CORRELATION OF THE UNDRAINED SHEAR STRENGTH AND PLASTICITY INDEX OF TROPICAL CLAYS

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ABSTRACT

This paper attempts to establish a relationship between the undrained strength and plasticity index of tropical clays. Its theoretical basis lies with the previous works of Skempton and Northey [1] and Atkinson and Bransby [2]. *The data obtained from independent laboratory tests on* so *some clay samples sourced from several actual project locations* in *Eastern Nigeria, were subjected to statistical least squares regression analysis after the samples had been grouped into* CL, *CI and* CH *using the Unified System of Soil Classification. The derived regression equations are shown to have high correlation coefficients thereby proving their viability. These equations can be used to estimate the undrained strength of clays encountered in Eastern Nigeria in lieu of the very expensive triaxial compression tests.*

NOTATION

C_u = Undrained Cohesion
C_c = volumetric compression index
CL = clay of low plasticity
CI = clay of intermediate plasticity
CH = clay of high plasticity
e = void ratio
G_{max} = initial tangent shear modulus
G_s = specific Gravity of soil particles
K_o, = coefficient of lateral earth pressure at rest
LI = Liquidity Index
LL = liquid limit
P = total mean stress (= $(\sigma + 2\sigma 3)/3$ for axis-symmetric compression)
P_o = initial total mean stress or preconsolidation pressure (= σ_3 for isotropic compression)
PI = plasticity index
PL = plastic limit
q_u, = undrained shear strength
S_r = degree of saturation
U = pore pressure
w = moisture content or natural moisture content (%)
σ = total normal stress
σ^1 = effective normal stress
σ_1 = major principal stress
σ_3 = cell pressure in the triaxial test (or minimum principal stress)

σ^1_{vo} = initial effective vertical stress (or effective overburden pressure)
ϕ_u = undrained angle of internal friction
γ=Unit Weight (with subscript *'w'* for water)

INTRODUCTION

Soil Mechanics arose to meet the need of a means of evaluating in a rational manner such soil engineering problems as: the bearing capacity of a foundation; the stability of natural slopes, embankments and excavations; the magnitude and the time-rate of settlement of a footing; the quantity of seepage through an earth dam or beneath a concrete dam or into an excavation; the force on a retaining wall. For each of the above purpose it is necessary for the soil engineer to furnish himself with the necessary soil parameters which he must then employ in some empirical or analytical formulae in order to get the desired solution. The needed soil parameters, invariably, must be obtained either through careful laboratory measurements or some other in-situ tests. The shear strength parameters viz., the cohesion and angle of internal friction are needed for the following purposes: the evaluation of the bearing capacity of a foundation; the assessment of the stability of a slope

Accurate measurement of shear strength parameters, coefficient of consolidation, and compressibility can be difficult, time consuming and costly. As a result of this there is now a tendency in countries all over the world towards building up correlation equations between the above soil properties and the so-called soil indices in order to speed-up the design process. This is most pertinent in third world countries where up-to-date testing equipment are lacking together with the trained manpower needed to operate them. For the plastic, clayey soils the Atterberg limits (which are indices of soil behaviour) have been found useful for this application. This is because the measurement of the Atterberg limits requires very simple apparatuses and takes up comparatively short periods of time.

It should be noted that correlation between soil properties and the Atterberg limits had been initiated more than an half a century ago and some of these will now be mentioned.

Terzaghi and Peck [3] had tabulated a correlation between the unconfined compressive strength of clays and their standard penetration resistances. But when the exhorbitant cost of the standard penetration test is considered no advantage is gained therefrom. For a truly cohesive soil the undrained shera strength is half the unconfined compressive strength.

There exists in the technical literature a number of empirical correlations between qu/σ^1_{vo} (the normalised undrained shear strength) and the Atterberg indices for normally consolidated clays, namely:

Skempton and Henkel [1] presented a curve for the variation of q_u/σ^1_{vo} with plasticity index which was later approximated by the linear equation.

$$q_u/\sigma^1 v_o = 0.11 + 0.37(PI/100) \quad (1)$$

Bjerrum and Simons (1960) gave the following regression equations as fitting their experimental data best.

$$q_u/\sigma^1 v_o = 0.45(PI/100)^{1/2} \quad (2)$$

For PI > 50%; which had a deviation range (or scatter) of ± 25% Or

$$q_u/\sigma^1 v_o = 0.18(LI/100)^{1/2} \quad (3)$$

For LI > 50%; which had a deviation range of±30%

Karlson and Viberg (1967) obtained the simple formula

$$q_u/\sigma^1 v_o = 0.5(LL/100) \quad (4)$$

For LL > 20%; with a deviation range of ± 30%

Osterman [4] presented of graphical

correlation – but no regression equation. between q_u/σ^1_{vo} and PI for normally consolidated soils. His plot indicated that q_u/σ^1_{vo} varied along a concave downward curve with PI for the soils he designed as special clays but increased almost linearly with PI for marine clays. An inspection of his plot indicated a substantial deviation (scatter) of the data points from the best-fit lines.

From the above it is obvious that all the listed researchers proposed a linear variation of q_u with σ^1_{vo} and by implication with depth in a stratum of normally consolidated clay in accordance with field observations. The constant of proportionality in their equations is a function of either PI, or LI or LL. They did not give any explanations for the relatively high scatter of their regression equations from the data points. These authors hereby suggest that it may have arisen because they did not distinguish between CL, CI and CH clays in their analyses.

A few of the correlation equations developed for the other soil parameters include: Terzaghi and peck [3] suggested that for virgin compression of normally consolidated soils

For remolded soil

$$C_c = 0.007\ (LL - 10\%) \qquad (5)$$

For undisturbed soil

$$C_c = 0.009(LL - 10\%) \qquad (6)$$

Alpan [5] recommended that of normally consolidated clay.

$$K_o = 0.19 + 0.233 \log(PI\%) \qquad (7)$$

More recently, Vucetic and Dobry [6] have shown that the shapes of the curves G/G_{max} (the modulus ratio) versus γ_C (the cyclic shear strain amplitude) and λ (the equivalent damping ratio) for a soil are primarily influenced by the plasticity index. They produced design curves which can be used for evaluating the dynamic response of a foundation soil under cyclic loading from such varied causes as machine vibration, earthquakes, pile driving, explosions etc.

Of particular interest to the soil engineer is the undrained shear strength of fine- grained soils. The shear strength of soils is required for the design of foundations and retaining walls and for calculating the stability of embankments, cuttings and natural slopes. It is common in Geotechnical engineering practice to use the undrained shear strength to evaluate the stability of a slope, or the bearing capacity of a foundation, in the short term the excess pore pressures, generated in the soil as a result of the surcharge loads, are still high and have not had the time to dissipate.

Skempton and Northey [1], and Atkinson and Bransby [2] showed that the undrained shear strength and liquidity index of remoulded saturated clays are related. In particular, Atkinson and Bransby investigated the behaviour of four different samples of clay soils and the results of the tests on three of the samples are listed below in Table 1.

Table 1

Clay type	LL (%)	PL (%)	PI (%)
Horten	30	16	14
London	73	25	48
shellhaven	97	32	65

By varying the moisture content of each clay sample the liquidity index is made to vary and the undrained shear strength corresponding to the different values of liquidity index were measured with the triaxial apparatus. The test results were presented graphically by plotting the liquidity index against the logarithm of the undrained shear strength. It this worthwhile at this stage to point out that the upper limits of the water contents used in these experiments were unrealistically high, being frequently in excess of 100%, while in reality the in-situ saturated moisture contents were usually in the range of 28 to 35%.

The plots of Atkinson and Bransby [2] indicated that for a remoulded soil sample log (q_u) varies inversely as the liquidity index. That is

$$\log q_u = a + \frac{\beta}{LI} \qquad (8)$$

Where a and β are constants are for a particular soil (which will not be the same for another soil)

But $LI = (W = PL)/PI$

$$\Rightarrow \log q_u = a + \frac{\beta}{(w=PL)^{PI}} \qquad (9)$$

If now β/(w-PL) is a assumed to be a constant for a set of soils then

$$\log q_u a + bPI$$

where $b = \beta/(w - PL)$ (10)

and a and b are constants which will be shown later to be functions of σ_3.

Subsequently if β/ (W-PL) approximates a constant then, a plot of log q_u versus plasticity index (P.I.) will be straight line in accordance with eqn. (6). Hence in this study a correlation was sought between the log of q_u, and the P.I. in accordance with eqn. (6).

The correlation of undrained shear strength (q_u) with plasticity index (PI) appears to be spurious as soon as it is realized that the PI is obtained from tests on fully disturbed and remoulded soil. But the manner in which q_u varies with PI can be anticipated as follows:

It is known that soils of high plasticity are composed of very small particles, which having relatively high surface area per unit weight, possess a large number of particles contact points. On the other hand, soils of low plasticity, which have larger particles, possess a fewer number of inter-particle contact points. Under an externally applied load, it is not difficult to visualize that for the soil with numerous contact points the average inter-particle contact stress will be relatively lower. As such, the shear stress that can be mobilized to resist sliding would also be lower. In that case a soil with higher P.I. (numerous contact points) will have a lower undrained shear strength than a soil with lower P.I. (fewer contact points).

So-far-as a soil is saturated the effective stress has the value

$$\sigma^1 = \sigma - U \qquad (11)$$

which, to an extent, is not influenced by the actual moisture content. In an undrained test the pore pressure, U will increase as the deviator stress, increases but for saturated specimens

Of the same soil U will be independent of the actual moisture contents.

Consequently, the undrained shear strength of saturated soils can be expected to be independent of their actual moisture contents (provided of -course that particle contacts still subsist). If the moisture content becomes so large as to prevent particle contact then the undrained strength could also be expected to vary with moisture content. This leads to the concept of an upper threshold value of water content below which strength is moisture-content independent. The lower threshold value is somehow below the saturated moisture content. This proposition awaits further investigation.

In partially saturated soils (with moisture contents below the saturation value), the effective stress is a variable quantity defined by Bishop et.al. [7] as

$$\sigma^1 = \sigma - U_a + \chi(U_a - U_w) \qquad (12)$$

Where U_w = pore water pressure

U_a = pore air pressure

χ = the ratio of area of water cross-section to the total area of cross-section taken along the points of contact.

The quantity U_a _ U_w is a measure of soil suction and χ is related primarily to the degree of saturation of the soil.

For a saturated soil (S_r = 1), χ = 1

For a completely dry soil (S_r =0), χ = 0

The behaviour of a partially saturated soil is quite complex. As the void ratio,e, of such a soil is reduced under loading, the pore air becomes compressed and its pressure increases (i.e. U_a increases). Also, χ increases as the degree of saturation increases as can be seen from the relations

$$S_r = \frac{wGs}{e} \qquad (13a)$$

$$\chi = \chi(S_r) \qquad (13b)$$

since the moisture content remains constant in an undrained test.

Depending on the relative magnitudes of ΔU_a and $\Delta\chi$ the effective stress can increase or decrease under increased loading in accordance with eqn (8). As can be seen from eqns. (9) $\Delta\chi$ is a function of the moisture content. As a consequence, the final value of effective stress in a sample of partially saturated soil will depend on its moisture content (through the factory). Since the shear resistance of a particulate system caries with the effective stress it becomes obvious that a partially saturated soil will have an undrained shear strength which is dependent on its moisture content (or initial degree of saturation).

SAMPLING OF MATERIALS AND LABOURATORY TEST METHODS

The data used in this study were those obtained by Kabia Engineering Services Ltd. And Geoprobe Geological and Engineering Services Ltd. from the sites of actual projects executed in several localities in Eastern Nigeria. The samples used for the test were obtained by means of hollow cylindrical cutters attached to boring rods. These samples were then trimmed to cylindrical shapes with height to diameter ratios of 2:1 and thereafter tested for undrained shear strengths in the triaxial test apparatus as per B.S. 1377 (1975).

In each case the test sample was enclosed in a rubber membrane with impervious plates at both ends and placed in the Perspex cell of the triaxial apparatus. The cell was then sealed up and water pumped in to fill the cell and thereby subject the samples in the rubber membrane to a confinement cell pressure (variable).

Subsequently, a deviator stress is applied through a proving ring at a constant rate of strain until the soil sample fails in shear. This test is performed on each sample at a minimum of three different initial cell pressures of 70, 140 and 210KN/m^2. From the results so obtained a series of Mohr's circles of stress are drawn from which the shear strength parameters of each sample are obtained i.e. the cohesion and internal friction angle.

For the determination of the Atterberg limits the Casangrande device was used to obtain the liquid limits of the various samples. The usual criterion was used to obtain the plastic limits. These tests were carried out on the samples in accordance with B.S. 1377 (1975).

Other tests carried out on the samples were the particle size distribution analyses and the determination of their natural moisture contents.

The various samples were then classified into CL, CI and CH soils based on the following three soil characteristics: particles size distribution, liquid limit and plasticity index. Casangrande's plasticity chart containing the A- line plays a major role in the classification.

It is relevant at this point to list the project sites from which the tested samples were obtained. They are as follows:

1. Federal Mortgage Bank, Enugu
2. NNPC, Emene
3. PRODA, Staff Housing Complex, Emene
4. Secretariat Complex, Owerri.
5. Federal School of Arts and Science, Aba
6. Eziorsu - Oguta Road
7. Eziorsu Bridge
8. Eastern Highway By-Pass, Port- Harcourt

EXPERIMENTAL RESULTS AND ANALYSIS

Table 2 (CL soils): shear strength parameters C_u and ϕ_u

	Site location	Bore Hole Number	Depth (meters)	C_u (KN/m^s)	ϕ_u (degrees)
1	Secretariat Complex, Owerri	2	3.00	28	25
2	Eastern Highway By-pass, Port Harcourt	1	7.00	33	24
3	Eziorsu Bridge Site	1	4.50	75	15
4	Federal School of Arts and Science, Aba	1	3.00	40	18

Table 3: (CI soils): shear strength parameters C_u and ϕ_u

S/no	Site location	Bore Hole Number	Depth (meters)	C_u (KN/m^s)	ϕ_u (degrees)
5	Eziorsu - Oguta Road	3	4.50	78	10
6	Secretariat Complex Owerri	3	3.50	65	11
7	Federal Mortgage Bank, Enugu	2	3.00	60	12
8	Eziorsu Bridge	3	3.00	45	13
9	NNPC, Emene	29	4.50	50	11
10	PRODA Staff Housing, Emene	3	1.50	45	12
11	PRODA Staff Housing, Emene	5	1.50	46	11

Table 4: (CH soils): shear strength parameters C_u and ϕ_u

S/NO	Site location	Bore Hole Number	Depth (meters)	C_u (KN/m^s)	ϕ_u (degrees)
12	NNPC, Emene	27	4.50	25	10
13	PRODA Staff Housing, Emene	3	4.50	26	9
14	NNPC, Emene	32	4.50	34	7
15	PRODA Staff Housing, Emene	5	6.00	24	8.5
16	PRODA Staff Housing, Emene	6	3.00	18	8

The test results, obtained by the aforementioned soil testing companies, are reproduced above in Tables 2 to 4. The natural moisture contents and Atterberg limits have been omitted in these tables to avoid duplication of Tables 5 to 7 where they have been included.

Evaluation of Undrained Shear Strength

The undrained shear strength, q_u is calculated from

$$q_u = \underline{1}(\sigma_1 - \sigma_3)\cos\phi u \qquad (14)$$

The minimum principal stress, σ_3 is known and corresponds to the triaxial cell confinement pressure. The major principal stress,σ_1, is calculated from the well known Bell's formula viz:

$$\sigma_{1=}\sigma_3 N\phi + 2C_u(N\phi)^{1/2} \qquad (15)$$

Where Nϕ, called the flow factor, is given by

$$N\phi = \frac{(1+\sin\phi_u)}{(1+\sin\phi_u)}\tan^2(45^0 + \phi_u/2) \qquad (16)$$

It should be noted that some authors propose that q_u be calculated from

$$q_u = \underline{1}(\sigma_1 - \sigma_3)$$

but these authors have preferred the formula given in eqn. (14) because, in the first instance it accords with theory and secondly it gives a conservation value.

In Tables 5 to 7 are presented the relevant data which were used for the regression analysis. The undrained shear

strengths were evaluated for cell pressures of σ_3 = 70, 140 and 210 KN/m^2. In Fig. 1 the experimental data have been presented as log q_u versus PI plots for the case of σ_3 210KN/ m^2 and these are seen to be matched by the straight line - regression equations within the limits of experimental error.

Table 5: (CL Soils): Atterberg Limits and undrained shear strengths.

						q_u (KN/m^2)		
S/NO	W (%)	LL (%)	PL (%)	PI (%)	W-PL (%)	σ_3 =70KN/m^2	σ_3=140 KN/m^2	σ_3 =(KN/m^2)
1	15.2	31.0	21.0	10.0	-5.8	86.3	132.7	179.1
2	16.9	31.0	17.0	14.0	-0.1	90.3	134.1	177.9
3	16.0	34.0	17.0	17.0	-1.0	118.0	141.6	165.2
4	10.7	34.0	14.4	-19.6	-3.7	82.1	111.9	141.6

Table 6: (CL Soils): Atterberg Limits and undrained shear strengths.

						Q_u (KN/m^2)		
S/NO	W (%)	LL (%)	PL (%)	PI (%)	W-PL (%)	σ_3=70 KN/m^2	σ_3=140 KN/m^2	σ_3 =201 (KN/m^2)
5	16.7	38.0	22.0	16.0	+0.7	106.0	120.5	135.0
6	13.0	37.0	19.6	17.4	-6.6	93.6	109.8	126.0
7	20.0	44.0	25.5	18.5	-5.5	90.4	108.4	126.4
8	11.9	41.0	22.0	19.0	-10.1	74.9	94.7	114.4
9	19.7	48.0	28.0	20.0	-83	75.8	92.0	108.2
10	22.0	45.0	23.0	22.0	-1.0	72.3	90.3	108.3
11	20.7	45.0	21.0	24.0	-0.3	71.0	87.2	103.4

Table 7: (CL Soils): Atterberg Limits and undrained shaer strengths

						q_u (KN/m^2)		
S/NO	W (%)	LL (%)	PL (%)	PI (%)	W-PL (%)	σ_3=70 KN/m^2	σ_3=140 KN/m^2	σ_3 201 (KN/m^2)
12	17.8	50.0	27.0	23.0	-9.2	43.8	58.3	72.8
13	26.5	55.0	28.0	27.0	-1.5	42.9	55.7	68.5
14	29.0	62.0	33.0	29.0	-4.0	47.8	57.5	67.1
15	24.7	58.0	25.0	33.0	-0.3	39.6	51.6	63.6
16	26.4	64.0	30.0	34.0	-3.6	31.7	42.9	54.1

The least squares analyses of the data in Tables 5 to 7 yielded the following regression equations:

For $\sigma_3 = 70\text{KN/m}^2$

For the CL group:

$$\log q_u = 1.930 + 0.263\ (\text{PI}/100) \quad (17)$$

With a correlation coefficient of + 15.4%

For the CI group:

$$\log q_u = 2.342 - 2.175(PI/100) \quad (18)$$

With a correlation coefficient of - 88.2%

For the CH group:

$$\log q_u = 1.911 - 1.028\ (\text{PI}/100) \quad (19)$$

With a correlation coefficient of -68.3%

For $\sigma_3 = 140\text{KN/m}^2$

For the CL group:

$$\log q_u = 2.196 - 0553\ (PI/100) \quad (20)$$

With a correlation coefficient of -51.6%

For the CL group:

$$\log q_u = 2.345 - 1.767\,(PI/100) \quad (21)$$

With a correlation coefficient of-91.0%

For $\sigma_3 = 140 KN/m^2$

For the CL group:

$$\log q_u = 2.011 - 0.986(PI/100) \quad (22)$$

With a correlation coefficient of-81.2%

For $\sigma_3 = 210 KN/m^2$

For the CL group:

$$\log q_u = 2.371 - 1.008(PI/100) \quad (23)$$

With a correlation coefficient of -87.8%

For the CI group:

$$\log q_u = 2.356 - 1.472(PI/100) \quad (24)$$

With a correlation coefficient of -92.5%

For the CH group:

$$\log q_u = 2.093 - 0.961(PI/100) \quad (25)$$

With a correlation coefficient of-88.4%

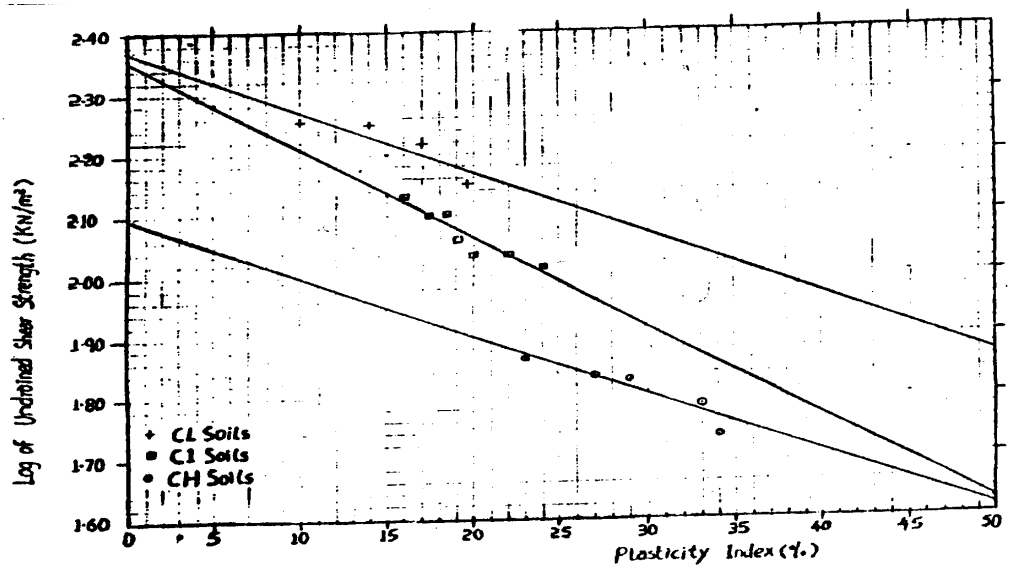

FIG.1 Plot of Log q_u versus P.I. for σ_3=210 KN/m²

The coefficients of the above correlation equations (17) to (25) can be seen to vary with the confinement pressure σ_3. Consequently the constants 'a' and 'b' in eqn. (10) are functions of σ_3. On the basis of a linear variation of a and b with σ_3 a more general regression equation was assumed to hold, namely:

$$\log q_u = a1_{\frac{\sigma_3}{100}} + \frac{a2\,\sigma 3^{+}}{100}\,\frac{PI}{}(b_1 + b_{2\frac{\sigma_3}{100}}) \quad (26)$$

On analysis the data in Tables 5 to 7 via the least square method the more useful and general regression equations below were obtained:

For the CL soil group

$$\log q_u = 1.725 + 0.315_{\frac{\sigma_3}{100}} + \frac{PI}{100}(0.834 - 0.906_{\frac{\sigma_3}{100}}) \quad (27)$$

which has a correlation coefficient of -92.7%, a mean standard % deviation of ± 9.5% and maximum % deviations of -19.3 to 17.5%.

For the CI soil group

$$\log q_u = 2.334 + 0.\,0.0094_{\frac{\sigma_3}{100}} + \frac{PI}{100}(2.508 - 0.504_{\frac{\sigma_3}{100}}) \quad (28)$$

which has a correlation coefficient of -96.0%, a mean standard % deviation of ± 5.3% and maximum % deviations of -6.6 to 13.9%.

For the CH soil group

$$\log q_u = 1.821 + 0.131_{\frac{\sigma_3}{100}} + \frac{-I}{100}(1.054 - 0.0457_{\frac{\sigma_3}{100}}) \quad (29)$$

which has a correlation coefficient of -94.0%, a mean standard % deviation of ± 7.7% and maximum % deviations of -13.6 to 16.1 %.

Also, an analysis of the data in Tables 5 to 7 shows that (w-PL) varied from +0.7 to - 10.1 % and as such is far constant. The mean value of (w-PL) = 3.77%. The standard deviation = 3.50%. On the basis of a normal distribution for (w-PL) the 90% confidence interval for (w-PL) is -3.77 ±-1.645*3.50 [i e. -9.53 % ≤ (w-PL) ≤ + 1.99%].

DISCUSSION OF RESULTS

An examination of the initially derived regression equations (17) to (25) reveals the following facts: The three regression equations for the CI group have very high correlation coefficients in excess of -88%. Those for the CH group also have very high correlation coefficients except for σ_3 =70KN/m^2 which is medium/high at - 68.4 %. The CL group displays the least level of correlation except for the equation for σ_3 =210KN/m^2. In fact for σ_3 =70KN/m^2 the CL group exhibits an unusually small and positive correlation of +15.4%. Also all three soil groups exhibit a high correlation at σ_3 =210KN/ m^2. To further mystify the whole situation the general regression equations (27) to (29) all have excellent correlation coefficients.

The above observations are not easy to explain but it appears that one of the reasons for the excel1ent correlation of the CI group was the greater number of samples in that group: 7 of them versus 4 for the CL and 5 for the CH. This same reason appears responsible for the improved correlation of the general equations in that the data used for their evolution are thrice those used previously.

Another reason is the saturation or non-saturation of the samples at failure. Unfortunately, due to lack of data, the degrees of saturation cannot be reliably quantified. Estimates only are made in Appendix 1. In Appendix 1 it is demonstrated that at a higher cell pressure of σ_3 =210KN/m^2 there is a greater likelihood for most of the samples to be saturated at failure than at the lower cell pressure of σ_3 =70K N / m^2. As postulated earlier samples which are saturated at failure should have undrained strengths that are independent of their moisture contents while those which are not will behave contrariwise. This partially explains the higher correlation of the loq q_u versus PI plots for σ_3 =210KN / m^2 than for σ_3=70KN/m^2.

The behaviour of the CL soils at σ^3 =70KN/ m^2 may be attributable to two more reasons. First, they are frictional-cohesive soils, that is granular material with plastic properties, as an examination of Table 2 will reveal that they have higher ϕ_u which range from 15 to 25°. Secondly, as explained above, because of their low water contents, they are very unlikely to be saturated at the failure in which case their q_u, would also depend on their water contents. The partial saturation of the granular CL soils can actually make them to have a higher than expected q_u Thssis is because negative pore pressures may be generated which according to eqn. (11) will result in increased effective stress and increased undrained strength.

From the plots of Fig. 1, which are for σ_3 =210KN/m^2, it is shown that the behaviours of the different classes of soils viz. CL, CI and CH are different. It is seen that for all classes of soils the undrained shear strength decreases with increasing value of plasticity index. Also, in general, for the range of plasticity index increases in the following order CH, CI, and CL. That is at any value of plasticity index, the CI. soil class possesses a higher undrained shear strength than the CI soil which in turn attains a higher undrained shear strength than the CH soil class. This result was earlier anticipated. Also a curious thing can be observed: the slope of the regression line for the CI soil class is greater than the slopes of the regressions lines for the CL and CH soil classes which are approximately equal. This indicates

that the CI soils undergo greate: decrease in strength with increasing plasticity index than they do the CI, and CH soils. No reasons can be readily put forth to explain this observation.

A question that arises is, "were the clay samples considered normally consolidated or overconsolidated?" In the absence of a volume versus log(P) diagram such a question would be difficult to answer. An overconsolidated sample has a volume- log (p) diagram which consists of approximately two straight lines with different slopes while a normally consolidated sample has a single straight line diagram.

As suggested above and also proved in Appendix 1 a majority of the soil samples would be saturated at the failure state even though they might have been unsaturated at the beginning of the tests (especially at σ_3 =210KN/m^2). Since pore water is prevented from escaping in an undrained test, this will result in increasing degree of saturation as the sample is compressed under increasing pressure, until at 100% saturated the sample can no more undergo volume change, and is thereby described to be in a critical state.

CONCLUSION AND RECOMMENDATIONS

In this paper the relationship between the undrained shear strength and the plasticity index of saturated tropicals clays of Eastern Nigeria has been investigated. It has been shown that the logarithm of the undrained shear strength is related to the plasticity index via the regression equations (27) to (29) for the three classes of soil considered with correlation coefficients of - 92.7%, -96.0% and - 94.0% for the CL, CI and CH soils respectively. These high values of correlation coefficient strongly confirm the suggested relationships. For the evaluation of the strength of a soil in-situ it is suggested that $p_o=\sigma vo\ (1 + 2K_o)/3$ be substituted for σ_3 in eqns.(27) to (29) as is usual in Soil Mechanics. Ko could be conservatively estimated using eqn (7). However, as had been pointed out above the soils that were investigated were at water contents that were mainly below their plastic limits. Therefore the authors recommend that the derived equations be used with confidence only when the natural moisture content of a soil sample is equal to or not more than 8% below the plastic limit. Should this condition not be met, then any evaluation of the undrained shear strength from the plasticity index using the above regression equations must be regarded as only an approximate guide. Ultimately, recourse must be made to the triaxial apparatus for the determination of the true undrained strength of a soil.

Notwithstanding the deficiencies of this study the authors believe that the formulae that have been presented would be found to be very useful for preliminary estimates of undrained shear strength of the tropical clays of Eastern Nigeria. Of course, it should be recognized that any correlation between a particular soil property and the Atterberg limits can only be approximate.

To improve the reliability of the correlation equations, research is being carried out on the incorporation of more variables which may significantly influence the undrained shear strength of saturated tropical clays namely; initial void ratio, overconsolidation ratio and crystalline structure.

Appendix 1: Minimum Water Content For Sample To Become Saturated At Failure.

A contention of this paper is that if a soil sample is nearly saturated or saturated then the undrained shear strength may be expected to be independent of the actual water contents. This is because, though the stress path used to approach failure may be complex, once the sample becomes saturated the failure pattern is likely to be independent of the initial approach path. Proof is hereby given that most of the soil samples in this paper could have achieved saturated at failure when σ_3 =210KN/ m^2 thereby explaining the high correlation

coefficients of eqns. (23) to (25). Note again that in the standard triaxial test *a3* is kept constant while σ_1 is increased. Also in the undrained test the moisture content remains constant while the degree of saturation varies.

The subscript '0' shall designate the initial state while 'f' will represent the final/ failure state. It can be derived that the minimum water content. W_{min}, for a partially saturated sample to become saturated at failure is

$$W_{min} = \frac{100[G_s\gamma w - \gamma o(1+e_o-e_f)]}{[G_s(\gamma_o - \gamma_w)} \quad (30)$$

where

$$e_o - e_r \cong C_c \log(Pf/\sigma_3) \quad (31)$$

$$Pf = (\sigma_{1f} + 2\sigma_3)/3 = \sigma_3 + 2q_u/(3\cos\phi_u \quad (32)$$

Cc is given by equ. (6) namely

$$C_c \cong 0.009(LL - 10\%)$$

Also, γo = the unit weight at void ration of e_o and mean stress $p_o = \sigma_3$

In addition the following relationship will hold

$$S_r = wGs/e \quad (13a)$$

$$e_r = e_{sat} = wGs/100 \quad (33)$$

$$\gamma = \frac{Gs\gamma w(100+w)}{100(1+e)} \quad (34)$$

$$e \cong eo - Cc\log(P/P_O) \quad (35)$$

Where eo and e are the void ratios at mean stress of p and p respectively.

Illustrative example: consider sample S/ No .7 for which LL = 44%, w = 20%, ϕ =12°. At σ_3 =70KN/m^2 q_u, = 90. KN/m^2 and at σ_3= 210KN/m^2 q_u, = 126.4KN/ m^2 Eqn. (6) gives Cc =0.306. Let it be assumed that Gs = 2.7 and that γ = 20. 5KN/m^3 at =210KN /m^2. Then from eqn . (34) e_o =0. 5505. Using eqns. (35) and (34) at σ_3 =70KN/m^2 e_o = 0.6965 and γ =18 .7KN/ m^3 by using eqn. (30) it can be calculated that W_{min} = 25.9% and 17.5% at a_3 =70KN/m^2 and 210 KN/m^2 respectively. The corresponding initial degrees of saturation are 89.3% and 91 .2% respectively.

Based on the above, since the actual moisture content was 20%, the simple reconsolidated at *a3* = 210KN/m^2 would have become saturated at failure while that at σ_3 =70KN/m2 would still remain partially saturated at failure: This analysis is of course only approximate since the actual Cc and y are not available.

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