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Development of Shear Strength Model for Banded Gneiss Derived Soils

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Abstract: *This study developed a model for determining the shear strength of lateritic soils in a Banded Gneiss geologic unit. This was with a view to determining the effects of index properties and geology on the shear strength of selected soils. Lateritic soil samples were collected from selected locations within the Banded Gneiss geologic unit of the Obafemi Awolowo University (OAU) campus, Ile-Ife, Southwestern Nigeria. Soil samples were subjected to laboratory analyses for index and other geotechnical properties relevant in the determination of shear strength. Relationships between shear strength and index properties were determined by developing and validating Multivariable Least Square Regression (MLSR) model. The empirical relationship (model) between shear strength, τ (kN/m^2) and index properties (natural moisture content w, in %; fines content, fc in %; coefficient of curvature, C_c; and plastic limit, PL in %), which was found to be valid for the studied geologic unit is, $\tau = 27.268+1.594w-0.242fc+0.092C_u+0.510PL$. This study concluded that the index properties have significant contribution to the shear strength of soils. The model could also be used to determine the shear strength of soils derived from Banded Gneiss geologic unit.*

Keywords: *banded, geology, gneiss, model, shear*

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I. Introduction

Soil supports different types of structures like buildings, roads, railway lines, pipelines, etc. Accurate determination of the geotechnical properties can therefore enhance good design of foundation of such structures (Too, 2012). One of the most important engineering properties of soil is the shear strength, which is its ability to resist sliding along internal surfaces within a mass, that is, the shearing resistance offered by the soil along probable surfaces of slippage (Roy and Dass, 2014).

Accurate determination of the soil shear strength is a major concern in the design of different geotechnical structures. This property can be determined either in the field or in the laboratory. The various methods of determining shear strength of soil include direct shear test, triaxial shear test, unconfined compression test and vane shear test. The triaxial, unconfined compression and direct shear tests are the most common laboratory tests. The direct shear test is commonly used for sandy soils and its procedure is simpler, in comparison with the triaxial and unconfined compression tests. Vane shear test is employed in the field (El-Maksoud, 2006; Murthy, 2008; Mousavi *et al.*, 2011a; Mollahasani *et al.*, 2011).

Estimation of shear strength of soil requires the preparation of numerous samples and the use of expensive laboratory equipment. In addition, the experimental determination of shear strength can provide inaccurate results, especially due to difficulties in obtaining undisturbed samples. Besides, the regularly used methods require expensive equipment and the laboratory tests are time-consuming. (Ersoy *et al.*, 2013). It is therefore obvious that the determination of shear strength of soil in easy and economical ways will be of considerable advantage to geotechnical engineers (Eid, 2005). Experimental determination of shear strength is cumbersome and costly. It is therefore desirable to find simpler, quicker and cheaper methods of determining shear strength of soils. And, since the index property tests of soil are relatively simple to perform, attempts have been made in the past to develop models for the determination of shear strength of soils from the index properties.

Several empirical procedures have been developed and proposed to predict the shear strength of soils, particularly unsaturated soils. Some of these procedures used the soil-water characteristic curve (SWCC) as a tool along with the shear strength properties, cohesion (c), and angle of internal friction (ϕ), to predict the shear strength function for unsaturated soils (Escario and Juca, 1989; Vanapalli, 1996; Oberg and Salfours, 1997; Miao *et al.*, 2002; Tekinstoy *et al.*, 2004; Nam *et al.*, 2011, Zhou *et al.*, 2012).

The ratio of undrained shear strength (S_u) to overburden stress (σi) of soft clay in Pontianak, Indonesia had been correlated with Atterberg limits by many researchers. Skempton (1957) proposed a linear relationship for this ratio value to the value of plasticity index (Ip): ($Su/\sigma i = 0.11 + 0.0037$ Ip). Bjerrum and Simons (1960) also presented a power equation for correlation between undrained shear strength and plasticity index (Ip) and liquidity index (I_L): ($Su/\sigma i = 0.045\ Ip^{0.5}$); ($Su/\sigma i = 0.18/I_L^{0.5}$). Karlsson and Viberg (1967) later proposed a linear equation for correlation of undrained shear strength with liquid limit (W_L): $Su/\sigma i = 0.005\ W_L$.

Wroth and Wood (1978) developed model for undrained shear strength (Su) of soils with respect to the liquidity index (I_L): $Su = 170e^{-4.6I_L}$. Subsequently, Leroueil *et al*. (1983) proposed another model for undrained shear strength of soil: $Su = 1/(I_L - 0.21)^2$. The authors indicated that the model is valid for $0.5<I_L<2.5$). Later, Locat and Demers (1988) sought a model that would be valid for higher values of I_L, and thus came up with the model $Su = (1.167/I_L)^{2.44}$, which is valid for the range of liquidity index $1.5<I_L<6.0$. Ojuri (2013) also developed statistical models to predict shear strength of some tropical lateritic soils, using maximum dry density (MDD) and group index (GI) values of soil. The author came up with a significant model for undrained shear strength ($S_u = -547.713 + 0.381MDD - 9.104GI$), and thus concluded that a quick evaluation of shear strength could be made for the selected soils using maximum dry density and group index. Recently, Vardanega and Haigh (2014) developed and proposed a relationship between liquidity index and undrained shear strength of soil. The model is $I_L = 1.150 - 0.283\ln(S_u)$.

According to Vanapalli (1996), the empirical procedures for estimating shear strength of soils may or may not be suitable for all types of soil. Relationships between geotechnical engineering properties, and more specifically, shear strength and simple soil index properties vary across regions (Roopnarine *et al.*, 2012). Previous attempts are also region-specific and thus necessitate the need for localised investigations (Adunoye, 2017). The specific contribution of geology has also not been studied. There is therefore the need to develop models, which will take cognisance of the geology, and that can be easily employed for determining shear strength of soil. This present study therefore identified and modelled relationships between shear strength and index properties of soils derived from Banded Gneiss geologic unit within the Obafemi Awolowo University (OAU) campus, Ile-Ife.

II. Materials and Methods

Lateritic soil samples were collected from 25 different locations within identified Banded Gneiss geologic unit in OAU campus. Preliminary, index property, compaction and triaxial tests were conducted on the samples. The tests were conducted using standard methods. The triaxial tests were conducted to determine the shear strength properties of the soil. Empirical relationship between shear strength and selected index properties was determined using multiple regression analysis. The developed models were subsequently validated.

Description and Geology of the Study Area

The study area is the OAU campus, Ile-Ife, Southwestern Nigeria. Ile –Ife lies between Latitudes 7°28’0’’N and 7°45’0’’N and Longitudes 4°30’0’’E and 4°34’0’’E. Figure 1 shows the map of Obafemi Awolowo University, Ile-Ife. The Obafemi Awolowo University is located within the Ife-Ilesha Schist Belt. The campus falls within the Basement Complex area of Nigeria (Durotoye, 1983). The rock types are primarily made up of Gneisses and Mica Schists into which some minor granitic and basic rocks have intruded. The main lithological units are: Banded Gneiss, Granite Gneiss and Mica Schist (Boesse, 1989 and Figure 2).

Materials and Equipment

The main material used are lateritic soil samples collected from Banded Gneiss geologic unit in the study area. The list of equipment used is contained in Table 1. They are all available at the Geotechnical Engineering Laboratory of Department of Civil Engineering, Obafemi Awolowo University, Ile-Ife.

Methods

The following are the methods employed in this study.

Soil sampling

Twenty-five sampling locations were identified within Banded Gneiss geologic unit in the study area. The soil samples were collected at approximately one sample per 200 m^2. Test pits were dug by hand and excavated with the aid of digger and shovel. The depth of sample collection was 0.5 m – 1 m (Arora, 1988; Roy and Das, 2014). 20 - 25 kg of each sample was collected into a nylon, sealed and immediately taken to the Geotechnical Laboratory of the Department of Civil Engineering, OAU, Ile-Ife, for analyses. After determining the initial moisture content, the samples were prepared for subsequent laboratory analyses by air-drying and grinding to pass a 2 mm sieve.

Preliminary and index property tests on soil samples

In the Laboratory, representative samples were taken and used for the determination of natural moisture content. Further geotechnical analyses/tests carried out on soil samples included: particle size analysis, specific gravity test, and Atterberg limits test.The tests and analyses were conducted following standard procedure as contained in BS 1377 (1990). Effectives sizes (D_{10}, D_{30} and D_{60}), uniformity coefficient (C_u) and Coefficient of curvature (C_c) were also determined from the particle size distribution curves and Equations 1 and 2.

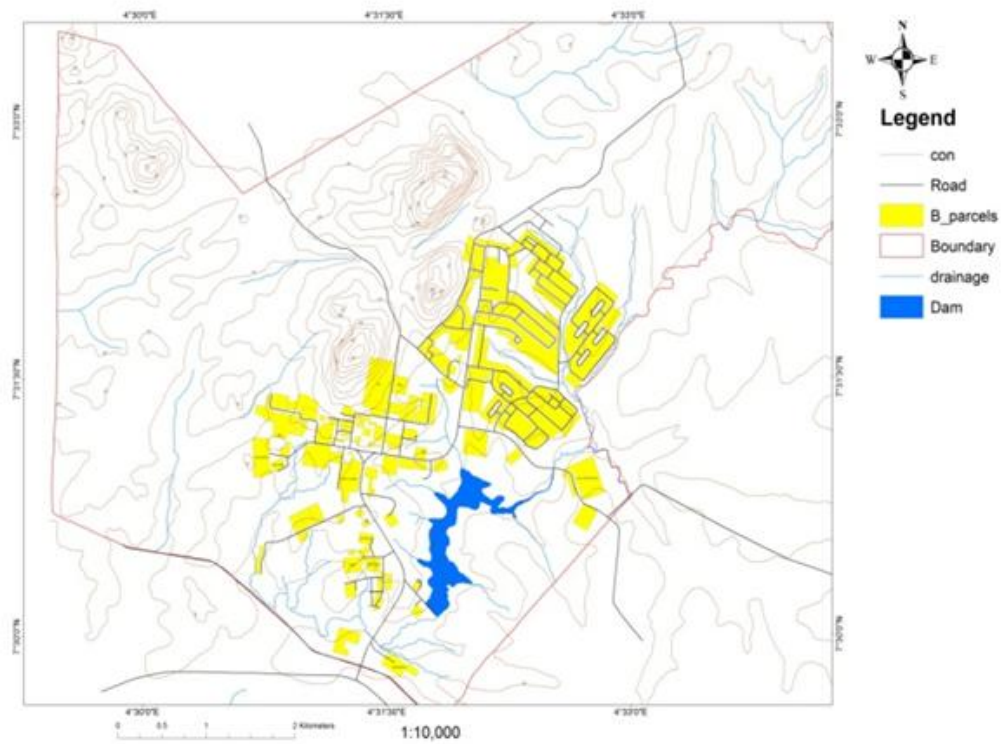


Figure 1: Map of Obafemi Awolowo University, Ile-Ife (OAU, Ile-Ife, 2015)

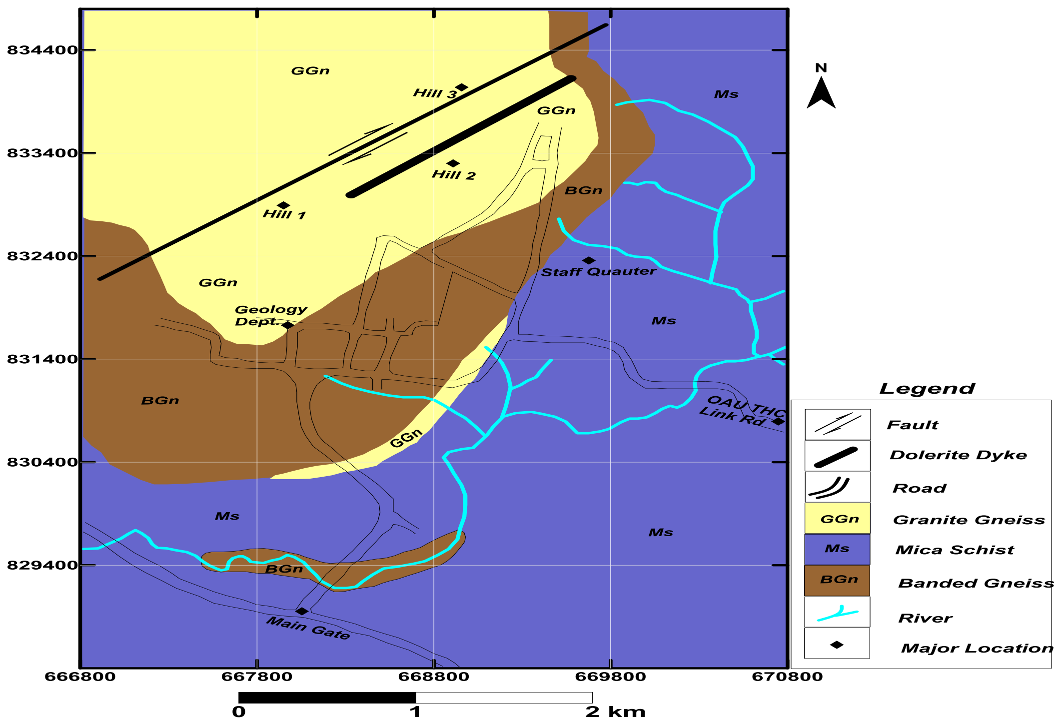


Figure 2: Geological Map of the Obafemi Awolowo University, Ile-Ife (After Boesse, 1989)

Table 1: List of equipment

Equipment	Purpose
Set of Sieves (4.5mm to 0.06mm)	Particle size analysis (coarse grain)
Sieve Shaker	Shaking of soil sieves
Hydrometer Bulb	Particle size analysis (fine grain)
Specific Gravity Bottle	Specific gravity determination
Atterberg Apparatus	Plastic and liquid limits determination
Electric Oven (Temp 105^{o}C to 110^{o}C)	Drying of moist soil sample
Weighing balance	Weighing of soil
Measuring Cans	Measurement
Compaction Moulds and Rammers	Compaction test
Triaxial Machine	Determination of shear strength parameters

$C_c = D_{30}^2/(D_{60} \times D_{10})$ (1)
$C_u = D_{60}/D_{10}$ (2)
Where D_{10} = Grain size for which 10% of the sample is finer
D_{30} = Grain size for which 30% of the sample is finer
D_{60} = Grain size for which 60% of the sample is finer

Compaction and triaxial tests on soil samples

The compaction test was conducted in accordance with BS 1377 (1990). The corresponding dry density (DD) and moisture content (MC) were eventually evaluated. A graph of DD versus MC was then plotted to obtain the appropriate maximum dry density (MDD) and the corresponding optimum moisture content (OMC) for the soil samples.

Unconsolidated-undrained (UU) triaxial tests were conducted on the soil samples. To produce wet samples for the test, the dry soil samples were remoulded using a known percentage of water (optimum moisture content obtained from the compaction test). UU triaxial tests were conducted on the remoulded soil samples in accordance with BS 1377 (1990). Mohr envelopes were subsequently developed, from which the shear strength parameters (cohesion, c and angle of internal friction, ϕ) and the normal stress were determined. The shear strength parameters were then used to compute the shear strength of soil samples, making use of Equation 3.

$\tau = c + \sigma tan\phi$ (3)

Where τ is the shear strength of soil

c is cohesion of soil
σ is the normal stress
ϕ is angle of internal friction of soil

Formulation of models to relate shear strength and index properties of soil

Formulation of models entailed the identification and choice of modeling tool - multivariable least squares regression (MLSR). After the identification of MLSR that is needed to develop the predictive model for shear strength of the soils, the simulation was performed using the selected index properties and shear strength values which had been determined earlier. The index properties for model development were considered based on literature review (Barends *et al.*, 1999; El-Maksoud, 2006; Murthy, 2008; Kayadelen *et al.*, 2009) The shear strength is the dependent variable, while index properties are the independent variables.

Stepwise regression was carried out and decision was made to remove the correlated parameters in order to eliminate the problem of multicollinearity (Dunlop and Smith, 2003) and thus maximise the reliability of the final model. Data from 20 sampling points were used for model development, while data from the remaining five locations were used for validation. Microsoft Excel multiple regression statistical package was used to develop multiple regression models. The general multiple linear regression (MLR) model is usually expressed by Equation (4).

$y = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \beta_3 x_3 + \ldots . + \beta_n x_n$ (4)
Where y = dependent variable
$x_1, x_2, x_3, \ldots x_n$ = independent variables of order n

$\beta_1, \beta_2, \beta_3, ., \beta_n$ = regression coefficients
β_0 = value of y when independent variables are zero, or the intercept on y-axis.

Validation of developed model

Validation of developed model was done using correlation coefficients, which were obtained from the results of the prediction. The coefficient of correlation is a measure that is used to determine the relative correlation and the goodness-of-fit between the predicted and observed data (Shahin *et al.*, 2009). The following guide has been suggested for values of /R/ between 0.0 and 1.0:

$/R/ \geq 0.8$ strong correlation exists between two sets of variables;
$0.2 < /R/ < 0.8$ moderate correlation exists between the two sets of variables; and
$/R/ \leq 0.2$ weak correlation exists between the two sets of variables.

III. Results and Discussion

Sample Locations

Sampling points for this study is shown in Figure 3.

Preliminary and Index Properties of Soil Samples

Table 2 presents the natural moisture contents (NMCs) of soil samples. The NMCs range between 4.11% (sample A9) and 13.1% (sample A19). Shear strength is expected to increase with the decrease in water content. This is because reduction of water content in clayey soils results in higher friction angle, due to the fact, that clay particles group into aggregates which have larger effective particle size (Brackley, 1973; 1975; Toll, 2000; Blahova et al., 2013).

Table 3 contains the characteristic points on the grading curves of soil samples. According to Ismail (2008), $C_u < 3$ indicates a uniform soil and $C_u > 5$ indicates a well-graded soil. Therefore, samples A5 and A6 are between uniform and well-graded, while the rest are well-graded. Most well-graded soils will have grading curves that are mainly flat or slightly concave, giving values of C_c between 0.5 and 2.0. C_c <0.1 indicates a possible gap-graded soil (Ismail, 2008). Thus, this confirms that the generality of the samples are well-graded.

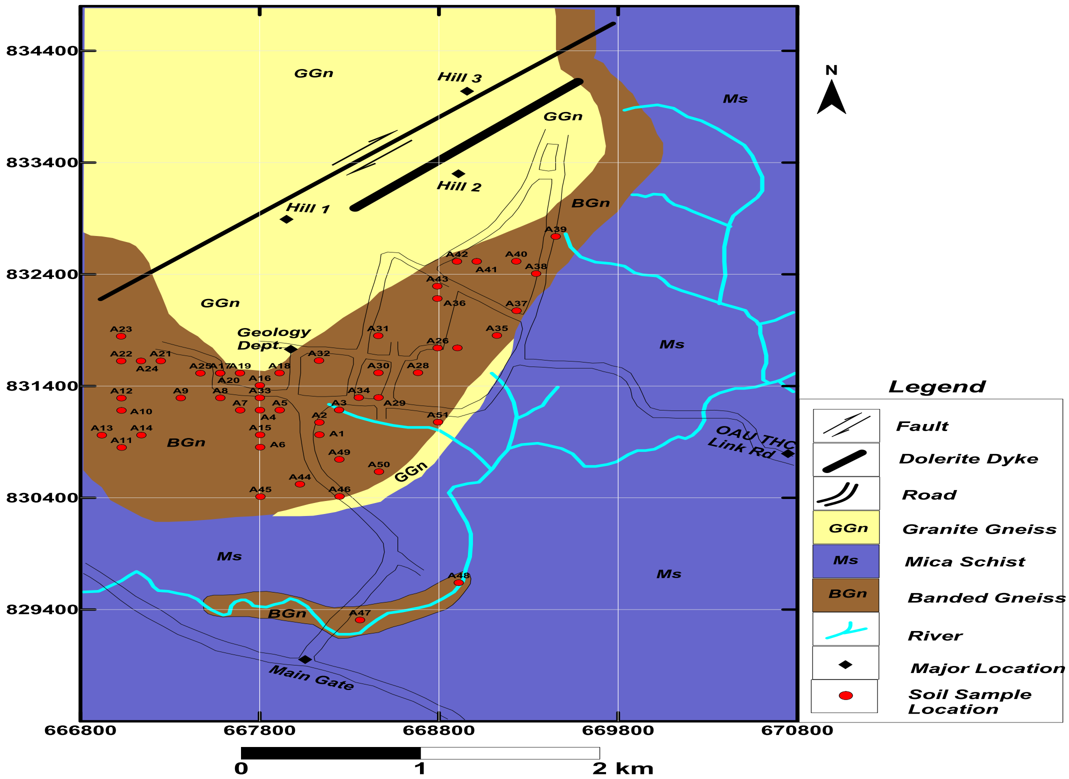


Figure 3: Geological map of the OAU, Ile-Ife showing sampling points

The specific gravity values for the soil samples are as shown in Table 4. Figure 4 is the graphical presentation of the variation of the specific gravity values. The standard range of values of specific gravity of soils lies between 2.60 and 2.80 (Wright, 1986). According to Das (1990), this range of values is for clay minerals. BS 1377 (1990) states that lower specific gravity values indicate a coarse soil, while higher values indicate a fine grained soil. Thus, it could be concluded that majority of the soil samples fall between coarse and

Table 2: Natural moisture contents of soil samples

Sample ID	w (%)	Sample ID	w (%)	Sample ID	w (%)
A1	6.81	A9	4.11	A17	6.04
A2	9.31	A10	10.74	A18	8.11
A3	7.11	A11	8.73	A19	13.1
A4	12.22	A12	7.29	A20	6.2
A5	8.8	A13	5.02	A21	6.13
A6	6.2	A14	7.11	A22	10.23
A7	9.08	A15	5.89	A23	5.11
A8	7.11	A16	11.02	A24	7.01
				A25	9.02

Table 3: Characteristic points on the grading curves of soil samples

Sample ID	D_{10}	D_{30}	D_{60}	C_c	C_u
A1	0.18	0.53	1.46	1.07	8.11
A2	0.12	0.48	1.05	1.83	8.75
A3	0.19	0.41	1.25	0.71	6.58
A4	0.28	1.42	2.5	2.88	8.93
A5	0.15	0.19	0.5	0.48	3.33
A6	0.29	0.91	1.28	2.23	4.41
A7	0.18	0.52	1.5	1.00	8.33
A8	0.21	0.53	1.75	0.76	8.33
A9	0.23	0.41	1.55	0.47	6.74
A10	0.02	0.15	1.1	1.02	55.00
A11	0.22	0.48	1.55	0.68	7.05
A12	0.17	0.21	1.2	0.22	7.06
A13	0.19	0.91	2.6	1.68	13.68
A14	0.28	1.15	2.45	1.93	8.75
A15	0.41	1.85	2.25	3.71	5.49
A16	0.38	1.25	2.15	1.91	5.66
A17	0.16	0.32	1.1	0.58	6.88
A18	0.14	0.55	1.15	1.88	8.21
A19	0.41	1.25	3.05	1.25	7.44
A20	0.13	0.89	1.75	3.48	13.46
A21	0.22	1.75	2.25	6.19	10.23
A22	0.38	1.25	2.05	2.01	5.39
A23	0.41	1.1	2.05	1.44	5.00
A24	0.16	1.3	1.9	5.56	11.88
A25	0.37	0.75	1.55	0.98	4.19

fine-grained. The results of Atterberg limits tests are graphically illustrated in Figure 5. Liquid limits less than 35% indicate low plasticity; between 35% and 50% indicate intermediate plasticity; between 50% and 70% high plasticity and between 70% and 90% very high plasticity. This implies that majority of samples (17 nos.) are of low plasticity; six samples are of intermediate plasticity; while only two samples are of high plasticity.

Table 4: Specific gravity of soil samples

Sample ID	Gs	Sample ID	Gs	Sample ID	Gs
A1	2.61	A9	2.76	A17	2.63
A2	2.56	A10	2.66	A18	2.62
A3	2.85	A11	2.78	A19	2.66
A4	2.60	A12	2.61	A20	2.81
A5	2.88	A13	2.87	A21	2.62
A6	2.57	A14	2.48	A22	2.79
A7	2.64	A15	2.57	A23	2.51
A8	2.71	A16	2.66	A24	2.63
				A25	2.81

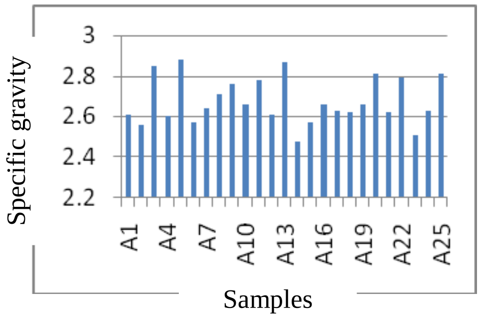
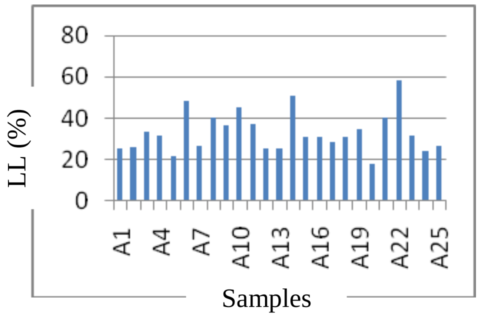
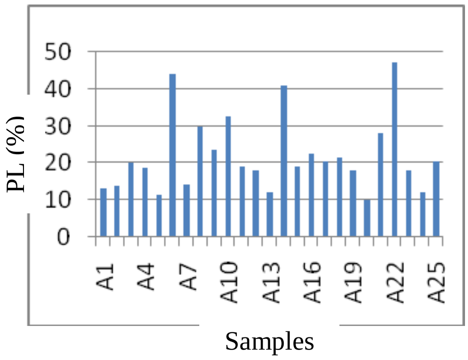


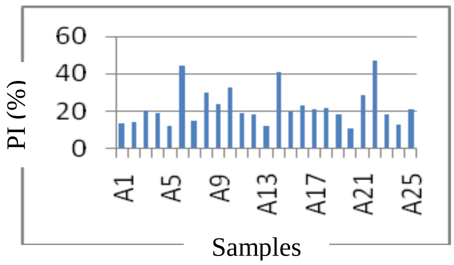
Figure 4: Specific gravity of soil samples



(a) Liquid limit of soil samples



(b) Plastic limit of soil samples



(c): Plasticity Index of soil samples

Figure 5: Graphical presentation of Atterberg limits of soil samples

Compaction test results

Figures 6 and 7 respectively show the graphical presentation of the MDD and OMC of the soil samples. MDD for the samples varies between 1.58 mg/m^3 and 3.11 mg/m^3, while optimum moisture content

(OMC) ranges between 14.58% and 20.01%. According to O'Flaherty (1988), the range of values that may be anticipated when using the standard proctor test methods are: for clay MDD may fall between 1.44 mg/m^3 and 1.69 mg/m^3 and OMC may fall between 20% and 30%.; for silty clay, MDD is usually between 1.70 mg/m^3 and 1.85 mg/m^3 and OMC range between 15% and 25%. For sandy clay, MDD usually range between 1.86 mg/m^3 and 2.17 mg/m^3 and OMC between 8% and 15% (Bello and Adegoke, 2010). Therefore, the soil samples could be generally described as falling between silty clay and sandy clay.

Results of triaxial test

The shear strength values of soil samples, which were obtained after the triaxial tests, are shown in Figure 8. Majority (9 no.) of the samples have shear strength ranging between 40 and 49 kN/m^2; six samples fall between 30 and 39 kN/m^2; six other samples also fall between 50 and 59 kN/m^2; while 4 samples have shear strength ranging between 60 and 69 kN/m^2.

Predictive shear strength model

Stepwise regression analysis of the data showed that there were correlations between fines content (fc) and coarce content (cc); D_{10}, D_{30} and D_{60}; PL and LL. Therefore, to guide against multicollinearity (Dunlop and Smith, 2003; Mollahasani *et al.*, 2011), only one variable was considered in each group of the mentioned variables. Also, it was shown that PI and MDD do not have significant contribution on shear strength of the soil samples. Therefore, after several trials, the employed variables for the development of shear strength models are w, fc, C_u and PL. Also, ratio of number of objects to number of variables (Mousavi *et al.*, 2011b) is 5.

The results of MLSR analyses and the generated performance metrics are summarised in Table 5. Table 5 shows that correlation exists between soil shear strength and the selected index properties. The value of correlation coefficient is within the range 0.2<R<0.8 (i.e. moderate correlation) (Shahin *et al.*, 2009). The resulting multiple regression analysis equation is:

$$\tau = 27.268+1.594w-0.242fc+0.092C_u+0.510PL \quad (R=0.72) \qquad (5)$$

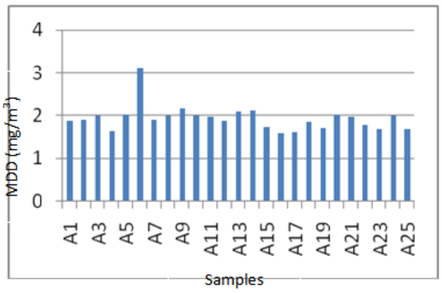


Figure 6: Maximum dry density of soil samples

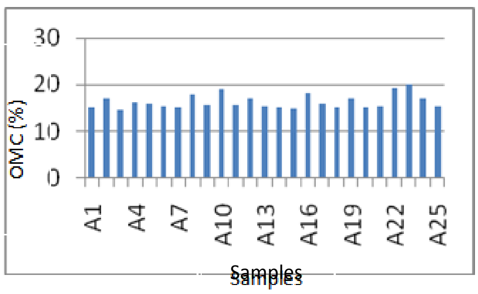


Figure 7: Optimum moisture content of soil samples

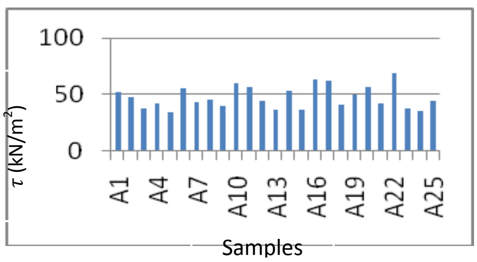


Figure 8: Share strength of soil samples

Table 5 : Performance metrics for developed model

Metrics	Value
Multiple R	0.72
R Square	0.52
Adjusted R Square	0.394
Standard Error	7.554
Observations	20

Validation of developed model

The observed correlation coefficient (0.720) is considered moderate. Therefore, the results of internal validation of the MLSR model is shown in Table 6 .The percentage difference in measured and predicted values of shear strength is 5.93, which is considered low, i.e. less than 6% (Ayininuola *et. al*, 2009). Therefore, the developed MLSR model could be said to be valid for the study area.

Table 6: Comparison between measured and model shear strength of soils

Sample ID	Measured τ (kN/m^2)	Model τ (kN/m^2)
A5	35	43.92
A11	57	44.38
A15	37	42.04
A20	57	36.29
A25	44	49.73
Mean	**46**	**43.27**
% Variance		**5.93**

IV. Conclusion

This study aimed at developing predictive model for the determination of shear strength of lateritic soils in a Banded Gneiss geologic unit. Following standard procedures, the study determined the index properties and shear strength of selected soils The index properties were analysed; and specific relationship was developed between selected index properties and shear strength. Natural moisture content (w), fines content (fc), coefficient of uniformity (C_u), and Plastic Limit (PL) showed useful relationship with the shear strength of the selected soils. Multilayer regression model was developed and subsequently validated. The developed model was found to be valid for the studied geologic unit.

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