



ORIGINAL RESEARCH ARTICLE

INVESTIGATION OF GEOTECHNICAL PROPERTIES OF COLLAPSIBLE SOILS
WITHIN MAIDUGURI METROPOLIS

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ARTICLE INFORMATION

Received: 30th April 2026

Revised: 8th May 2026

Accepted: 9th May 2026

Keywords:

Collapsible soil

Geotechnical properties

Oedometer

Quantitative

Sampling points

ABSTRACT

This work presents the experimental study to investigate the geotechnical properties of the collapsible soils within Maiduguri Metropolis, Borno state, Nigeria. Ten Soil sampling points (test pits) were considered from five different locations within the town and labelled as A₁-A₅ (Dikwa, Damboa, Baga, Jos and Bama roads). The samples were obtained with the aid of a hand auger, shovel, 100mm Polyvinyl chloride (PVC) pipes, scrapper and preserved in Polyethene bags and prepared wooden boxes to avoid moisture loss. Particle size distribution revealed that the soils were predominantly sand with traces of gravel and classified as A-3(0) according to the American Association of State Highway and Transportation Officials (AASHTO) and SP in accordance with the Unified Soil Classification System (USCS). The compaction characteristics showed that optimum moisture content (OMC) ranged between 9.0-15.1% and maximum dry densities (MDD) ranging between 1.57-1.92g/cm³ while colours ranged from brown to rich brown. The oedometer test affirmed the existence of collapsible soils in the studied locations while collapse potential (CP) ranged between 1.50-12.20%. The shear strength test revealed low values of cohesion (c) which ranged between 0-1 kPa with an angle of internal friction (φ) ranging between 5-9°. The ultimate bearing capacity result revealed a range between 14.40-86.80 kN/m² and safe bearing capacity ranging between 20.20-86.80 kN/m² with a mean safe bearing capacity of 46.26 kN/m². Hence the soils were classified as cohesionless.

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1.0 Introduction

Collapsible soils are regarded as one of the problematic soils found predominantly in arid and semi-arid regions of the world (Seiphoori and Zamanian, 2022). Collapsible soils typically consist of 50 to 90% silt, 7 to 82% fines and 15 to 75% sand. These wide ranges of percentages indicate that components depend on location (Lutenecker, 2019). Some of its basic features include; an open structure, low moisture content, high void ratio, low dry density, high porosity, geologically young or recently altered deposit, high sensitivity and, low inter-particle bond strength (Opukumo *et al.* 2022). These soils exhibit high strength in their natural-moisture state, but undergoes substantial settlement, when their moisture content increases or upon application of additional loads (Fatemeh *et al.* 2025). Construction activities on collapsible soil pose significant geotechnical challenges such as differential settlement, earth cracks, landslides, and embankment failures in road construction as well as loss of human lives in certain scenario (Anwer and Sarbast, 2019; Opukumo *et al.* 2022; Hamdy, 2024).

Presently, some parts of Maiduguri metropolis and environs is covered with many problematic soils such as deposits of light brown to yellowish brown ferruginous sand-stones which is popularly referred to as Bama ridge soils (Zarma, 2000; Kundiri *et al.* 2011; Hassan *et al.* 2012; Kundiri *et al.* 2016). In order to promote infrastructural development, the need to carry out a proper site investigation and propose appropriate mitigation measures is expedient.

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2. Materials and Method

2.1. Materials

Disturbed and undisturbed soil samples were collected from ten sampling points at different designated locations within Maiduguri Metropolis (Dikwa, Damboa, Baga, Jos and Bama roads respectively) at intervals of 20m and average depths of 1.5m for each of the points. The samples were obtained using hand auger, shovel, 100mm Polyvinyl chloride (PVC) pipes, scrapper and preserved in Polyethene bags and prepared wooden boxes and transported to the Civil and Water Resources Engineering Laboratory of the University of Maiduguri for testing.

2.2. Method

Preliminary tests conducted included; natural moisture content, Atterberg limits, particle size distribution, specific gravity and compaction. Furthermore, quantitative method using double oedometer in accordance with BS 1377 (2022) was adopted for the evaluation of collapse potential and direct shear box was considered in ascertaining the shear strength parameters of the soil.

2.2.1 Natural moisture content

This test was carried out in accordance with BS 1377:2: (1990) in order to ascertain the amount of water content present in the soil. Three moisture content cans were weighed to nearest 0.1g as m_1 and representative samples were crumbled and loosely placed in the cans. The cans with sample were weighed as m_2 and placed in the oven for 12-24 hours for drying and then the cans and its contents were weighed as m_3 . The moisture content of the soil was calculated using Equation 1;

$$w = \left(\frac{m_2 - m_3}{m_3 - m_1} \right) \times 100\% \quad 1$$

where w = Water content (%), m_1 = Weight of container (in g), m_2 = Weight of container with sample before oven drying (in g), m_3 = Weight of container with sample after oven drying (in g)

2.2.2 Atterberg limits

The Atterberg limits test was carried out to define the water content corresponding to different behavior conditions of silt and clay. The test was conducted in concordance with the American Society for Testing and Materials (ASTM) D 423 and D 424 (1998).

(i) Liquid Limit Determination: The liquid limit device was adjusted using adjustment plate on the device and the gauge on the handle of the grooving tool and ensured that the centre of the water point on the bottom of cup was lifted exactly 1 cm above the base. 100 grams of prepared soil was placed on the evaporating dish and 15 to 20 ml of water was added and mixed thoroughly with the soil sufficiently and formed a uniform mass of stiff consistency. The mixture in the cup was placed over the spot where the cup rested on the base and divided with a firm stroke of the grooving tool along the diameter through the centerline of the cam. The crank was turned to raise, dropped the cup twice per second until the two sides of the grooved sample came in contact at the bottom of the groove for a distance of 12.5 mm, and recorded the number of blows. The moisture content was adjusted by mixing, with or without the addition of water until all blows fell within the required range. The liquid limit device cup was cleaned after each trial test, the moisture content was determined by placing the sample in a tared aluminum can and weighed, and then, the test was repeated. Finally, the sample was oven dried at a temperature between 105 - 110° C to a constant weight and weighed after cooling. However, the numerical difference between the wet and dry weights was recorded as weight of moisture. Repetition of two to three trials was plotted against the number of blows. The moisture content and corresponding number of blows for the repeated trials of liquid limit were used to determine the liquid limit (W_L) at 25 blows.

(ii) Plastic Limit Determination: Twenty (20) grams of the soil sample was placed on evaporating dish and mixed thoroughly with water until the mass becomes plastic enough to be shaped into a ball. About 8 to 12 grams of the moistened soil was used and formed into a uniform mass roughly elliptical in shape and rolled the ball of soil by hand on a flat surface with just enough pressure to form an elongated thread as rolling proceeds. Hence, the soils cannot be rolled to a thread of 3mm.

(iii) Plasticity Index Determination (I_p): This is the numerical difference between its liquid limit and its plastic limit as shown in Equation 2;

$$(I_p) = W_L - W_P \quad 2$$

Where I_p = Plasticity index, W_L = Liquid limit (%), W_P = Plastic limit (%).

2.2.3 Particle size distribution

The test was conducted by using sieves to determine the amount and distribution of the particle size of the soils. The test was carried out in accordance with the American Society for Testing and Material (ASTM) D 6913-4 (2009). Representative air-dried samples were obtained and pulverized as finely as possible using mortar and pestle and obtained 500g of the soil sample as w_0 (g). Then, the set of sieves was arranged in descending order and a pan was placed under the last sieve to collect the portion that passed through sieve No (200). The sample was poured onto the sieves and shaken manually for 20 minutes and the mass of each sieve with retained soil were measured. The percentage retained and passing from each sieve were computed and finally values of D_{10} , D_{30} and D_{60} which are the diameter that corresponded to percentage finer of 10, 30 and 60% were obtained from the grain size distribution curve. Additionally, the values coefficient of uniformity C_u and coefficient of gradation C_c were computed using Equations 3 and 4;

$$C_u = \frac{D_{60}}{D_{10}} \quad 3$$

and

$$C_c = \frac{D_{30}^2}{D_{60} \times D_{10}} \quad 4$$

Where C_u = Coefficient of uniformity, C_c = Coefficient of gradation, D_{10} = Diameter that correspond to percentage finer of 10, D_{30} = Diameter that correspond to percentage finer of 30, D_{60} = Diameter that correspond to percentage finer of 60

2.2.4 Specific gravity (Gs)

This test was conducted according to B.S 1377:2: (1990) to determine the weight or density of the liquid as compared to the density of an equal volume of water at a specified temperature. The bottle with stopper was dried and weighed to the nearest 0.01g as m_1 , the first portion of the soil sample was transferred to the bottle directly from the sealed container and the weight of the bottle and its content was recorded to the nearest 0.01g as m_2 . Thereafter, air-free distilled water was added so that the soil in the bottle was covered, the bottle containing soil and water was placed in the vacuum desiccator and evacuated gradually. The bottle was allowed in the desiccator for at least one hour until no further loss of air was observed, then the vacuum was released and the lid of the desiccator was removed. The bottle was vibrated until no more air was released. The lid was replaced and evacuated again as specified above. The bottle was then removed from the desiccator and air-free water was added until the bottle was full. The stopper was inserted and immersed the bottle up to neck in the water bath for at least one hour. The bottle was removed from the bath again and the content was allowed to attain a constant temperature. Thereafter, bottle was taken out of the bath, wiped, dried carefully and weighed to nearest 0.01g as m_3 . Cleaned bottle was filled with air-free water and inserted the stopper and immersed in constant temperature water bath for an hour. The bottle was removed from the bath, wiped, dried carefully and weighed to the nearest 0.01g as m_4 . The process was repeated for the second and third specimens of the same soil and obtained 2-3 values of the particle density. The specific gravity (Gs) of the soil sample was computed based on Equation 5;

$$G_s = \frac{m_2 - m_1}{(m_4 - m_1)(m_3 - m_2)} \times 100 \quad 5$$

Where;

G_s = Specific gravity, m_1 = Weight of bottle with stopper (in g), m_2 = Weight of the bottle and its content (in g), m_3 = Weight of bottle without stopper removed from the bath after drying (in g), m_4 = Weight of bottle with stopper removed from the bath after drying (in g).

2.2.5 Compaction

The standard proctor or British standard Light compaction test was conducted in the laboratory in order to obtain the maximum dry densities and the optimum moisture contents. The test was conducted according to ASTM- D 1557-98: (2000). An empty mould and the base plate attached was weighed to the nearest 1g as m_1 , extension collar was attached and placed on the mould on a solid base. Moist soil sample was placed in the mould such that when compacted it occupied at least 1/3 of the height of the mould and a rammer with guide on the material was placed in the mould. The handle of the rammer was lifted until it reached the top of the guide, then released allowing to drop freely from a height of 300mm on to the sample. The position of the guide was shifted and dropped the rammer again and repeated the process, systematically covering the entire surface of the sample. A total of 27 blows was applied, and a subsequent layer of sample in the mould up to a third layer. When all the three layers had been compacted, the extension collar was removed and the excess soil was trimmed and levelled using the straight edge. Coarse particles encountered in the levelling process were replaced by finer materials from the sample of the soil and the mould with base plate attached weighed to the nearest 1.0g as m_2 . The compacted sample was removed from the mould and a representative sample of minimum of 300g was obtained for determination of moisture content and the process was carried out for all the portions of the sample, and the bulk and dry densities were calculated from the Equations 6 and 7;

$$\text{Bulk density, } \rho = \frac{m_2 - m_1}{V} \times 100 \text{ (in kg/m}^3\text{)} \quad 6$$

$$\text{Dry density, } \rho_d = \frac{100\rho}{100 + W} \quad 7$$

where m_1 = Mass of mould and base plate (in g), m_2 = Mass of mould, base plate and compacted soil (in g), V = Volume of the mould (in cm^3), W = Moisture content of soil (in %).

The graph of dry densities against the corresponding moisture contents was plotted on which a curve of best-fit points was plotted and identified the positions of the maximum densities (mdd) on these curves and the corresponding values of the moisture contents were considered as the optimum moisture contents (omc) to three significant figures.

2.2.6 One-dimensional double oedometer

One-dimensional double Oedometer test was conducted on the specimens according to BS 1377:5:1990 in order to ascertain the settlement behaviour of soil. The height of the specimen was measured using a vernier caliper to 0.05 mm as H_0 and the specimen was placed in the ring on the tray and weighed to 0.1g as m_1 . The initial mass of the specimen m_0 was obtained based on Equation 8;

$$m_0 = m_1 - m_{ring} - m_{tray} \quad 8$$

where m_0 = Initial mass of the specimen (in g), m_1 = Weight of specimen in the ring and the tray (in g), m_{ring} = Weight of the ring (in g), m_{tray} = Weight of the tray (in g).

The initial moisture content of the soil sample was also computed. The porous plates used in the test were cleaned and ensured that the pores were not clogged with fine soil particles, and were readily permeable. Consolidation cell was placed in position on the bed of the loading apparatus and adjusted the counter balance using loading beam to allow the load-transmitting members just make contact with the loading cap. Additional weight was added to the beam hanger to maintain contact between the load transmitting members while final adjustments were made. The compression gauge was securely clamped into position so that it can measure the relative movement between the loading cap and the base of the cell gauge to for measurement of an amount of swelling of the specimen. The greater part of the range of travel was allowed for compression as the initial reading of the gauge was recorded. However, a range of pressure selected was 12.5, 25, 50, 100, 200, kN/m^2 (kPa) as stated on the machine's manual used. Compression gauge reading was considered as the initial reading for the load increment stage as d_1 . The required pressure was applied to the specimen at a convenient moment (zero time) by adding the appropriate weight to the beam hanger as stated above without jolting and the weight used for the seating load was removed. One of the consolidation cells was filled with

water after applying the pressure, while the specimen was kept for 24 hours before proceeding to the next higher pressure. The readings of the compression gauge were taken at suitable intervals of time using periods of elapsed time from zero as convenient, in the following sequence: 0, 15, 30 seconds, 1, 2, 4, 8, 15, 30 minutes and 1, 2, 4, 8, 12 and 24 hours for the last load increment considered. The compression gauge readings against square root of time were plotted, while the test was in progress, manually and the pressure maintained for 1 hour with the plotted readings confirming to t_{90} have been reached. The time and compression gauge reading at the termination of the load increment stage was recorded as d_1 . This reading became the initial for the next stage. Then, the pressure was increased to the next value in the selected sequence. By and large, unloading was done in one step (to P_0 or to the initially applied pressure). The final readings was recorded, plotted and proceeded to "Dismantling". The pressure was reduced to a value not less than the last, but one value of the loading sequence at a convenient moment (zero time) and recorded the reading of the compression gauge at convenient intervals. The readings were plotted so that the completion of swelling can be identified and the reading of the compression gauge recorded. The water from the cell was drained off and allowed to stand for 15 minutes to enable free water to drain from the porous plates. The excess water from within the cell was mopped up, and the load was removed from the specimen and the consolidation cell. The cell was dismantled while the specimen in its ring were placed on the watch glass or tray and weighed. The specimen and the ring were transferred on the tray to the oven and maintained at a temperature of 105°- 110° C. The dried mass of the specimen was determined to 0.1g as m_d .

The initial moisture content, W_0 (in %) was calculated from the specimen trimmings using Equation 9;

$$w_0 = \frac{m_0 - m_d}{m_d} \times 100 \% \quad 9$$

The initial bulk density, (ρ) was calculated based on Equation 10;

$$\rho = \frac{m_0 \cdot 1000}{A H_0} \quad 10$$

Where m_0 = Initial mass of the specimen (in g), A = Area of the specimen (in mm²), H_0 = Initial height of the specimen (in mm)

In addition, the initial dry density, ρ_d (Mg/m³) was calculated based on Equation 11;

$$\rho_d = \frac{100\rho}{100\rho + w_0} \quad 11$$

The initial voids ratio e_0 was calculated using Equation 12;

$$e_0 = \frac{\rho_s}{\rho_d} - 1 \quad 12$$

Where ρ_s = Particle density (in Mg/m³).

Initial degree of saturation S_i was calculated as a percentage using Equation 13;

$$S_i = \frac{w_0 \rho_s}{e_0} [in \%] \quad 13$$

Where S_i = Initial degree of saturation (in %), w_0 = Initial moisture content (in %), ρ_s = Particle density (in Mg/m³), e_0 = Initial voids ratio

However, the coefficient of volume compressibility was obtained using the stages of computation starting with the equivalent height of solid particles, H_s (mm), as in Equation 14;

$$H_s = \frac{H_0}{1 + e_0} \quad 14$$

Where H_s = Height of solid particles (mm), H_0 = Initial height of the specimen (mm), e_0 = Initial voids ratio

Then, the height of the specimen, H (mm), at the end of each loading or unloading stage was calculated based on the Equation 15;

$$H = H_0 - \Delta H \quad 15$$

Where H = Height of the specimen (mm), H_0 = Initial height of the specimen (mm), ΔH = Cumulative compression of the specimen (reduction in height) from the initial height as recorded by the compression gauge (mm)

The void ratios, e, at the end of each loading or unloading stage, which was obtained from Equation 16;

$$e = \left(\frac{H - H_s}{H_s} \right) \quad 16$$

Finally, coefficient of volume compressibility, m_v (m^3/MN), for each loading increment was obtained from the Equation 17;

$$m_v = \left(\frac{H_1 - H_2}{H_1} \right) \cdot \frac{1000}{(p_2 - p_1)} \quad 17$$

Where H_1 = Height of the specimen at the start of a loading increment (in mm), H_2 = Height of the specimen at the end of that increment (in mm), P_1 = Pressure applied to the specimen or the previous loading stage (in $kN/m^2 = kPa$), P_2 = Pressure applied to the specimen for the loading stage (in $kN/m^2 = kPa$) being considered.

The values of the void ratios as ordinate against applied pressure on the logarithmic scale as abscissa were drawn with smooth curves through the points for both the loading and unloading portions as well as the swelling pressure were measured. Such curves will start and terminate at swelling pressure, indicating the value of the initial void ratios, e_0 , on the vertical axis. Finally, the collapse potential C_p for each pit was calculated based on Equation 18;

$$C_p = \frac{\Delta ce}{(1 + e_0)} \times 100 \quad 18$$

Where C_p = Collapse potential (%), Δce = Void ratio during saturation, e_0 = Initial void ratio.

2.2.7 Direct shear test

This test was carried out in accordance with B.S 1377:2:1990 to ascertain shear strength behaviour of the soils. The inner dimension of the soil sampler and the parts of the direct shear apparatus was checked and arranged together, such that the volume and the weight of the sampler were also calculated, then the sample was placed in three smooth layers (approximately 10mm thick each before tamping). The soil specimen was placed inside the direct shear apparatus and the upper porous stone, pressure pad and loading block placed on top of soil. The dial gauge and proving ring were adjusted to zero position after setting up the specimen. The desired normal stress of $0.5kg/cm^2$ was applied, water at the top of direct shear box set up was added and then waited for at least 20 minutes to ensure saturation and the shear pin was removed. The final vertical dial gauge reading was measured for the deformation in vertical direction due to saturation. The initial reading of the dial gauge and proving ring values were recorded before starting the shearing. All adjustments were checked to ensure that there was no connection between the two parts except sand/ soil. The strain-controlled frame was fixed to the required strain rate and the reading of the shear force in proving ring with respect to the change in horizontal dial gauge reading and vertical deformation in vertical dial gauge till failure were recorded. Finally, the process was repeated for another two normal stress ($1.0 kg/cm^2$ and $1.5kg/cm^2$).

The shear stress (τ) on the horizontal failure plane was calculated as in Equation 19;

$$\tau = S/A \quad 20$$

Where S = Shear force (kN); and A = Cross-sectional area of the sample, which decreases slightly with the horizontal deformation (m^2)

The corrected Area (A_{corr}) was calculated for shear stress at failure using Equation 20;

$$A_{corr} = A_0 - (6 * \delta) \quad 20$$

where δ = Horizontal displacement due to shear force applied on specimen, A_0 is the initial area of the specimen. A_0 and δ (in cm).

It could be inferred that:

- a. Shear stress = (Proving ring reading \times proving ring constant/A_{corr})
- b. Horizontal displacement = Horizontal dial gauge reading \times least count of vertical dial gauge.
- c. Vertical displacement = Vertical dial gauge reading \times least count of vertical dial gauge
- d. Shear stress at failure needs was calculated for all the three tests performed at three different normal stresses and the failure envelope was plotted.

2.2.8 Analysis of soil bearing capacity

Bearing capacity of the soil was computed based on the Terzaghi's equation for strip foundation, while the bearing capacity factors which are functions of shearing resistance of the soil sample obtained from direct shear box test as shown in Equation 21 and 22;

i. Ultimate Bearing capacity calculation

$$q_u = cN_c + \gamma D_f N_q + 0.5 B N_{\gamma} \quad 21$$

where, q_u = Ultimate bearing capacity, c = Cohesion, γ = Unit weight of water, D_f = Depth of foundation, B = Width of foundation, N_c , N_q , N_{γ} = Bearing capacity factors depending on the angle of internal friction.

ii. Safe Bearing capacity

$$q_{(safe)} = \frac{q_u}{F.S} \quad 22$$

where $q_{(safe)}$ = Safe bearing capacity, q_u = Ultimate bearing capacity, F.S = Factor of safety (Usually between 2.5-3)

3. Results and Discussion

3.1 Index Properties Tests

Gradation characteristics of the soils revealed that the soils were predominantly sand and gravel and classified according to the American Association of States Highway and Transportation Officials (AASHTO 1986) as A-3 and SP in accordance with the Unified Soil Classification System (USCS) ASTM D2487 (2006). The compaction characteristics showed that optimum moisture content (OMC) ranged between 9.0-15.1% and maximum dry densities (MDD) range of between 1.57-1.92g/cm³. While the colour of the soil ranged from brown at Dikwa, Damboa and Baga roads to reddish brown at Jos and Bama roads respectively (Kalsari *et al.* 2024). Some of the properties of the soil are shown in Table 1.

The grain size distribution analysis depicted in Figure 1. showed that the soil consists of 8-44% gravels and 56-92% sand. Sampling point A₄T₁ of Jos road showed highest percentage passing while point A₅T₁ of Bama road revealed lowest percentage passing.

3.2 Quantitative Method of Evaluating Collapsible Soils

The Severity of collapse was achieved in accordance with Jennings and Knight, (1975) by determining the differences in void ratio between the saturated and unsaturated samples at a maximum loading of 200kN/m²

as shown in Table 2, while Figures 2-6 illustrated the stress history experienced by the soils which revealed that the unsaturated soil samples had higher void ratios and lower dry densities than the saturated soil.

Table 1: Geotechnical properties of soils

Location	Sampling point	Grading (AASHTO) characteristics				Classification	Atterberg limits			Compaction	
		Gravel (%)	Sand (%)	Silt (%)	Clay (%)		AASHTO	USCS	LL	PL	OMC (%)
Dikwa Road	A ₁ T ₁	36	64	0	0	A-3(0)	SP	21.0	NP	9.0	1.92
	A ₁ T ₂	20	80	0	0	A-3(0)	SP	19.0	NP	11.0	1.57
Damboa Road	A ₂ T ₁	20	80	0	0	A-3(0)	SP	20.0	NP	12.0	1.91
	A ₂ T ₂	11	89	0	0	A-3(0)	SP	19.9	NP	10.0	1.82
Baga Road	A ₃ T ₁	24	76	0	0	A-3(0)	SP	15.0	NP	10.0	1.75
	A ₃ T ₂	30	70	0	0	A-3(0)	SP	20.0	NP	12.5	1.84
Jos Road	A ₄ T ₁	16	84	0	0	A-3(0)	SP	16.0	NP	13.0	1.91
	A ₄ T ₂	8	92	0	0	A-3(0)	SP	15.9	NP	12.0	1.92
Bama Road	A ₅ T ₁	44	56	0	0	A-3(0)	SP	17.0	NP	15.0	1.87
	A ₅ T ₂	28	72	0	0	A-3(0)	SP	16.5	NP	15.1	1.90

Legend: A_{1, 2, 3, 4} and A₅ = Test Locations while T₁ and T₂ = Test pits, LL= Liquid Limits, PL= Plastic Limits, NP= Non plastic

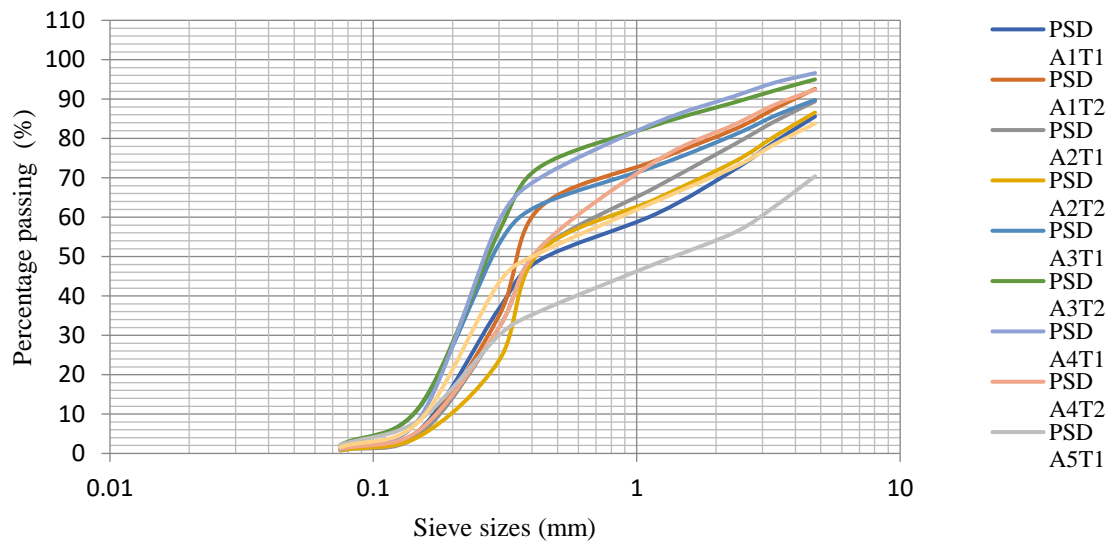


Figure 1: Particle size distribution plots of soil from the study area

Table 2: Severity of Collapse potential using double oedometer test

Location	Sampling point	Collapse potential CP (%)	Severity of collapse
Dikwa Road	A ₁ T ₁	4.60	Medium collapsibility
	A ₁ T ₂	3.00	Medium collapsibility
Damboa Road	A ₂ T ₁	1.50	Medium collapsibility
	A ₂ T ₂	12.20	Very highly collapsible
Baga Road	A ₃ T ₁	1.80	Medium collapsibility
	A ₃ T ₃	4.60	Medium collapsibility
Jos Road	A ₄ T ₁	11.80	Very high Collapsible
	A ₄ T ₂	18.10	Very high Collapsible
Bama Road	A ₅ T ₁	3.20	Medium collapsibility
	A ₅ T ₂	18.10	Very high Collapsibility

The variation in void ratio between the saturated and unsaturated samples at sampling point A₁T₁ and A₁T₂ for Dikwa road were observed to be 0.081559 and 0.050469 with a collapse potential of 4.60 and 3.0%. Thus, the soil was considered as medium collapsible soil, which might be as a result of high natural moisture content of the soils. Similar scenarios were earlier reported by (Nanine, 2010; Awolabi and Ola, 2014 and Kalsari, 2024). This is presented in Figure 2.

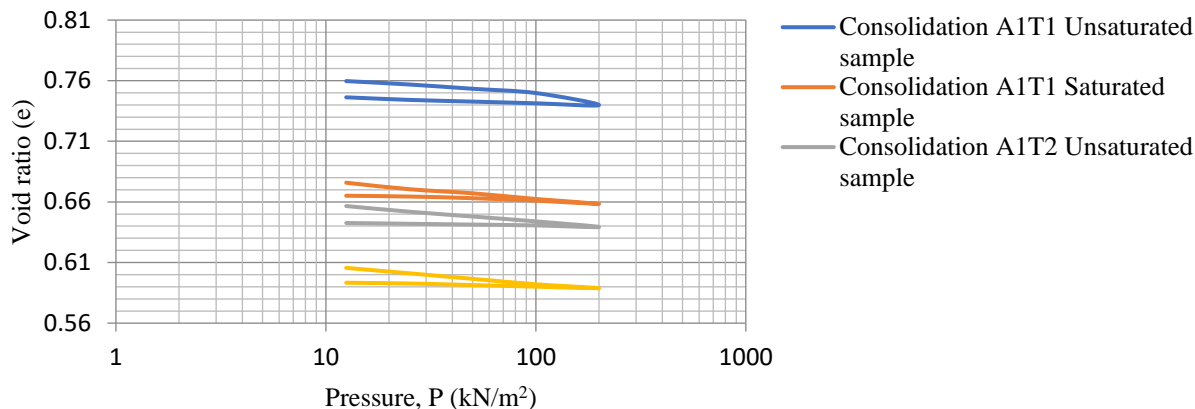


Figure 2: Void ratio- Pressure curve for Dikwa road at station (A1)

Sampling point A_2T_1 for Damboa road revealed a variation of 0.044263 in void ratio between the saturated and unsaturated samples and 1.50% collapse potential, which made the soil to be medium collapsible soil. Similarly, A_2T_2 gave the void ratio difference of 0.308677 with collapse potential of 12.20%, which implied that soil is very highly collapsible soil. This might be as a result of high void ratio and lower initial moisture content of the soils as confirmed by (Kalsari, 2024). This could be seen in Figure 3.

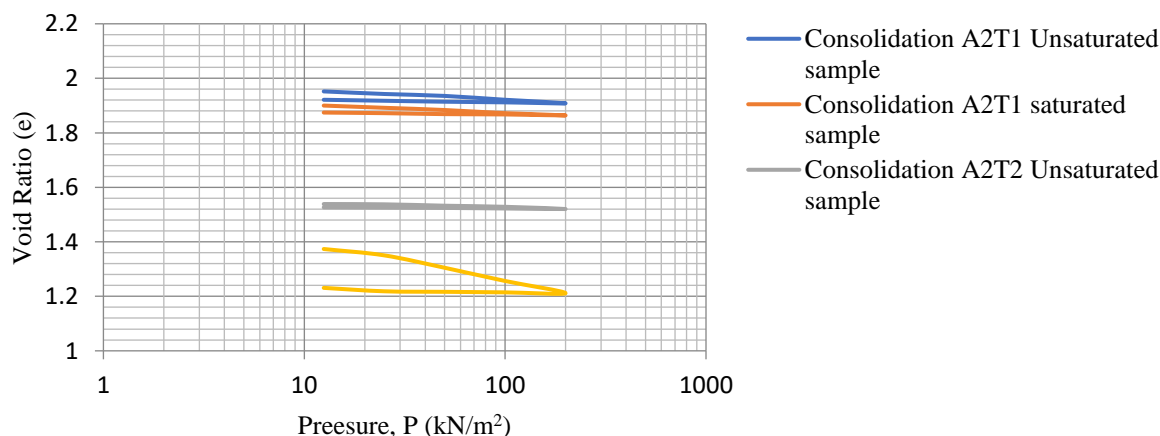


Figure 3: Void ratio- Pressure curve for Damboa road at station (A2)

Sampling Points A_3T_1 and A_3T_2 for Baga road yielded changes in void ratio between the saturated and unsaturated samples was 0.057407 and 0.045517 with a collapse potential (C_p) of 1.80 and 4.60% respectively and classified soil as medium collapsible soils which might be as a result of good bonding and cementation agents of the soils as depicted in Figure 4.

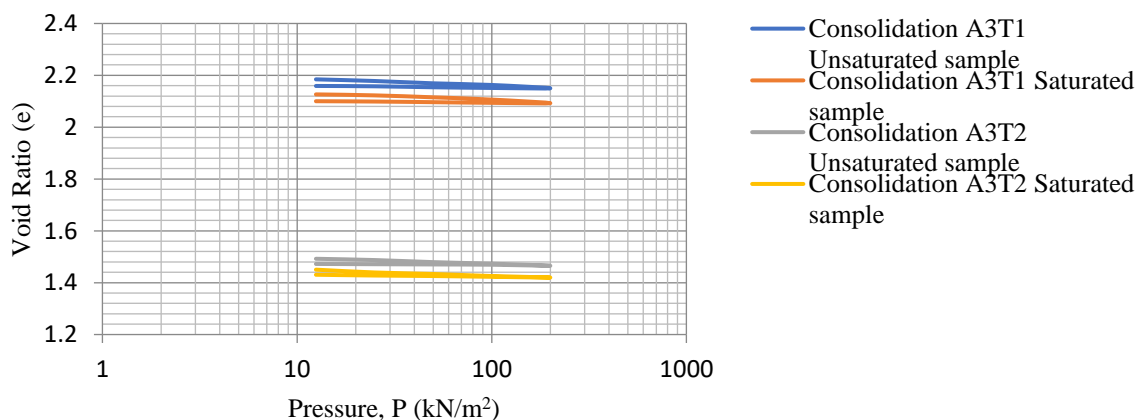


Figure 4: Void ratio- Pressure curve for Baga road at station (A3)

The differences in void ratio between the saturated and unsaturated samples at points A_4T_1 and A_4T_2 for Jos road gave 0.363756 and 0.469978 with collapse potential (C_p) of 11.80 and 18.10% respectively, which classified the soil as very highly collapsible. This might be as a result of high percentage finer of the soils as shown in Figure 5 and is in concordance with the study by (Kalsari, 2024).

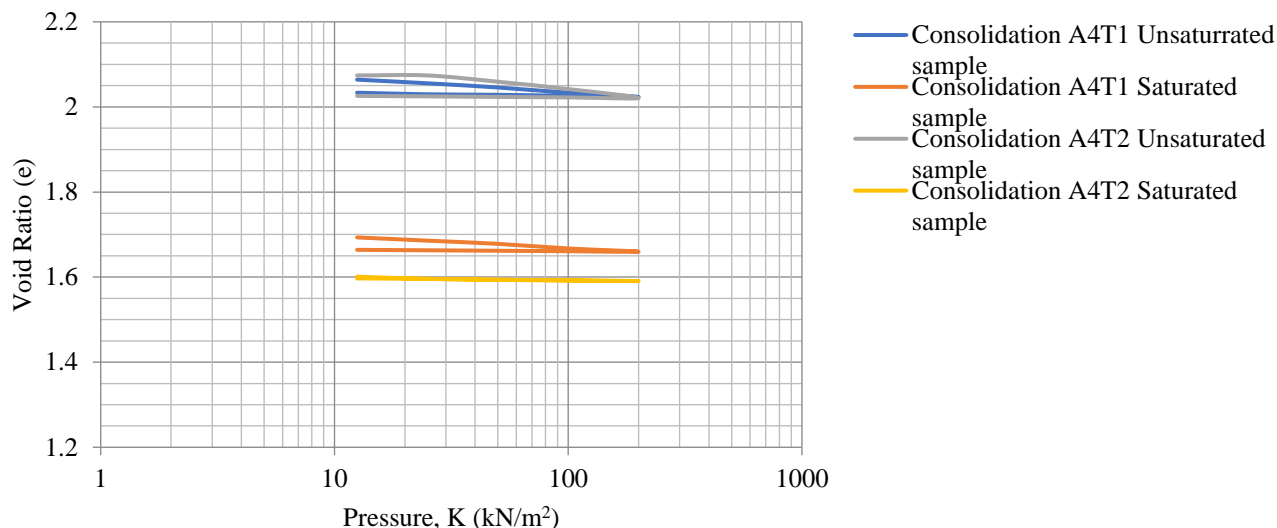


Figure 5: Void ratio- Pressure curve for Jos road at station (A4)

The double oedometer test results obtained for Bama road at sampling points A₅T₁ and A₅T₂ showed that the void ratio changed between the saturated and unsaturated sample at maximum loading of 200kN/m² as 0.082366 and 0.469978 with collapse potential of 3.20 and 18.10%. This soil was classified as medium and very highly collapsible soils according to (ASTM, 2003), which might be as a result of low initial void ratio and high percentage finer of the soil. This is presented in Figure 6.

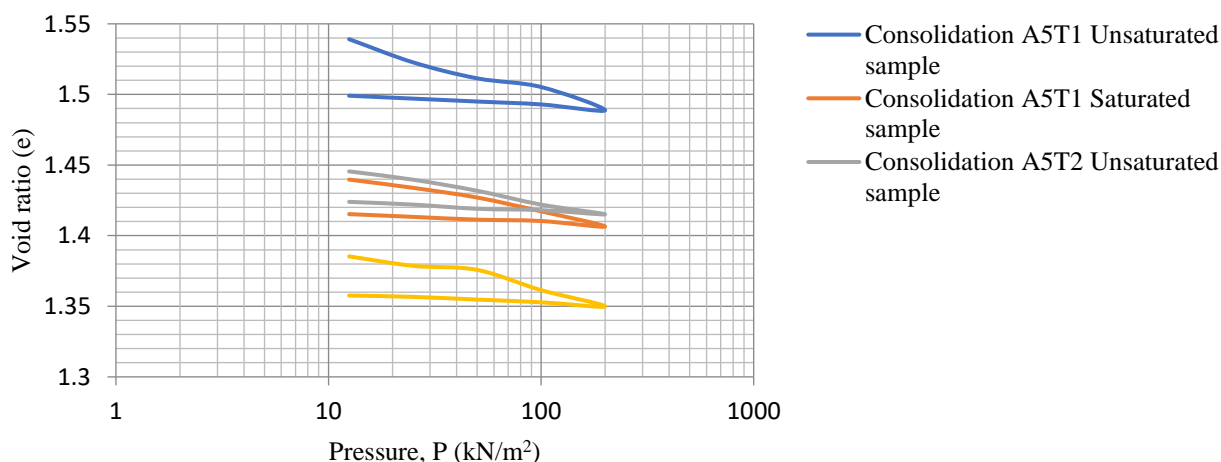


Figure 6: Void ratio- Pressure curve for Bama road at station (A5)

3.3 Shear strength parameters of Collapsible soils

The shear strength parameters results of the entire sampling points showed low values of cohesion (c) and hence the soils are regarded as cohesionless which is in agreement with the findings of (Nanine, 2010; Awolabi and Ola, 2014). The shear strength parameters for the soil were shown in Table below.

Table 3: Shear Strength Parameters

Location	Sampling Points	Cohesion (c) kPa	Angle of Internal friction (ϕ)
Dikwa Road	A ₁ T ₁	1	7
	A ₁ T ₂	1	9
Damboa Road	A ₂ T ₁	0	9
	A ₂ T ₂	1	8
Baga Road	A ₃ T ₁	0	5
	A ₃ T ₂	1	7
Jos Road	A ₄ T ₁	0	7
	A ₄ T ₂	1	6
Bama Road	A ₅ T ₁	1	9
	A ₅ T ₂	0	8

The bearing capacity values obtained were illustrated in Table 4.

Table 4: Bearing Capacity values

S/No	Location	Sampling points	Ultimate bearing capacities (kN/m ²)	Safe bearing capacities (kN/m ²)
1	Dikwa Road	A ₁ T ₁	66.50	86.80
		A ₁ T ₂	22.20	28.90
2	Damboa Road	A ₂ T ₁	62.10	70.45
		A ₂ T ₂	20.70	23.50
3	Baga Road	A ₃ T ₁	43.30	67.90
		A ₃ T ₂	14.40	22.60
4	Jos Road	A ₄ T ₁	55.80	60.60
		A ₄ T ₂	18.60	20.20
5	Bama Road	A ₅ T ₁	86.80	61.20
		A ₅ T ₂	28.90	20.40
Mean safe bearing capacity				46.26

4. Conclusion

The index properties revealed that the soils were cohesionless and categorized as A-3(0) according to American Association of States Highway and Transportation Officials and SP in conformity with the Unified Soil Classification System (USCS) with colour ranging from brown to rich brown. The application of the quantitative method using the oedometer test confirmed the existence of collapsible soils with the severity of collapse varying from medium to highly collapsible soil. The shear strength test showed an indication for collapsibility potentials in the soil with low shear strength parameters of 0-1kPa cohesion and 5-9° angle of internal friction with a mean safe bearing capacity of 46.26kN/m². Therefore, for the medium collapsible soils, it is recommended to propose remediation techniques such as prewetting (presoaking), compaction (mechanical densification), chemical stabilization, soil replacement and grouting with foundation designs of raft (mat), strip or spread footing (after improvement), pier or under-reamed as well as piles foundations, while for the highly collapsible soils, mitigation measures such as removal and replacement, deep compaction (dynamic compaction), chemical stabilization (cement lime and fly-ash), grouting, prewetting with controlled compaction as well as stone column/vibro-replacement with foundation designs of pile (most common), pier, raft (mat), pile-raft foundation were recommended.

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