Nigerian Journal of Applied Physics (NJAP)
ISSN Print: 3115-5871
ISSN Online: 3115-588X
DOI: https://doi.org/10.62292/njap-v2i1-2026-39
Volume 2(1), March 2026

Index Properties and Shear Strength Characteristics of Sub-Grade Soil Along Mariam-Awa Road, Southwest Nigeria

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ABSTRACT

This study evaluates the index properties and shear strength characteristics of subgrade soils along the Mariam–Awa Road, Ago-Iwoye, Southwestern Nigeria. Thirty-one soil samples (18 disturbed, 13 undisturbed) were collected from 18 locations at depths between 0.4m and 1.5m. Laboratory tests conducted include grain size analysis, specific gravity, natural moisture content, Atterberg limits, compaction, California Bearing Ratio (CBR), and shear strength analysis. Grain size curves were plotted for classification and interpretation. Results showed that fines passing the No. 200 sieve ranged from 20% to 61%. Only 7 of the 18 disturbed samples met the Federal Ministry of Works and Housing (FMWH) specification of <35% fines. Specific gravity averaged 2.66, consistent with the recommended standard. Natural moisture content (6.5%–10.7%) was found to be within the permissible limits. However, Atterberg limits exceeded FMWH thresholds, with liquid limits of 41%–47%, plastic limits of 13%–16%, and plasticity indices of 28%–31%, indicating high plasticity. Maximum Dry Density (MDD) ranged from 1.808–1.898 Mg/m³, and Optimum Moisture Content (OMC) ranged from 9.8%–16%. All MDD values were satisfactory, while 14 out of the treated 18 samples had acceptable OMC (≤15%). Unsoaked and soaked CBR values ranged from 46%–86% and 22%–70%, respectively. According to approved and permissible National and International Highway and Transport Standards, only 10 samples met the unsoaked CBR requirement (≥ 80%), and 11 met the soaked requirement (≥ 30%). In conclusion, the majority of soils (68%) along the route are suitable for subgrade use, though localized weak zones require treatment.

Keywords:
Subgrade Soil,
Index Properties,
Shear Strength,
Grain Size Distribution,
CBR,
AASHTO classification.

INTRODUCTION

Soils are natural dynamic bodies on the uppermost layer of the earth, exhibiting distinct organization of their mineral and organic components, including water and air. They formed in response to atmospheric and biological forces acting on various kinds of parent material under diverse topographic conditions over a period. Soil is a natural body, differentiated into horizons of mineral and organic constituents, usually unconsolidated, of variable depth; it differs from parent material below in morphology, physical properties and constitution, chemical properties and composition, and biological characteristics. Subgrade soil is the natural ground or prepared soil layer that serves as the foundation for pavement structures such as roads, highways, and airfields. It is the soil immediately beneath the pavement system and is responsible for bearing and distributing the load impossible by vehicular traffic. According to Das (2010), subgrade soil is defined as the bottom layer of the pavement system that provides the foundational support

to all the other layers above it, including the sub-base, base and surface course. Its properties determine how well it can withstand an imposed stresses without undergoing excessive deformation. The stability and strength of subgrade soil are critical to the overall performance and durability of any civil engineering structures. In road construction, the subgrade plays a fundamental role in distributing loads from traffic efficiently to the underlying ground weak or unstable subgrade can result in differential settlement, rutting, cracking and even total pavement failure (Huang, 2004). For this reason, detailed investigations of the geotechnical properties – especially the index and shear strength parameters-are essential during the planning and design stages of construction. The stability of subgrade soil determines the overall structural integrity of roads, buildings, and other engineering constructions. If the subgrade is weak, it can undergo excessive settlement, deformation, or even complete collapse under load, leading to failure of the superstructure. Poor subgrade conditions are often responsible for common structural problems such as cracking, rutting, and deflection in pavements and foundations. As Craig (2004) notes, proper evaluation and treatment of subgrade materials can significantly reduce maintenance costs and extend the service life of structures. Hence, a stable and well- prepared subgrade is essential for safe and durable construction. In foundation engineering, subgrade soil serves as the base upon which foundation systems are designed. The type and quality of subgrade soil determine the nature of the foundation to be adopted; whether shallow or deep foundations. For example, soils with low bearing capacity or high compressibility may require pile foundations, while stronger, more stable soils may permit shallow foundations. Tomlinson and Woodward (2015) emphasize that understanding subgrade properties helps in preventing differential settlement and structural instability. In areas like Ago-Iwoye, where lateritic soils predominate, assessing the strength and behavior of the subgrade is critical before foundation construction begins.
Shear strength is a soil's resistance to shearing forces, or its ability to resist sliding along internal surfaces. It is one of the most important parameters in soil mechanics and is influenced by factors such as cohesion, internal friction angle, water content, and soil type. According to Das and Sobhan (2018), shear strength is the maximum resistance a soil can offer before failure occurs due to shear stress. In engineering practice, understanding the shear strength of subgrade soil is crucial in assessing slope stability, bearing capacity, and the design of retaining structures. In the context of subgrade analysis, adequate shear strength ensures that the pavement or foundation built above does not experience shear failure under load. The shear strength of subgrade soil, defined by parameters like cohesion and angle of internal friction, is essential for assessing the stability and load-bearing capacity of the pavement structure (Terzaghi *et al.,* 1996). Poor subgrade conditions, including low shear strength and high plasticity, are leading causes of road failure, particularly in tropical regions like South-Western Nigeria, where weathering and seasonal variations affect soil performance (Ola, 1983).
Index properties and shear strength characteristics collectively determine how suitable a soil is for use as subgrade material. Index properties such as moisture content, Atterberg limits, particle size distribution, and specific gravity are used to classify soils and predict their engineering behavior. These parameters influence workability, compaction, strength, and volume change tendencies of the soil (Head, 2006). For example, high liquid limit and plasticity index values may indicate expansive clay soils that are prone to swelling and shrinkage, making them unsuitable for subgrade unless stabilized. Shear strength parameters (cohesion and angle of internal friction) directly affect the soil's load-bearing capacity and its resistance to deformation under applied stresses. High shear strength implies that the subgrade can sustain heavier loads without failure, which is essential for the stability of pavements and foundations. Conversely, low shear strength may result in instability and uneven settlement. In engineering design, both index and shear strength properties are used in modeling and selecting soil improvement techniques (Gidigasu, 1991). Several studies have been undertaken on the Index properties and shear strength of sub-grade soils. Adebisi *et al*., (2014); Ayotunde *et al*., (2015); Olorunfemi *et al.*, (2016); Anukwu *et al., (*2017); Ogundipe (2017); Adeniyi *et al.*, (2018); Sulaimon *et al*., (2019); Akinbile and Fapohunda *et al.,*(2019); Toriola *et al.* (2019); Ogunleye *et al*.,(2019); Bello *et al*., (2020); Idowu and Alabi (2020); Owolabi *et al*., (2020); Ayeni and Omotayo (2021); Afolabi *et al.,* (2021); Obasaju *et al.,* (2022); Adebayo *et al.*, (2023); Akinmoladun and Adedeji (2023); Olufemi *et al*., (2024). There have been poor road conditions and observable signs along Mariam-Awa road that necessitate testing for the index properties and shear strength characteristics. Testing was crucial because of the visual signs of distress most notably rutting, potholes and patching indicating that the subgrade or pavement layers are not supporting traffic loads; for road construction projects such as the Awa–Mariam Road in Ogun State, comprehensive evaluation of these properties provides the data needed to predict the behavior of subgrade soils under traffic loads and environmental conditions. From this study, the following objectives were obtained namely, the subsurface characteristics of the representative subgrade soil samples from selected locations; the index properties of the representative subgrade soils in the study area namely natural moisture content, specific gravity, Atterberg limits, and particle size distribution; grading and categorization the soil samples in the study area based on the AASHTO classification system; shear strength

parameters (cohesion and angle of internal friction) of the soil samples in the study area using direct shear tests and the recommendations of the appropriate ground improvement techniques based on the output of the investigation. Therefore, the index properties and shear strength characteristics of the subgrade soil along the Mariam-Awa road, Ago-Iwoye, South-West, Nigeria were determined with the consequent assessment of its suitability for road construction in the study area.

Location and Geologic Setting of the Study Area

The Mariam–Awa Road is situated within Ago-Iwoye, a prominent town located in the Ijebu North Local Government Area of Ogun State, in the southwestern region of Nigeria (Onakomaya et al., 1992; Adenuga et al., 2025; Ishola, 2026). Mariam-Awa road lies at the North-South boundaries approximately within latitude 6°57′01’’N to 6°57′24’’N and at the West-East boundaries within longitude 3°55′32’’E to 3°55′58’’E. The study area is accessible by network of both major and minor roads as well as foot (Figure 1).

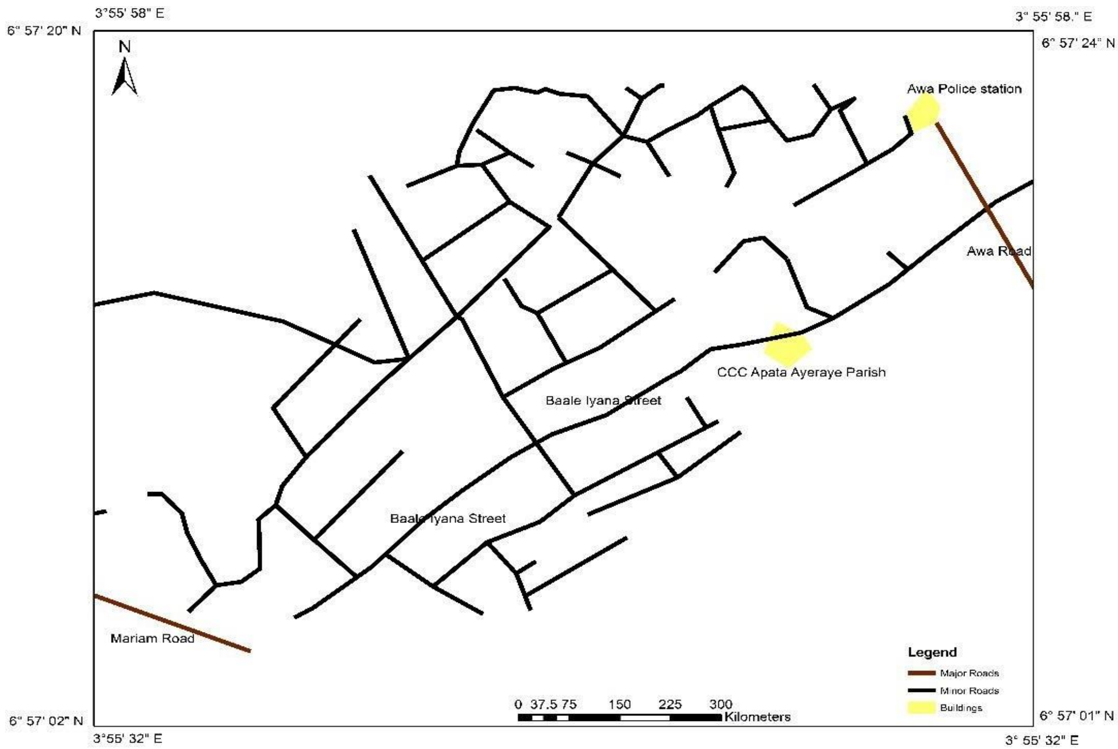


Figure 1: Map showing the location and accessibility of the study area

The raining seasons in the study area ranges between Mid-March and early November with double maxima rainfall whose peak occur between June to September while the dry season last from Mid-November to Early March with the month of December and January being relative dry . The vegetation is characterized by the rain forest type which acts as a pedogenic factor affecting the soil type in the study area. The study area is characterized by gently undulating topography, typical of the southwestern basement complex terrain (Oyawoye, 1972; Onakomaya et al., 1992; Omasanya and Akinbodewa, 2012). Elevations generally range between 70 and 150 meters above sea level, with subtle slopes that gradually dip towards surrounding lowland areas. This terrain is shaped primarily by prolonged weathering and erosion. The drainage pattern along the road is predominantly dendritic, reflecting the influence of the homogenous geologic structure and absence of major structural controls. However, poor drainage infrastructure along the road, coupled with inadequate slope stabilization, has led to surface runoff accumulation, erosion and waterlogging. These drainage issues have a direct impact on the stability of the subgrade soil, often leading to saturation, loss of strength and subsequent road pavement failure.

The Mariam-Awa Road traverses a region underlain by the Precambrian Basement Complex, predominantly composed of crystalline igneous and metamorphic rocks such as granite, gneiss, and schist (Ishola, S.A and Olufemi, 2024). (Figure 2). These rocks have undergone extensive weathering, resulting in the formation of lateritic soils rich in iron and aluminum oxides. The degree of weathering and the mineralogical composition significantly influence the engineering properties of these soils (Akintola et al., 2012; Folorunsho et al., 2013; Oladunjoye et al., 2023). The typical soil profile in this area comprises of topsoil which are rich in organic matter, generally unsuitable for engineering purposes; lateritic horizon which are highly weathered, iron-rich layer with varying degrees of plasticity and strength; saprolite which are Partially weathered rock retaining the structure of the parent rock but with reduced strength and fresh bedrock which are unweathered, competent rock providing a stable foundation. The engineering behavior of these soils is influenced by factors such as mineralogy, grain size distribution, and moisture content. For instance, soils derived from granite tend to be sandy with low plasticity, while those from schist may exhibit higher clay content and plasticity (Akintola et al., 2012; Oladunjoye et al., 2023; Adenuga et al., 2025).

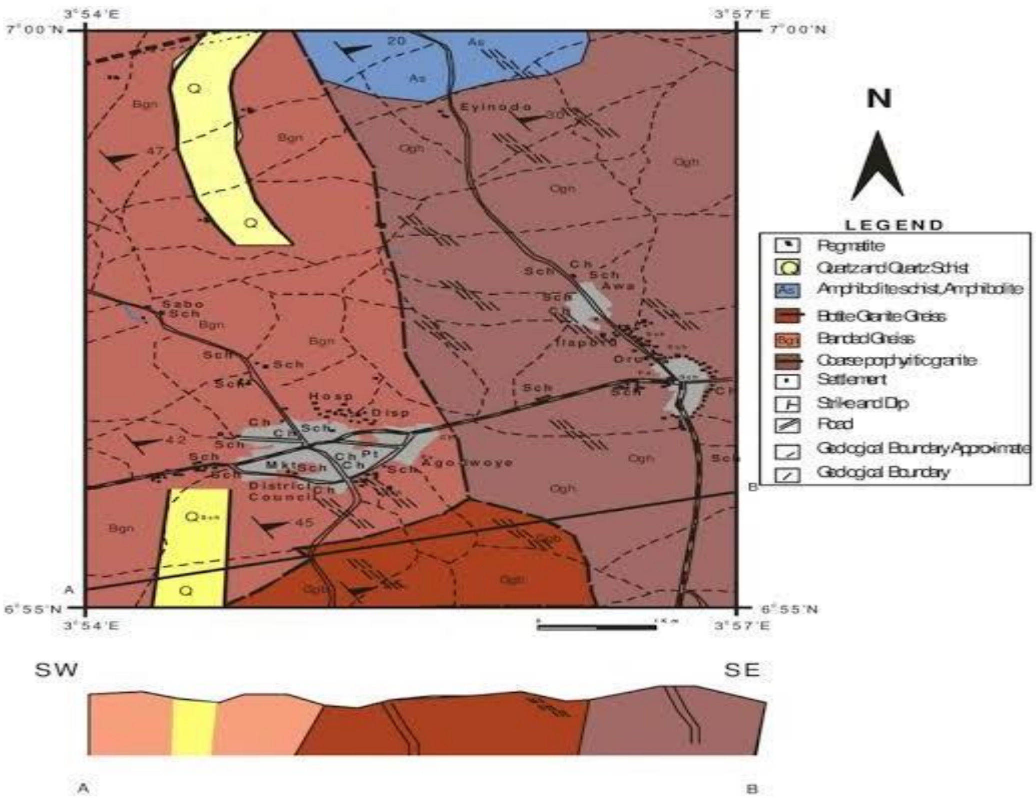


Figure 2: Map showing the Geology of Ago-Iwoye (Akintola et al., 2012)

MATERIALS AND METHODS

Theoretical Background

Sub-grade soil strength is defined by shear parameters (s), cohesion (c) and the internal friction angle (ϕ) which are related to index properties like plasticity index (PI) and moisture (w) via Coulomb-Terzaghi equation as expressed in equation 1 (Gao et al., 2019; Cai et al., 2022; Lacasse, 2023).

$s = c + \sigma \tan\phi$ (slope stability)

And $s = c_0 + \mu\sigma'$ (effective stress)} (1)

Coulomb-Terzaghi shear strength equation is widely cited in geotechnical literature to define the shear shear strength of soil based on effective stress, particularly in studies reviewing soil mechanics foundations and stability analysis, including seminal work by Terzaghi and subsequent analyses (Gao et al., 2019; Cai et al., 2022; Lacasse, 2023; Zing and Zhang, 2024); the equation combines cohesion (c) and the product of effective normal stress (σ') and the tangent of the friction angle (tanϕ) to define soil failure. (The key relationships include Plasticity Index, consistency index and empirical correlations where higher PI reduces shear strength in wet conditions while higher compaction increases shear strength and expression of PI is simplified in equation 2 (Assadi, 2014; Gao et al., 2021; Gao et al., 2023; Zing and Zhang, 2024).

$PI = LL - PL$ (2)

Where LL and PL represent Liquid Limit and Plastic Limit respectively (O'Kelly, 2021; Habte et al., 2024). The shear strength (s) of the sub-grade soil is primarily determined by its cohesion (c) and internal friction angle (ϕ) acting under normal stress (σ) as earlier expressed in equation 1. According to Mohr-coulomb equation (Chaparro et al., 2021; Islam et al., 2021; Du et al., 2022; Abanc et al., 2022; Mohammed et al., 2022; Chavez-Torres et al., 2024); it was further revealed that, cohesive soils typically clays are often analyzed using undrained strength where

$s = C_U$ (ϕ = 0) (3)

While for cohensionless soils that are typically sandy (sands and sand particles), we have

s = σ tanϕ (ϕ = 0) (4)

(Patel and Darji, 2013; Assadi et al., 2014; Mohammed et al., 2022; Chavez-Torres et al., 2024).

Showing that the limit conditions for c on both cases is zero. These properties dictates the potential behaviour and strength of the soil (Zheng and Hryciw, 2015; Rouse, 2018; Xiao et al., 2019; Zhao et al., 2021; He et al., 2021a; He et al., 2021b; Gao et al., 2025; ABG, 2026) where plasticity index (PI) as expressed in equation (2) where higher (PI) as expressed in equation 2 where higher PI generally indicates higher clay content and lower strength when wet. The Plasticity index as expressed in equation 2 were as higher PI generally indicates higher clay content and lower strength when wet (d). Liquidity Index (LI) is expressed in equation 5 as

$$LI = \frac{W-PL}{PI} \quad (5)$$

LI is a key parameter used in determining the state of soil such that when LI is greater than one (LI); it serves as an indication of liquid state (Cox and Bhudu, 2010; Tsomikos and Georgiannou, 2010; Sadrekarimi and Olson, 2011; Cabalar et al., 2013; Ghafghazi et al., 2014; Maki et al., 2014; Yang and Luo, 2015; Zheng and Hryciw, 2016; Afzali-Nejad et al., 2017. Also, consistency index (CI) is expressed in equation 6 as

$$CI = \frac{LL-W}{PI} \quad (6)$$

While the relative density (D_r) is expressed in equation 7 as

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\% \quad (7)$$

The expression in equation 7 is highly crucial for granular subgrade strength (Sujatha et al., 2018; Singh and Jamatia, 2020; Sujatha et al., 2020; Tana et al., 2020; Kannan and Sujatha, 2021; Qu and Zhu, 2021; Agarwal et al., 2021). Also, there are significant relationships between index properties and shear strength as often encountered in undrained shear strength(C_U), void ratio (e) and moisture content (w). In undrained shear strength(C_U) and Plasticity Index (PI), the empirical correlations for cohesive soils suggest

$$\frac{C_U}{\sigma/} = (0.11 + 0.0037 \times PI) \quad (8)$$

Equation (8) serves as Skemptons correlation, linking strength to plasticity but lower void ratio (e) is an indication of higher compaction and directly result in higher ϕ and c. also, as moisture content (w) increases, liquidity index increases leading to decreasing cohesion (c) and decreasing shear strength (s) (Terzaghi, 1967; Terzaghi et al., 1996; Zunrawi, 2016; Ju and Yin, 2020; Zhang et al., 2020a; Zhang et al., 2020b; Bodour et al., 2021; Zhang et al., 2021). Furthermore, the other related parameters that are of significant contributions are modulus of sub-grade reaction (k) and average shear stress (τ) consecutively displayed in equation 9 and 10.

$$k = \frac{q}{\delta} \quad (9)$$

Where q is the applied pressure and δ is the deflection thereby serving as a representation of soil's stiffness rather than ultimate strength while τ is expressed as an applied force over an area

$$\tau = \frac{F}{A} \quad (10)$$

It should be noted that in pavement design, the sub-grade soil is often assessed by California Bearing Ratio (CBR) which is an indirect measure of shear strength rather than direct shear testing (Zunrawi, 2016; Ju and Yin, 2020; Zhang et al., 2020b; Bodour et al., 2021; Zhang et al., 2021).

Equation for estimating the shear strength of saturated soil

Schnellmann et al., 2015 reported the Terzaghi worked on equation for shear strength of unsaturated soil in 1943 where he presented the Mohr-Coulomb shear strength theory in terms of effective stresses ($\sigma - \mu_w$) for a saturated soil as shown in equation (11).

$$\tau = c^1 (\sigma - \mu_w) tan\phi^1 \quad (11)$$

Where τ represents the shear strength of the soil (Kpa), c^1 represents the effective cohesion (Kpa), ($\sigma - \mu_w$) represents the effective stress (Kpa) and ϕ^1 represents the effective frictional angled (0^0). Also, the constitutive equations was proposed for unsaturated soils in 1977 by Fredlund and Morgenstern could be formulated as an extension of the constitutive equations for saturated soils (Fredlund et al., 2012; Schnellmann et al., 2015; Haliu, 2018; Cai et al., 2022; Sing and Zhang, 2024). The constitutive equations describe the relationships between shear strength or volume change with two independent stress state variables. The constitutive equations for unsaturated soils show a smooth transition to the constitutive equations for saturated soils when the degree of saturation approaches 100% or matric suction approaches zero as shown in equation 12 and 13.

$$\tau = c^1 (\mu_a - \mu_w) tan\phi^b + (\sigma - \mu_a) tan\phi^1 \quad (12)$$

$$c = c^1 (\mu_a - \mu_w) tan\phi^b \quad (13)$$

where ($\sigma - \mu_w$) represents the normal stress (Kpa) and ($\mu_a - \mu_w$) represents the matric suction (Kpa), c represents the total cohesion and is ϕ^ban angle that indicates the rate of increase in shear strength with respect to a change in matric suction (Goh et al., 2010; Fredlund et al., 2012).

Equation for estimating the shear strength of unsaturated soil

Goh et al., (2010) developed an equation to estimate the drying and wetting shear strength of an unsaturated soil. They assumed the shear strength of soil to be linearly proportional for matric suction where $tan\phi^b$ is equal to $tan\phi^1$at matric suctions lower than air-entry value (AEV), the relationship between shear strength and matric suction is non-linear where the calculated logarithmic difference of matric suction and the air entry value of the soil increases non-linearly when matric suction increases.

$\tau = c^1 + (\sigma - \mu_a)tan\phi^1 + (\mu_a - \mu_w)tan\phi^1$ (14)

$if\ (\mu_a - \mu_w) \leq$ AEV

Where $\phi^b = \phi^1$

$\tau = c^1 + [(\sigma - \mu_a) + AEV]\ tan\phi^1 + (\mu_a - \mu_w) - AEV]\ b\theta^c tan\phi^1$ (15)

$if\ (\mu_a - \mu_w) >$AEV

Where

X = [log $(\mu_a - \mu_w)$ − log AEV]

Y = 0.502ln (PI +2.7) −0.307

b_c = − 0.245 [ln [nd (PI + 4.4)]^2 + 2.114 [ln [nd (PI + 4.4)] – 3.522 (16)

Schnelimann et al. (2015) further developed equation to estimate the shear strength of an unsaturatred soil. They observed that the effective degree of saturation play significant role in controlling parameter for unsaturated soil property functions. They also concluded that the shear strength due to soil suction is interpreted to become zero at residual state.

$\tau = c^1 + (\sigma - \mu_a)tan\phi^1 + (\mu_a - \mu_w)(\frac{S - S_r}{I - S_r})tan\phi^1$ (17)

$for\ (\mu_a - \mu_w) < \phi$ and

$\tau = c^1 + (\sigma - \mu_a)tan\phi^1$ (18)

$for\ (\mu_a - \mu_w) \geq \phi$

Where ϕ represents the suction at residual state.

Field Data Acquisition

Field mapping took the duration of one (1) day and it involves the collection of subgrade soil samples; thirty-one (31) samples of subgrade soil were collected each at 10m intervals with which eighteen (18) were disturbed soil samples and thirteen (13) were undisturbed soil samples. The undisturbed samples were collected using core cutter while disturbed samples were collected in sample bags (Figure 3 and 4). The samples were collected at depths which ranged from 0.4m to 1.6m. The samples were carefully sealed to prevent moisture loss and also labeled for easy identification. The collected soil samples were further subjected to geotechnical testing in the laboratory where certain geotechnical parameters were measured namely, Specific gravity, Grain size distribution, Consistency limit, California Bearing Ration (CBR) and Compaction (West African Standard) adopting standard laboratory procedures. Core-cutter was used for the collection of the undisturbed subgrade soil samples; Polythene bag for the gathering subgrade soil sample; Ground Positioning System was used for the recording coordinates and elevation of each of sampled location and the trial-pit; Measuring tape was used for determining the depth and intervals of the trial pit; Digger was used for the excavation of the burrow pits; Hand-Auger was used for packing disturbing soil sample into polythene bag; Paper Tape was used for sealing core cutter to prevent moisture loss and also for the labeling of each subgrade soil sample; Hammer was for the driving of the core-cutter into the ground at each burrow pit while handheld Camera was used for the capturing and storage of images for each of the procedures of sample collections.

Figure 3: Field Photograph showing the collection of undisturbed soil sample in the study area

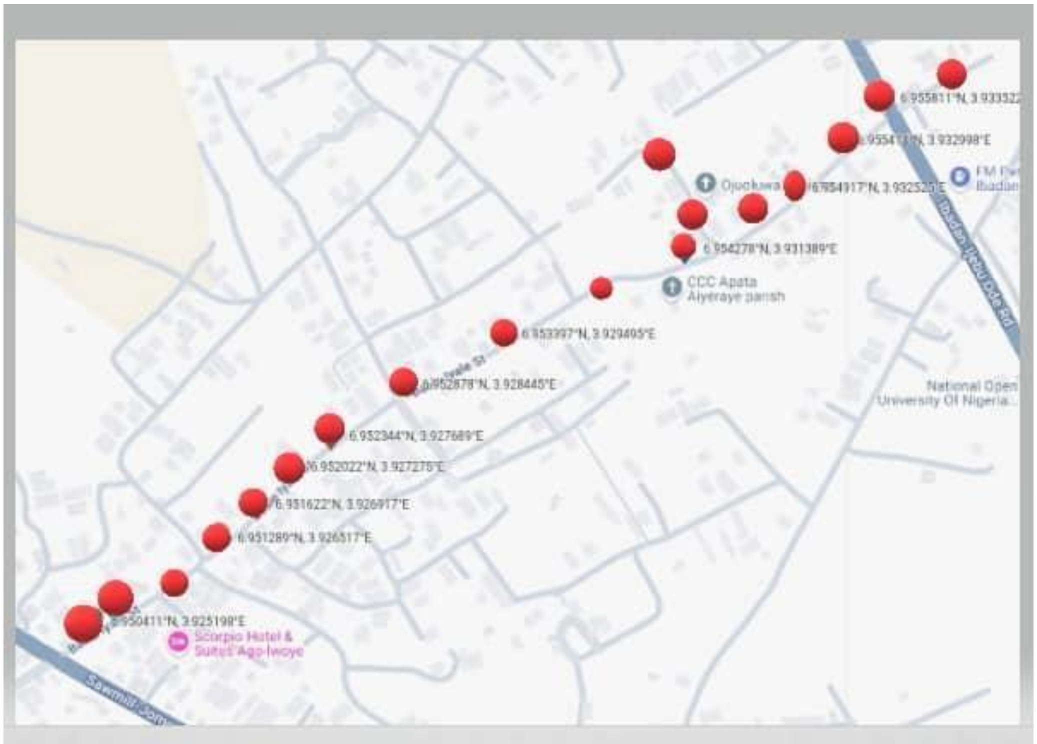


Figure 4: Map of the study area showing the sampling points with their respective coordinates

Laboratory analysis and analytical procedures

The collected soil samples were processed and analyzed for certain geotechnical parameters at the Department of Geosciences, University of Lagos, Nigeria. The parameters analyzed were grain size distribution, specific gravity, consistency limit, compaction, california bearing ratio and shear strength with all the testing procedures adhering to the approved standards set by the West African.

Grain-size Distribution

This is used to obtain the grading of the soil sample in terms of the description of the relative content of the entire various fractions such as boulders, pebbles, sand, silt, and colloid size. Since most of the physical properties of soils that are impacted by grain size are relatively small, this method is therefore utilized for the separation of soils into different fraction based on particle size distributions. For instance, permeability and the rate of deformation of saturated soils are largely dependent on the grain size while others are not (Gidigasu, 1994; Abrar and Hassan, 2025). The grain size distribution analyses notably consist of two (2) tests, which include Sieve Analysis and Hydrometer Technique.

Sieve Analysis and Laboratory procedures

The sieve analysis was used to determine the grain size distribution of soils that were greater than 0.075mm in diameter using sieves that were made of woven wires with square openings primarily applied to classifying the soil and grading aggregates (Figure 5). The result was used to determine the compliance of the particle size distribution with applicable specification requirements and to provide necessary data for controlling the production of various aggregate products and mixtures containing aggregate. The acquired data were utilized in developing relationships concerning porosity and packing. The laboratory tools used were stack of sieves with a cover, Mortar and pestle, a mechanical soil pulverized device, a balance sensitive to 0.1g, an Oven, Mechanical sieve shaker, and a Brush.

A representative oven-dried soil sample was pulverized and made as finely as possible, using a mechanical soil pulverizer. A soil sample of about 500g was collected with its mass Wo (g) determined**.** The sieves were stacked such that those with larger openings (lower numbers) were placed above those with smaller openings (higher numbers). A pan was placed under the last sieve (#200) to collect the portion of soil passing through it while ensuring that both the #4 and #200 sieves were always included in the stack. The sieves were ensured to be clean and when the soil particles were stuck in the openings, brush was used to poke them out. The prepared soil sample was poured from the top into the stack of sieves with the cover placed on it. Put the stack in the sieve shaker, affix the clamps, set a timer for 10-15 minutes, and start the shaker. Stop the sieve shaker and measure the mass of the sieve and retained soil.

Hydrometer Analysis and Laboratory procedures

The hydrometer analysis is one of the commonly used methods to accurately determine particle size distribution in a soil sample. Hydrometer is an instrument used to measure specific gravity of a fluid. The basis for hydrometer test is Stoke's Law for falling spheres in a viscous fluid in which the terminal velocity of the fall depends on the grain diameter, the densities of the grains in suspension and the density of the fluid. The grain diameter can then be calculated from knowledge of the distance and time of the fall. Also, the hydrometer determines the specific gravity (or density) of the suspension, which enables the percentage of particles of a certain equivalent particle diameter to be calculated.

The hydrometer analysis is useful principally for measuring particles with a grain diameter of 2mm or less (sands, silts and clays). Based on Stoke's law, it is known that sand size particles (0.05mm – 2mm) fall from suspension rapidly. Smaller silt-sized particles (0.002mm to 0.05mm) remain in suspension longer, but eventually fall from suspension while Clay-sized particles (less than 0.002mm) are small enough to remain in suspension indefinitely. Therefore, two hydrometer readings were necessary to determine particle size distribution. The first hydrometer reading gives a measure of the percent of silt and clay in suspension while the second reading gives a measure of the percent of clay in suspension. By subtracting the second reading from the first, percent silt can quickly be obtained. Also, by knowing that the sample must add to 100% the sand percentage can also be quickly determined.

The equipment and laboratory apparatuses used were, Soil Spatula, 100mLgraduated cylinder, Top load balance, R-O waterbottle, Sharpiemarker, Weighing paper, Cylinder plunger, Calculator, 250mL Erlenmeyer, 100mL graduated cylinders, Fahrenheit thermometer, Buoyoucose Hydrometer calibrated to 68F, Soil samples, 5% Calgon solution (Sodium hexametaphosphate-$Na_6(PO_3)_6$) was created by mixing 50 grams of Calgon powder into 1 liter (100mL) distilled water. 50 grams of dried, grounded, and sieved soil samples were placed in a 250mL Erlenmeyer flask. 100 mL of 5% Calgon solution was added to the sample, cap flask, and was swirled until the observed solution and soil were well mixed (several times). The mixture was allowed to rest overnight for a period of 12 hours required to be minimum in order to allow the solution to effectively disperse the soil separates (sand, silt, clay).

Soil-Calgon mixture was later transferred from the flask into an electric mixer cup. Water bottle was used to completely rinse all material from the flask into the mixing cup. The mixing cup was thereafter filled with water to about 3inches from the top while the cup was thereafter attached to a mixer and was consequently stirred for 3 minutes. The mixing cup was slowly removed and lowered so that the mixer propeller could just be above the water-level. A water-bottle was used to rinse all the soil mixture remaining on the mixing rod and propeller into the cup. The mixing cup of the soil, Calgon and water were emptied into 1000 mL graduated cylinder while remaining residue was completely washed out of the mixing cup with a water bottle into the graduated cylinder and the graduated cylinder was completely filled to 1000 mL mark while all soil materials were ensured to be below the 100 mL mark.

The plunger was inserted into the graduated cylinder and was allowed to gently mix the soil until a uniform suspension was obtained within a period of 30 seconds while ensuring that a clock with a second hand was really made visible with a clean hydrometer nearby. As soon as the plunger was removed, the exact time was checked and consequently recorded. The plunger was quickly rinsed into the graduated cylinder using a little water as as fast as possible, the hydrometer was gently inserted into the suspension. The plunger was later removed, read and recorded after a period of 40seconds had elapsed from the time the last 40-second hydrometer reading. The hydrometer was rinsed and wiped wipe to dryness, and was put back in its protective case. To correct for temperature effects and density of the dispersion agent, 100 mL of 5% Calgon and 880 mL of distilled water were mixed in a clean 1000 mL graduated cylinder and were allowed to rest for two (2) hours. (NOTE: 100 mL + 880 mL = 980mL); the missing 20mL accounted for the approximate volume occupied by 50 grams of soil). After the expiration of two (2). Another hydrometer reading from the soil solution and the hydrometer readings were recorded in 2-hour after which the hydrometer was removed and consequently cleaned.

The clean hydrometer was placed into water-calgon solution and blank hydrometer reading was recorded. This allowed for hydrometer calibration to account for temperature differences. Likewise, the thermometer was placed into water-calgon solution and temperature was read. When the temperature was above 68°F, 0.2 units was added to blank hydrometer reading for each degree above 68°F. When the temperature was below 68°F, 0.2 units was subtracted from the blank hydrometer reading for each degree below 68°F; this was recorded as the corrected hydrometer reading. Corrected blank hydrometer reading from 40 seconds and 2 hours hydrometer reading to calculate calibrated 40 seconds and 2 hours reading. The percentage of sand, silt and clay in the soil sample using the following equations was mathematically determined as

% clay = (calibrated 2-hour reading) × (100/sample weight) % Silt = (calibrated 40 second reading) × (100/sample weight) – (% Clay), % Sand= 100 – (% silt + % clay).

Figure 5: Photograph showing the set of sieves used for the determination of coarse grain analysis

Specific Gravity

The specific gravity of the soil is the ratio of the mass of a given volume of soil to the mass of an equal amount of water at 4. Specific gravity has been identified as an index for the identification and evaluation of lateritic soil. It correlates well with their mechanical strength features (De Graft-Johnson Model). Specific gravity can be influenced by parent rock composition, iron content, particle size distribution and oil sample position in soil profile. The soil has a specific gravity between 2.66 to 2.69. The lists of the apparatuses and materials used were Pycnometer (Figure 6), Weighing balance, Wash bottle, Distilled water and Desiccator.

The procedures follow thus: the soil sample was sieved through a 2.36mm sieve size, thoroughly cleaned, dried and measured with the pycnometer (M1). The test used an 100g of soil and the pycnometer was consequently filled with 100g of dried dirt recorded as (M2), this was followed by the addition of roughly two-third of distilled water to saturate the soil. The pycnometer was placed in a desiccator after letting the saturated soil to sit and settle for 24hours after which the desiccator was sealed. A vacuum of around 15kpa was then applied for a period of about 10 minutes as to remove the air. The pycnometer was filled with distilled water and the combined mass of water, soil and pycnometer (M3) was then recorded alongside the mass of distilled water and pycnometer (M4). The specific gravity of the soil was determined using the relation in equation 19:

Specific Gravity of soil by Pycnometer

$$G = \frac{(M2-M1)}{(M2-M1)-(M3-M4)} \quad (19)$$

Where, M1=mass of empty pycnometer

M2=mass of pycnometer with dry-soil

M3=mass of the pycnometer a soil and water

M4=mass of pycnometer filled with water only.

G=specific gravity of soil.

Consistency Limit

Consistency limit is used to filter particles, such as clay and silt, which pass through a 200mm sieve size. These fine particles cannot be fully characterized by grain size analysis or hydrometer analysis alone. The consistency limit of soil varies with its state form solid, semi-solid and plastic to liquid. There are different types of consistency limit which include the basic limit which entails both plastic limit and liquid limit then the derived limit from the name derived they are obtained from the basic limit test and they include Plasticity index, liquidity index, toughness index, flow index and consistency index. Plasticity index is the range of moisture content over which the soil remains plastic and is calculated as the difference between Liquid Limit (LL) and the Plastic Limit (PL).

Plastic Test Limit

The plastic limit of a fine soil is the moisture content at which the soil begins to break apart or crumble. The plastic limit is reached when the soil rolled into threads and is about to crumble. This moisture content can be expressed in %. The apparatus for plastic limit test is shown in Figure 7. The list of laboratory materials and apparatuses used were Weighing balance, Moisture can, Glass plate, Distilled water, Spatula and Mixing bowl. The procedures followed that, 15g of oven dried soil was mixed with distilled water and was edallow to rest for approximately 24 hours before conducting the test. The fines were gradually rolled into thin threads of approximately 3mm in diameter, usually done on a glass plate. The mass of the can was measured and recorded, then the soil that had reached its crumbling state was emptied into the can and placed in an oven for 24 hours. The moisture and the rolled wet soil that has reached it plastic limit were weighed and recorded. After the expiration of 24 hours, the mass of the moisture can with the dry soil was weighed and recorded in order to measure the water content. The average value for the three test was later utilized as the plastic limit of the soil (Figure 7 and 8).

Figure 6: Photograph of a calibrated pycnometer used in the determination of specific gravity

Figure 7: Apparatus for Plastic limit test

Liquid Limit

Liquid Limit (LL) of a soil is the moisture content at which the soil flows under its own weight i.e the moisture content of a fine soil when it passes from plastic state to liquid state. The lists apparatuses used were Casagrande machine, Grooving tool, Weighing balance, and Moisture tin. The process involves first, the placement of wet soil into the Casagrande cup, creating a groove in the center using a grove tool with the number of bowls required to close the groove. Each soil sample underwent four tests. The first sample took 42, 32, 22 and 17 blows respectively, the second sample took 42, 32, 23 and 17 blows respectively, the third sample took 43, 33, 23 and 17 blows respectively, the fourth sample took 43, 32, 23 and 17 blows respectively, and so on for the remaining samples. The reduction in blows during the successive test was attributed to adding distilled water after the first test, which accelerates the groove. After testing, the moisture can was weighed along with soil closed in Casagrande cup to determine the water content. The samples were then dried in an oven for 24 hours and reweighed. By plotting the water content against the number of blows, the liquid limit can be identified on the graph.

Figure 8: Apparatus for consistency limit test

Compaction

There are three types of compaction test, but the method adopted in this study was the West African Standard, which serves as the most recent technique. Soil compaction involves reducing the volume of soil by expelling pore air and water, resulting to a closely packed soil particles. It can be calculated using the expression in equation 20.

$$\text{Dry Density (pd)} = \frac{\text{Wet Density (pw)}}{1+\text{Moisture content (w)}} \quad (20)$$

Where pd represents the dry density of the soil, pw represents the wet density of the soil (including water content) and w represents the moisture content of the soil expressed as decimal. The test is valuable for assessing subgrade soils. The compaction test produces a compaction curve from which Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) can be determined. The energy used in the laboratory soil compaction test was influenced by five parameters: the weight of the rammer, the height of the fall, the number of the blows per layer and the volume of the mould. According to Federal Republic of Nigeria General Specification for Roads and Bridges in 1997, the West African method uses a 4.5kg rammer dropping from a height of 450mm, comparing soil in five layer within a

mould of 2210cm volume. Figure 9 shows the mold and rammer.
The list of apparatuses used in this study were Mould diameter of 10.2cm and height of 11.7cm. Rammer weighing 4.5kg, No 4 sieve size, Steel-straight edge, Moisture containers, Graduation cylinder, Mixer and Oven. For this study, the procedures involved using a 4.5kg rammer and 949cm³ (1×10^{-3} cm³) mould for the test. A measured 3kg air dried soil sample, which was first passed through a 20mm sieve size was mixed with approximately 5% water by total mass at the initial stage. The water content was then increased gradually to 7%, 9%, 11% and 13% based on the water retention capacity of each sample. The soil was later compacted in five layers, using 25 evenly distributed and at a height drop of 450mm (0.450m). The dry density of soil was plotted against the moisture content, and the Maximum Dry Density (MDD) and the Optimum Moisture Content (OMC) were consequently determined from the peak of the graph.

California Bearing Capacity (CBR)
The CBR test measures and compare the load bearing capacity or strength of soil with a maximum particle size of 20mm. The CBR test is essential for evaluating the subgrade strength of roads and pavements. The CBR formula is expressed in equation 21.

$$\text{CBR} = \frac{\text{Load of Soil}}{\text{Standard Load}} \times 100 \qquad (21)$$

CBR test is a form of strength test which involves a plunger passing through the soil sample by 2.5mm under a load of 13.24kN and by 5.0mm under a load of 19.96kN. The CBR value is typically expressed as %. The two types of CBR test were carried out; Unsoaked CBR and Soaked CBR. The unsoaked CBR test was carried out on a soil sample a few days after it has been compacted and soaked in water. Soaked CBR values was found to be lower than the unsoaked CBR value and this indicates the reduction in the soil strength when in contact with water. The CBR test is significant in determining the extent of reduction in soil strength upon water contact, the water absorption capacity of the soil and its swelling potential when exposed to water. The lists of apparatuses used were Loading machine which can operate at constant rate of 1.255mm per minute, Cylindrical moulds (150mm diameter and 175mm height), Baseplate with thickness of 10mm, collar of 50mm height, Cylindrical plunger (50mm diameter, at least 100mm long), Dial gauge (for measuring penetration values), Proving ring (for measuring load), Metal disc spacer, Annular metal weight and slotted weight, Compaction rammer, Surcharge weight of 2.5kg and 147mm diameter, IS sieve of 20mm, Weigh balance and Coarse filter paper.
The procedure first involved the preparation of remoulded soil sample at proctor maximum dry density, which was required for the CBR test. The sample was compacted dynamically which began by sieving approximately 5kg of soil through a 20mm IS sieve. The soil should pass through the 20mm IS sieve but be retained on the 475mm IS sieve. For heavy compaction, the soil was compacted in five (5) layers each receiving 55 blows from a rammer weighing 4.5kg and dropping from a height of 450m, after compacting each layer, the process was repeated until the fifth layer was compacted, the extension collar was removed, and the excess soil was trimmed with a straight edge. The mould containing the soil sample was placed on the testing machine and 2.5kg annular metal weight was positioned on the soil surface. The penetration plunger was set in contact with the soil surface, a 4g load was applied and a 2.5kg slotted weigh was placed on the oil surface so that the total surcharge weight equal 5kg. The dial plunge was set to zero, while the load was applied at a rate of 1.25mm per minute. The load penetrations of 0.00, 0.25, 0.50, 0.75, 1.00, 1.25, 1.50, 1.75, 2.00, 2.25, 2.50, 2.75, 3.00, 3.25, 3.50, 3.75, 4.00, 4.25, 4.50, 4.75, 5.00, 5.25, 5.50, 5.75, 6.00, 6.25, 6.50, 6.75, 7.00, 7.25 and 7.50mm were sequentially recorded. This procedure was repeated for the soaked tests; where the sample was soaked with the surcharge weight in a water tank for 4 days before being subjected to the CBR test.

Figure 9: Photograph showing mold and rammer used for compaction test

Soaked soil is the more suitable and conservative standard for subgrade design as it stimulates the worst-case scenario (saturation) of the subgrade materials while unsoaked soil tests provide higher strength values, soaked CBR tests are generally used to determine the minimum required subgrade capacity to ensure stability and prevent failures (Sorum et al., 2025). Subgrade soils are often exposed to water from heavy rainfall, high water tables or poor drainage reducing their strength. Testing the soil in a soaked condition for four (4) days provides a safe, durable design that holds up under these field conditions. The unsoaked CBR value is generally higher but is only reliable if the pavement will never experience moisture infiltration, which is rare in practice. It is only suitable for dry environments. Soaked soil properties ensure long-term performance and reduced maintenance, making it the preferred choice for defining subgrade suitability (Khatti and Grover, 2023; Quanbin et al., 2023; Sorum et al., 2025).

Triaxial Strength and Direct Shear Strength

Triaxial strength refers to the maximum shear resistance of a soil sample measured while it is subjected to three-dimensional stresses-a confining pressure (minor principal stress σ_3) and an additional axial vertical stress (deviator stress, Δ_σ or σ_d). This test method simulates real-world in situ stress conditions better than most other laboratory tests, determining both the cohesion (c) and the angle of internal friction (ϕ). The triaxial test determines the shear strength of a soil sample. In this test, a cylindrical soil sample is placed in a rubber membrane and subjected to a uniform hydrostatic pressure (confining pressure). A vertical deviatoric stress is applied until the soil fails. Unlike the direct shear test, the failure plane is not predetermined; the soil is allowed to fail on its weakest plane or bulge. It measures the principal stresses ($\sigma_1 = \sigma_3 + \Delta_\sigma$) and minor principal stress (σ_3) at failure from which the shear strength is calculated. The test also allows for strict control of drainage allowing for the measurement of pore water pressure. While both tests determine shear strength parameters, the triaxial test is considered more accurate for critical design due to its stress control making it highly significant in failure mechanism where direct shear forces a failure plane horizontally, triaxial allows for natural failure along the weakest plane, generally resulting in more accurate parameters; stress state, in direct shear, the distribution is not uniform across the shear box, leading to stress concentration. The triaxial test provides a uniform stress distribution across the failure plane; drainage and pre pressure evaluation, triaxial tests allow measurement of pore water pressure (u) and volume changes, enabling effective stress analysis (c^l, ϕ^l), which is not possible in standard direct shear tests. Comparative analyses from research studies have shown that direct shear tests often yield higher cohesion values but both tests generally give similar effective shear strength parameters for standard soils **(**Quanbin et al., 2023; Sorum et al., 2025).

). Basically, there are three different types of triaxial tests namely the unconsolidated undrained (UU) which is quick, suitable for low permeability soils (clay) in short term loading; the consolidated undrained (cu), where though consolidation occurs but not during shearing, it is principally utilized for analyzing rapid drawdown of slopes, and the consolidated drained (CD) which ios a slow test that makes allowance for complete consolidation and dissipation of pore pressure. It is used for long-term stability analyses.

Procedures for obtaining Triaxial Strength

The test was conducted on a cylindrical sample sample with a height to diameter ratio of 2:1. The undisturbed specimen was prepared with its dimension and weight measured. The sample was assembled by placing the sample on the pedestal within the triaxial cell surrounded with a rubber membrane and secured with O-rings. The drainage valves were closed, the top cap and loading ram were put in place with and the cell was filled with water. The next stage was the testing stages which comprises of stage 1 and stage 2. Stage 1 involves the determination of confining pressure (Consolidation) where a desired cell pressure ($\boldsymbol{\sigma_3}$) was applied to the water in the chamber. When consolidation was allowed (CD/CU), the drainage valve was opened to let out escaping pore water until volume changes cease but when no consolidation (UU) was allowed, the drainage valves were kept closed. Stage 2 involves the determination of deviator stress (shearing) where a vertical axial stress was applied (Q) was applied to the sample through the piston at a controlled rate of strain. For the drained (DD), the drainage was kept open to allow water to escape while for the undrained (UU/CU), the drainage was kept closed and pore pressure was measured. The set-up was left to continue loading until failure occurs (failure is defined as the maximum deviator stress or 20% axial strain). The test was repeated at three times on identical samples using different confining pressures to construct the Mohr circles and draw the failure envelope. The data were analyzed by computing the corrected area of the specimen at failure due to bulging, computation of the deviator stress as seen in equation 22.

$$\Delta_\sigma = \frac{\boldsymbol{P}}{\boldsymbol{A_{corrected}}} \qquad 22$$

Mohr's circles at failure ($\boldsymbol{\sigma_1 and\ \sigma_3}$)were consequently plotted and the failure envelope was drawn to determine cohesion (c) and friction angle ($\boldsymbol{\phi}$).

The shear strength characteristics of the subgrade soils were determined using the Direct Shear Test, in accordance with the procedures specified in BS 1377: Part 7 (1990). Aside from BS 1377 (British Standards), the primary international standards for determining the shear strength characteristics of soil are ASTM International (American Society for Testing and Materials) and ISO (International Organization for Standardization), both of which are commonly utilized in geotechnical engineering to define cohesion (c) and friction angle (ϕ). Other key standard group testing method are differentiated and specified for direct shear test which are ASTM

D3080/D3080M-23 for direct shear test of soils under consolidated drained conditions; ISO17892-10-2018 for geotechnical investigation and laboratory testing of soil; ISO/TS 17892-10:2004 for former version of the ISO direct shear test standard and IS 2720-13 for Indian Standard Methods of tests for direct shear test for soils. The triaxial shear test standards are namely ISO 17892-8: 2018 for consolidated undrained triaxial compression tests on water saturated soils; ISO 17892-9: 2018 for consolidated triaxial compression tests on water saturated soils; ASTM D2850 for unconsolidated-undrained triaxial compression test on cohesive soils and ASTM D4767 for consolidated drained triaxial compression test on cohesive soils. The interface shear (Soil Geosynthetic) standards, ASTM D5321/D5321-12 for determining the shear strength of soil geosynthetic and geosynthetic-geosynthetic interfaces by direct shear; ASTM D6243 for determining the shear strength of geosynthetic clay liners (GCLs) and EN ISO 12957 for geotechnical investigation and laboratory testing of geosynthetics. ASTM D2573/D2573M for field vane shear test in cohesive soil; ISO 22476-16:2024 for geotechnical investigation and testing, field testing-part 16 for borehole shear test (philcometer test) and ASTM D5778 for electronic friction cone and piezocone penetration testing of soils (CPT). Eurocode code 7 of BS EN 1997-1 and 2 which provides the framework for geotechnical design and determining characteristic shear strength values (c^l, ϕ^l) based on the limit state design approach. Other laboratory shear tests include ASTM D6528-17 for consolidated undrained direct simple shear testing of fine-grained soils; ASTM D2166/D2166M for unconfined compressive strength of cohesive soil and ISO 17892-7:2018 for unconfined compression (SPTL, 2010).

This test provides the two fundamental shear strength parameters: cohesion (c) and angle of internal friction (φ), which are essential for analyzing the stability and bearing capacity of soil materials used in geotechnical structures such as pavements and embankments. The lists of apparatuses used were Direct shear test apparatus, Shear box with a square cross-section (usually 60mm×60mm), Proving ring or load cell, Dial gauge or digital displacement transducer, Weights and loading frame, Tamper or spatula, Balance (for weighing soil), Sample preparation tools (e.g. cutter, tray). The procedures involved collection of Disturbed soil from the field which were first air-dried, then pulverized and passed through a 4.75mm sieve. The soil was mixed with water to its natural moisture content and compacted into the shear box in three layers to ensure uniform density and to simulate field conditions as closely as possible. After the soil specimen was placed in the shear box, it was carefully leveled and a porous stone was placed above and below the sample to allow drainage. The test was performed under consolidated drained conditions (CD test), where the sample was allowed to consolidate under a normal load before shear was applied.

Normal stresses of varying magnitudes (namely 50,100, 150, and 200 kN/m²) were then applied one at a time to different samples to replicate field stress conditions. After consolidation under each normal load, horizontal shear force was gradually applied at a constant rate until the specimen failed along the pre-defined shear plane. The maximum shear force at failure was recorded along with the corresponding displacement. The maximum shear stress (τ) for each normal load (σ) was calculated from the recorded data. These values were plotted on a graph of shear stress versus normal stress. The Mohr–Coulomb failure envelope was drawn, from which the cohesion (c) and angle of internal friction (φ) were determined as the intercept and slope of the best-fit line, respectively. The shear strength parameters were computed using the Mohr–Coulomb failure criterion expressed in equation 23.

$$\tau = c + \sigma \tan\phi \quad (23)$$

Where τ represents the shear strength, c represents cohesion, σ represents the normal stress and ϕ represents the angle of internal friction. These parameters were essential for the understanding of the resistance of the subgrade soils to shearing forces, which is a critical factor in determining slope stability, pavement performance, and bearing capacity.

Quality Control and Precautionary Measures in Laboratory Testing

Quality control (QC) measures and strict adherence to precautions in laboratory testing of sub-grade soil are essential to ensure accurate design parameters for pavement construction during the testing for index properties (Atterberg limits, sieve analysis) and shear strength (CBR, Triaxial, Direct Shear). Quality Assurance was consequently observed in the following areas.

Sample Preparation**:** the acquired disturbed samples were properly identified and the undisturbed samples from the thin wall tubes were carefully handled to avoid disturbance.

Calibration: the equipment used such as the balances, oven temperatures, compression machines and CBR apparatus were calibrated according to standard protocols after the order of ASTM/AASHTO).

Moisture-Density Control**:** it was ensured that the compaction tests (Standard/Modified Proctor) was used to determine the optimum moisture content (OMC) and Maximum Dry Density (MDD) to guide specimen preparation.

Replication**:** a minimum of three (3) specimens was tested to ensure consistency in results especially for the tests undertaken for shear strength.

Standardization: the established test methods like IS: 2720, and AASHTO were followed in order to ensure consistency in the observed results.

Observed Precautions in Laboratory Testing
Precautionary measures undertaken during the testing procedures in this study were

Sample Disturbance**:** undisturbed samples were handled with extreme care in order to prevent the

Identified changes in structures of some samples especially sensitive clays

Sample Preparation Environment: trimming of samples for shear tests were performed in a humid room to prevent moisture loss while ensuring that the trimming tools used were well sharpened and cleaned.

Sieving**:** during the grain size analysis, the samples were thoroughly washed over the 75 micron sieve to ensure that all the fines were removed particularly for fine-grained soils.

Soaking (CBR Test)**:** it was ensured that the samples for the CBR test were soaked for 96 hours to stimulate the worst-case moisture conditions in the field.

Direct Shear/Triaxial Testing**:** in Rate of Loading, the shear load was applied at a very slow rate for clayey soils to allow for pore water pressure dissipation especially in drained tests; in Porous Stones, it was monitored to ensure that the porous stones were made clean and saturated to permit drainage in consolidated tests; in Failure Plane, the shear plane was predetermined and the specimen was properly placed with the screws removed only after applying the normal load in direct shear tests; in consistency Limits, Casagrande apparatus for liquid limit was calibrated to a 10mm height of drop to ensure accurate results (ASTMD3080/3080M(2011); ASTMD5311/D5311(2013); ASTMD6467(2013).

Pitfalls and Mitigation
Common pitfalls and Mitigation were avoided in this study in order to avoid possible errors and ensure accuracy.

Error in Moisture Content: allowing samples to dry out before weighing or improper oven temperature regulation was avoided in order to ensure that significant errors in index properties were avoided. Non-utilization of Proper Moisture: When preparing compacted samples, failing to maintain the intended moisture density was avoided in order to ensure that the intended moisture density yield reliable shear strength values. Misalignment of shear box: In direct shear tests, the misalignment of the shear box was avoided because it can cause increased resistance, leading to an overestimation of shear strength. Proper Documentation: Proper recording of the soil's physical characteristics such as colour and consistency during testing were upheld because of its utmost significance in identifying potential issues with the sub-grade (ASTMD6528(2017) ASTMD4767(2020); ASTMD7181(2020) .

RESULTS AND DISCUSSION

Natural Moisture Content
Moisture content affects the strength of the soil and the ability of the pavement to support the load. Natural moisture content indicates the level of water in an area. The natural moisture content of the soil in this study ranged from 6.5% to 10.7% and this can be seen in Table 1. The natural moisture content ranges as specified by Federal Ministry of Work and Housing (FMWH, 1997) for engineering construction is 5% to 15% and this is used to classify soils with low moisture content. High moisture content reduces the strength of the soil, whereas low moisture content increases the strength of the soil. The soil samples ranged from 9.8% to 16% indicating that the soil in the study area is suitable for road construction and good sub-grade material; since it falls within the FMWH permissible limits; implying that the soil is of low moisture content.

Grain-size Distribution
The grain size distribution is an important evaluation parameter for highway subgrade and sub- base material because most road failures due to geotechnical factors result from the shrinking of clay as a result of withdrawal of water and expansion of clay when there is ingress of water. As a result of this, with other factors being constant, a soil largely made up of fine particle (clay and silt size particles) is likely to have worse geotechnical properties as highway subgrade than a soil largely made up of coarse particles (gravel and sand particles).

Table 1: The Natural Moisture Content of the Soil Samples in Mariam-Awa Road

Location	Natural Moisture Content (WC %)
1	9.4
2	8.8
3	9.4
4	8.3
5	10.7
6	6.7
7	8.7
8	6.5
9	8.2
10	10.7
11	9.1

12	10.2
13	9.8
14	9.5
15	9.3
16	10.6
17	9.7
18	10.1

The particle size distribution of the soil samples is presented in Table 2 and 3. Using the Unified Soil Classification System (USCS; ASTM D2487-17: R2025), British Soil Classification System (BSI, 2020); the European Soil Classification System (ISO, 2017; the German Soil Classification System (DIN, 2023), the American Soil Classification System (ASCS, 2017), the Japanese Soil Classification System (JGS, 2009), Chinese Soil Classification System (GIB, 2007), and Swiss Soil Classification (SNV, 1959) and so on (Moreno-Manito et al., 2021). The percentage of material passing through the 0.075mm (No. 200) sieve varies between 20% and 61% with a sample from location 11 having the lowest percentage and sample from location 6 having the highest percentage (Table 3). According to specifications by the Federal Ministry of Works and Housing, soils intended for subgrade use should have no more than 35% passing the No. 200 sieve. Based on this criterion, samples 2, 3, 4, 7, 11, 16, and 18 satisfy the requirement, indicating their suitability for use as subgrade material. On the other hand, samples 1, 5, 6, 10, 14, 15, and 17 exceed the recommended limit and are therefore less ideal for subgrade purposes. This suggests that the soils are prone to repeated shrinkage and swelling abilities during alternate dry and wet seasons typified of the climatic conditions of the study area. High amount of fine grain had been related to abundance of clay content which makes them mechanically unstable. Therefore, they are unsuitable for subgrade road construction materials. The grain size distribution curves for the samples were plotted with test sieves against total percentage passing and as illustrated in Figures 10 to 23 revealed that samples 2, 3, 4, 7, 11, 16, and 18 are well graded but poorly sorted, indicating a mix of particle sizes including coarser materials. This gradation generally enhances mechanical stability. In contrast, samples 1, 5, 6, 10, 14, 15, and 17 are poorly graded and well sorted, dominated by fine particles. Such soils tend to have low shear strength and are more susceptible to deformation, making them less suitable for subgrade layers in road construction (Figure 10-23). The common objective of all these classification systems is to provide the means to describe soils through a recognized grouping system so that the soils within a given category may be expected to exhibit similar engineering behaviour (Moreno-Manito et al., 2021).

Table 2: Grain Size Distribution by Wet-Sieving In Mariam-Awa Road

Sample Number	Natural Water Content (WC %)	Percentage Particle Passing by Dry Wt				
		2.0mm	0.60mm	0.425mm	0.30mm	0.075mm
1	9.4	98.0	73.0	70.0	55.0	38.0
2	8.8	96.0	60.0	50.0	35.0	24.0
3	9.4	97.0	70.0	65.0	50.0	23.0
4	8.3	93.0	60.0	55.0	30.0	26.0
5	10.7	97.0	72.0	70.0	60.0	44.0
6	6.7	98.0	75.0	70.0	68.0	61.0
7	8.7	96.0	70.0	65.0	50.0	23.0
8	7.5	80.0	50.0	40.0	25.0	5.0
9	8.2	88.0	53.0	50.0	25.0	13.0
10	10.7	95.0	70.0	65.0	45.0	37.0
11	9.1	97.0	70.0	65.0	40.0	20.0
12	10.2	96.0	68.0	60.0	30.0	18.0
13	9.8	92.0	60.0	55.0	30.0	16.0
14	9.5	95.0	75.0	70.0	60.0	42.0
15	9.3	98.0	73.0	60.0	45.0	40.0
16	10.6	91.0	66.0	60.0	40.0	21.0
17	9.7	97.0	73.0	70.0	55.0	43.0
18	10.1	98.0	72.0	70.0	50.0	28.0

Table 3: Particle Size Distribution by Wet-Sieving and Sedimentation Analysis using

Sample No.	Percentage Particles Pass by Dry Wt					% Gravel	% Sand	% Silt	%
	2.0mm	0.60mm	0.425mm	0.075mm	0.002mm				
1	98.0	73.0	70.0	38.0	19.0	2.0	60.0	19.0	19.0
2	96.0	60.0	50.0	24.0	10.0	4.0	72.0	14.0	10.0
3	97.0	70.0	65.0	23.0	12.0	3.0	74.0	11.0	12.0
4	93.0	60.0	55.0	26.0	12.0	7.0	67.0	14.0	12.0
5	97.0	72.0	70.0	44.0	25.0	3.0	53.0	19.0	25.0
6	98.0	75.0	70.0	61.0	41.0	2.0	37.0	20.0	41.0
7	96.0	70.0	65.0	23.0	12.0	4.0	73.0	11.0	12.0
8	95.0	70.0	65.0	37.0	17.0	5.0	58.0	20.0	17.0
9	97.0	70.0	65.0	20.0	10.0	3.0	77.0	10.0	10.0
10	95.0	75.0	70.0	42.0	23.0	5.0	53.0	19.0	23.0
11	98.0	73.0	60.0	40.0	20.0	2.0	58.0	20.0	20.0
12	91.0	66.0	60.0	21.0	10.0	9.0	70.0	11.0	10.0
13	97.0	73.0	70.0	43.0	25.0	3.0	54.0	18.0	25.0
14	98.0	72.0	70.0	28.0	10.0	2.0	70.0	18.0	10.0

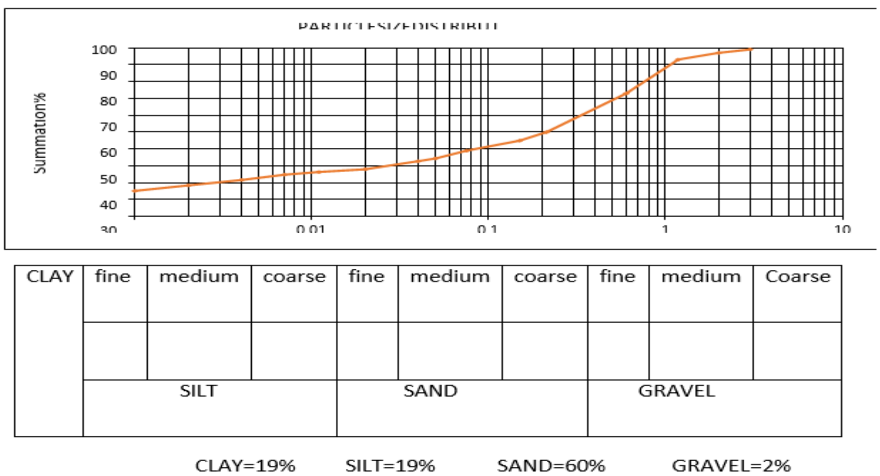


Figure 10: Grain-size distribution curve showing soil sample 1 in Mariam-Awa Road

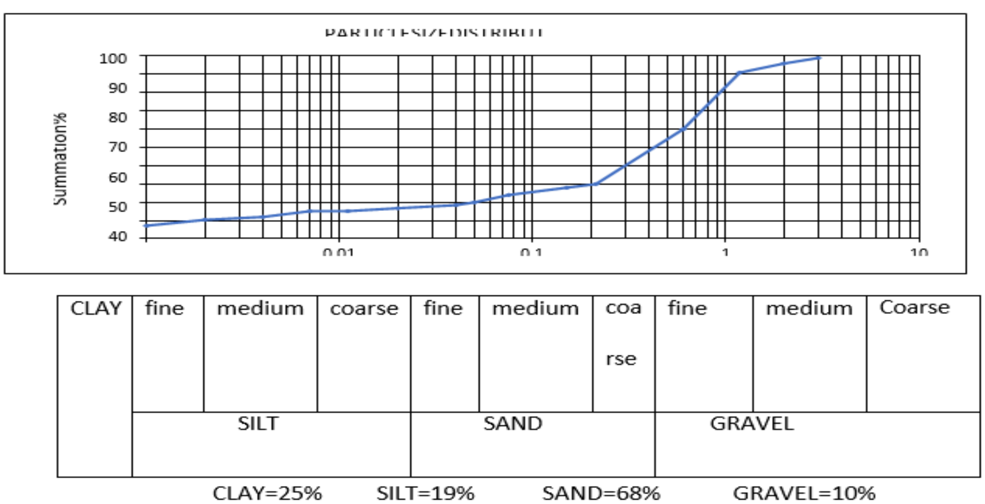
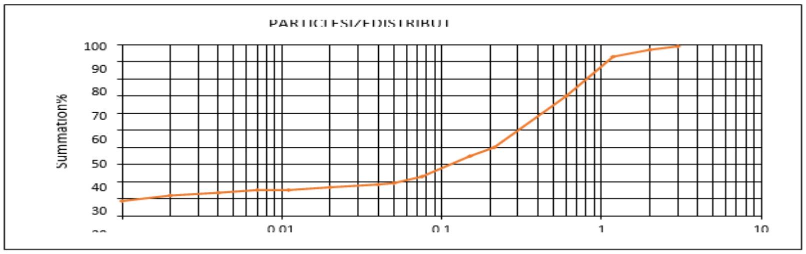


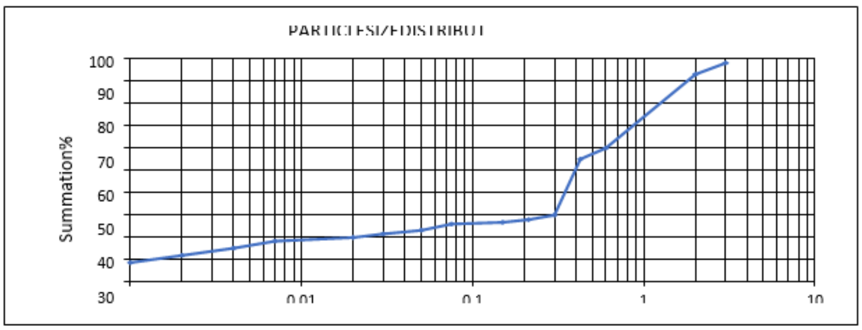
Figure 11: Grain-size distribution curve showing soil sample 2 in Mariam-Awa Road



CLAY	fine	medium	coarse	fine	medium	coarse	fine	medium	Coarse	
	SILT			SAND			GRAVEL			

CLAY=12% SILT=11% SAND=74% GRAVEL=3%

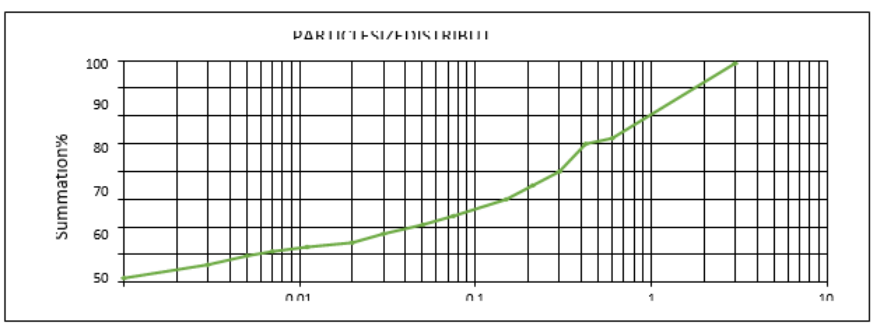
Figure 12: Grain-size distribution curve showing soil sample 3 in Mariam-Awa Road



CLAY	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse
	SILT			SAND			GRAVEL		

CLAY=12% SILT=14% SAND=67% GRAVEL=7%

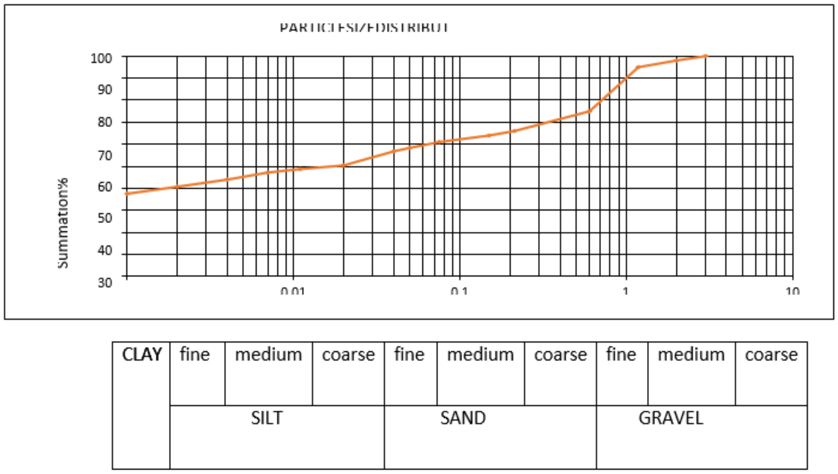
Figure 13: Grain-size distribution curve of soil sample 4 in Mariam-Awa Road



CLAY	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse
	SILT			SAND			GRAVEL		

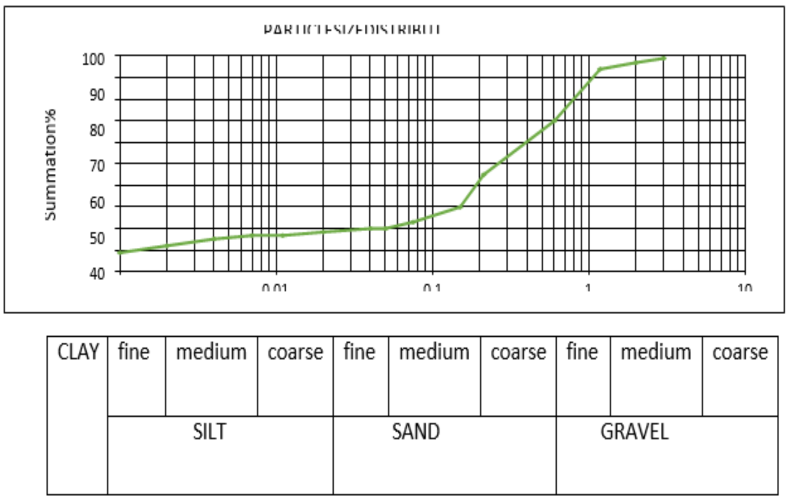
CLAY=25% SILT=19% SAND=53% GRAVEL=3%

Figure 14: Grain-size distribution curve of soil sample 5 in Mariam-Awa Road



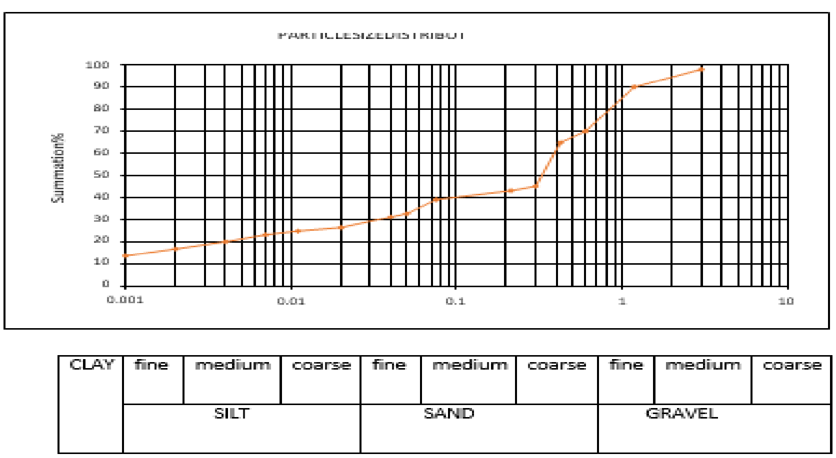
CLAY=41% SILT=20% SAND=39% GRAVEL=2%

Figure 15: Grain-size distribution curve of soil sample 6 in Mariam-Awa Road



CLAY=12% SILT=11% SAND=73% GRAVEL=4%

Figure 16: Grain-size distribution curve of soil sample 7 in Mariam-Awa Road



CLAY=17% SILT=20% SAND=58% GRAVEL=5%

Figure 17: Grain-Size distribution curve of soil sample 10 in Mariam-Awa Road

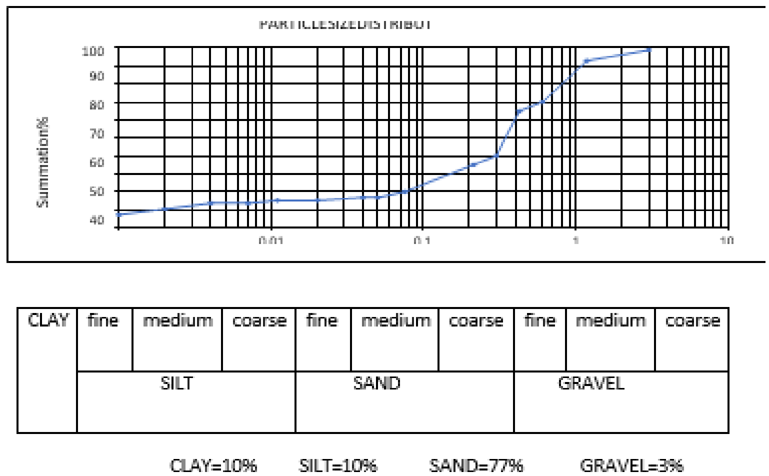


Figure 18: Grain Size distribution Curve of Soil Sample 11 in Mariam-Awa Road

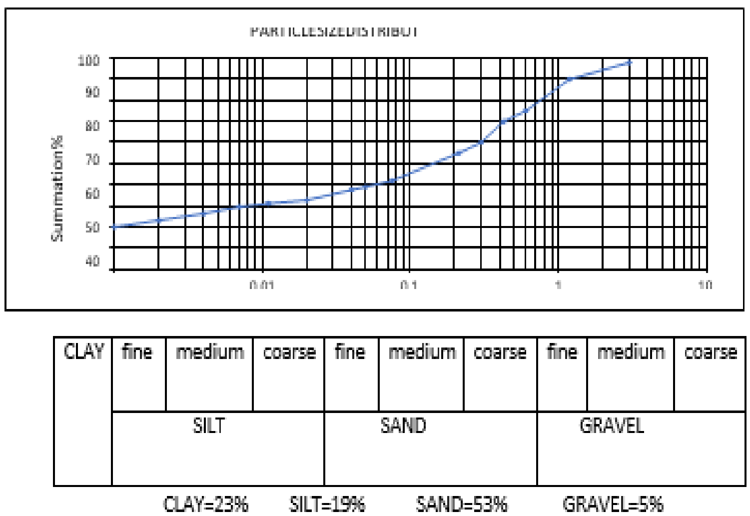


Figure 19: Grain-size distributions Curve of Soil Sample 14 in Mariam-Awa Road

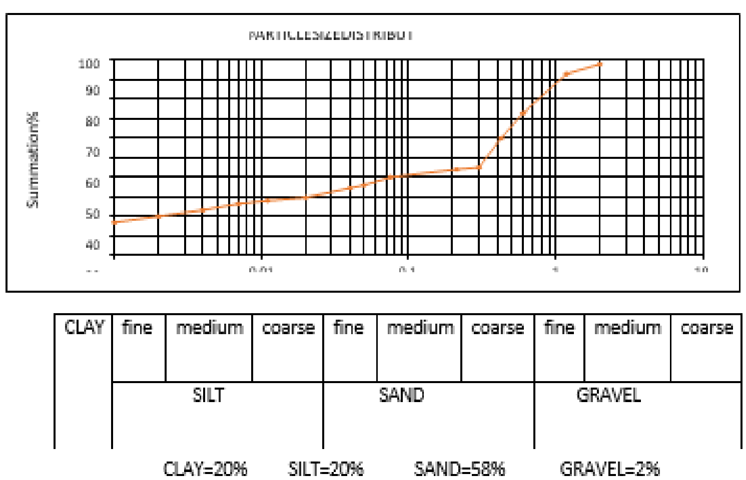
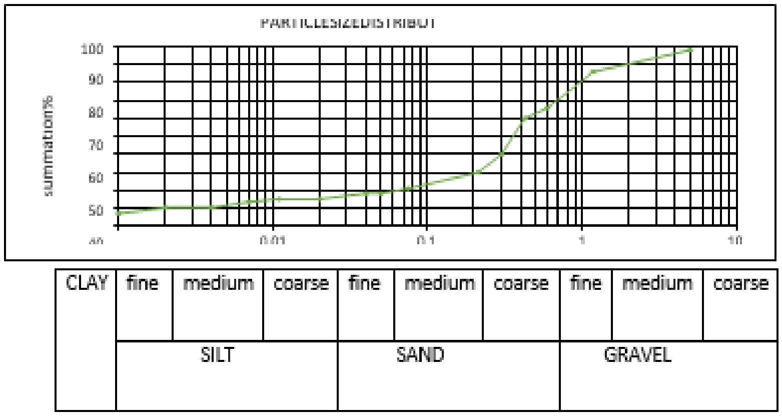
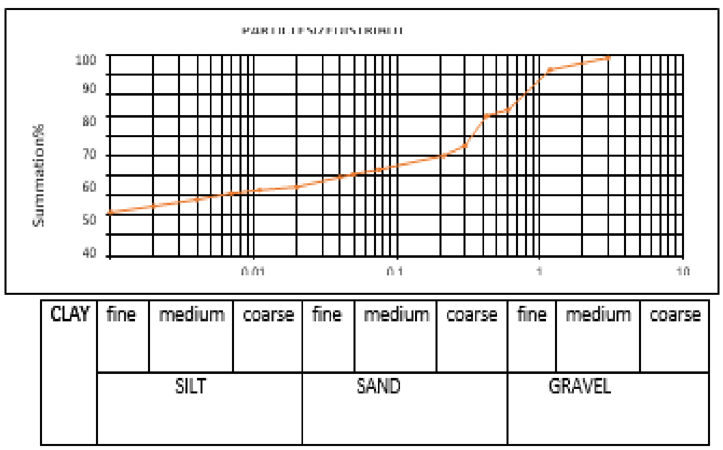


Figure 20: Grain-size distributions Curve of Soil Sample 15 in Mariam-Awa Road



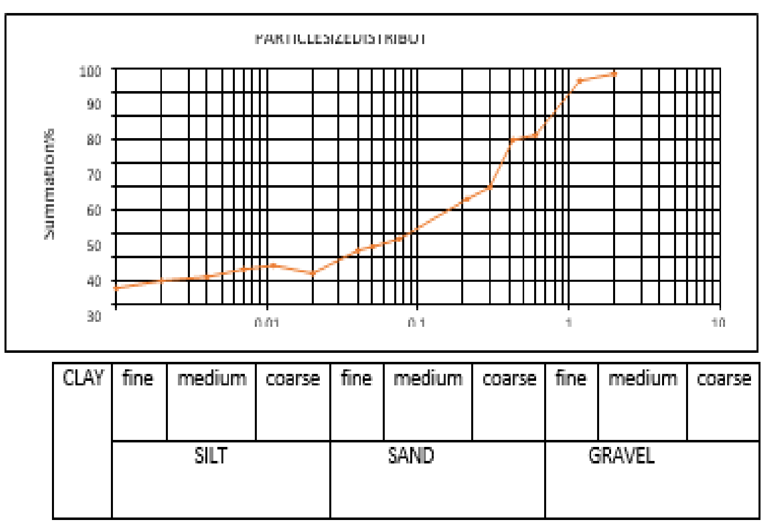
CLAY=10% SILT=11% SAND=70% GRAVEL=9%

Figure 21: Grain-Size distributions curve of soil sample 16 in Mariam-Awa Road



CLAY=25% SILT=18% SAND=54% GRAVEL=3%

Figure 22: Grain-Size distributions Curve of Soil Sample 17 in Mariam-Awa Road



CLAY=10% SILT=15% SAND=70% GRAVEL=5%

Figure 23: Grain-Size Distribution Curve of Soil Sample 18 in Mariam-Awa Road

Specific Gravity

Specific gravity of the soil is influenced by its mineral composition and formation of the soil. The specific gravity of the soil sample of the study area are of the same value 2.66 and this can be seen in Table 4. A soil described as a good subgrade if it falls between 2.60 to 4.60 (Gidigasu, 1991). This implies all samples met the standard minimum value for road construction. The higher the specific gravity the higher the load-bearing capacity of subgrade soil and stability for road construction project. Lower specific gravity indicates lower stability which can leads to failure (Terzaghi and Peck, 1967). An increase in specific gravity has been found to be associated with a decrease in void ratio (Daramola et al., 2015). The findings have made specific gravity an important parameter for validation of soils as subgrade material.

Consistency/Atterberg's Limits

Atterberg's limits estimates the strength and settlement characteristics of soils for road construction. The consistency limits of the soil which relates to the relative ease to which a soil can be deformed based on the relationship with water are presented in Table 5. Soils with high liquid limit encourage high linear shrinkage (Adeyemi, 2000). The liquid limit value ranged from 41%- 47%. The lowest liquid limit occurred at location 11, 12 and 16 which can be considered good for sub-grade soil while the highest liquid limit occurred at location 6 and can be considered poor for use as sub-grade soil.

Table 4: Table Showing the Specific Gravity of the Studied Samples in Mariam-Awa Road

Location	Specific Gravity
1	2.66
2	2.66
3	2.66
4	2.66
5	2.66
6	2.66
7	2.69
8	2.66
9	2.66
10	2.66
11	2.66
12	2.66
13	2.66

For heavy traffic road, for sub-grade liquid limit shall not be more than 35% and plasticity index not more than 12% and for light road, liquid limit shall not be more than 35% and plasticity index not more than 12% (FMWH, 1997). The liquid limit and the plasticity indices of the soil samples are higher than the Nigerian Federal Ministry of Works and Housing specification (FMWH, 1997), therefore soil samples here have poor engineering geological properties and are not suitable as sub-grade material. Soils with high plastic limit could lead to road failure when used for road construction. For a light traffic road, soils less than or equal to 50% plastic limit is suitable for use as sub-grade. The plastic limit value ranged from 13%-16%. The lowest plastic limit occurred in location 11, 12 and 16 while the highest in location 1, 5, 6, 10, 14, 15 and 17. Soil samples in the study area can be considered good subgrade material for road construction since they fall below the specified limit of 50%.

The plasticity index of the studied soil samples ranged from 28% -31% (Table 5). Soil samples in locations of 3, 7, 11, 12 and 16 has the lowest plasticity index of 28% and location 6 has the highest plasticity index of 31%. Hence, they fall within the range of soil with low cohesion, which will reduce compaction, since the specification for plasticity index for sub-grade soil is less than or equal to 12%. The higher the value of the plasticity index, the poorer the value of the soil. The failure of this highway can be inferred from the plasticity index values greater than 12%, which is an indicator of the instability of the pavement. It is advisable to excavate the less stable materials and replace with stable ones.

Table 5: Consistency Limit of the Study Area

Sample No.	Natural Water Content (wc %)	Liquid Limit (L.L)	Plastic Limit (P.L)	Plasticity Index (P.I)	Liquidity Index (I.L)	Consistency Index (I.C)	Flow Index (F.I)	Toughness Index (T.I)
1	9.4	45.0	16.0	29.0	-0.23	1.23	15.3	1.90
2	8.8	43.0	14.0	29.0	-0.18	1.18	15.3	1.90
3	9.4	42.0	14.0	28.0	-0.16	1.16	12.4	2.26

4	8.3	43.0	14.0	29.0	-0.20	1.20	14.9	1.95
5	10.7	46.0	16.0	30.0	-0.18	1.18	14.9	2.01
6	6.7	47.0	16.0	31.0	-0.3	1.3	16.3	1.90
7	8.7	42.0	14.0	28.0	-0.19	1.19	13.2	2.12
10	10.7	45.0	16.0	29.0	-0.18	1.18	18.3	1.58
11	9.1	41.0	13.0	28.0	-0.14	1.14	21.7	1.29
12	10.2	41.0	13.0	28.0	-0.14	1.14	21.7	1.29
14	9.5	45.1	16.0	29.1	-0.22	1.22	15.86	1.83
15	9.3	45.0	16.0	29.0	-0.23	1.23	19.02	1.52
16	10.6	41.0	13.0	28.0	-0.99	1.09	24.81	1.13
17	9.7	45.0	16.0	29.0	-0.22	1.22	15.86	1.83
18	10.1	43.0	14.0	29.0	-0.13	1.13	16.30	1.78

Compaction

Compaction is used to establish a dry density/moisture content relationship of a soil under controlled condition which can form a standard for comparison with field specification. According to Bello (2013) soils which are characterized by high maximum dry density and low optimum moisture content are suitable as excellent subgrade soils.

The results for the Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) for the subgrade samples are shown in Table 6. The Optimum Moisture Content (OMC) ranges from 9.8% to 16% with location 8 having the lowest OMC value of 9.8% and location 6 having the highest OMC value of 16%. The Maximum Dry Density (MDD) ranges from 1.808 to 1.898 Mg/m³ using the West Africa Standard with location 8 having the highest MDD value of 1.898Mg/m³ and Location 6 having the lowest MDD value of 1.808 mg/m³. According to FMWH (2000) stated that OMC values should be less than 18% while the MDD values should be greater than 1.8mg/m³.The soil samples have OMC values lesser than1 8% and MDD values that are greater than 1.8mg/m³, this implies that all soil samples met the requirements and are suitable for construction purposes. These values shows that the soils respond gradually to compaction. Therefore, Compaction suggests that the foundation of pavement structures must always be compacted above MDD and OMC values to yield the maximum strength, prevent ingress of water and distribute wheel loads uniformly into the pavement structures.

According to O'Flaherty (1998), the ranges of values that may be anticipated when using the standard proctor test methods are clay, Maximum Dry Density fall between 1.440 mg/m³ and 1.685mg/m³ and Optimum Moisture Content may fall between 20-30%. Forsilty clay MDD is between 1.6mg/m³ and 1.845mg/m³ and OMC ranges between 15-25%. For sandy clay MDD usually ranges between 1.705g/dm³ and 2.165mg/m³ and OMC between 8-15%. Looking at the results of soil samples 5,6,14 and 17 it could be deduced that they are silty clay while the results of 1, 2, 3, 4, 7, 8, 9, 10, 11, 12, 13, 15, 16 and 18 shows that they are sandy clay.

California Bearing Ratio (CBR)

As per the standards set by the American Association of State Highway and Transportation Officials (AASHTO**)**, subgrade soils are expected to achieve a minimum unsoaked CBR of 80%, or at least 30% soaked CBR when tested using the West African Standard Compaction method. For sub-base materials, an unsoaked CBR of 80% is also the benchmark, while a minimum of 20% soaked CBR is acceptable using the same compaction standard.

The results, shown in Table 4.6, indicate that unsoaked CBR values for the soil samples range from 46% to 86%, while soaked CBR values range between 22% and 70%. The lowest CBR value for the soaked which is 22% occurred at location 6 while the highest CBR value of 70% occurred at Location 8. For the unsoaked, the lowest CBR value of 46% occurred at Location 6 and highest CBR value of 86% occurred at Location 13.

Table 6: Table Showing the OMC and MDD of the Samples in Mariam-Awa Road

Sample No.	Depth	Optimum Moisture Content (OMC %)	Maximum Dry Density (MDD)(mg/m^3)
1	0.4	14.5	1.866
2	0.7	11.8	1.886
3	0.7	11.2	1.888
4	0.7	11.9	1.884
5	0.6	15.2	1.812
6	0.5	16.0	1.808
7	0.9	11.1	1.889
8	0.8	9.8	1.898
9	1.2	10.0	1.895

10	1.0	12.0	1.880
11	1.1	11.0	1.891
12	1.5	10.8	1.893
13	1.6	10.5	1.894
14	1.4	15.0	1.819
15	0.5	14.8	1.822
16	0.6	11.6	1.891
17	0.5	15.2	1.813
18	0.4	13.5	1.826

Based on the unsoaked results, samples 2, 3, 4, 7, 8, 9, 11, 12, 13, and 16 exceeded the 80% threshold and are thus considered suitable for use as both subgrade and sub-base materials. In contrast, samples 1, 5, 6, 10, 14, 15, 17, and 18 fell below the required standard and are therefore less suitable for these purposes.

For the soaked CBR test, samples 2, 3, 4, 7, 8, 9, 11, 12, 13, 16, and 18 achieved values above the 30% requirement, confirming their adequacy as subgrade materials under soaked conditions. However, samples 1, 5, 6, 10, 14, 15, and 17 did not meet this criterion and may require soil stabilization or improvement before they can be used effectively in road construction

Triaxial Shear Strength

The values of undrained cohesion (Cu) range from 47 to 80 kN/m², while the angle of internal friction varies from 10 to 21 degrees as shown in Table 7. These parameters reflect the soil's ability to resist shear deformation and are essential for evaluating its suitability for pavement and foundation design. Most of the samples have cohesion values above 60kN/m², with a few such as samples 5 and 14 reaching the highest values of 80kN/m².According to Schemertmann's 1969 classification, soils with cohesion greater than 50 kN/m² are considered to have moderate to high strength, and therefore are suitable for use as subgrade material, especially under moderate to heavy loading conditions. Cohesion in this range implies that the soil has the ability to hold together even in the absence of confining pressure, a desirable property for subgrade soils in road construction.

Table 7: California Bearing Ratio (CBR) of Soil Samples of Mariam-Awa Road

Sample No.	Depth (m)	Moisture Content (MC%) before soaking	Moisture Content (MC%) after soaking	Unit Weight before soaking (Mg/m^3)	Unit Weight after soaking (Mg/m^3)	Percentage Swelly Potential	CBR Value (%)	
							Unsoaked	Soaked
1	0.4	14.5	16.1	2.137	2.220	0.397	56.0	27.0
2	0.7	11.8	12.5	2.108	2.159	0.288	84.0	44.0
3	0.7	11.2	12.1	2.106	2.155	0.285	85.0	45.0
4	0.7	11.9	12.3	2.107	2.158	0.286	83.0	42.0
5	0.6	15.2	17.1	2.087	2.190	0.403	50.0	23.0
6	0.5	16.0	18.1	2.097	2.189	0.403	46.0	22.0
7	0.9	11.1	12.0	2.099	2.157	0.284	85.0	46.0
8	0.8	9.8	10.3	2.084	2.117	0.0078	86.0	70.0
9	1.2	10.0	10.5	2.085	2.162	0.0068	84.0	68.0
10	1.0	12.0	13.8	2.105	2.212	0.396	62.0	30.0
11	1.1	11.0	11.8	2.099	2.159	0.285	84.0	46.0
12	1.5	10.8	11.3	2.098	2.162	0.278	85.0	46.0
13	1.6	10.5	10.9	2.093	2.167	0.269	86.0	48.0
14	1.4	15.0	17.1	2.092	2.198	0.398	52.0	25.0
15	0.5	15.2	16.6	2.091	2.196	0.405	53.0	23.0
16	0.6	11.6	12.2	2.112	2.180	0.283	84.0	46.0
17	0.5	15.2	17.5	2.089	2.196	0.403	51.0	24.0
18	0.4	13.5	15.1	2.073	2.167	0.376	64.0	36.0

The internal friction angle, which complements cohesion in assessing overall shear strength, mostly falls within the range of 13 to 21 degrees. These values indicate that the soil has a moderate level of interparticle friction. Friction angles above 15 degrees, as observed in most samples, suggest that the soil has sufficient resistance to sliding and deformation under load. For example, sample 6, which has a cohesion of 75 kN/m², and a friction angle of 21 degrees, would offer excellent performance as a subgrade material due to both its cohesive and frictional strength components.

Sample 8, with the lowest cohesion value of 47 kN/m², falls slightly below the acceptable threshold recommended by Schemertmann, 1969. This indicates that in some isolated zones, the soil may be weaker and potentially less suitable for supporting loads without modification. Likewise, samples with lower friction angles, such as sample 1 with 10 degrees and sample 10 with 11 degrees, may be less stable, particularly when exposed to moisture or traffic loads. Such areas may benefit from soil improvement techniques such as mechanical compaction or chemical stabilization to enhance their shear strength parameters (Table 8). Generally, the combination of moderate to high cohesion and friction angle values across the majority of the samples points to a lateritic subgrade soil that is largely cohesive in nature, with fine-grained particles likely influenced by the underlying basement complex geology. These soils typically perform well in subgrade applications if adequately compacted and protected against moisture variation.

As observed in the result of the soil sample test possessing a high PI alongside a moderate CBR is a common, though challenging scenario in geotechnical engineering studies. This occurred in the study area because the soil is a well graded material containing a significant amount of plastic clay (leading to high PI) but also a substantial proportion of coarse grained materials (like sand or gravel) that maintain shear strength leading to moderate CBR. The implications of the high PI is high clay content since PI is the difference between the liquid limit and plastic limit. A high PI >17% or higher indications a high percentage of clay particles which are cohesive and sensitive to water; the mineralogical component of the study area is also a contributive factor since the presence of clay minerals like Montmorillonite commonly found in expansive soils can lead to high PI because these minerals attract one another; moisture sensitivity is another factor because high PI soils are sensitive to moisture. They are very hard but turn soft and loose strength when wet due to their potential swelling (Praven et al., 2021; Pule and Yendaw, 2024; Shen and Hassan, 2025; Habal et al., 2025). Furthermore, despite the high PI, areas of moderate CBR are due to certain factors that maintain the strength of the soil; coarse fraction (Sand/Gravel), even if the fine grained part is highly plastic, if the sample contains a decent amount of sand or Gravel (coarse aggregates), this aggregate skeleton provides inter-particle friction and interlocking which increases bearing strength (Meddah et al., 2022; Koirala et al., 2023; SC, 2025a; SC, 2025b). High compaction is another factor since the soil samples were tested at high compaction (OMG, the particles are forced together reducing the void space which increases the CBR value even if the clay itself is plastic; Low moisture content (unsoaked versus soaked) also contributes to the moderate CBR because if the test was unsoaked, the soil might appear relatively strong because the moisture content is low, keeping the clay stiff. However, if that same soil is soaked, it is highly likely to fail or display low CBR because the clay would absorb water, expand and loose its cohesive strength. It should therefore be noted that such soils are usually considered poor subgrade material for road construction because their strength would significantly drop once they are exposed to water and attain saturation (Rahman et al., 2021; SC, 2025a; SC, 2025b).

Table 8: Shear Strength of the Soil Samples of Mariam-Awa Road

Depth	Undrained	Cohesion, Cu	Angle of Internal Friction (^{0})
1	0.4	78	10
2	0.7	60	16
3	0.7	71	13
4	0.7	63	13
5	0.6	80	15
6	0.5	75	21
7	0.9	65	16
8	0.8	47	17
9	1.2	58	19
10	1.0	64	11
11	1.1	60	21
12	1.5	65	16
13	1.6	76	17
14	1.4	80	12
15	0.5	65	19
16	0.6	73	13

17	0.5	55	15

As observed from this study (Table 8), fine-grained laterite soils can exhibit a high undrained direct shear strength of 80kPa or even higher due to a combination of certain prevailing geological conditions in the study area namely high sesquioxide cementation where the laterite soils are rich in Iron (Fe_2O_3) aluminium (Al_2O_3) oxides, which act as natural cementing agents between fine particles. This cementation bonds fine clay particles into larger, aggregated structures, transforming the behaviour from that of a soft clay clay to a more granular, stiff material; significant high particle interlocking, lateritic soils possess a granular microstructure due to the presence of these iron aggregates which significantly enhances particle interlocking and increases the friction angle; over consolidation and aging is another significant factor where the geological aging and dry-wetting cycles commonly experienced by laterites act as an over-consolidation process resulting in a higher, stiff, and pre-stressed structure that resists shear deformation; high presence of Goetithe (a form of iron oxide) improves inter-aggregate interlocking, further increasing the shear strength; Negative Pore pressure, in undrained conditions, a dense, fine-grained, over-consolidated laterite tends to dilate (expand) during shear. If the soil is saturated, this dilation causes negative pore water pressures which in turn increases the effective stress along the failure plane and temporary strength; Compaction energy where the strength of lateritic soils is also highly dependent on the compaction energy applied with higher compaction reducing the void ratio and increasing both friction and cohesion (Akinniyi et al., 2018; Al-Adhadh et al., 2021; Gao et al., 2024). It is worthy of note that while fine-grained laterites can possess high strength, their shear strength parameters are highly sensitive to moisture content, typically reaching a maximum strength at lower saturation levels (Tiwari and Ajimera, 2023; Achu et al., 2020).

Soil Classification

The American Association of Housing and Transportation Officials (AASHTO) is the most common use of highway purpose. AASHTO Classification of the soil samples are based on Grain size distribution, liquid limit and plasticity Index. Soils are classified from A-1 to A-7 (Table 9) with: A-1 being the best for subgrade (strongest) and A-7 being the clayey and least suitable. Sample 2, 3, 4, 7, 8, 9, 11, 12, 13, 16 and 18 are categorized as A-2-7 and it is used to classify silty or clayey gravel and sand general rating as excellent subgrade soil. Samples 1, 5, 6, 10, 14, 15 and 17 are categorized as A-7-5 and they are used to classify clayey soil general rating as fair to poor sample.

Table 9: AASHTO Standard Soil Classification System (AASHTO, 2012)

General Classification	General Materials (35% or less passing 0.075mm)							Silt-Clay Materials (More than 35% passing 0.075mm)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis Percentage Passing											
2.0mm (No. 10)	50 max										
0.425mm (No. 40)	30 max	50 max	51 min								
0.725mm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of Fraction Passing	6 max										
Liquid Limit			N.P	40 max	41 min	40 max	41 max	40 max	41 min	40 max	40 max
Plastic Index				10 max	10 min	11 min	11 min	10 min	10 max	11 min	11 min
Usual Types of Significant Constituent Materials	Stone Fragment Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General Rating	Excellent to Good							Fair to Poor			

This study was conducted to assess the Index Properties and Shear Strength of the subgrade soils along Mariam–Awa Road, Ago-Iwoye, and Southwestern, Nigeria. Laboratory tests were carried out to determine both the index properties and strength parameters of the soils. The tests included grain size distribution, Atterberg limits, specific gravity, compaction, California Bearing Ratio (CBR) and triaxial shear strength tests. These tests were conducted following the United States Method of classification in compliance with the regulatory standard of Nigeria Federal Ministry of Work and Housing (1972) for road construction and aimed at evaluating the engineering behavior of the subgrade soils in relation to road construction and pavement stability.

Results of the index properties showed that the soils have moderate to high specific gravity values (average 2.66), suggesting the presence of lateritic material with iron oxide cementation. The Atterberg limits indicated moderate plasticity, consistent with clayey lateritic soils. Compaction results revealed maximum dry densities

(MDD) between acceptable ranges and optimum moisture contents (OMC) suitable for field compaction. The CBR values demonstrated that most of the soils fall within the acceptable range for subgrade material, with a few requiring stabilization. Shear strength parameters obtained from triaxial compression tests showed undrained cohesion values ranging from 47 to 80kN/m², and angles of internal friction between 10°and 21°. Based on Schemertmann's (1969) classification, most samples showed good to moderate shear strength characteristics suitable for engineering purposes. Only a few samples with relatively low cohesion or friction angle may require attention during design and construction.

CONCLUSION

The engineering classification and strength analysis of the subgrade soils along Mariam–Awa Road has confirmed that the area is underlain by lateritic residual soils derived from weathered Basement Complex rocks. The soil possesses generally favorable engineering properties, particularly in terms of density, shear strength, and bearing capacity. The shear strength data showed that most sections of the road alignment are suitable for subgrade use without extensive stabilization. However, isolated areas with lower strength values should be identified and treated to prevent settlement or pavement failure over time. The study also shows that index properties such as plasticity, specific gravity, and compaction characteristics correlate well with the shear strength behavior of the soils. This supports the practical use of these basic soil tests for preliminary assessment of subgrade quality in similar geological settings. The observed high PI indicates that the soil is problematic regarding swelling and shrinkage but the moderate CBR suggests that in its current compacted, dry or well-graded state, it is capable of providing temporary or moderate structural support. This study therefore recommends that proper field compaction be carried out to achieve maximum dry density and ensure subgrade stability, weak soil sections be stabilized using lime or cement to enhance their strength and reduce compressibility, effective drainage systems be installed to prevent water infiltration and maintain soil integrity, geotechnical investigations be conducted regularly to identify weak zones and inform design decisions and lastly, this study therefore recommended that reinforcement materials like geotextiles be used in low-strength areas, and that long-term monitoring of subgrade performance be implemented for future improvements.

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