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SEMI-EMPIRICAL METHOD OF INTERPRETATION OF CPT DATA

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ABSTRACT. Cone penetration test (CPT) is a reliable technique for measuring subsurface soil properties. In most CPT measurements the mobilized cone tip resistance is representative of the formation conditions. A Semi-empirical method of interpretation of such data is presented. The method was developed from a theoretical concept. Angle of internal friction, cohesion and ultimate bearing capacity of soils can be directly obtained from observed tip resistance. The results of the interpretations are in good agreement with observed conditions. The method is particularly suitable in cases where the accuracy of the measured skin friction is in doubt.

1. Introduction

Cone penetration testing is a method of determining the in-situ mechanical properties of soils. The test method has gained wide preference over the years because of its rapid procedure, relatively cheap operational cost and continuous profile reproduction of results. Following the standardization of test procedure[1] and improvement on the method of data interpretation[2,3,4], its reliability is found to be excellent[5].

Mechanical cone, Electric cone, and Piezocone are the devices commonly used in cone penetration testing. The mechanical type is least efficient and least sensitive to changes in soil conditions. Generally, test results are generated as cone resistance, q_c, skin friction, q_s, and pore pressure, u (for piezocone only) that are functions of depth of penetration, h. The interpretation of the field data to establish the mechanical properties of a soil involves a combination of two or more types of data. Thus the accuracy of the input data controls the reliability of the final results.

When a mechanical cone device is used, especially in clay deposits, the value of q_s is not as reliable as q_c[6]. Biddle[7] has shown that, in general, q_s is not as repeatable as q_c. Therefore results obtained with q_c only may be more reliable. In this report semi-empirical criteria for the interpretation of CPT/CPTU data from observed cone resistance (q_c) is presented. The criteria followed an original work[8,9] and field experience.

2. Theoretical Background.

Functional expressions for the theoretical cone resistance $*q_c$ and theoretical skin friction, $*q_s$ were established[8,9], as:

$$* q_c = 2.26(c \sec\alpha + (9_c + {}_h\gamma(1+\sin\phi)e^{\pi\tan\phi}\frac{1+\sin\phi}{1-\sin\phi})\left(\tan\alpha\tan\phi + 1\right) + 2\sec\alpha\sin\phi(\gamma h\frac{1+\sin\phi}{1-\sin\phi} + 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}})) \quad \ldots(1)$$

$$* q_s = 2.26(c + 2\sin\phi(\gamma h\frac{1+\sin\phi}{1-\sin\phi} + 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}})) \quad \ldots(2)$$

where, $*q_c = q_c + 1.13\, w_a$ ... (3)

$*q_s = q_s + 1.13\, w_a$ ... (4)

Also c = soil cohesion, $\propto$ = semi-apex angle of cone, h = depth of penetration, γ = unit weight of soil, ϕ = angle of internal friction, w_a=net weight of the penetrometer. If the penetrometer factor is represented by E, then,

$E = 1.13 w_a$ ... (5)

$W_a = (\gamma_p - \gamma_a)\, L$ ... (6)

Where γ_p = specific weight of the penetrometer (= 79 KN/m^3), L = limiting depth of penetration (= 139B) beyond which L remains constant, B = diameter of the penetrometer and γ_a = an assumed average unit weight of the particular soil under consideration .

Equations (1) and (2) were programmed, solved and calibrated with field data to obtain the model chart presented in fig. 1 ($^*q_c$ versus $^*q_s$), fig. 2 (σ_u / $^*q_c$ versus ϕ), and fig. 3 ($^*q_c$ / $^*q_s$ versus ϕ), where σ_u = ultimate bearing capacity of pile foundation and $^*q_c$ / $^*q_s$ = I^*_{ft} = theoretical friction ratio [8,9] .

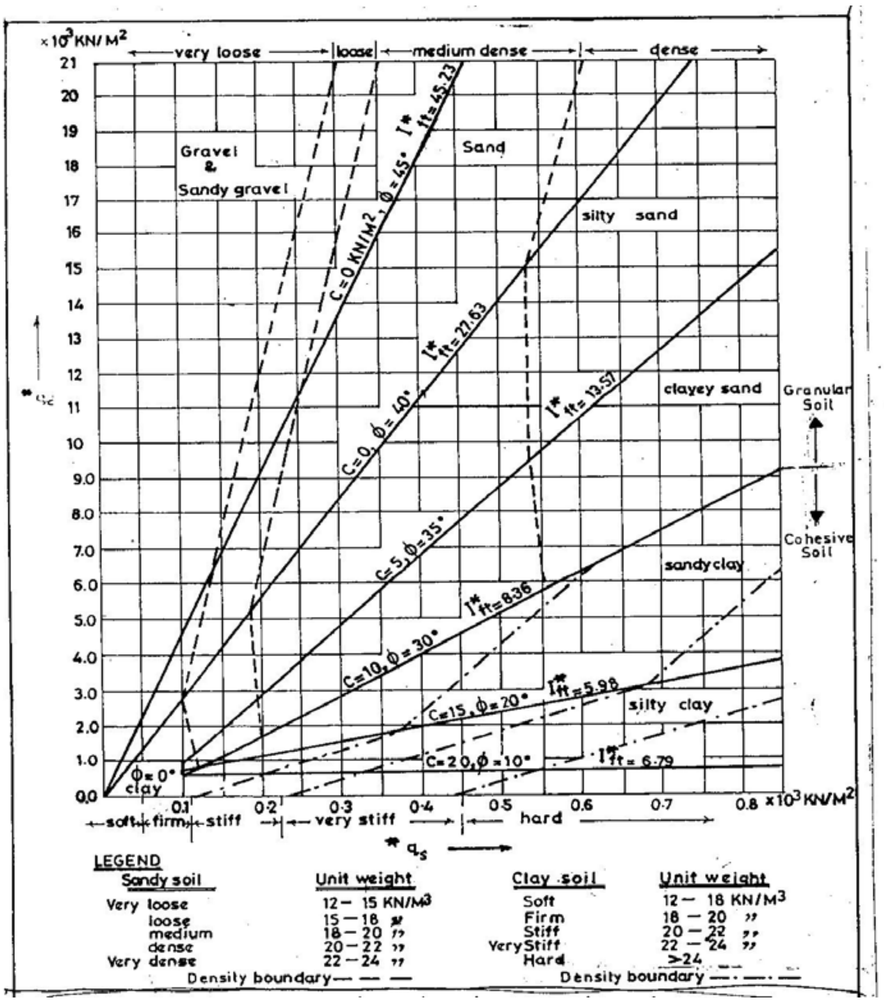

Fig. 1 : A model chart for the interpretation of penetration resistance in soils.

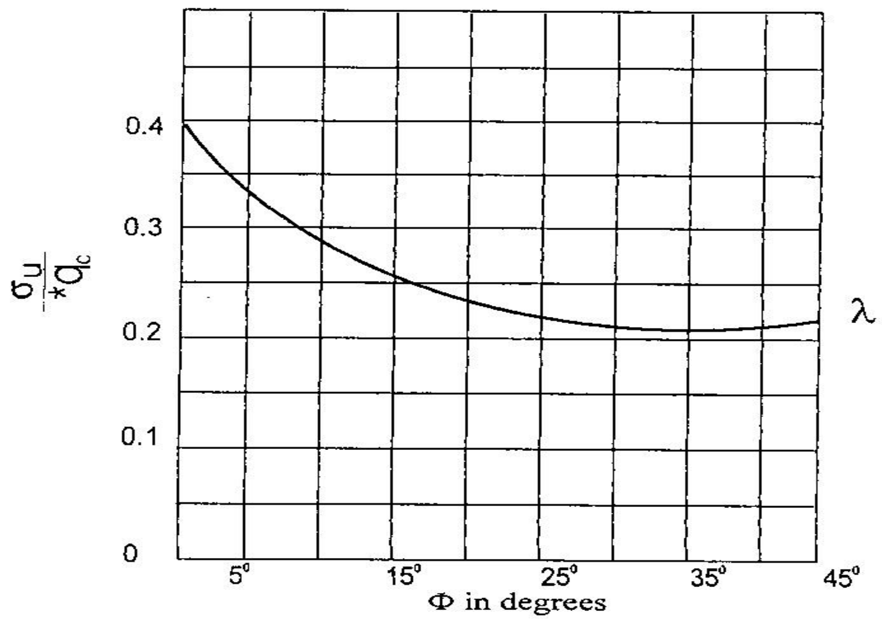

Fig. 2 : Relationship between bearing pressure factor λ (= σu / *q_c) and friction Φ, (Owuama 1994)

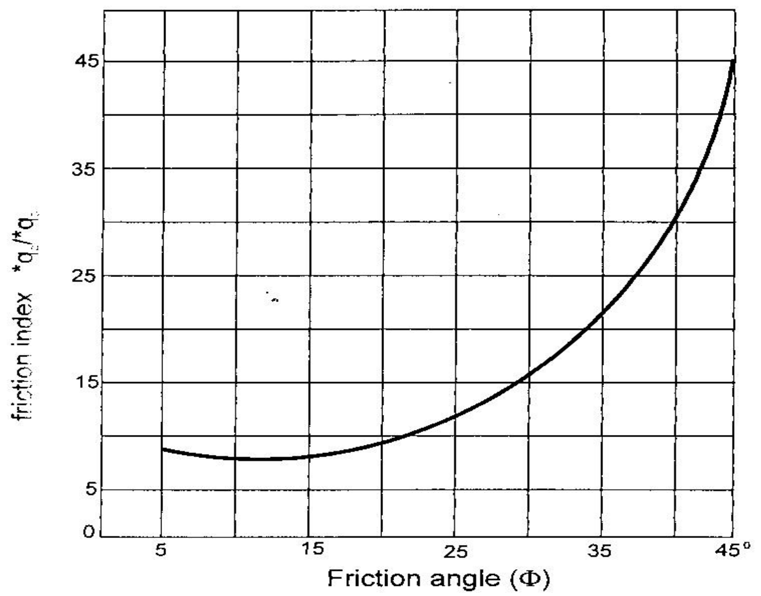

Fig. 3 : Theoretical relationship between friction index *q_c/*q_s and friction angle (Φ).

From observed q_c, q_s and u (where applicable), $*q_c$ and $*q_s$ can be determined from equations (6), (5), (4) and (3) in succession. By interpolation in fig. 1 the soil type, consistency, ϕ and c can be obtained. If the angle of friction ϕ is known the ultimate bearing capacity σ_u can then be obtained from fig. 2.

3. Semi-Empirical Criteria

In order to develop semi-empirical criteria for the purpose of simplicity, over 240 CPT data points from the Niger Delta area of Nigeria were studied, analyzed and interpreted with the model charts (fig.1, 2 and 3). The data were generated from sites at Eleme, Port Harcourt, New Finima and Etelebou in Rivers state Nigeria using a 20 KN mechanical cone penetrometer of the following specifications: cone diameter = 35.7mm, surface area = 10 cm^2, apex angle = 60^o, length of mantle extension above cone = 110 mm, and thickness of cone base = 5mm. Boreholes were drilled at adjacent points to establish soil profiles.

The average q_c and q_s for each depth was obtained by matching similar data at the same depth range. From the arrays of data available 20 value points were established. The data were converted to their theoretical equivalents $*q_c$ and $*q_s$ using equations (3) and (4), respectively. With these corrected data, values of ϕ and c were determined from fig. 1. The results are presented in Table 1. Then from known ϕ and $*q_c$ the corresponding values of σ_u were obtained following fig. 2. The results are shown in Table 1. It was observed that similar ϕ values were obtained when fig. 3 was used in the interpretation. Generally, Table 1 displays the relationship between the results obtained with the theoretical charts, and the corresponding average field values of cone resistance, q_c. From the table, figs. 4 and 5, were prepared.

Geologically, the Niger Delta area represents a quarternary deposit. It comprises shell sands of beach ridge origin intercalated with interdunes and lagoonal mud, dark silty sands and black organic mud at the coastal margin. They extend to thick alluvial sands within the broad valleys of rivers, creeks, and lagoons and lateritic sandy clay at some depths. The subareal deposits consist of coarse layered sands, silts and clays deposited in the river channels, point bars, backswamps and levees.

Table 1 : Summary of soil properties obtained from fig. 1, 2, & 3.

Depth (m)	Average q_c MN/m^2	Φ Degrees	c KN/m^2	σu MN/m^2
1	1.475	23.30	13.33	0.431
2	1.925	30.00	10.00	0.556
3	2.503	26.25	11.88	0.751
4.	2.600	25.26	12.37	0.804
5.	2.750	23.46	13.27	0.865
6	3.650	23.89	13.06	1.117
7.	4.105	27.86	11.07	1.224
8	5.790	31.47	8.33	1.655
9	5.800	32.19	7.81	1.658
10	5.650	33.04	6.96	1.545
11	8.250	36.04	3.95	2.276
12	7.250	35.42	4.50	2.011
13	7.083	35.33	4.67	1.906
14	7.417	35.76	4.24	2.049
15	9.028	38.18	1.82	2.474
16	9.861	38.09	1.91	2.672
17	11.500	39.79	0.21	3.090
18	9.778	38.56	1.44	2.641
19	10.417	39.56	0.44	2.808
20	10.056	38.56	1.44	2.725

(Source: Owuama 1994).

With the results obtained from the CPT interpretations some fundamental relationships are established thus: q_c versus ϕ and c, fig.4: this is a parabolic curve from which the friction angle ϕ and cohesion c can be determined from observed cone resistance; σ_u versus q_c, fig 5: this is a linear relationship, and the governing equation is such that

$$\sigma_u = 0.12 + 0.26\, q_c \qquad \text{... (7)}$$

The ultimate bearing capacity . σ_u) of the soil for pile foundation can therefore be obtained directly from fig. 5 or eqn. (7) when the observed cone resistance is known. The equivalent bearing capacity for shallow foundation (σ_{us}), following Vesic [10], can be estimated from

$$\sigma_{us} = \sigma_u (1 + \sin \phi) \qquad \text{... (8)}$$

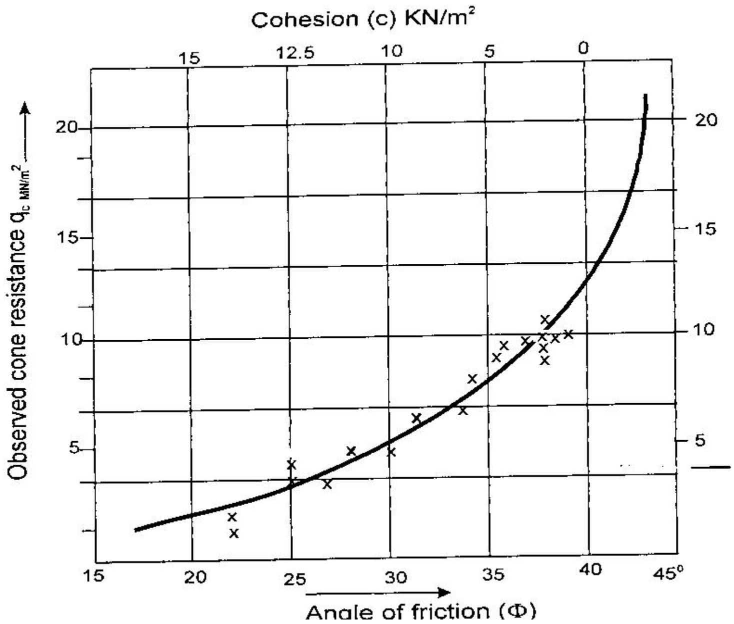

Fig. 4 : Relationship between observed cone resistance q_c and angle of friction Φ and cohesion c.

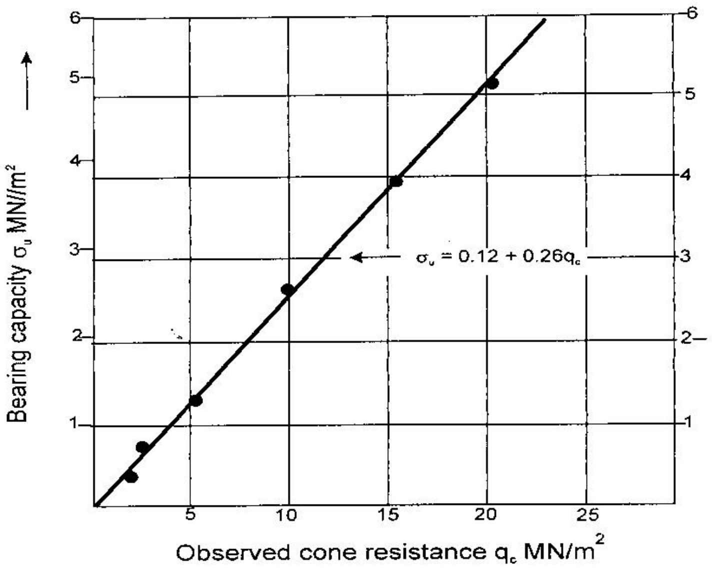

Fig. 5 : Relationship between observed cone resistance q_c and bearing capacity for deep foundations, σ_u.

The relationships, fig. 4 and 5 represent the semi-empirical (SE) criteria for the interpretation of cone resistance measurements.

4. Validation of Criteria

To assess the reliability of the proposed criteria average CPT data from some sites (Bonny south, Etelebou, Zarama) in the Niger Delta region of Nigeria were considered. Boreholes were sunk adjacent to the test holes. Data from about 10 CPT locations spaced at 5m intervals were averaged for each site. The average results are displayed in Table 2.

Table 2 : Soil types at Bonny south, Etelebou and Zarama following fig.1
(a) Average Cone Resistance, (b) Borehole log.

Depth in meters	Bonny	South	Etelebou		Zarama	
	(a) q_c MN/m²	(b) BH log	(a) q_c MN/m²	(b) BH log	(a) q_c MN/m²	(b) BH log
1	2.43	Top	3.33	Clayey	1.00	
2	5.84	Soil	4.02	Sand	1.40	
3	11.82		0.88	Silty	1.00	Clayey
4	14.94	Silty	1.52	Clay	14.20	
				to		
5	14.30		1.06	Silty	15.44	
6	13.21	Sand	1.25	Sand	17.88	
7	8.11		9.83		20.78	
8	8.53		15.16	Fine	17.60	
9	10.42		15.60	Sand	17.68	Sandy
10	8.42		12.37		18.86	
11	7.20	Silty Clay	10.85		20.72	
12	10.27		8.08		19.12	
13	13.68		11.03	Sand	18.68	
14	14.78	Silty	11.10	and	17.04	
15	15.52	Sand	8.20	gravel	18.40	
16	15.90		11.41		22.28	
17	18.70		12.43		22.08	
18	7.50		11.43		19.60	
19	2.25	Silty	15.20		14.28	
20	5.00	Clay	18.33		29.06	

The data were generally interpreted with the SE-criteria. Meyerhof [11], and Roberston and Campanella[12] criteria were employed to obtain ϕ values. Also, observed laboratory and standard penetration test (SPT) results were considered.

Meyerhof [13] and Vesic[14] criteria were used to determine the equivalent values of σ_u. The overall results are compared in figs 6,7, and 8 for all the sites.

From fig. 6, ϕ (SE) overestimates ϕ (Meyerhof) but underestimates ϕ (Robertson and campanella), at depths shallower than 16m in all the sites investigated. On the average ϕ (SE) = 40.2^o , ϕ (Meyerhof) = 38.3^o, ϕ (Robertson and campanella) = 41.4^o, and ϕ (SPT) = 34.5^o.

Fig. 7 is a display of c (SE). There are no corresponding results from any other criterion.

From fig 8, it is apparent that σ_u (Vesic) is at the low side and σ_u (Meyerhof) is at the high side when compared with σ_u (SE). Generally σ_u (SE) = 3.82 MN/m^2, σ_u (Meyerhof) = 7.02 MN/m^2 and σ_u (Vesic) = 1.05 MN/m^2.

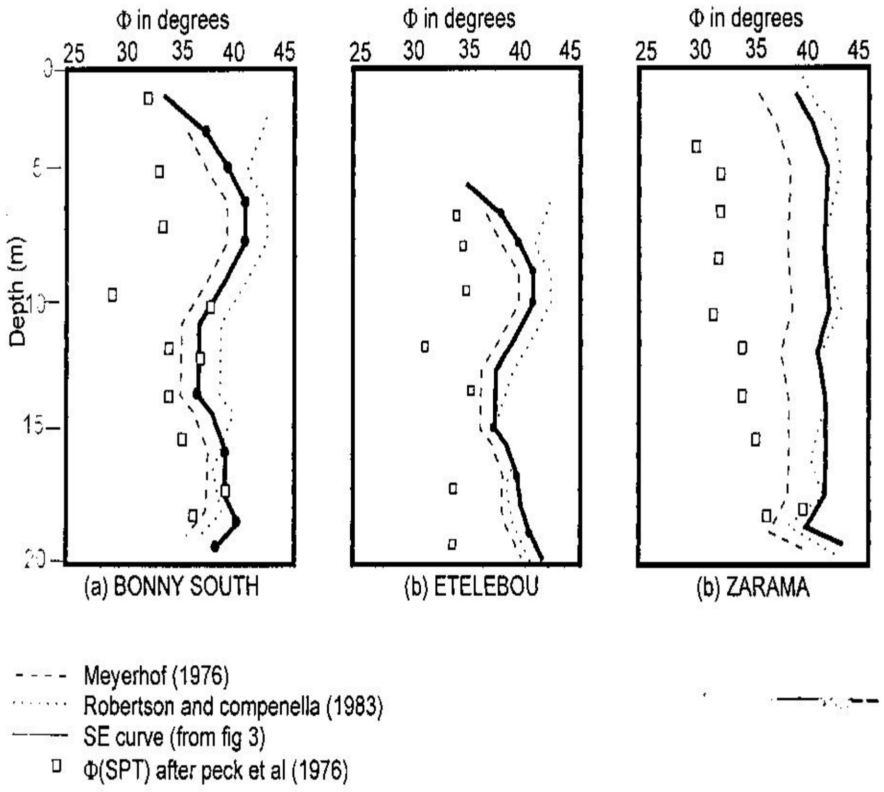

Fig. 6 : Variations of friction angle, Φ with depth.

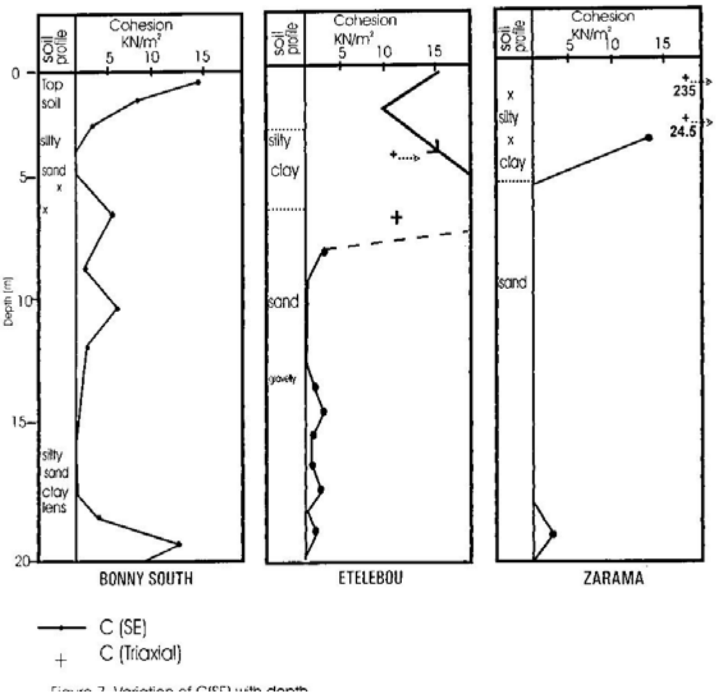

Fig. 7 : Variations of C(SE) with depth.

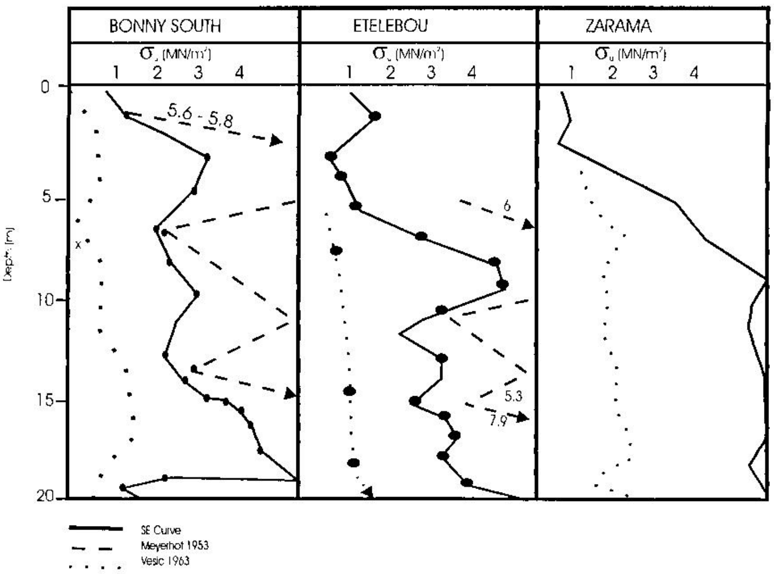

Fig. 8 : Variations of bearing capacity σ_u with depth for deep foundation.

5. Conclusion

The proposed SE criteria, fig. 4 and fig.5, can be used to interpret CPT data using the cone resistance q_c as the only input variable. This method is significant especially in a case where the accuracy of the skin friction q_s is in doubt. The measured q_c can be corrected for pore pressure(u) to q_{ct}, eqn. (9) and then used in the chart. However, its validity is a subject of verification.

$$q_{ct} = q_c + u(1-a) \qquad \text{... (9)}$$

where a = effective area ratio of cone.

It shall be noted that Fig. 3 was prepared from data obtained with mechanical cone device although, fundamentally, the figure was based on a theoretical concept. Measurements with electric cone devices generate more reliable data. It is envisaged that good results would be obtained if an electric cone data are interpreted with fig. 3. However, the level of accuracy of the result therefrom is a subject of further research.

6. Limitations

The applicability of SE-criteria may be valid for deposits with similar dispositions as the Niger Delta region of Nigeria. Its adaptability to other geologic environments is a subject of further investigation.

7. Acknowledgement

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References

[1] **ISOPT. 1, (1988).** "Cone penetration test". Int. Ref test procedure. Proc. *Int. Symp. on penetration testing I, Orlando California*. 20 - 24 March.

[2] **LUTENEGGAR, A.J., M.G. KABIR, (1989)**. Interpretation of piezocone results in overconsolidated clays, *Proc. Pen. test in U.K. Inst. of civil Engrs*, Birmingham. pp 147 - 150, 6 - 8 July.

[3] **HUIJZER, G.P. (1990**). Automated stratigraphic classification of CPT data. *Proc. 6th Int. IAEG, congress, Amsterdam*. pp 309 - 311, 6-10 August.

[4] **JUANG, C.H.; HUANG, X.H., HOLTZ, R.D**. and **J.W. CHEN (1996).** Determining relative density of sands from CPT using fuzzy sets. *Journ Geotech Engrg, ASCE*, Vol. **122m,** No.**1** Jan.

[5] **HARDER, H AND G. VON BLOH, (1988**). Determination of representative CPT parameters. Penetration testing in the UK. *Proc. Geotech Conf. Birmingham. 6-8 July*. pp 237 - 240.

[6] **BROMS, B.B. (1986)**. Penetration tests. *4th international Geotech seminar. 25 - 27 November*. Nanyang Tech. Inst. singapore.

[7] **BIDDLE, D.W. (1989)**. Penetration testing in the UK. *Proc. Geotech. Conf. Birmingham, 6-8 Jully, ICE*, pp 189 - 191.

[8] **OWUAMA, C.O. (1994)**. A model for interpretation, of penetration resistance in soils. *PhD Thesis, University of Benin*, Benin City.241 p.

[9] **OWUAMA, C.O. (2000)**. A comprehensive method of interpretation of static cone data. *NSE Technical Transactions*, vol. **35** No. **1** Jan. - Mar. pp 53 - 66.

[10] **VESIC A.S (1967**). Ultimate loads and settlement of deep foundations in sand. *Proc. Symp. bearing capacity and settlemet of foundations. Duke University*. pp 53-68.

[11] **MEYERHOF, G.G (1976)**. Bearing capacity and settlement of pile foundations. Journ. *Geotech Engrg, ASCE* vol. **12**, No. **GT3**. pp 194 - 228.

[12] **ROBERTSON, P.K.** and **R.G. CAMPANELLA (1983)**. Interpretation of cone penetration test part 1-sand, *Canadian Geotech. Journ*., vol **30**, No.**4**, pp 734 -745.

[13] **MEYERHOF, G.G. (1953)**. The bearing capacity of foundations under eccentric and inclined loads. *Proc. 3rd Int. Conf. Soil Mech. Zurich.* Vol. **1**, pp 440 - 45.

[14] **VESIC, A.S (1963)** Bearing capacity of deep foundation in sand. *Highway Research Method*. No. **39** p 112 - 153.

طريقة شبه عملية لتفسير البيانات المتحصل عليها من اختبار (CPT)

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المستخلص : يعتبر اختبار المخروط (CPT) من الطرق التي يمكـن الاعتماد عليها لقياس خصائص طبقة تحت التربة . وفـي معظـم قياسات اختبار (CPT) تمثل المقاومة لحركة رأس المخروط ظروف التكوين . تم توضيح طريقة شبه تجريبية لتفسير البيانات المتحصل عليها من خلال مثل هذا الاختبار . وقد تم تطوير هذه الطريقة من المفاهيم النظرية . ويمكن الحصول على كل من زاويـة الاحتكـاك الداخلي وقوة التماسك ، وسعة التحميل النهائي للتربة من المقاومـة الملحوظة عند رأس المخروط . وتعتبر النتائج المتحصل عليها جيدة مع الظروف المشهودة ، كما تعتبر هذه الطريقة مناسبة لا سيما في الحالات التي تكون فيها الدقة في القياس غير واضحة أو مشـكوك فيها .