geotechnical guidance.

4.1 SIEVE ANALYSIS

This discussion analyzes the grain size distribution, also known as sieve analysis, performed on the soil sample taken from 1.0 meter below the ground surface. This test is essential for classifying the coarse-grained fraction of the soil and assessing its suitability for use in civil engineering applications.

Table 4.1 showing sieve analysis results of sand sample 1m below ground surface.

SAMPLE NO: **WARRI, 1m, SAMPLE 1**
DATE: **29/10/2025**
TESTED BY:

Table 4.1

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	0.28	99.72	99.72
10	2	0.14	99.58	99.58
14	1.18	0.65	98.93	98.93
25	0.6	3.02	95.91	95.91
36	0.425	2.05	93.86	93.86
52	0.3	2.05	91.81	91.81
72	0.212	14.22	77.59	77.59
100	0.15	4.55	73.04	73.04
200	0.075	15.72	57.32	57.32

The sieve analysis was performed on the soil passing the 3.35 mm sieve, as the initial 100% passing for the coarser sieves indicates no gravel-sized material was present in the tested sample. Based on the percentage of material passing the No. 200 sieve (0.075 mm), the soil can be classified according to the Unified Soil Classification System (USCS) and its engineering behavior.

i. Fine Content: A critical finding is that 57.32% of the soil sample passes the No. 200 sieve.

ii. Since more than 50% of the material passes the No. 200 sieve, the soil is classified as a Fine-Grained Soil (Silt and/or Clay).

iii. The remaining 42.68% of the material is Sand-sized or coarser.

Coarse Fraction: The material retained on the No. 200 sieve, which is predominantly Sand, shows:60% finer is approximately 0.17 mm (interpolating between 77.59 and 57.32),The sand fraction appears to be uniformly graded, as a high percentage is concentrated in the fine-to-medium sand range (between 0.425 mm and 0.075 mm).

Engineering Significance of soil grade:The predominance of fine-grained particles has direct implications for the soil's behavior and engineering design:

Permeability: Fine-grained soils have very small voids, leading to a low coefficient of permeability. This will result in slow drainage, which can prolong consolidation settlement time.

Plasticity and Compressibility: The high percentage of fines indicates that the soil's properties will be controlled by cohesion and plasticity, not friction. This confirms the likelihood of high compressibility and significant long-term settlement risk.

Table 4.2 showing sieve analysis results of sand sample 2m below the ground.

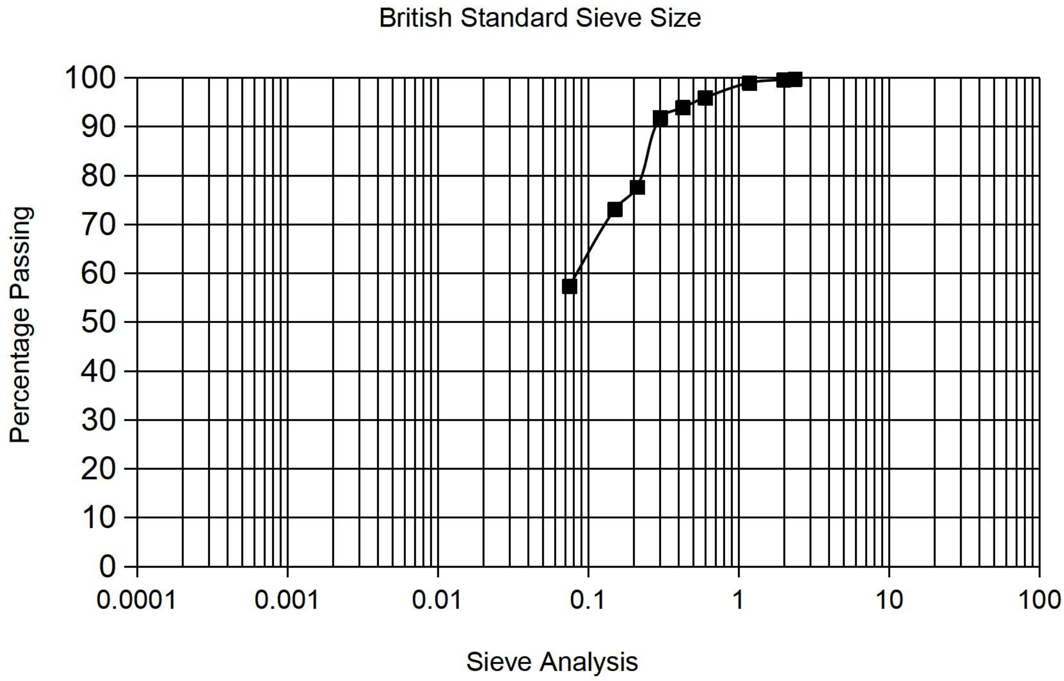
SAMPLE NO: **WARRI, 1m, SAMPLE 1**
DATE: **29/10/2025**
TESTED BY:

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	0.1	99.9	99.9
10	2	0.07	99.83	99.83
14	1.18	0.67	99.16	99.16
25	0.6	3.25	95.91	95.91
36	0.425	1.45	94.46	94.46
52	0.3	3.14	91.32	91.32
72	0.212	18.44	72.88	72.88
100	0.15	4.44	68.44	68.44
200	0.075	19.97	48.47	48.47

The grain size analysis, conducted via sieving on the soil sample recovered from 2.0 meters below the ground surface, This test specifically evaluates the distribution of the soil's coarse and fine fractions, directly impacting the anticipated engineering behavior of the stratum.

The critical observation from the sieve analysis is the percentage of material passing the No. 200 sieve (0.075 mm), which is determined to be 48.04%. According to the Unified Soil Classification System (USCS), a soil where less than 50% passes the No. 200 sieve is classified as a Coarse-Grained Soil (Sands or Gravels). Therefore, the soil behavior at the 2.0 m depth is primarily governed by the frictional resistance of its granular particles, although the high percentage of fines is a significant factor.

The remaining 51.96% retained on the No. 200 sieve constitutes the coarse fraction, predominantly in the sand size range. This finding is in direct contrast to the 1.0 m stratum, where the fine content exceeded 50%. The soil at 2.0 m is thus tentatively classified as a Silty Sand (SM) or a Clayey Sand (SC), pending the determination of the fines' plasticity (Atterberg limits).



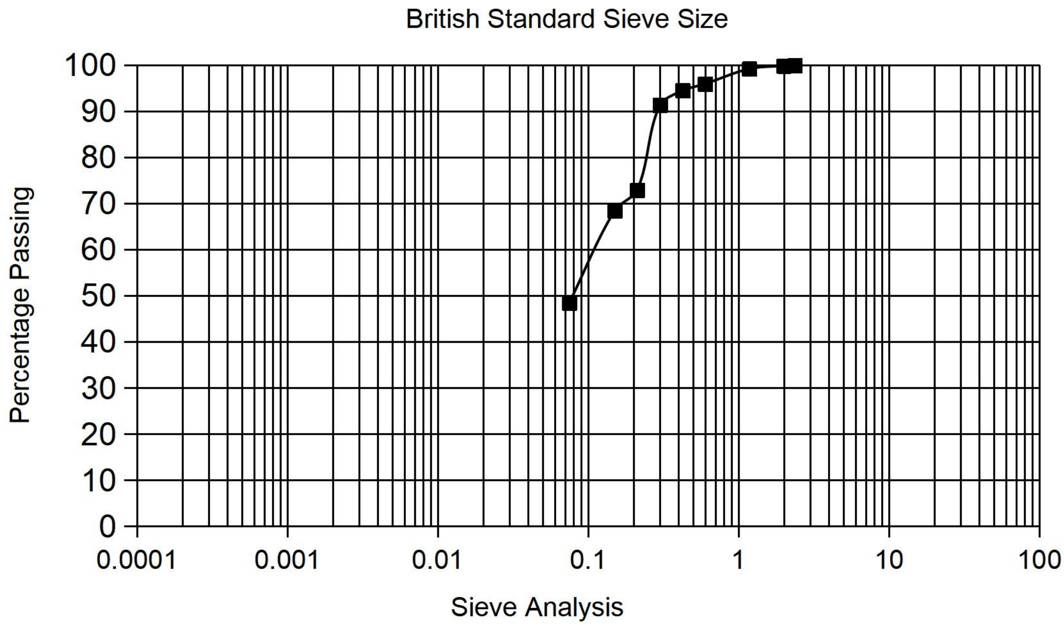


Fig 4.1 a and b showing particle size distribution graph at 1 and 2 meters below ground surface respectively.

4.2 SPECIFIC GRAVITY.

The Specific Gravity (G_s) test was performed on the soil sample from the 1.0 meter stratum to determine the density of the solid soil particles relative to water. This value is fundamental for establishing the phase relationships (void ratio, degree of saturation) and inferring the composition of the soil.

Table 4.3. Table showing specific gravity data of sample 1 meter below ground surface.

SPECIFIC GRAVITY AT 1M		
BOTTLE NUMBER	DN	EI
BOTTLE WEIGHT (g) M1	23.87	23.36
BOTTLE + DRY SOIL (g) M2	58.25	58.85
BOTTLE + SOIL + WATER (g) M3	96.39	98.23
BOTTLE + WATER (g) M4	76.24	76.82
MASS OF DRY SOIL (g) (M2 - M1)	34.38	35.49
MASS OF EQUAL VOLUME OF WATER (g) (M4 – M1) – (M3 – M2)	14.23	14.08
SPECIFIC GRAVITY $G_s = \frac{M2-M1}{M}$	2.416022	2.520597
AVERAGE SPECIFIC GRAVITY	2.46831	

The test yielded an Average Specific Gravity OF 2.468.This Specific Gravity value is significantly low when compared to the typical range of 2.65 to 2.80 for common inorganic mineral soils (e.g., quartz, silicates). This deviation is a direct and strong indicator that the soil is not a pure inorganic mineral composition. The low Gs suggests the presence of lightweight solids, which is characteristic of:

* Organic Matter: A high content of organic material, which has a specific gravity typically ranging from 1.2 to 1.6.

* Porous or Lightweight Mineral Grains: Highly weathered or porous primary minerals.

The low G_s value of 2.468 immediately flags the stratum as potentially problematic. Soils with such low specific gravity generally correlate with high compressibility, low maximum dry density, and potential for long-term consolidation settlement, classifying them as poor quality for foundation support. This test result is the first critical warning regarding the site's subgrade quality and is crucial for guiding subsequent engineering decisions.

Table 4.4. Table showing specific gravity data of sample 2 meter below ground surface.

SPECIFIC GRAVITY AT 2M		
BOTTLE NUMBER	GLE	P31
BOTTLE WEIGHT (g) M1	24.62	23.04
BOTTLE + DRY SOIL (g) M2	57.82	56.91
BOTTLE + SOIL + WATER (g) M3	98.06	96.97
BOTTLE + WATER (g) M4	77.97	76.49
MASS OF DRY SOIL (g) (M2 - M1)	33.2	33.87
MASS OF EQUAL VOLUME OF WATER (g) (M4 – M1) – (M3 – M2)	13.11	13.39
SPECIFIC GRAVITY $G_s = \frac{M2-M1}{M}$	2.532418	2.5295
AVERAGE SPECIFIC GRAVITY	2.530959	

The Specific Gravity test results for the soil sample taken from the 2.0 meter stratum, providing a direct interpretation of the engineering implications without referring to past work.

The Specific Gravity test yielded an Average Specific Gravity of 2.531.

This Gs value is low compared to the 2.65 to 2.80 range typical of standard inorganic soils. While this value is slightly higher than that of the 1.0 m stratum, it still strongly indicates the presence of low-density components, such as organic matter or highly weathered, porous mineral grains. The low Gs suggests that the solid particles themselves are light, which intrinsically limits the maximum dry density and is associated with higher compressibility and lower stability than desired for structural subgrades.

4.3 COMPACTION TEST

Table 4.5. Table showing compaction test results at 1m depth.

DETERMINATION OF THE MOISTURE/DENSITY RELATION OF SOIL USING THE HEAVY COMPACTION/PROTOR METHOD.										
Wt of mould & wet soil(W2)g	6412.00		6634.00		6712.00		6675.00		6500.00	
Wt of mould (W1)g	4646.00		4646.00		4646.00		4646.00		4646.00	
Wt of wet soil (W2-W1)	1766.00		1988.00		2066.00		2029.00		1854.00	
BulkDensity(Pb)(W2-W1)/x g/cm^3	1.77		2.00		2.07		2.04		1.86	
MOISTURE CONTENT DETERMINATIONS										
Container No	BH	UJU	BH	R4	OI	XU	O4	2G	ZOI	AXB
Wt of wet soil & container (g)	52.30	51.28	56.00	47.80	47.20	46.20	58.60	52.50	62.50	64.60
Wt of dry soil & container (g)	49.75	49.62	53.19	45.01	44.64	43.35	55.44	48.36	57.80	58.63
Wt of Container (g)	23.28	23.02	21.96	21.96	21.99	21.79	23.48	23.71	22.24	21.09
Wt of dry soil (Wd) g	26.47	26.60	31.23	23.05	22.65	21.56	31.96	24.65	35.56	37.54
Wt of Moisture (Wm) g	2.55	1.66	2.81	2.79	2.56	2.85	3.16	4.14	4.70	5.97
Moisture Content 100(Wm/Wd)%	9.63	6.24	9.00	12.10	11.30	13.22	9.89	16.80	13.22	15.90
Average Moisture Content (m)%	7.94		10.55		12.26		13.34		14.56	
Dry Density = Pb/1+(m/100)(g/cm^3)	1.64		1.81		1.85		1.80		1.63	

The compaction test was conducted on the soil sample from the 1.0 meter stratum to determine the relationship between moisture content and density achieved under a standard compactive effort. This relationship is defined by the Maximum Dry Density and the corresponding Optimum Moisture Content (Wopt), both of which are critical for field control of engineered fills.

The magnitude of the maximum dry density is a crucial indicator of the soil's quality. For common inorganic mineral soils, values typically exceed 1.9g/cm^3 under standard Proctor effort. The observed value of 1.76g/cm^3 is low, which strongly supports the previous findings from the specific gravity test (Gs = 2.468) that this soil has a low-density composition, likely due to a significant content of organic matter. Low maximum dry density correlates directly with low shear strength and high compressibility, confirming the poor quality of the material for use as structural fill or as a shallow foundation bearing stratum.

Furthermore, the optimum moisture content of nearly 16% is relatively high. Utilizing the calculated specific gravity, the theoretical void ratio (e) and degree of saturation (S) at Pd max can be determined;

$$e = \frac{Gs\ .\ Pw}{Pd\ max} - 1 = \frac{2.468\ x\ 1.00}{1.76} - 1 = 0.402$$

Equation 4.1

$$S = \frac{Wopt\ .Gs}{e} = \frac{0.1593\ x\ 2.468}{0.402} = 0.975 = 97.5\%$$

The resulting Degree of Saturation (S) of approximately 97.5% at the peak density confirms that the soil is highly saturated when fully compacted. This proximity to the Zero Air Voids line indicates that the soil's density is extremely sensitive to minor fluctuations in moisture content during field compaction, making field control challenging and demonstrating a high risk of pore pressure buildup and subsequent stability issues if over-wetted.

The compaction results therefore characterize the 1.0 m stratum as a high sensitive and low strength fine-grained material, mandating careful engineering assessment before any construction activities commences.

Table 4.6. Table showing compaction test results at 2m depth.

<table>
<tr><td colspan="11">DETERMINATION OF THE MOISTURE/DENSITY RELATION OF SOIL USING THE HEAVY COMPACTION/PROTOR METHOD.</td></tr>
<tr><td>Wt of mould & wet soil(W2)g</td><td colspan="2">6400.00</td><td colspan="2">6529.00</td><td colspan="2">6593.00</td><td colspan="2">6534.00</td><td colspan="2">6460.00</td></tr>
<tr><td>Wt of mould (W1)g</td><td colspan="2">4557.00</td><td colspan="2">4557.00</td><td colspan="2">4557.00</td><td colspan="2">4557.00</td><td colspan="2">4557.00</td></tr>
<tr><td>Wt of wet soil (W2-W1)</td><td colspan="2">1843.00</td><td colspan="2">1972.00</td><td colspan="2">2036.00</td><td colspan="2">1977.00</td><td colspan="2">1903.00</td></tr>
<tr><td>BulkDensity(Pb)(W2-W1)/x g/cm^3</td><td colspan="2">1.85</td><td colspan="2">1.98</td><td colspan="2">2.04</td><td colspan="2">1.99</td><td colspan="2">1.91</td></tr>
<tr><td colspan="11">MOISTURE CONTENT DETERMINATIONS</td></tr>
<tr><td>Container No</td><td>EH</td><td>OZ</td><td>AZE</td><td>0Z1</td><td>SSG</td><td>VEG</td><td>JJBT</td><td>VI4</td><td>1ZO</td><td>LUP</td></tr>
<tr><td>Wt of wet soil & container (g)</td><td>38.10</td><td>49.40</td><td>56.10</td><td>59.10</td><td>59.50</td><td>72.10</td><td>66.20</td><td>60.70</td><td>71.00</td><td>66.80</td></tr>
<tr><td>Wt of dry soil & container (g)</td><td>36.50</td><td>47.00</td><td>52.30</td><td>55.00</td><td>54.20</td><td>65.40</td><td>59.20</td><td>55.10</td><td>62.90</td><td>59.90</td></tr>
<tr><td>Wt of Container (g)</td><td>21.40</td><td>23.00</td><td>24.40</td><td>23.50</td><td>22.00</td><td>21.90</td><td>21.30</td><td>24.10</td><td>21.40</td><td>24.70</td></tr>
<tr><td>Wt of dry soil (Wd) g</td><td>15.10</td><td>24.00</td><td>27.90</td><td>31.50</td><td>32.20</td><td>43.50</td><td>37.90</td><td>31.00</td><td>41.50</td><td>35.20</td></tr>
<tr><td>Wt of Moisture (Wm) g</td><td>1.60</td><td>2.40</td><td>3.80</td><td>4.10</td><td>5.30</td><td>6.70</td><td>7.00</td><td>5.60</td><td>8.10</td><td>6.90</td></tr>
<tr><td>Moisture Content 100(Wm/Wd)%</td><td>10.60</td><td>10.00</td><td>13.62</td><td>13.02</td><td>16.46</td><td>15.40</td><td>18.47</td><td>18.06</td><td>19.52</td><td>19.60</td></tr>
<tr><td>Average Moisture Content (m)%</td><td colspan="2">10.30</td><td colspan="2">13.32</td><td colspan="2">15.93</td><td colspan="2">18.27</td><td colspan="2">19.56</td></tr>
</table>

Dry Density = $Pb/1+(m/100)(g/cm^3)$	1.68	1.75	1.76	1.68	1.60

The compaction test was conducted on the soil sample from the 1.0 meter stratum to determine the relationship between moisture content and density achieved under a standard compactive effort. This relationship is defined by the Maximum Dry Density and the corresponding Optimum Moisture Content (Wopt), both of which are critical for field control of engineered fills.

The data shows the dry density for five distinct moisture contents (W), allowing for the approximation of the characteristic compaction curve, the maximum dry density achieved is 1.76 g/cm^3, which occurs at an approximate optimum moisture content of 15.93%. For common inorganic mineral soils, Max dry density values typically exceed 1.9g/cm^3 under standard Proctor effort. The observed value of 1.76g/cm^3 is low, whicgh strongly supports the previous findings from the specific gravity test (Gs = 2.468) that this soil has a low-density composition, likely due to a significant content of organic matter. Low maximum dry density correlates directly with low shear strength and high compressibility, confirming the poor quality of the material for use as structural fill or as a shallow foundation bearing stratum.

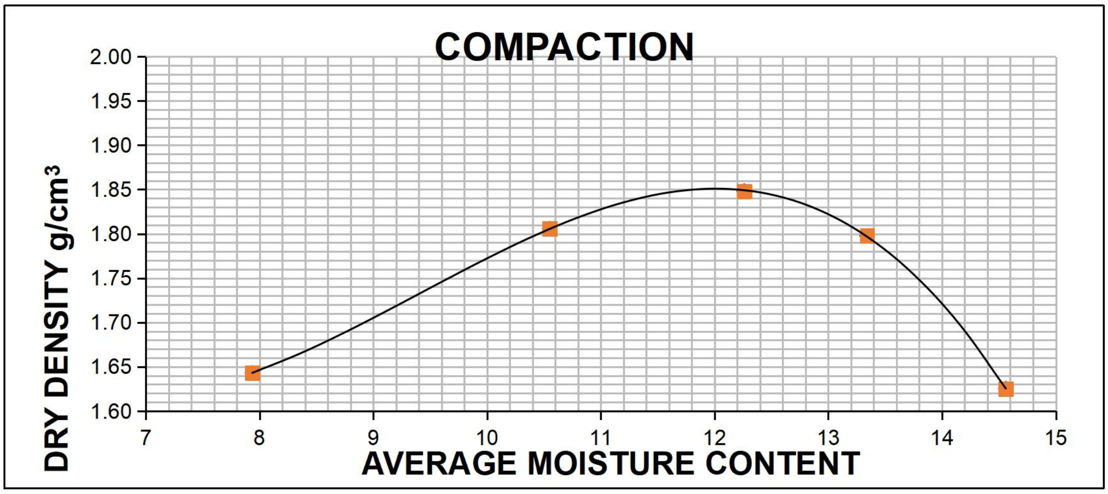
Furthermore, the optimum moisture content of nearly 16% is relatively high. Utilizing the calculated specific gravity, the theoretical void ratio (e) and degree of saturation (S) can be determined:

$$e = \frac{Gs \cdot Pw}{Pd\ max} - 1 = \frac{2.531 \times 1.00}{1.76} - 1 = 0.438 \qquad \text{Equation 4.2}$$

$$S = \frac{Wopt\,.Gs}{e} = \frac{0.1593\ x\ 2.531}{0.438} = 0.920 = 92\%$$

The Degree of Saturation of 92.0\% at the peak density indicates that this layer, while slightly less moisture-sensitive than the 1.0 m stratum (approx 97.5%), remains highly vulnerable to small increases in water content, which could lead to stability problems due to rapid pore pressure development during construction.

The compaction data thus characterizes the 2.0 m stratum as marginally better than the upper layer, primarily due to its lower organic content and slightly better gradation, but ultimately confirms it as a low-strength, highly moisture-sensitive material that requires careful handling and likely extensive remediation for structural use.



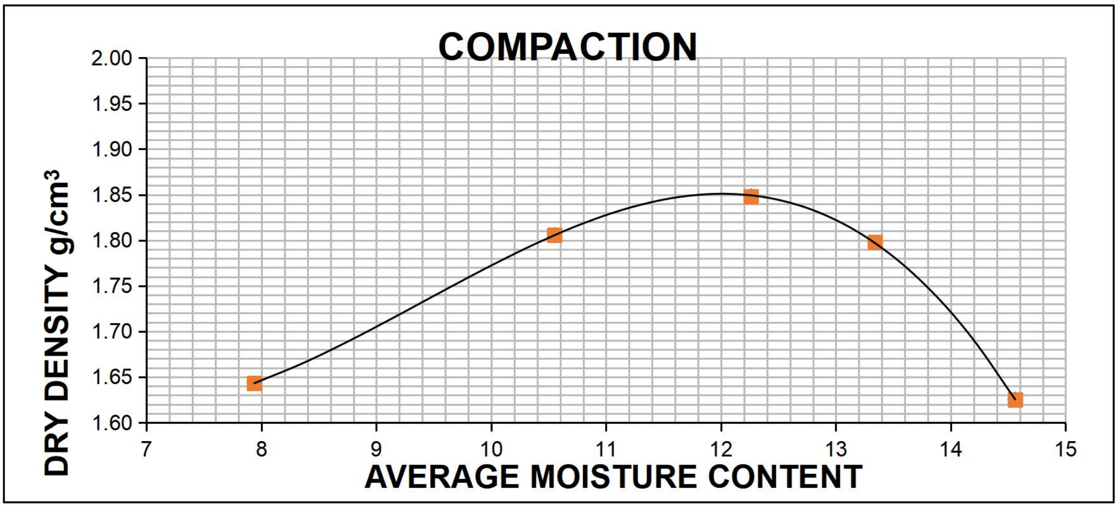


Fig 4.2 showing compaction curve at 1 and 2 meters below ground surface respectively.

4.4 WATER TABLE DATA

Groundwater data from Effurun–Warri (Akpoborie, Uriri & Efobo, 2014) reveal that the depth to water level (DWL) ranges from 0.75 m to 3.8 m, with hydraulic head values between 3.45 m and 23.6 m. These results indicate a generally shallow groundwater table, averaging about 2.2 m across the metropolis.

The shallow aquifer reflects Warri's low relief, high rainfall, and deltaic soil conditions, which promote high recharge and limit deep percolation. Locations such as Ubeji, Aladja, and Ugbomro show DWL less than 2.0 m, while deeper levels around Effurun and Ekpan may result from local elevation or soil permeability differences.

Such variations strongly influence the shear strength of soils. When the water table rises, pore water pressure increases, reducing effective stress and therefore decreasing shear strength and bearing capacity. Conversely, a lower water table increases effective stress, improving strength but possibly causing settlement in fine-grained soils.

Differences in hydraulic head across the area also indicate uneven groundwater flow, which

ATTERBERG LIMIT (LIQUID AND PLASTIC LIMIT TEST)					
SAMPLE NO :1m	LIQUID LIMIT : 16.6935%			PLASTIC LIMIT : 14.63%	
DATE: 27/10/2025	PLASTIC INDEX : 2.06%				
SOIL DESCRIPTION :				SITE : WARRI	
TYPE OF TEST:	LIQUID LIMIT				
No of blows/shrinkage %	45.00	31.00	26.00	18.00	10.00
Container no	V13	T11	XU	XB	OZ
Wt of wet soil and Container(g)	44.24	49.67	41.11	44.24	44.46
Wt of dry soil and container (g)	41.33	45.89	38.44	41.33	41.26
Wt of container(g)	21.92	23.22	21.83	24.55	23.45
Wt of Dry soil(Wd)(g)	19.41	22.67	16.61	16.78	17.81
Wt of moisture(Wm)(g)	2.91	3.78	2.67	2.91	3.20

may generate localized seepage and stability issues. In persistently saturated zones, especially around Ubeji and parts of Aladja, soils are likely weaker and more compressible, requiring careful geotechnical design considerations.

In summary, the Effurun–Warri area exhibits a shallow, spatially variable water table that significantly affects soil behavior. This theory is what we would be considering for predicting our water table level and its effect on the shear strength and long-term stability of structures within the study area.

Moisture content 100x(Wm/Wd)	14.99	16.67	16.07	17.34	17.97
ATTERBERG LIMIT (LIQUID AND PLASTIC LIMIT TEST)					
SAMPLE NO :2m	LIQUID LIMIT : 17.573%		PLASTIC LIMIT : 17.06%		
DATE: 27/10/2025	PLASTIC INDEX : 0.50%				
SOIL DESCRIPTION :			SITE : WARRI		
TYPE OF TEST:	LIQUID LIMIT				
No of blows/shrinkage %	41.00	35.00	21.00	18.00	14.00
Container no	R4	FIDE	BHD	O9	0I
Wt of wet soil and Container(g)	47.24	56.16	60.59	50.14	51.30

PLASTIC LIMIT

Container no	QTA	STO	N
Wt of wet soil & Container(g)	25.20	25.50	33.88
Wt of dry soil and container (g)	23.98	24.45	32.46
Wt of container(g)	16.00	17.47	21.99
Wt of Dry soil(Wd)(g)	7.98	6.08	10.48
Wt of moisture(Wm)(g)	1.22	1.05	1.42
Moisture content	15.29	15.04	13.56

4.5 ATTERBERG LIMIT DATA

Table 4.7 showing Atterberg Limit results at 1m depth.

Table 4.8 Table showing Atterberg Limit results at 2m depth.

Wt of dry soil and container (g)	43.81	51.45	55.10	46.08	46.48
Wt of container(g)	21.92	23.04	23.89	23.66	21.69
Wt of Dry soil(Wd)(g)	21.89	28.41	31.21	22.42	24.79
Wt of moisture(Wm)(g)	3.43	4.71	5.49	4.06	4.82
Moisture content 100x(Wm/Wd)	15.67	16.58	17.59	18.11	19.44
PLASTIC LIMIT					
Container no	03	37.00	DO4		
Wt of wet soil & Container(g)	31.28	26.03	38.86		
Wt of dry soil and container (g)	29.88	24.52	37.52		
Wt of container(g)	21.62	16.35	29.04		
Wt of Dry soil(Wd)(g)	8.26	8.17	8.48		
Wt of moisture(Wm)(g)	1.40	1.51	1.34		
Moisture content 100x(Wm/Wd)	16.95	18.48	15.80		

The Atterberg Limits tests (Liquid Limit, Plastic Limit, and resulting Plasticity Index) were conducted on the fine-grained fraction of the soil samples from the 1.0 m and 2.0 m strata. These limits define the boundaries of the soil's consistency states and are crucial for classifying fine-grained soils and assessing their behavior regarding volume change, compressibility, and strength.

Summary of Atterberg Limits

1. Low Plasticity Index (PI)

The most striking feature of the results is the extremely low Plasticity Index (PI) for both strata (2.06\% at 1.0 m and 0.50\% at 2.0 m).

i. Classification: According to the Unified Soil Classification System (USCS), soils with a PI of less than 4% are typically classified as having low plasticity. When combined with the high fines content previously observed (57.32% fines at 1.0 m and 48.04% fines at 2.0 m), this low PI suggests the fines are inorganic silt (ML) or organic silt (OL/OH), but with a predominantly silty (non-plastic) character, or that the material is non-plastic and non-cohesive.
ii. Behavior: Soils with very low PI are often described as non-plastic or borderline plastic. This indicates that the soil's cohesive strength and water-holding capacity are minimal, meaning its consistency does not change significantly over a wide range of moisture contents.

2. Extremely Low Liquid Limit (LL)

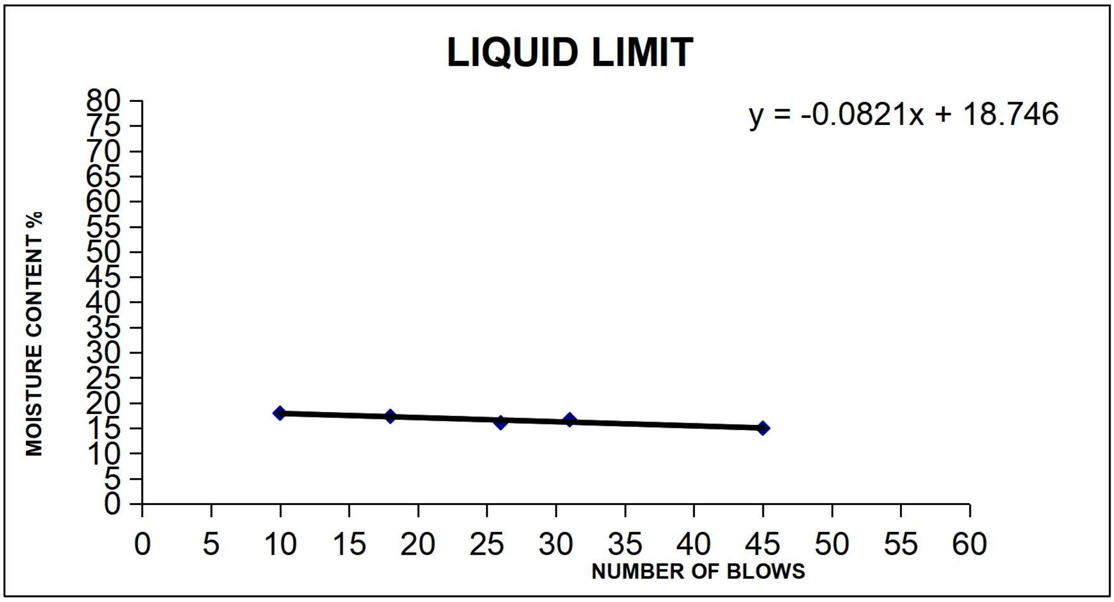
The Liquid Limit (LL) values (16.69% and 17.57%) are also extremely low for typical fine-grained soils. High LL values generally point to high compressibility. The low LL values support the conclusion of a silty or non-plastic fine-grained matrix. However, the low LL, combined with the previously established low Specific Gravity (G_s = 2.468 and 2.531), is a known characteristic of highly compressible, lightweight (likely organic) silts. These soils can have a deceptively low LL but still be highly compressible and problematic due to the low density of the organic solids, a factor that the LL test alone does not fully capture.

3. Comparison of Strata

i. The 2 meter has a lower PI (0.50%) than the 1.0 m stratum (2.06%). This suggests the 2.0 m layer is even less cohesive and more silt-like than the 1.0 m layer, reinforcing its coarse-grained classification (Silty Sand).
ii. The proximity of the LL and PL values (resulting in low PI) confirms the marginal classification and potential difficulty in accurately testing these near-non-plastic materials.

Conclusion on Consistency and Compressibility

The Atterberg Limits confirm the fine-grained fraction of both strata has a very low degree of plasticity. This indicates that the soil will have minimal cohesive strength and that its engineering behavior will be highly dependent on interparticle friction (in the 2.0 m sand) and void ratio collapse (in the 1.0 m silt). Combined with the low Gs and low dmax, the soil is likely a highly compressible organic silt/sand with limited ability to hold shape or strength upon wetting, posing a significant risk for immediate bearing failure and long-term settlement.



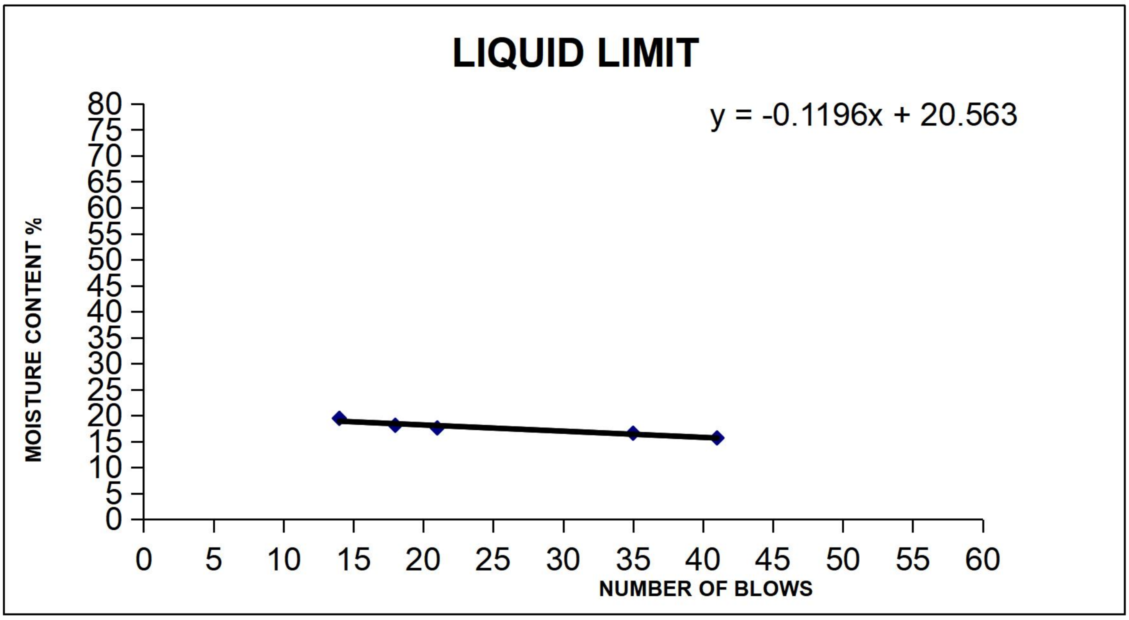


Fig 4.3 showing Atterberg limits graph at 1 and 2 meters below ground surface respectively.

4.6 TRIAXIAL TEST DATA

The Triaxial Compression Test was performed on the soil from the 1.0 meter stratum to determine its fundamental shear strength parameters: cohesion (c) and the angle of internal friction (∅). These parameters are derived from the failure envelopes (Mohr's circles) plotted against the normal stress (σ). Shear strength is the primary factor governing slope stability, bearing capacity, and overall foundation performance.

Analysis of Mohr-Coulomb Parameters

The failure envelope, represented by the linear regression line tangent to the Mohr circles, is expressed by the equation $\tau = c + \sigma \cdot \tan(\emptyset)$.

The equation provided on the chart, y = 0.3525x, represents the slope of the failure envelope, where y = τ (Shear Stress) and x = σ (Normal Stress).

i. Cohesion (c): The failure envelope appears to pass through the origin of the graph (or very close to it). This indicates that the cohesion (c) is approximately zero (0kPa). This finding is highly consistent with the Atterberg Limits results, which showed an extremely low Plasticity Index (PI = 2.06%), classifying the material as non-plastic or silty, which inherently possesses negligible cohesive strength.
ii. Angle of Internal Friction (∅): The angle of internal friction is derived from the slope of the envelope (tan(∅)).

$\tan(\emptyset) = 0.3525$

$\emptyset = \arctan(0.3525) = 19.4^{\circ}$

Engineering Significance and Soil Behavior

The determined shear strength parameters (0kPa and $\emptyset$ approximately 19.4°) definitively classify the 1.0 m stratum as a friction-dominant soil with very poor strength characteristics.

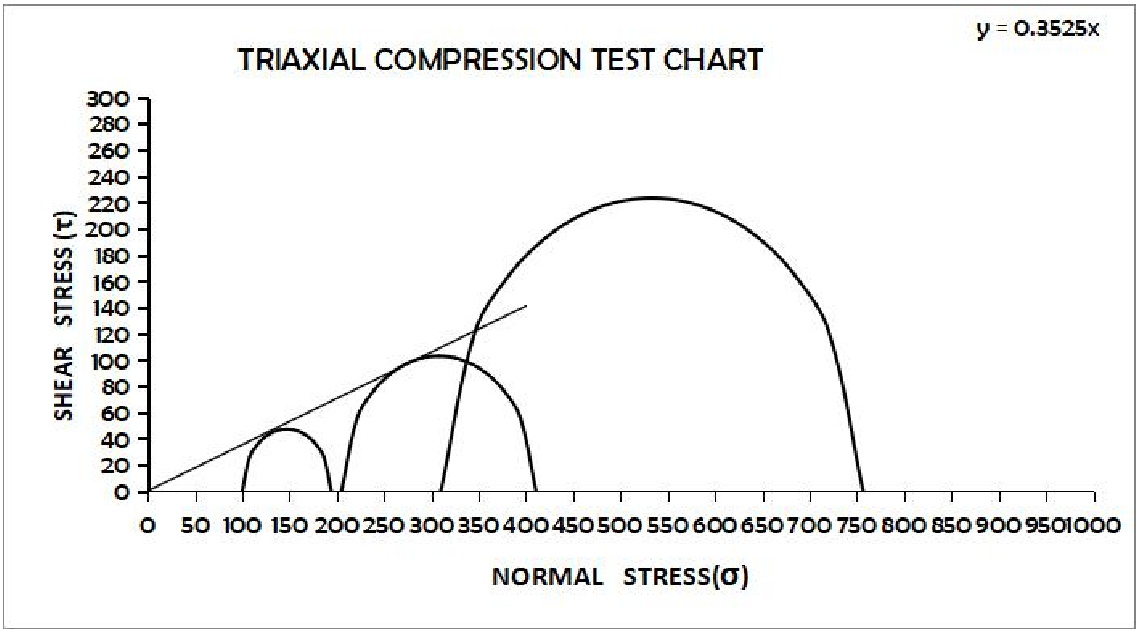
i. Low Frictional Angle: A friction angle of 19.4° is notably low for most engineering soils. Standard inorganic sands and well-compacted granular fills typically exhibit $\emptyset$ values greater than 30°. This low value confirms that the soil matrix is loose and highly compressible, possessing poor particle interlock and minimal load-transfer capability. This aligns perfectly with the previously established findings of low Specific Gravity (Gs = 2.468) and low Maximum Dry Density (dmax 1.76 g/cm^3), all of which point to a soil with a highly unstable structure, likely due to a high content of low-density organic or silty material.

ii. Implications for Bearing Capacity: Since the cohesion is negligible, the soil's strength is entirely reliant on the effective normal stress (\sigma'). The ultimate bearing capacity of any shallow foundation placed in this layer will be severely restricted by the low friction angle, confirming the layer is unsuitable for structural foundations.

Relationship to Water Table Variation (Objective i)

The fact that the soil's strength is purely frictional (c = 0) means the strength is directly proportional to the effective stress. A rise in the water table will cause a significant reduction in effective stress , which, in turn, will cause a direct and severe proportional reduction in the available shear strength and consequently the bearing capacity (Objective ii). This confirms the high sensitivity of the 1.0 m stratum to water table fluctuations.

The Triaxial Test results quantify the poor soil quality, transitioning the analysis from descriptive inference to quantitative assessment for the purpose of foundation design and risk assessment.



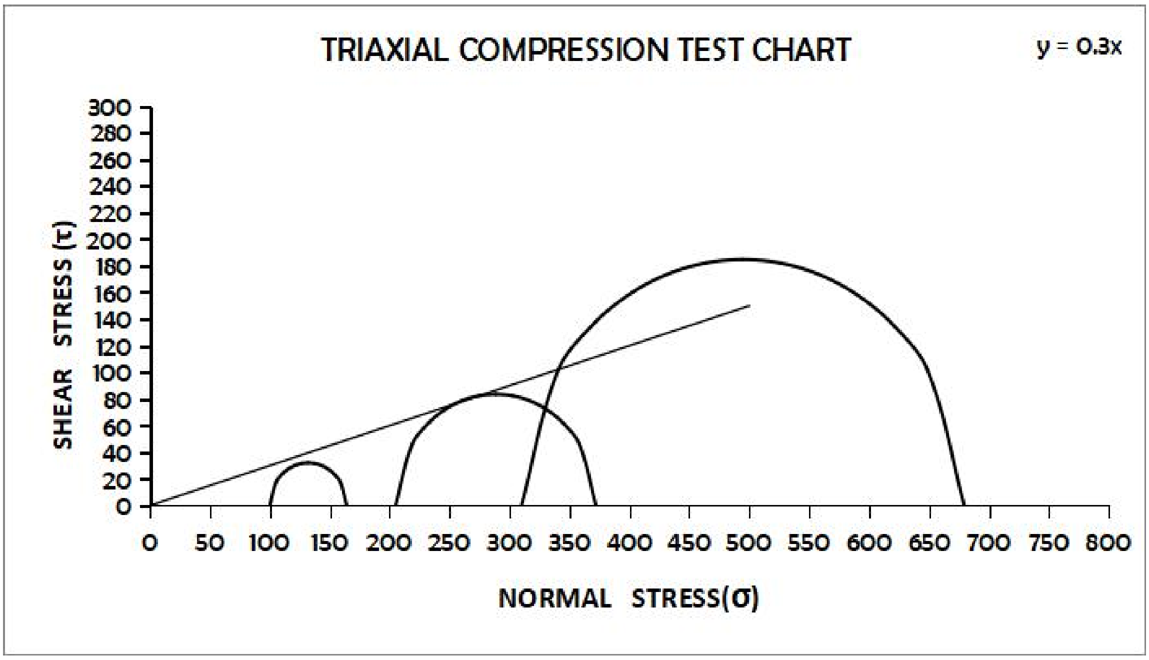


Fig 4.4 showing Triaxial chart at 1 and 2 meters below ground surface respectively

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATION

The laboratory tests Sieve analysis,Specific Gravity (Gs), Compaction, Atterberg Limits, and Triaxial Compression demonstrate that the soil profile from the 1.0 meter to 2.0 meter depth is critically weak and unsuitable for structural foundation support.

5.1 CONCLUSION

1. Soil Composition and Classification

Both strata are characterized by exceptionally low Specific Gravity (Gs: 2.468 at 1.0 m; 2.531 at 2.0 m), which is highly indicative of the presence of lightweight, likely organic, materials. This composition results in low Maximum Dry Density (dmax 1.76 g/cm^3 for both), confirming the soil's poor quality and high compressibility. The Atterberg Limits further confirm the low plasticity of the fines, consistent with a highly compressible organic silt/sand mix.

2. Shear Strength and Stability

The Triaxial Compression tests revealed the most critical issue: negligible cohesion (c as 0 kPa) in both strata, placing the entire load-bearing capacity solely on frictional resistance. The measured angle of internal friction ($\emptyset$) is critically low ($\emptyset$= 19.4 at 1.0 m; $\emptyset$= 16.7 at 2.0 m). This low friction angle confirms that the soil matrix is loose and highly unstable, resulting in extremely low ultimate bearing capacity.

3. Water Table Risk

The c as 0 condition means the soil's strength is 100% dependent on effective stress (σ). The high moisture sensitivity (near full saturation at dmax) coupled with the low frictional strength confirms that any rise in the water table will cause an immediate, severe, and proportional loss of effective stress and shear strength, resulting in a high risk of bearing failure and substantial volume instability.

5.2 RECOMMENDATION

These recommendations directly address the mitigation of the poor soil parameters (Objective iii) and provide necessary guidance for engineers (Objective iv).

1. Foundation Design and Ground Improvement

Due to the critically low shear strength (\phi < 20^\circ) and high compressibility of the entire 1.0 m to 2.0 m zone, the following is recommended:

i. Bypass or Replacement: The most reliable and effective technique is to remove and replace the entire unstable stratum (down to 2.0 m) with non-plastic, well-graded granular structural fill. The fill must be compacted to a verifiable dry density significantly higher than the natural dmax of 1.76 g/cm^3.
ii. Deep Foundations: For structures sensitive to settlement or carrying heavy loads, deep foundations (piles or piers) are the only secure option to bypass the weak organic stratum and transfer loads to a deeper, uncharacterized, competent bearing layer.
iii. Chemical Stabilization: As a secondary measure for stabilizing the existing fines, cement or lime stabilization can be employed to improve the Plasticity Index and increase the in-situ stiffness, though this requires careful laboratory verification for organic soils.

2. Site Characterization and Risk Management

Engineers must adopt the following parameters and risk management practices:

i. Risk Assessment: The primary engineering risk is long-term consolidation settlement (due to the inferred organic content) and shear failure during the wet season (due to water table rise).

ii. Field Control: Compaction field control must enforce the low Optimum Moisture Content (Wopt as 15-16%) meticulously, though the dmax of 1.76 g/cm^3 should not be accepted as a target density for imported structural fill.

iii. Further Testing: Given the high risk associated with organic content, consolidation testing and an organic content determination test must be performed immediately to accurately quantify the magnitude and rate of long-term settlement.

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APPENDIX

Fig 3.3 Soil Compaction test (Mishra, 2015)