



## A Comparative Study of the Geotechnical Properties of Subsoils Along Failed and Unfailed Pavement Sections of Parts of the Enugu-Makurdi Expressway, Southeastern Nigeria

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### Abstract

Considering the frequent road failures along parts of the Enugu-Makurdi expressway in Igbo Etili Local Government Area, Enugu State, southeastern Nigeria, this study aimed to perform a comparative analysis of the geotechnical properties of failed and unfailed sections of the road. The investigation involved reconnaissance surveys, fieldwork, and the collection of soil samples from both failed and unfailed road sections. Soil samples were manually collected using a hand auger from three different locations at depths ranging from 1 to 3 meters. The geotechnical index properties analyzed in the laboratory included moisture content, specific gravity, bulk density, grain size distribution, Atterberg limits, compaction, and California Bearing Ratio (CBR) tests. Moisture content in the failed sections ranged from 12.4% to 14.9%, while the unfailed sections ranged from 12.7% to 21%. The liquid limit of the unfailed sections ranged between 18 and 20, whereas the failed sections had values between 20 and 28. The plastic limit of the failed sections ranged from 13 to 22, while the unfailed sections ranged from 16 to 18. The plasticity index for the failed sections ranged from 8 to 15, while the unfailed sections ranged from 1 to 4. Sieve analysis classified the soil in the study area as poorly graded. The optimum moisture content of the failed sections ranged from 11.7% to 13.7%, compared to 10.9% to 12.0% in the unfailed sections. The maximum dry density of the failed sections ranged from 114.8 kg/m<sup>3</sup> to 130.6 kg/m<sup>3</sup>, while the unfailed sections ranged from 118.1 kg/m<sup>3</sup> to 119.0 kg/m<sup>3</sup>. The CBR values for the failed sections ranged from 120.5% to 175.7%, compared to 93.7% for the unfailed sections. The study concludes that factors such as poor design and maintenance, government and user mismanagement, inadequate geotechnical assessments, poor sub-base and base materials, heavy traffic, gully erosion, and improper drainage systems contributed to the frequent road failures observed. It is recommended that proper geotechnical investigations and drainage systems be implemented to mitigate future failures.

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### 1.0 Background of the Study

A road pavement should be a continuous stretch of asphalt laid for a smooth ride. Potholes, cracks, depressions, and total road failure may disrupt the smooth ride. Such disruptions are generally regarded as road failures. Road failures can also be defined as discontinuities in the road pavement resulting in cracks, potholes, bulges, and depressions (Aigbedion, 2007; Obianwu *et al.*, 2015). According to the Federal Ministry of Works and Housing (FMWH, 1970), a failed road is characterized by potholes, pavement surface erosion, longitudinal cracks, drainage collapse, depressions or sinking of roadways, flooding of carriageways, gullies, and trenches. All of these are evident along the Enugu-Makurdi and Opi-Onitsha expressways.

In Nigeria, land-use development, such as road network alignment, is hindered by a myriad of problems, ranging from surficial geomorphic processes that shape the terrain to topographic constraints and collapsing structures. Neglect of road conditions during the planning, design, construction, and maintenance phases of engineering structures like highways can lead to road failures, often due to catastrophic morpho dynamics. Verstappen (1993) and Ollier (1978) warned against land utilization without appropriate geotechnical investigations. Singh (2008) explains that continuing engineering projects on unsuitable sites, especially where soils are not stabilized, can have adverse geological effects, such as shear zones and poor strength in expansive soils.

Laboratory tests carried out by Agada (1980), Mesida (1981), and Ajayi (1987) showed that road failures can arise due to poor or inadequate knowledge of the geotechnical behavior of residual soils on which the roads are built, without considering the influence of geology and geomorphology during the design and construction phases. The treatment of

problematic materials, such as clayey materials (e.g., montmorillonite), is often overlooked by construction engineers, leading to issues. This was supported by the works of Gidigasu (1983), Akpokodge (1986), and Adiat *et al.* (2009), who found that geological factors influencing road failures include the geologic sequence, the existence of geological structures like fractures and faults, the presence of laterites, ancient stream channels, shear zones, and problematic materials such as montmorillonite. Faulty designs, poor construction, and poor maintenance, as shown in the works of Paul & Radnor (1976) and Gordon (1976), contribute significantly to road failures.

The present study emphasizes the need to incorporate geotechnical studies into road and highway construction. Furthermore, the use of shale and clay as sub-base materials in highway construction is problematic, as reported by Arora & Singh (2008). Geology plays a significant role in road construction, as roads are built on geological materials and are significantly influenced by them (George *et al.*, 2008b). The main reasons why Nigerian roads fail, as identified in this study, include insufficient preliminary geological investigation (George *et al.*, 2010; Atat *et al.*, 2012), poor road design and construction, inadequate monitoring and workmanship, lack of routine and regular maintenance, poor drainage systems (Akan *et al.*, 2019), use of low-quality materials, traffic congestion, overuse, incorrect use, and inadequate enforcement of accountability for roadway failures (George *et al.*, 2008a). This study aims to conduct a comparative analysis of the geotechnical properties of subsoils along both failed and unfailed pavement sections of the Enugu-Makurdi Expressway. Key parameters examined include moisture content, specific gravity, bulk density, particle size distribution, Atterberg limits, compaction

characteristics, and California Bearing Ratio (CBR). By identifying differences in these properties between failed and unfailed sections, the study seeks to uncover the causes of pavement failure by comparing the measured values with established standards. This investigation highlights the critical role of geotechnical site analysis in construction, as it provides essential data on subsurface conditions, soil characteristics, groundwater levels, and potential engineering challenges. These insights guide design, foundation, and stabilization decisions, ensuring the success and longevity of infrastructure projects.

## 2.0 Location and Geology of the Study Area

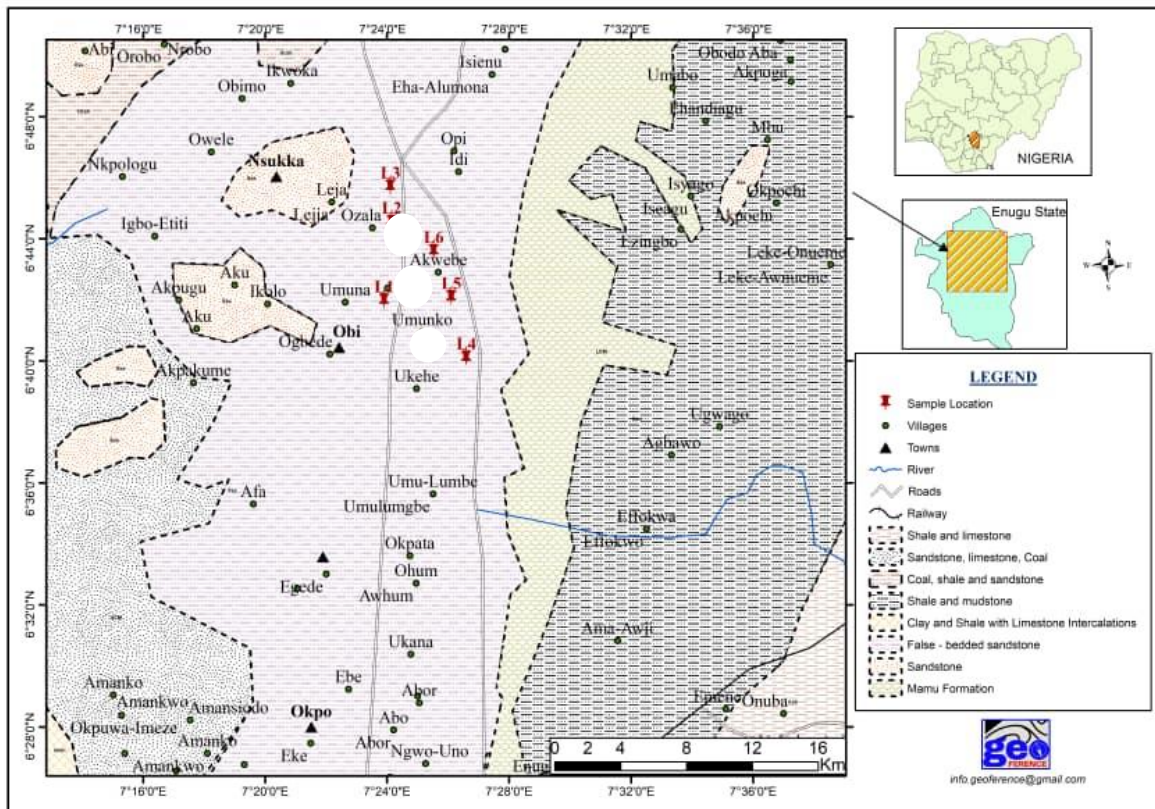
The study area is located between Latitude  $6^{\circ} 42' 39.11''$  N to  $6^{\circ} 42' 56.11''$  N and Longitude  $7^{\circ} 23' 21.11''$  E to  $7^{\circ} 25' 18.11''$  E in the Anambra Basin of the Nigerian Sedimentary Basin, in southeastern Nigeria. The Anambra Basin became a major site of deposition following the Santonian folding in southeastern Nigeria (Ibuot et al., 2019). The compressional uplift of the lower Benue Trough succession (Albian to Coniacian) along the northeast-southwest axis was accompanied by tectonic inversion and the down warping of the Anambra Basin (Obiora et al., 2016). Estimates of total sediment thickness in the Anambra Basin, based on gravity measurements, range between 1,000 to 4,500 meters, with between 3,000 to 500 meters deposited during the Late Cretaceous (Upper Campanian to Maastrichtian). According to Reyment (1965), the geologic map of the study area is given in Figure 1. Geological descriptions of the study area have been well documented by Reyment (1965). The dominant geological formation within the area is the Nsukka Formation, consisting of successions that range from unconsolidated to poorly consolidated sands (310 m thick), with thin intercalations of claystone and siltstone bands, lenses, and false beds, alongside poorly sorted,

medium to coarse-grained sands. The Nsukka Formation is underlain by the thick Imo Formation of Paleocene age (Reyment, 1965). The geological rock systems that outcrop in the study area include the "Upper Coal Measures" and False-Bedded Sandstone (Tattam, 1944). Reyment (1965) later renamed the geological unit to the Nsukka Formation (Upper Maastrichtian-Paleocene). The lithology of the Nsukka Formation is characterized by carbonaceous shale, sandstone, siltstone, and coal, with ridges and dome-like outliers capped by ironstones. This formation overlays the Ajali Formation, which predominantly consists of medium to coarse, poorly sorted, and poorly cemented sandstones, with fine sand at the base (Obiora et al., 2015 a). The topmost part of the Ajali Formation comprises reddish to whitish sands, formed by alterations due to weathering near the surface, influenced by iron-bearing aqueous solutions (ferruginization) of the formation (Onwuemesi, 1990). The red sand is underlain by whitish sands; the whitish sandstone is more susceptible to gully erosion than the red sand, which is more clayey and cohesive (Onwuemesi, 1990; Obiora et al., 2015b).

## 2.1 Topography and Drainage Pattern of the Study Area

The study area features two major landform types: a high relief zone with undulating residual hills and valleys, and lowland areas. The residual hills, remnants of the Nsukka Formation, comprise highly weathered and eroded surface layers that overlie the Ajali Sandstone. Ofomata (1967) identified five types of these residuals based on their shapes: dome, conical, ridge, flat-topped (mesa-like), and Cuesta-like. The dry valleys, characterized by wider, flat bottoms, spread across the plateau's surface. Together, the dry valleys and residual hills form an extensive escarpment known as the Nsukka-Okigwe Cuesta (Ezimet et al., 2017).





**Figure 1:** Showing the geologic map of the study area Reymt, (1965)

In contrast, the lowlands, most prominent in the northwestern part of the study area, serve as collecting centers for runoff during the rainy season. The drainage pattern in this area may indicate outflows from the aquifer or recharge/inflow from outside the system, delineating the catchments of the aquifer system and indicating surface water/groundwater divides (Amah, 2006; George et al., 2015). Surface drainage is sparse in the central and much of the western areas, with only one river (River Adada) crossing the Nsukka Plateau near Lejja (Ozoko, 1996). As a scope this work involves drilling six boreholes to a depth of 3 meters at various locations using a hand auger. Disturbed soil samples will be collected from each borehole for further analysis. The laboratory will determine key index properties of the soil, including moisture content, specific gravity, bulk density, particle size distribution, Atterberg limits, compaction characteristics, and California Bearing Ratio (CBR). This data will aid in assessing the geotechnical properties of

the soil, which is crucial for understanding its behavior and suitability for construction and engineering applications.

### 3.0 Materials and methods

#### 3.1 Materials

The following materials were used in the field to collect the sample: Hand-Auger, shovel, sack bags, digger, cylindrical tube, drying oven, moisture content can, weighing balance, pycnometer bottle or density bottle, Numbered Sieves and Mechanical sieve shaker, Liquid limit device, Casagrandes grooving tool, mixer spatula, Proctor mould or compaction mould, detachable base plate, removable collar, graduated jar, mixing pan, trimming knife, 4.5kg hammer or rammer, CBR machine, CBR mould and soil samples.

#### 3.2 Methods

##### 3.2.1 Determination of Specific Gravity of Soil

The specific gravity of a soil is the ratio of the unit weight of the soil's solid particles to the unit weight of water. It is a key parameter in geotechnical engineering as it helps in determining soil properties like void ratio, porosity, and degree of saturation. The test was performed using an oven-dried soil sample and a pycnometer or density bottle.

#### Procedure

**Soil Preparation:** The soil sample was oven-dried for about 24 hours at a temperature of 110°C to remove moisture.

#### Weighing:

The weight of the empty pycnometer bottle was measured and recorded as  $W_1$ . Approximately 100 grams of soil were weighed. The bottle was filled with water, and the combined weight of the bottle and water was recorded as  $W_2$ .

**Calculation of Water Weight:** The weight of the water was calculated by subtracting the weight of the empty bottle from the weight of the bottle plus water through the expression,  $W_{\text{water}} = W_2 - W_1$ . Again, the volume of the pycnometer bottle,  $V_1$  was determined using Equation 1.

$$V_1 = \frac{W_2 - W_1}{G_w} \quad 1$$

$V_1$  = Volume of the pycnometer bottle,  
 $W_2 - W_1$  = Weight of water,  $G_w$  = Specific gravity of water (typically 1 at standard conditions). This method ensures accurate determination of soil specific gravity, which is crucial for analyzing the soil's behavior under various environmental conditions. The procedure was performed in the laboratory using the pycnometer as shown in Figure 2.



**Figure 2:** Laboratory procedures for determination of specific gravity

### 3.3 Particle Size Distribution Test

The particle size distribution of a soil measures the distribution of individual soil/sediment particles, such as sand, silt, and clay (Figure 3). This analysis helps in understanding soil genesis, classifying soil, or defining soil texture. A sieve analysis was performed to determine the distribution of coarser and larger-sized particles. The distribution of different grain sizes affects

the engineering properties of the soil. Grain size analysis provides the grain size distribution, which is essential for classifying soil types (see Figure 2). The Apparatus used was mechanical sieve shaker. A 500-gram soil sample was oven-dried for 24 hours before being subjected to sieving. The sieves were assembled in ascending order based on sieve numbering, after which the sieve stack was placed in a mechanical sieve

shaker and shaken for 10 minutes. The fractions of the sample retained on each sieve were weighed. The percentage of soil retained and the percentage passing through each sieve were calculated. Additionally, the weight of the bottom pan with its retained soil was recorded. The percentage of soil passing was then plotted against the sieve diameter, and the percentage retained and passing through the sieves was computed using Equations 2 and 3.

$$\% \text{ of soil retained} = \frac{\text{Mass of Soil retained}}{\text{Initial mass}} \times \frac{100}{1} \quad 2$$

The percentage of soil passing was calculated using the Equation 3.

$$\% \text{ of soil passing} = \frac{\text{Mass of Soil passing}}{\text{Initial mass}} \times \frac{100}{1} \quad 3$$

### 3.4 Grading of soil particles

The Coefficient of Uniformity (CU) or Uniformity Coefficient is a parameter used in soil mechanics to express the range of particle sizes in a given soil sample. It was calculated as the ratio of two specific particle diameters,  $D_{60}$  and  $D_{10}$ , obtained from a particle size distribution curve, which is created by performing a sieve analysis or other particle size grading method. The CU was determined using Equation 4:

$$CU = \frac{D_{60}}{D_{10}} \quad 4$$

where  $D_{60}$  is the particle diameter for which 60% of the soil mass is finer (i.e., 60% of the particles are smaller than this diameter) and  $D_{10}$  is the particle diameter for which 10% of the soil mass is finer (i.e., 10% of the particles are smaller than this diameter). The Coefficient of Uniformity (CU) was interpreted as either high or low based on the following criteria: High CU ( $CU > 4$ ) indicates well-graded soil, meaning the soil contains a wide range of particle sizes. A higher CU value suggests that the particles vary in size, which is beneficial for soil compactness and stability in construction projects. Low CU ( $CU \approx 1$  to 3) indicates poorly graded or uniform soil, where the particle sizes are relatively similar. A

low CU suggests that the soil contains a narrow range of particle sizes, potentially leading to issues such as poor compaction, low shear strength, or susceptibility to erosion. Soil classification based on CU helps distinguish between well-graded soils (widely varying particle sizes) and poorly graded soils (similar-sized particles). In geotechnical engineering, well-graded soils generally exhibit better compaction characteristics and greater strength, making them more desirable for construction purposes, such as road subgrades or foundations. In terms of permeability and drainage, soils with low CU values, consisting of nearly uniform particle sizes, may have higher permeability but lower stability, while soils with high CU values may have lower permeability and better stability. Regarding compaction, soils with higher CU values typically compact better since smaller particles fill the voids between larger particles, resulting in denser packing. Strength and stability were assessed using well-graded soils with high CU values, which often exhibit better mechanical properties such as shear strength and resistance to liquefaction under load, compared to poorly graded soils with low CU values. In summary, the Coefficient of Uniformity (CU) provides insights into a soil's gradation, influencing its engineering behavior in terms of compaction, drainage, and overall stability. Specifically, for this study, soil was classified as uniformly graded if the CU was less than 2, well-graded if the CU was greater than 5, and poorly graded if the CU was between 2 and 5.

### 3.5 Moisture Content Test

The moisture content of soil is a crucial geotechnical parameter that directly influences its engineering properties, such as strength, compaction, compressibility, and permeability. Definition: Moisture content is the ratio of the mass of water contained within the pore spaces of the soil to the dry mass of the soil particles. Moisture content is the ratio of the mass of water contained in the pore spaces of soil to the solid



mass of particles in those materials, expressed in a percentage.

The moisture content of water was calculated using Equation 5:

$$MC = \frac{\text{mass of water}}{\text{mass of solid (dried soil)}} \times \frac{100}{1} \quad 5$$

$$MC = \frac{m_2 - m_3}{m_3 - m_1} \times \frac{100}{1} \quad 6$$

where  $m_1$  = mass of empty can,  $m_2$  = mass of wet soil + can,  $m_3$  = mass of dry soil + can, mass of water =  $m_2 - m_3$  and mass of solid ( $m_s$ ) =  $m_3 - m_1$ .



Figure 3. Showing lab procedures of particle size analysis

Soil samples collected for moisture content determination were sealed to prevent moisture loss during measurement. The natural moisture content is the ratio of the mass of water contained in the soil to the mass of solids, expressed as a percentage. This measurement has applications in stability and strength assessment, compaction control, drainage design, hydraulic conductivity, soil behavior prediction, construction material selection, agricultural engineering, environmental impact assessments, and geotechnical investigations.

As shown in Figure 4, the method employed for determining the natural water content of the sample is outlined in the American Association of State Highway and Transportation Officials (AASHTO) A-2-4. The weight of the wet soil is measured using different cans on a weighing balance. Then, the soil is oven-dried for 24 hours, after which the weight of the dried soil is recorded (see Figure 4). The weight of the empty cans is also measured with the weighing balance.



**Figure 4:** Showing lab procedures of moisture content

### 3.6 Bulk Density Test

The bulk density of a soil is the mass per unit volume of the deposit including any water it contains. The bulk density ( $Y$ ) in  $\text{gcm}^{-3}$  of the soil was calculated using the expression in Equation 7:

$$Y = \frac{W}{V} = \frac{m_2 - m_1}{V} \quad 7$$

where  $W$  = mass of wet soil,  $V$  = internal volume of the cylindrical mould,  $m_1$  = mass of empty can,  $m_2$  = mass of wet soil + can,  $m_2 - m_1$  = mass of wet soil,  $V$  = internal volume of the cylindrical mould. For the cylindrical mould of radius  $r$  and height  $h$ ,  $Y$  was estimated using Equation 8.

$$Y = \frac{\text{mass of soil}}{\pi^2 h} \quad 8$$

In measuring bulk density, a cylindrical tube approximately 10 mm in height was used to collect a soil sample. The tube was sealed at both ends to prevent moisture loss before reaching the laboratory. The procedure involved weighing the tube with the soil and recording the weight using a balance. The weight of the empty tube was also recorded. The cylindrical volume of the tube was then calculated using the appropriate Equation 9.

$$V = \pi r^2 h \text{ cm}^3 \quad 9$$

### 3.7 Atterberg Limit Test

The Atterberg limits are basic measures of the critical water content of fine-grained soil. They define its shrinkage limit, plastic limit, and liquid limit, depending on the water content. The soil can exist in one of four states: solid, semi-solid, plastic, or liquid (see Figure 5). This test is part of soil classification and is generally conducted to identify a soil sample. As the water content of a thick suspension of soil is gradually reduced, the soil-water mixture passes from a liquid state, through a plastic state, and finally into a solid state. The Atterberg limit test is conducted using a device called a liquid limit device (see Figure 6). The significance of the limiting water content for each physical state was first proposed by Swedish agricultural scientist Atterberg (Atterberg, 1971). Hence, these limits are commonly referred to as the Atterberg limits. They include the liquid limit test, plastic limit test, and plasticity index test. The objective of the test is to determine the moisture content of the soil at the points where it transitions from one state to the next.



### 3.7.1 Determination of Liquid Limit Test

The liquid limit (LL) of a soil is the moisture content, expressed as a percentage of the oven-dry weight of the soil, at which the soil changes from a liquid state to a plastic state. It marks the boundary between the liquid and plastic phases of the soil's behavior. The apparatus used for the liquid limit test typically includes Casagrande's apparatus, which consists of a cup, a standard grooving tool, a mixing spatula, moisture content cans, and a precise weighing balance. The procedure for determining the liquid limit is outlined in various standards, including British Standard B.S. 1377 (1975). In this method, about 100 grams of air-dried soil, which has been sieved to remove coarse particles, is mixed with water to form a uniform paste. A portion of

this paste is placed into the cup of Casagrande's apparatus and leveled so that the surface is parallel to the base. The Casagrande grooving tool is then used to cut a standard groove through the center of the soil. Next, the handle of the Casagrande apparatus is cranked at a standard rate of two revolutions per second. The number of blows required to close the groove over a distance of 13 mm (or ½ inch) is recorded. This process is repeated several times with different moisture contents until the number of blows required to close the groove consistently falls between 15 and 35. A small sample of soil is taken from the center of the closed groove for moisture content determination. This moisture content, corresponding to 25 blows, is defined as the liquid limit



**Figure 5:** Showing lab procedures of liquid limit

### 3.8 Determination of Plastic Limit (PL)

The plastic limit (PL) of a soil is defined as the moisture content at which it transitions from a plastic state to a semi-solid state. This limit is determined through a laboratory test in which the soil is rolled into threads with a diameter of 3 mm until it crumbles. Atterberg originally

defined this boundary as the moisture content at which a soil sample begins to crumble when rolled into a thin thread under the palm of the hand.

In this procedure, approximately 40 grams of the soil sample is mixed thoroughly with a small

quantity of water. The mixture is then rolled gently between a smooth glass surface and the palm until a thread forms. The thread is inspected for cracks when it reaches a thickness of about 3.0 mm. The necessary apparatus for estimating the PL includes a mixer spatula, moisture content can, and a weighing balance. The PL is calculated using Equation 10, where the weight of water is the difference between the weight of the wet soil and the weight of the dry soil.

$$PL = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100 = \quad 10$$

$$\frac{W_{\text{wet}} - W_{\text{dry}}}{W_{\text{dry}}} \times 100$$

### 3.9 Formula for Liquid Limit (LL)

The Liquid Limit (LL) of a soil is the moisture content at which it transitions from a plastic to a liquid state. This point signifies when the soil begins to flow or loses its strength due to excessive water content. The LL can be determined using a logarithmic relationship based on the Casagrande method, though it may also be calculated with the cone penetrometer method. In the Casagrande method, the liquid limit is defined as the moisture content at which the soil flows together after 25 blows in the Casagrande cup. The corresponding moisture content is represented in Equation 11.

$$LL = \frac{W}{W_s} \times 100 \quad 11$$

where  $W$  is the weight of the water in the soil sample (grams) and  $W_s$  is the weight of the dry soil (grams). In the plotting method, the moisture content (WW) is plotted against the semi-logarithm of the number of blows (NN) used to close the groove. The liquid limit (LL) is then read off the curve as the moisture content at 25 blows. The LL represents the point at which the soil begins to flow or loses strength due to high water content. This parameter is significant in geotechnical engineering for classifying fine-grained soils, as it provides insights into the

soil's behavior when saturated and its ability to retain water. Generally, high liquid limits indicate clay-rich soils with greater plasticity and compressibility.

In testing, the soil sample is placed in a cup, and a groove is made down the middle. The cup is then subjected to repeated impacts until the groove closes. The moisture content at which the groove closes at exactly 25 blows is recorded as the liquid limit. This property helps engineers assess soil suitability for construction and evaluate potential issues such as settlement, stability, and compaction.

### 3.10 Determination of Plasticity Index

A commonly used value in conjunction with the liquid and plastic limits is the plasticity index (PI). The PI of a soil is the numerical difference between the liquid limit (LL) and plastic limit (PL), as shown in Equation 12:

$$PI = LL - PL \quad 12$$

The plasticity index helps classify soils based on their clay content and behavior. Higher PI values typically indicate clay soils, which have significant potential for swelling, shrinkage, and settlement. These soils can undergo considerable volume changes with fluctuations in moisture content, potentially affecting foundation stability and leading to issues such as cracking in buildings or roadways. In contrast, soils with a lower PI are generally more stable and present fewer risks to foundation structures. Therefore, the PI is a valuable factor in foundation design. It is also essential for determining bearing capacity and settlement. Soils with high PI values tend to have lower bearing capacity and are more susceptible to differential settlement, making it crucial to consider these characteristics when selecting foundation types (e.g., deep foundations for high-PI soils). In summary, the PI provides engineers with insights into soil behavior under loading and

moisture variations, which is critical for ensuring safe and stable foundation design.

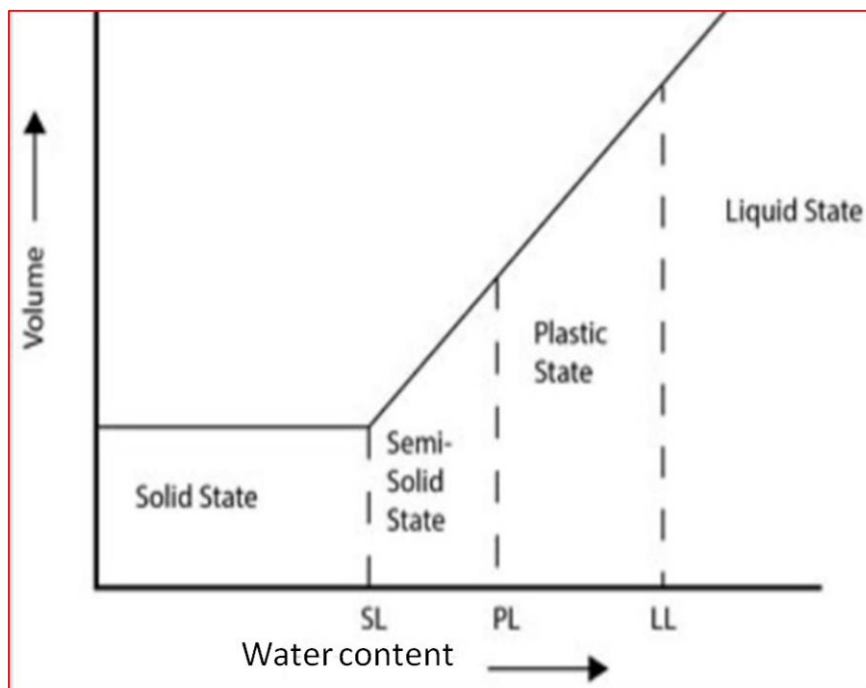
### 3.11 Liquidity Index

The Liquidity Index (LI) is a geotechnical parameter used to evaluate the consistency of a soil sample and its proximity to the liquid or plastic state. It is a dimensionless number that helps engineers assess how close a natural soil's water content is to its liquid or plastic limit. This assessment is vital in foundation engineering for predicting soil behavior under load, particularly when designing foundations and evaluating the potential for settlement or failure. The LI offers an estimate of how close the natural moisture content is to the liquid limit.

The formula for the Liquidity Index is provided in Equation 13:

$$LI = \frac{w-PL}{LL-PL} \times 100 = \frac{w-PL}{PI} \times 100 \quad 13$$

Where  $w$  is the natural water content of the soil. If  $LI > 1$ : The soil is in a liquid state (very soft and flowable). For  $LI = 1$ , the soil is at its liquid limit. If  $LI$  between 0 and 1, the soil is in a plastic state (can deform without breaking) and if  $LI < 0$ : The soil is in a semi-solid or solid state (more stable, less deformable). In foundation engineering, the LI helps in understanding the workability and stability of soils, especially clays, under varying moisture conditions. It is useful in evaluating the risk of settlement or bearing capacity issues in soft or saturated soils. A cohesive soil can exist in any of the states called, semi-solid, plastic and liquid state, depending on the moisture content of the soil. The boundary between the liquid and semi-solid state is known as shrinkage limit, depicted as SL. According to Atterberg (1931), If the LI or a soil is greater than zero but less than or equal to one, the soil is in its liquid limit state. But if the liquidity index is equal to zero, then the soil is in the plastic state. The LI of a soil is illustrated in Figure 6. Liquidity index of a soil, was calculated using the formula in Equation 13.



**Figure 6:** Showing stages of liquidity index of a soil (Atterbege, 1991)



### 3.12 Compaction Test

The Proctor compaction test, named after Ralph Roscoe Proctor (1933), established the relationship between the dry density of a soil and the moisture content for a given compactive effort. This test, known as the standard Proctor compaction test, was later modified to create the modified Proctor compaction test, which involves higher compactive efforts. Both tests are vital for understanding soil compaction behavior and are used to evaluate how well soils can support pavement structures.

In this study, a sample preparation method similar to that described by Proctor was used. Approximately 3 kg of air-dried soil was sieved through a 4.75 mm sieve, and an initial water content of about 8% was applied to each sample. Before compaction, the interior of the mold and the base plate were cleaned to ensure consistency in the results. The first layer of soil was placed into the mold, and 61 blows of a standard rammer were applied, compacting the soil to about one-third of the mold's height. The second layer was scratched, and soil samples were taken from the top, middle, and bottom to determine moisture content. This procedure was repeated for water contents of 10%, 12%, 14%, and so on. The soil is compacted in layers, with each layer receiving a controlled number of

blows from a standard hammer dropped from a specific height. This process is repeated for different moisture contents, and the corresponding dry densities are measured. By plotting dry density against moisture content, a compaction curve is generated. The peak of this curve represents the maximum dry density (MDD) and the corresponding optimal moisture content (OMC). These parameters are crucial for determining the soil's suitability for use in pavement subgrade.

The apparatus used in the Proctor compaction test includes a Proctor mold or compaction mold, a detachable base plate, a removable collar, a drying oven, a graduated jar, a large mixing pan, a trimming knife, a weighing balance, and a 4.5 kg hammer or rammer. Approximately 3 kg of soil is typically used for each test. The dry density of the soil is related to its bulk density, maximum dry density (MDD), and optimal moisture content (OMC), as expressed mathematically in Equation 14.

The dry density ( $\rho_d$ ) relates with MDD, OMC and  $w$  of a material with bulk density of the soil  $\rho$  as given in Equation 14

$$\rho_d = \frac{\rho}{1 + w} \quad 14$$



**Figure 7:** Showing lab procedure of compaction test

### 3.13 California Bearing Ratio (CBR)

In this comparative study of the geotechnical properties of subsoils along failed and unfailed pavement sections of parts of the Enugu-Makurdi Expressway, Southeastern Nigeria, the California Bearing Ratio (CBR) test plays a crucial role in assessing subgrade suitability. Developed by the California Division of Highways in 1929, the CBR test evaluates the strength and load-bearing capacity of subgrade, sub-base, and base course materials used in road construction. In this study, soil samples for the CBR test were collected from a depth of 2.5 meters below the surface. The apparatus setup included a CBR machine, CBR mould, mixing pans, hammer or rammer, a detachable extension collar, and a 6kg soil sample. The procedure began by weighing 6kg of the soil sample, which was then transferred into the cylinder, and the optimum moisture content (OMC) was added. The sample was compacted in the CBR mould using the rammer according to the approved specification. This compaction involved dividing the soil into three layers, with each layer receiving 52 blows from the rammer. Once compacted, the collar was removed, and the soil was trimmed to align with the mould. The sample, along with the mould, was then weighed to determine its unit weight. Following this, the mould and soil were placed in the CBR machine. A surcharge was applied to the soil, and a plunger load was introduced through an opening in the center of the surcharge. The test load was applied, and the sample was penetrated by the plunger at a strain rate of 1.25 mm per minute. Measurements were taken at regular intervals, such as at penetrations of 25, 50, 75, 100, 125, and 150 mm, until the soil sample failed.

Upon completion, the soil sample and mould were gently removed from the machine, and the base plate was detached. The void left by the plunger was refilled with soil of the same

moisture content. The mould was reassembled, the base plate was replaced, and penetration tests were carried out at both the top and bottom of the sample.

This CBR test followed the guidelines outlined in the British Standards (B.S., 1975). The results provide valuable insights into the comparative strength and stability of subsoils in both failed and unfailed pavement sections, contributing to a deeper understanding of the factors that influence pavement performance along the Enugu-Makurdi Expressway.

## 4.0 Results and Discussion

The geotechnical analysis of the subsoils from both failed and unfailed pavement sections along parts of the Enugu-Makurdi Expressway is presented in Table 1 for depths of 1m, 2m, and 3m at the study locations. The results demonstrate that the moisture content (MC) of soils varies according to soil type, with sand exhibiting moisture content ranging from 10 to 15%, silt from 15 to 30%, and clay from 30 to 50%, as observed by Sidi et al. (2015). These results indicate that the soils in the study area are predominantly sandy and clayey.

The MC data suggest that certain areas absorb significant amounts of water despite having low permeability. This is particularly critical for clayey soils, as high-water absorption can decrease shear strength and increase compressibility, reducing their bearing capacity. Such soils become less capable of supporting overlying structures, leading to potential issues for engineering works, especially due to the expansive and swelling nature of clay.

On the other hand, areas with higher permeability and high-water absorption could be susceptible to liquefaction under extreme conditions, particularly if the soil exhibits very high moisture content. Liquefaction, triggered

by dynamic loading such as tremors or strong vibrations, poses a significant risk to sandy or silty soils. While less common, clayey soils may also be affected in certain conditions.



**Figure 8:** Showing laboratory procedures for California Bearing Ratio test

In sandy regions of the study area, the high permeability could exacerbate engineering challenges, such as gully erosion, where inadequate drainage systems could result in the washing away of subgrade materials (Adejuwon *et al.* 1990). Rainfall and temperature are identified as the main factors influencing moisture changes in the soils. Variations in soil properties, precipitation patterns, groundwater levels, and the characteristics of overlying vegetation can all affect soil moisture content. In the study area, soil moisture increased sharply following heavy rainfall, highlighting the role of precipitation in controlling moisture levels.

At Point 1, the MC at a 2m depth was higher than at 1m and 3m, with the lowest value recorded at

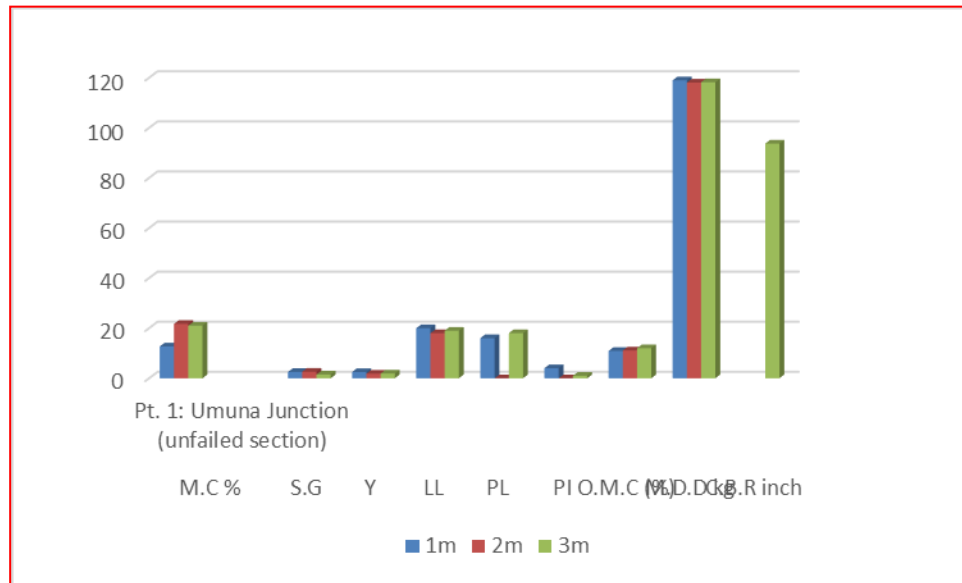
1m (12.7%). At Point 2, the highest MC was found at 1m, while values at 2m and 3m were equal. At Point 3, the 3m depth had the highest MC, followed by 2m, with the lowest value again at 1m. The pictorial descriptions of the parametric displays are given in Figures 9 to 11.

The high moisture content in certain locations can be attributed to an elevated water table following significant rainfall. Conversely, the lower values of natural moisture content suggest that the water table fluctuates during drier seasons. As noted by Ramamurthy (2005), moisture content is a key factor influencing the dry density of soils, and this variation is critical in understanding the geotechnical behavior of the subsoils in this region.

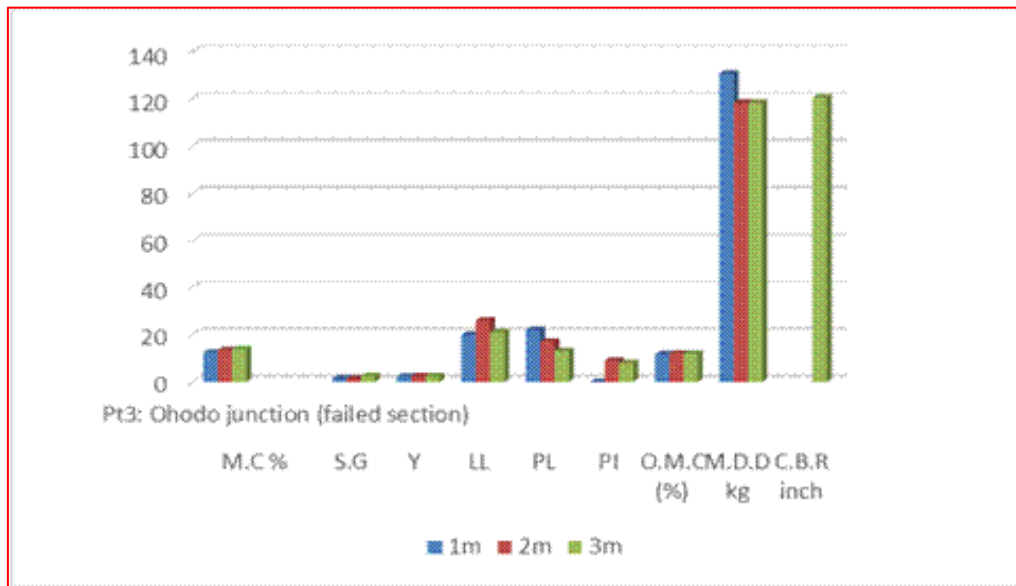


**Table 1:** Results of geotechnical characteristics of soil samples collected along parts Enugu-Markudi expressway

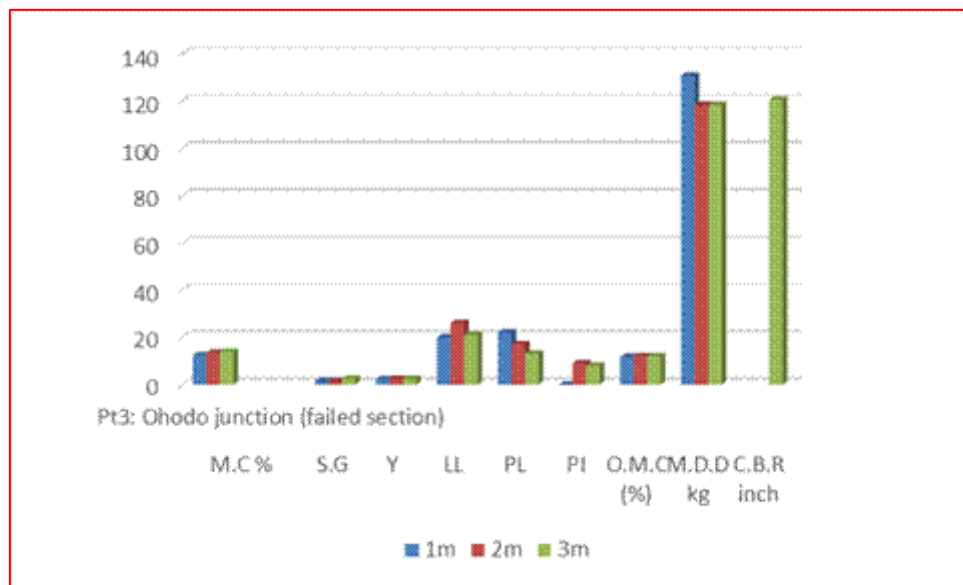
	M.C %	S.G	Y	LL	PL	PI	GROUP NAME (Grain size)	CU	GROUP SYMBOL	O.M.C (%)	M.D. D kg	C.B.R inch
<b>Pt. 1: Umuna Junction (unfailed section)</b>												
1m	12.7	2.56	2.5	20	16	4	Poorly graded sand	3.09	Sp	10.9	119.0	
2m	21.7	2.63	1.85	18	NP	NP	Poorly graded sand	3.13	Sp	11.1	118.1	
3m	21.0	1.51	1.88	19	18	1	Poorly graded sand	3.15	Sp	12.0	118.2	93.7
<b>Pt.2: Ozalla Junction (failed section)</b>												
1m	14.9	1.51	2.40	28	13	15	Poorly graded sand	3.56	Sp	13.7	120.3	
2m	14.7	1.52	2.21	26	17	9	Poorly graded sand	3.56	Sp	13.0	114.8	
3m	14.7	1.51	1.97	21	13	8	Poorly graded sand	3.85	Sp	12.5	115.3	175.7
<b>Pt3: Ohodo junction (failed section)</b>												
1m	12.4	1.51	2.18	20	22	NP	Poorly graded sand	3.13	Sp	11.7	130.6	
2m	13.4	1.50	2.34	26	17	9	Poorly graded sand	3.44	Sp	11.9	118.3	
3m	13.8	2.49	2.42	21	13	8	Poorly graded sand	3.03	Sp	11.9	118.3	120.5



**Figure 9:** Showing the variations of geotechnical parameters of Umuna junction



**Figure 10:** Showing the variations of geotechnical parameters of Ozalla junction



**Figure 11:** Showing the variations of geotechnical parameters of Ohodo junction

The bulk density of soil is influenced by several factors, including organic matter content, soil structure, compaction, and porosity. At Point 1, the bulk density at a depth of 1m was observed to be higher than at 2m and 3m, with 2m having the lowest bulk density. Similarly, at Point 2, the bulk density at 1m was higher than at 2m and 3m, with 3m having the lowest value. However, at Point 3, the bulk density at 3m was higher than at 1m and 2m, with 1m being the lowest (See Figure 9 to 11). Excessive tillage and disturbance of the soil can reduce its organic

matter content, thereby weakening its natural structure and making it prone to erosion. This erosion clogs pore spaces, reducing porosity and increasing bulk density (see Table 1). High bulk density is a critical indicator of soil compaction, which reduces soil porosity. This affects essential soil properties, such as water-holding capacity, root growth, and the movement of air and water through the soil. Compaction can significantly hinder plant growth and water infiltration, leading to reduced structural stability of the soil.

According to the specifications provided by the Federal Ministry of Works and Housing (FMWH, 2010), a good soil should have a specific gravity (SG) ranging from 2.57 to 2.67. The laboratory tests show that most samples from the failed pavement sections of the Enugu-Makurdi Expressway fell below this acceptable range. Some samples, however, fell within or exceeded the specification; with higher SG values also found predominantly in failed sections (see Table 1). At Point 1, the SG of soil at 2m was higher than at 1m and 3m, with 3m being the lowest. At Point 2, the SG at 1m and 3m were equal but lower than at 2m. At Point 3, the SG at 3m was higher than at 1m and 2m, with 2m being the lowest (See Figure 9 to 11). Sieve analysis revealed the particle size distribution of the soils, with grain size plotted

against the percentage of soil passing through the sieve. According to the American Association of State Highway and Transportation Officials (AASHTO) classification system, soils are divided into seven major groups, A-1 through A-7. Soils in groups A-1 to A-3 are granular or sandy materials with 35% or less of particles passing the No. 200 sieve, while soils in groups A-4 to A-7 have more than 35% passing and are typically silt and clay materials. The soil in the study area was predominantly classified as poorly-graded sand (SP), based on both AASHTO and the Unified Soil Classification System (USCS) criteria. The grain size distribution of unfailed section of Enugu-Markudi expressway at Umuna junction is represented in the Figure 12.

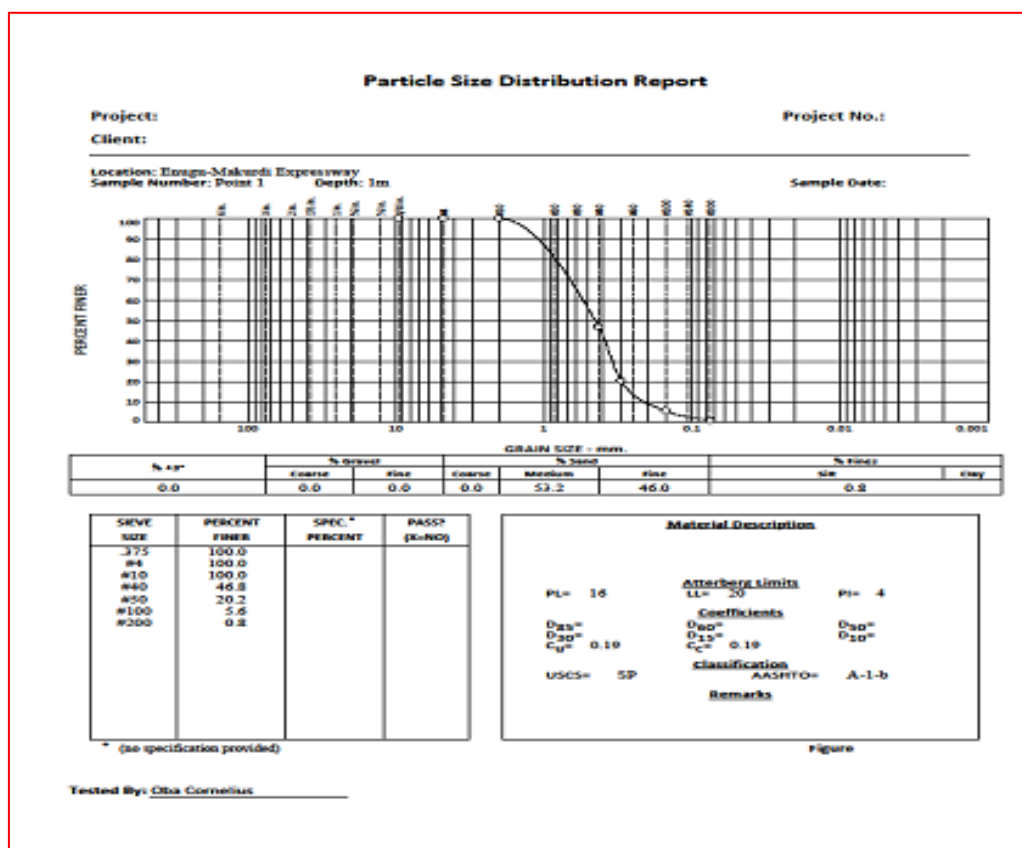


Figure 12: Showing particle size distribution of Umuna junction

The uniformity of particle sizes in soil is expressed by the uniformity coefficient (CU), calculated as the ratio of the soil diameter at

which 60% of particles are finer (D<sub>60</sub>) to the diameter at which 10% are finer (D<sub>10</sub>). A soil is considered well-graded if CU > 5, poorly-graded



if  $2 < CU < 5$ , and uniformly graded if  $CU < 2$ . In the unfailed sections of the expressway, CU values ranged from 3.09 to 3.15, while in the failed sections, CU values ranged from 3.03 to 3.85 (see Table 1). Therefore, the soil in the study area was classified as poorly-graded sand, which consists mainly of sand and clay.

Geotechnically, poorly-graded soils are non-compressible and have poor bearing capacity, making them unsuitable for use as base course materials in road construction. However, such soils can facilitate drainage due to their large void spaces. Poorly-graded sands consist of a limited range of particle sizes, which can lead to poor structural integrity in construction applications, such as road pavements. This type of soil lacks the varied particle sizes that provide a stable foundation, resulting in weak bearing capacity and excessive settlement. In construction, poorly-graded sands can cause challenges such as road pavement failure and building settlement, underscoring the

importance of soil investigation and improvement techniques to enhance its engineering properties, such as shear strength.

The Atterberg limits, which measure the plasticity and moisture content of soil, further support these findings. The liquid limit (LL) for the failed sections of the expressway ranged between 20% and 28%, while for the unfailed sections it was between 16% and 18%. The plastic limit (PL) for failed sections ranged between 13% and 22%, while for unfailed sections it was between 16% and 18%. The plasticity index (PI), which represents the difference between the liquid and plastic limits, ranged from 8% to 15% for the failed sections and from 1% to 4% for the unfailed sections (see Figure 9 to 11). Some soil samples, due to their high silt and clay content, were classified as non-plastic (NP). The Liquid Limit Graph of failed section of Enugu-Markudi expressway at Ozalla junction is represented in Figure 13.

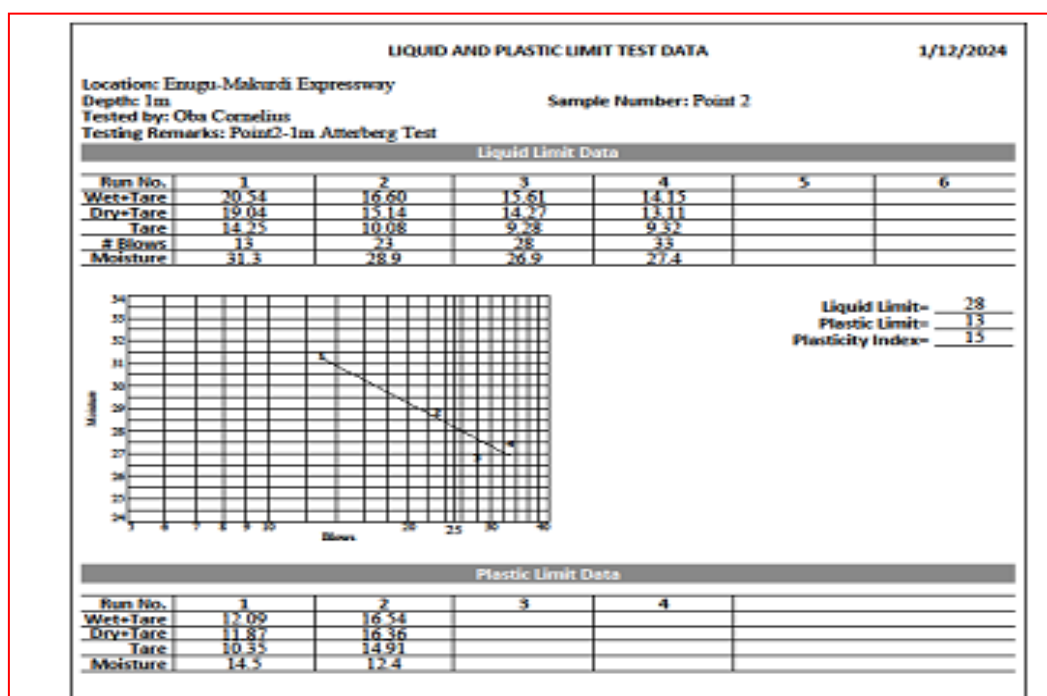


Figure 13: Showing the Liquid Limit Graph of Ozalla junction

According to Federal Ministry of Works and Housing 2010, (FMWH 2010), for roads and bridges, the liquid limit should be less than 35%,

and the plasticity index should not exceed 10%. Most samples from Points 1 to 6 fell within this specification, except for those that could not be

grooved. The plasticity index is especially critical for assessing road strength, foundation design, and suitability for other construction applications. Soils with a PI > 10 are considered clayey and may exhibit high plasticity and cohesive behavior, leading to issues such as cracking, differential settlement, and structural deformation, particularly in pavements and retaining walls. (Prakash & Jain 2002)

In conclusion, the geotechnical analysis of the subsoils along the Enugu-Makurdi Expressway reveals significant variations between the failed and unfailed sections. The failed sections predominantly consist of poorly-graded sands with low specific gravity, high bulk density, and poor Atterberg limit values, all of which contribute to the pavement failure observed. Proper soil stabilization and subgrade improvement are necessary to enhance the engineering properties of these soils, ensuring the longevity and stability of the expressway.

The geotechnical properties of subsoils along both failed and unfailed sections of the Enugu-Makurdi Expressway reveal interesting trends, especially when comparing compaction characteristics. Regarding the compaction test results, several observations were made across three different points at varying depths. At Point 1, the Optimum Moisture Content (OMC) at 3 meters was higher than at 1 meter and 2 meters, with the OMC at 1 meter being the lowest (see Figure 9 to 11). In contrast, at Point 2, the OMC at 1 meter was higher than at 2 meters and 3 meters, with 3 meters showing the lowest OMC. At Point 3, the OMC at 3 meters and 2 meters were equal, and both were higher than the OMC at 1 meter (see Figure 9 to 11).

In terms of Maximum Dry Density (MDD), at Point 1, the MDD at 1 meter was higher than at 2 meters and 3 meters, with 2 meters being the lowest. Similarly, at Point 2, the MDD at 1 meter was again higher than at 2 meters and 3 meters, with 2 meters showing the lowest MDD. At Point 3, the MDD at 1 meter was higher than at

2 meters and 3 meters, with the values at 2 meters and 3 meters being equal (see Figure 9 to 11). These compaction characteristics are summarized in Table 1. When comparing the failed and unfailed sections of the Enugu-Makurdi Expressway, the OMC for the failed section ranged from 11.9% to 13.7%, while for the unfailed section, it ranged from 10.9% to 12.0%. Similarly, the MDD for the failed section varied between 114.8 and 130.6 kg/m<sup>3</sup>, whereas the unfailed section ranged from 118.1 to 119.0 kg/m<sup>3</sup> (see Figure 9 to 11). The variations in MDD and OMC were influenced by factors such as soil type, organic matter content, particle size distribution, and compactive effort, which directly affect soil compaction. (O'Flaherty, 2001). The Compaction Test Graph of failed section of Enugu-Markudi expressway at Ohodo junction is represented in the Figure 14.

The California Bearing Ratio (CBR) test further highlights the differences between the failed and unfailed sections. The CBR for the failed section ranged from 120.5% to 175.7%, while the unfailed section recorded a CBR of 93.7% (see Figure 9 to 11). The variation in CBR values was influenced by soil properties such as density, cohesion, internal friction angle, applied load, soil moisture content, and the level of compaction. According to the Federal Ministry of Works and Housing (FMWH, 2010), the recommended CBR value for sub-base and base course materials ranges between 10% and 40%. However, the CBR values for the failed section of the expressway exceed this range, suggesting that the subsoils do not meet the recommended standards for road construction. The elevated CBR values observed in the study area may indicate the presence of silty or clayey soils, which typically have higher CBR values. However, due to the expansive nature of these soils, their performance under load may be poor if the CBR does not match the expected values for well-graded sand. In such cases, it may be necessary to excavate and replace the soil with

more suitable material. Alternatively, if the soil contains sufficient lime, it can be stabilized to enhance its bearing capacity and meet the necessary structural requirements. This comparative analysis of the geotechnical properties underscores the importance of soil evaluation in road construction, particularly in

assessing the suitability of subgrades and identifying potential issues that could lead to pavement. The California Bearing Ratio Graph of failed section of Enugu-Markudi expressway at Ohodo junction, is represented in the figure below Failure 15.

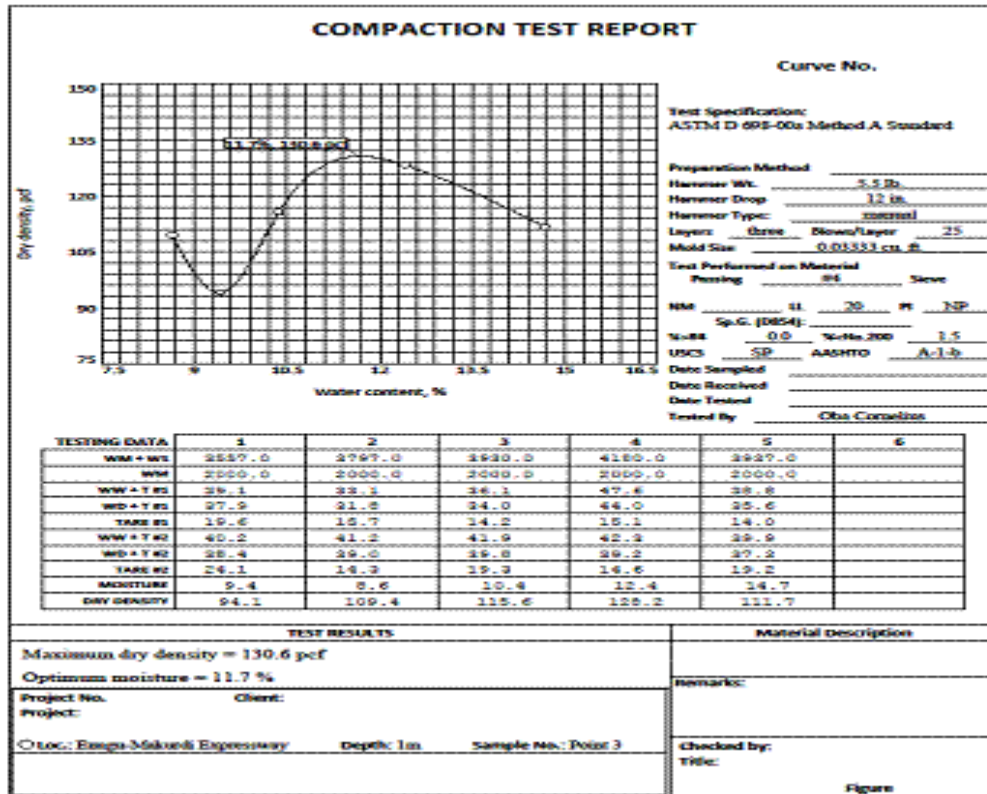


Figure 14: Showing the Liquid Limit Graph at Ozalla junction

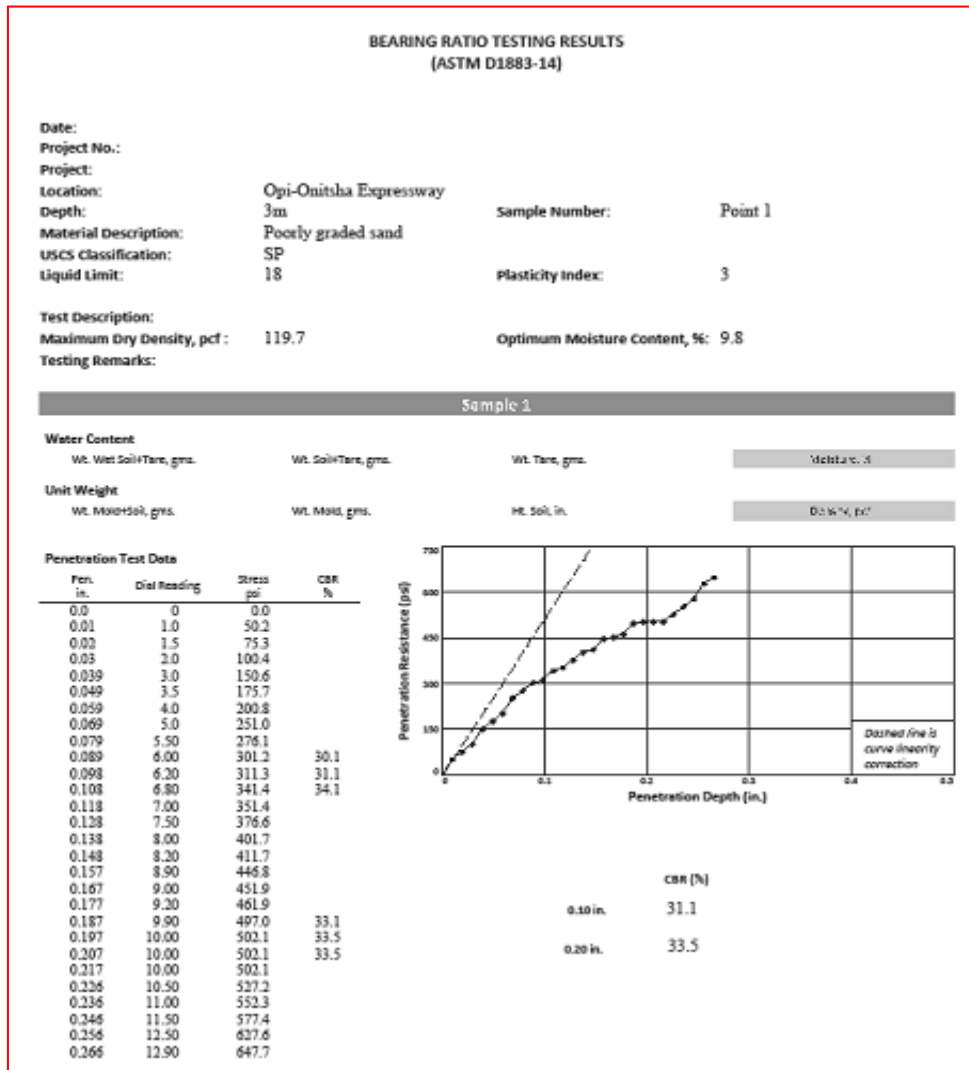


Figure 15: Showing the California Ratio Graph at Ohodo junction

**5.0 Conclusion and Recommendations**

**5.1 Concluding remarks**

In conclusion, the results of the tests unequivocally reveal that the repeated failure of the road pavement can be attributed primarily to the poor geotechnical properties of the underlying subsoil. The use of substandard materials in the road construction significantly compromised the integrity of the pavement, highlighting the critical importance of thorough geotechnical investigations prior to any construction project. Without proper soil evaluation and understanding of the material properties, roads remain vulnerable to early deterioration, especially under constant stress from traffic. To mitigate these issues and ensure the longevity of such infrastructure, soil

improvement techniques like stabilization must be employed to enhance the strength of the subsoil. These methods would not only improve the load-bearing capacity of the pavement but also reduce the risk of structural failure under heavy and continuous traffic. Furthermore, the analysis identifies additional factors contributing to road failure, including deficiencies in construction practices, improper design, and inadequate maintenance. Heavy traffic loads that exceed the designed capacity, coupled with the old age of the original construction materials, further exacerbate the situation. Excessive stress on the pavement from such conditions accelerates the deterioration process, especially when compounded by insufficient geotechnical studies during the



planning phase. Given that the road does not traverse a low-lying area prone to flooding, the provision of efficient drainage systems is crucial. Proper drainage would prevent the ingress of water into the pavement structure, which can weaken the already compromised subsoil and further shorten the road's lifespan. Implementing adequate drainage measures would help preserve the strength and integrity of the road, minimizing the adverse effects of water-induced soil weakening. In summary, addressing the fundamental geotechnical issues through appropriate soil stabilization, enhancing construction techniques, and ensuring adequate drainage systems will significantly extend the durability and resilience of the road. A comprehensive and well-executed approach to road construction and maintenance is essential for preventing recurrent failures and ensuring the sustainable use of infrastructure.

## 5.2 Recommendations

Based on the findings and conclusions, several recommendations are made. First, a geotechnical study should precede roadway design and construction to ensure suitability and reduce long-term maintenance costs. Since the soil properties vary along the roadway, zones with high clay content must be treated carefully during reconstruction to prevent premature failure. Failed sections with troublesome clays should be excavated and replaced with competent materials before asphaltting. Understanding the geotechnical and geological characteristics is crucial for stable foundations. Additionally, geotechnical data for major highways should be readily available for better planning of road projects. Proper drainage systems are necessary in hilly areas to prevent erosion, and areas with excessive clay should be stabilized to enhance soil capacity.

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