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# Evaluation of the Behavior of Sudanese Soils using Cone Penetration Test (CPT) Method; The State of the Art

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## ABSTRACT

This paper reviews the main findings and conclusions drawn from research studies that dealt with the application of the CPT method for estimating important geotechnical characteristics of Sudanese soils. Great efforts were made in previous research works to develop correlation relationships between CPT data and soil parameters determined from laboratory testing of samples obtained from different states in the country. Generally, the CPT has proved to be useful in the characterization of studied soil behavior. Sound relationships have been proposed for predicting the SPT, undrained shear strength, compressibility, over-consolidation ratio and swelling potential of local soils from CPT data. It has been shown that the relationships between CPT and the soil parameters considered are affected by some factors such as the type, moisture condition and stress history. The relationships based on CPT proposed for the SPT and undrained soil strength have been revised and modified using further databases and more rational analysis techniques made available in recent times. The developed correlation relationships are useful for evaluation of important soil parameters that are needed for the design and construction of foundations in future engineering projects.

## المستخلص

تقدم هذه الورقة بياناً بالأحوال التقنية الراهنة و اهم مخرجات و نتائج الدراسات البحثية التي اجريت بغرض استخدام نتائج اختبار المخروط المتعادل لاختراق التربة الاستاتيكي لتقدير خواص جيوتقنية هامة لأنواع مختلفة من التربة السودانية. بذل مجهود كبير في اعمال البحث السابقة لأنشاء علاقات ارتباط رياضية بين نتائج اختبار المخروط المتعادل و خواص التربة المتحصل عليها من نتائج

اختبارات معملية اجريت علي عينات تم اخذها من مواقع عديدة في ولايات مختلفة من البلاد. لقد ثبت بصفة عامة أن اختبار المخروط المتعادل مفيد في تحديد طبيعة خواص و سلوك انواع التربة التي تمت دراستها. تم استنباط علاقات رياضية لتقدير اهم خواص التربة و التي شملت مقاومة اختراق التربة القياسي، قوة قص التربة الغير جاف ، أنضغاطية التربة، درجة تصلب التربة المفرطة و قابلية التربة للتمدد. اوضحت نتائج البحوث التي اجريت ان العلاقة بين اختبار المخروط المتعادل و الخواص المشار اليها اعلاه تتأثر ببعض العوامل الاخرى مثلاً نوع التربة و درجة رطوبتها و تاريخ اجهادها. تم اجراء تعديلات و تحديث العلاقات بين اختبار المخروط المتعادل من جانب و اختباري مقاومة اختراق التربة القياسي، قوة قص التربة الغير جاف من الجانب الاخر بإضافة قاعدة بيانات جديدة و استخدام طرق تحليل حديثة. العلاقات الرياضية المستنبطة يمكن استخدامها في تقدير خواص التربة الجيوتقنية المطلوبة لأعمال تصمييم و تنفيذ الاساسات للمشاريع الهندسية المستقبلية.

**Keywords:** CPT, Sudanese soils, SPT, Undrained shear strength, Compressibility, OCR, Swelling potential.

## 1 Introduction

The cone penetration test “CPT” is a site investigation tool commonly used in geotechnical engineering for classification and characterization of soils. The main advantages which make the method superior to the other site investigation techniques include:

- The equipment can be easily and quickly mobilized to the site
- The test is relatively quick, simple and economical
- The test results provide information on soils in their undisturbed or natural conditions
- The test provides a continuous record of data measurement for investigated soil depth
- The test provides repeatable and reliable data i.e. not operator dependent
- There are strong theoretical basis for CPT data interpretation

The main disadvantages of the method are that no soil samples could be retrieved during testing and the penetration can be restricted in gravelly and highly cemented soil layers [1].

The development and wide application of the CPT method is mostly due to the fact that the test has yielded a considerable amount of valuable information needed in the design of foundations. The CPT can be performed using a mechanical or electrical cone device but generally the soil correlation relationships developed using data derived from both techniques may still be useful. The results of the CPT have been applied for soil classification and the determination of soil properties required for estimation of the bearing capacity and settlements of foundation soils and the design of shallow and deep foundations. Worldwide, the method has proved its reliability in solving some of these foundation problems in regions where a sufficient experience has been gained in the interpretation of the test results.

The CPT method was introduced in Sudan in 1976 when the Building and Road Research Institute (BRRI), University of Khartoum was involved with a Dutch consulting firm in carrying out extensive geotechnical investigations for Jonglie canal project in Southern Sudan. The first research in which the CPT was applied for aspects related to the geotechnical behaviour of some Sudanese soils was started in 1978 and since then several studies have been undertaken at BRRI and their findings and conclusions were published in national, regional and international journals and conferences.

This paper reviews the experience gained in Sudan on the CPT through documentation of the main findings of the various studies undertaken to evaluate the behaviour of local soils. It is considered as a continuation of the recently published paper on the application of the CPT method for the classification of local soils [3].

## 2 Use of the CPT method for estimating soil properties; a brief review

### 2.1 Preamble

The importance of establishing correlation relationships between the soil properties or parameters determined from conventional testing methods and the CPT data is that some theoretical, semi-empirical and empirical solutions of foundation engineering problems are based on the CPT data. Numerous empirical methods and correlations have been developed in many countries for evaluation of soil parameters which are needed for geotechnical design and quality control from the CPT data. A brief review of the widely accepted methods using the CPT method to evaluate important soil parameters is presented in the following sections.

## 2.2 Standard penetration test (SPT)

The standard penetration test, SPT is used in most countries as a routine test for estimating the relative density ($D_r$) of sand soils which is closely related to the angle of internal friction, bearing capacity and settlement of soils supporting foundations. However, despite continued efforts to standardize the SPT procedure and equipment there are still problems associated with repeatability and reliability [4]. Because of its widespread use, many relationships have been developed in several countries between the SPT N value and the CPT cone resistance, $q_c$ to enable estimating either parameter when the other is known and allows the application of solutions of the foundation problems based on the CPT and SPT methods.

The first correlation in which the SPT N value, defined as the soil resistance to the penetration of a 50mm diameter split spoon tube driven by a standard hammer into the soil, expressed in blows per foot (30.5cm) tube penetration and the CPT cone resistance $q_c$ were directly related by the following simple equation suggested in 1956 for fine or silty, loose to medium dense sand [5]:

$$q_c(kg/cm^2) = 4N(blows/30.5\ cm) \tag{1}$$

However, it has been reported that using Eq. 1 without taking into account the types of test equipment used and soils tested might lead to a serious error. A more flexible $q_c - N$ relationship was subsequently proposed [6] wherein the fixed constant of 4 was replaced by a variable constant “n”. The ‘n’ value reported in literature ranges widely from 2 to 18 depending on soil type, equipment and testing method.

A more general relationship that may apply to any soil type was proposed between N and the CPT parameters $q_c$ and $R_f$ through the constants $A$ and $B$ [7] as follows:

$$N = \left(A + B \times R_f\right)q_c \tag{2}$$

Several other CPT-SPT relationships have also been developed in various countries [8, 9] probably the most popular is the graphical correlation by Robertson et al. based on numerous SPT and CPT data collected from 18 sites in the USA [10]. However, in most studies undertaken the CPT machines used were equipped with electrical cone tips for measuring $q_c$ and $R_f$.

## 2.3 Undrained shear strength of cohesive soils

The undrained shear strength, $S_u$ at a point on a particular plane of a given soil is expressed according to the Coulomb's theory as a linear function of the normal stress at failure $\sigma_f$, undrained cohesion, $c_u$ and undrained angle of internal friction, $\varphi_u$ as:

$$S_u = c_u + \sigma_f \tan \varphi_u \tag{3}$$

Various methods mostly based on Terzaghi bearing capacity theory [11] have been developed in some studies to correlate the CPT cone resistance $q_c$ and the undrained shear strength of cohesive soils $S_u$ with most formulae of the following equation form:

$$S_u = (q_c - \sigma_v)/N_c \tag{4}$$

$N_c$ is a bearing capacity factor, defined as the cone factor of the clay soil and $\sigma_v$ is the effective overburden pressure. It has been shown that the $N_c$ factor cannot be imagined as a simple constant but depends on several factors such as the cone shape and roughness and the soil properties thus, the use of a certain value for all soils and penetrometers leads to a serious error [12] such as gross overestimation or underestimation of shear strength and thus the bearing capacity of foundation soils. In fact, the $N_c$ values reported in published literature varied widely from 5 to 70 for clay soils [6]. Nevertheless, the general relationship in Equation 4 was used by several researchers to develop $S_u - q_c$ correlations with $N_c$ values matching the local clay soils.

Many similar relationships have been proposed worldwide for cohesive soils between $q_c$ and $S_u$ but with the $N_c$ in Equation 4 replaced by the cone factor $N_k$ expressed in Equation 5 with reported values given in Table (1).

$$S_u = (q_c - \sigma_v)/N_k \tag{5}$$

In general, the $S_u - q_c$ relationships proposed by many researchers in various countries were mostly developed for soft to stiff and saturated cohesive soils while similar research works on unsaturated and over-consolidated clays are rare in published literature.

**Table 1: Typical $N_k$ values reported for various soil types from different countries**

| Soil types and source | $N_k$ range | Reference |
|---|---|---|
| Various soil types (worldwide) | 5 - 70 | Sanglerat, G., 1972 [6] |
| Alluvial clays, Malaysia | 12 - 19 | Abdel Rahman, I.,2007 [13] |
| Various NC soft clays, Germany | 8 - 29 | Gebreselassie, B., 2003 [14] |
| Quaternary clay and clay stone, Germany | 89.3 | Gebreselassie, B., 2003 [14] |
| Klang Clay, Indonesia | 5 - 12 | Chen, C. S.,2001[15] |
| Stiff fissured clays (general) | 11 - 30 | Terzaghi et al [16] |
| Soft to firm saturated clays, Nigeria | 34.2-57.2 | Otoko and Isoteim, [17] |
| Busan clay, Korea | 7-20 | Hong et al, 2010[18] |
| Soft Holocene clays, Hungary | 12 - 32 | Zsolt Rémai, 2012[19] |

## 2.4 Soil Compressibility

The soil compressibility characteristics may be determined from oedometer testing and are normally expressed in terms of certain parameters such as the compressibility constant $C$, compression index $C_c$, coefficient of volume compressibility $m_v$ and the constrained modulus $E_s$.

The first correlation of CPT data and soil compressibility was proposed as early as 1940 by Buisman [21] who estimated the $C$ constant of loose sand soil as 1.5 times the ratio of the cone resistance $q_c$ to the effective overburden pressure $\sigma_o$. However, subsequent research works have indicated that the 1.5 value must be replaced by a variable denoted as α which depends on the nature of soil tested and the relationship was modified accordingly as follows:

$$C = \alpha(q_c/\sigma_o) \quad (6)$$

The compressibility constant $C$ is related to $C_c$ and the void ratio, e by the expression [20]:

$$C = 2.3\,(1 + e)/C_c \quad (7)$$

The constant $C$ is also related to the modulus $E_s$ and the coefficient $m_v$ by the equation:

$$C\sigma_o = E_s = 1/m_v \tag{8}$$

From Equations 6 and 8, $E_s$ and $m_v$ may be related to $q_c$ as follows:

$$m_v = \frac{1}{E_s} = \frac{1}{(\alpha\, q_c)} \tag{9}$$

The Buisman method originally developed for sand soils was further extended for normally and under-consolidated cohesive soils by re-arrangement in Equations 6 and 8 and substitutions of $m_v$ in Equation 9 to express $C_c$ in terms of $q_c$, initial void ratio $e_o$ and $\sigma_o$ as given below:

$$C = \frac{2.3(1+e)\sigma_o}{\alpha\, q_c} \tag{10}$$

For over-consolidated clay soils, the equation was modified by replacing $\sigma_o$ and $e_o$ by the preconsolidation pressure $\sigma_c$ and corresponding void ratio $e_c$ [22].

Equations 6 to 10 furnish suitable mathematical forms that can be experimentally verified by comparison of field CPT data with the $C_c$ (or $C$) and $m_v$ soil parameters determined in the laboratory. Following this approach, several relationships have been developed for various soil types in different countries. However, it has been indicated that no universally accepted relationship can be established between CPT data and soil compressibility; thus applying of a correlation developed for soils in a specific region would be questionable for those from other regions.

## 2.5 Stress History

The CPT method was also used for the evaluation of stress history of clay soils expressed in terms of the pre-consolidation pressure $\sigma'_p$ and over-consolidation ratio OCR parameters. Several theoretical models in which the $\sigma'_p$ or OCR were related to the CPT data were proposed in previous studies [23, 24]. A typical relationship showing the variation of $\sigma'_p$ with the net cone resistance $(q_c - \sigma_{vo})$ is shown in Figure 1 [25]. Similarly, the relationship between the OCR, the normalized cone resistance and plasticity index was proposed for intact and fissured clay soils in Figure 2 [26].

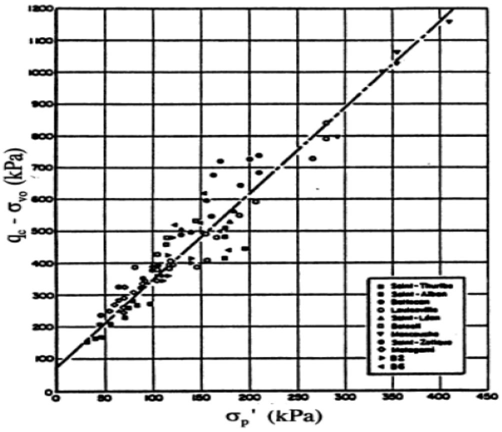


**Figure 1: Relationship between pre-consolidation pressure and net cone resistance**

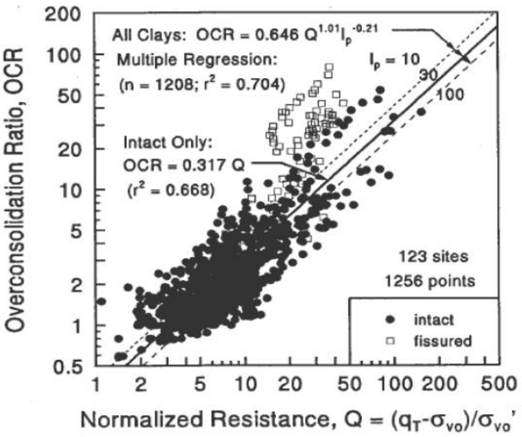


**Figure 2: Relationship between over-consolidation ratio and normalized cone**

## 3 Applications of CPT method for Sudanese soils

### 3.1 Techniques and Methodology

The main research topics investigated in the studies undertaken in Sudan since 1980 and covered in this review paper included CPT-SPT correlation and the evaluation of the undrained shear strength, compressibility, stress history and swelling potential of fine grained soils. The soil types investigated were collected from many sites mainly located in Khartoum State but test data from other parts of the country were also included as reported in few works.

The equipment used for CPT testing was hydraulically operated sounding machines with 100 to 200 kN rated capacities and the soil penetration device regularly used was the standard mechanical adhesion jacket cone with 10cm$^2$ area and 30° apex angle.

For the purpose of making useful comparisons with laboratory test results, the CPT soundings were made very close to the locations of boreholes drilled to obtain representative soil samples and determination of required soil parameters. The penetration testing was done according to the ASTM standard procedures [27] and the cone penetration $(q_c)$ and skin friction $(f_s)$ were determined at the same soil depths were the SPT was made or representative soil samples were retrieved for laboratory testing.

The main conclusions drawn from the research works reported on the comparison of the CPT data and the investigated parameters of local soils are presented in the subsequent sections.

## 3.2 CPT- SPT correlation

A comparative study was undertaken in 1978 between the CPT cone resistance $q_c$ and SPT blow counts $N$ to examine the validity of relationships published in previous studies and consider developing new $(q_c - N)$ correlation that might be applicable to Sudanese soil types and conditions [2]. The study program covered a variety of soil types collected from many sites located mainly in Khartoum State and along the proposed route of Jonglei canal project in Southern Sudan.

For the purpose of data analysis, the soil types tested were divided into four main groups according to the USCS Scheme as shown in Table (2) and for each soil type the values of $q_c$ (kg/cm$^2$) and $N$ (blows per 30.5cm penetration) were plotted against each other. Generally, a proportional relationship was found between $q_c$ and N for each soil group but the data scatter was significant. The $q_c/N$ ratio and friction ratio $R_f$ were computed for each soil group, and a summary of their ranges and average values is given in Table (2). It was found that the $q_c/N$ ratio is dependent on the soil type such that higher $q_c/N$ ratios correspond to cohesionless soils and lower $q_c/N$ ratios pertain to cohesive soils. Since different soil types and conditions were considered it was deemed important to consider the variability effects on the $q_c - N$ correlation to be developed for local soils.

**Table 2: Average values of $q_c/N$ ratio and friction ratio $R_f$ ratio for different soil groups**

| Soil group | $q_c/N$ ratio | Average $R_f$ (%) |
|---|---|---|
| Clays | < 2.0 | 5.8 |
| Silty clays and sandy clays | 2.0 - 3.0 | 4.5 |
| Clayey sands and sand-silt mixtures | 3.5 - 4.5 | 3.5 |
| Sands | > 5.0 | 1.7 |

To study the soil type effect on the $q_c - N$ relationship and thus establish a more generalized CPT-SPT correlation that can be applied for all soils, the friction ratio $R_f$ was introduced as an indicative parameter as suggested in some previous studies [28]. Linear relationships were established between $q_c$ and N for soil types having similar $R_f$ values.

An update of the developed relationships was made in 2002 for the soil groups in Table (1) to improve the degree of data matching by including the CPT-SPT test results made available after 1980 [29]. Statistical analysis was carried out on 138 data points pertaining to various soils using mathematical models to establish the best $q_c - N$ relationship. It was found that the relationship between cone resistance $q_c$ and the ratio of $N$ to $R_f$ $(N/R_f)$ can best be expressed by the following polynomial equation.

$$q_c = -0.0038(N/R_f)^2 + 1.6(N/R_f) + 10.3 \quad (R^2 = 0.64) \tag{11}$$

The suitability of Equation 11 was checked by applying the same mathematical model for soils of different origin using database of a study performed in the USA [30] and in relationship of better correlation was derived as expressed by Equation 12. This suggests the reliability of the developed polynomial equation for describing the $q_c - N/R_f$ relationship for soils tested in a country from a different continent.

$$q_c = -0.0865(N/R_f)^2 + 11.56(N/R_f) - 1.23 \quad (R^2 = 0.74) \tag{12}$$

A graphical solution of Equation 11 derived for Sudanese soils was made as depicted in Figure 3.

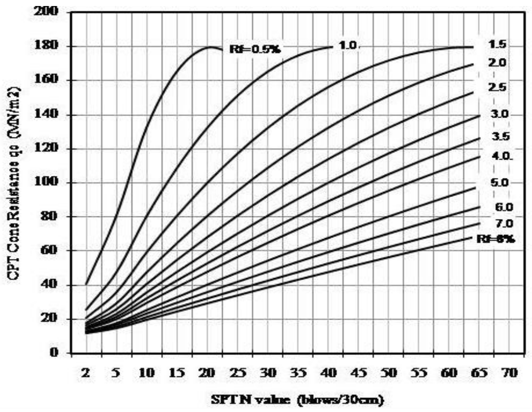


**Figure 3: Combined $q_c$-$R_f$-N chart for local soils**

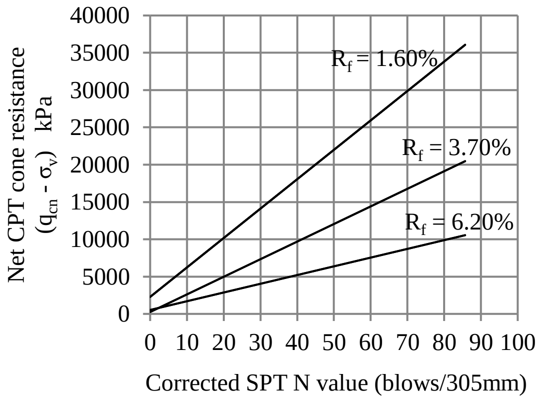


**Figure 4: Best fit lines of $q_{cn}$ versus $N_{corr}$ relationships for sands, mixed and clay**

Using Figure 3 the SPT *N* value may be estimated from $q_c$ or vice versa for a given $R_f$ which can be directly obtained from CPT data. On the other hand, the $R_f$ value may be assumed for a given soil groups in Table (2) then the $q_c$ may be estimated from *N* value. Therefore, either Equation 11 or the chart in Figure 3 can be used in conjunction with the data in Table (1) in order to estimate either parameter for the various soil types.

More recently, a modification was introduced to the CPT-SPT chart developed in 2002 by dividing the analyzed database in three main groups representing sandy soils, mixed clay-silt-sand soils and clay soils [31]. The SPT and CPT results were corrected for the overburden pressure effect to obtain the corrected SPT $N_{corr}$ and the net cone resistance $q_{cn} = (q_c - \sigma_v)$ parameters which were then considered in analysis instead of the measured gross $N$ and $q_c$ values. The corrected variables were then plotted against each other for three soil groups having assumed $R_f$ ranges of 0.5 to 2.5%, 2.6 to 4.5% and 4.6 to 7.5% as shown in Figure 4. As may be noted, linear relationships of fairly high correlation ($R^2$ =0.76 to 0.80) were developed between $q_{cn}$ (in kPa) and $N_{corr}$ (blows per 30.5cm) from analysis as given by Equation 13 for the three soil groups.

Sandy soils ($R_f = 0.5 \ to \ 2.5\%$ ): $$q_{cn} = 390\ N_{corr} + 2657 \quad (13a)$$

Mixed clay-silt-sand soils ($R_f = 2.6 \ to \ 4.5\%$): $$q_{cn} = 236\ N_{corr} + 248 \quad (13b)$$

Clay soils ($R_f = 4.6 \ to \ 7.5\%$): $$q_{cn} = 118\ N_{corr} + 558 \quad (13c)$$

A combined chart was then established as shown in Figure 5 to enable direct conversion from $q_{cn}$ to $N_{corr}$ or vice versa. This was made by interpolation and extrapolation of $q_{cn}$ and $N_{corr}$ data points using the regression lines in Equation 13 for making computations required to produce the best fit lines of equal $R_f$.

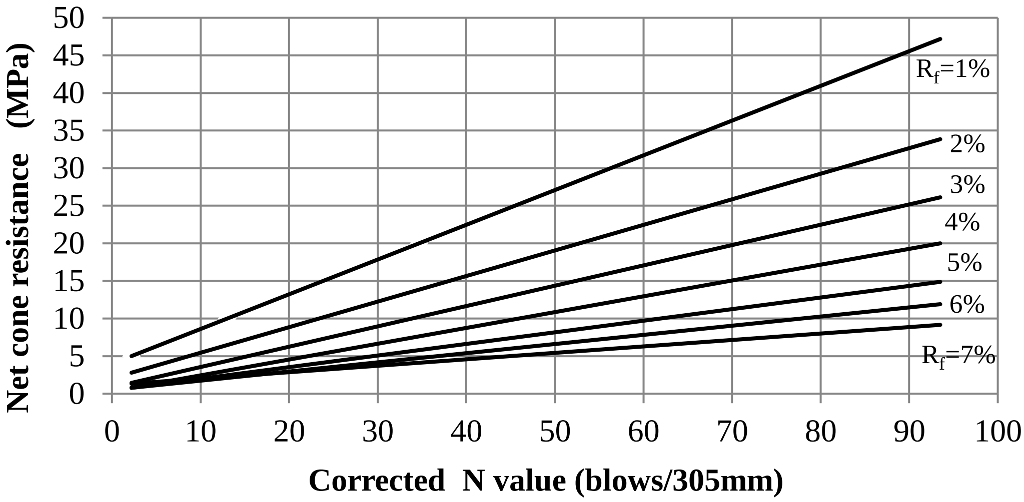


**Figure 5: Modified CPT- SPT correlation chart for Sudanese soils**

### 3.3 Relationship between CPT and undrained shear strength of cohesive soils

The first correlation between undrained shear strength $(S_u)$ and the CPT data was reported in 1980 for the alluvial silty clay and clayey silt deposits stretching along the Blue Nile left bank in Khartoum city [2]. Fifty undisturbed samples mostly representing highly plastic clay and silt soils where conditions of full and partial saturation existed were considered for analysis. The undrained cohesion $(c_u)$ and angle of internal friction $(\varphi_u)$ shear strength parameters were determined from UU triaxial test results and then the $S_u$ values were computed using the following formula derived as explained in the study.

$$S_u = c_u + \tan {\varphi_u}^2[R(1 - \sin \varphi_u) + (1 + \sin \varphi_u)] \qquad (14)$$

The total stress ratio denoted by $R(= \sigma_1/\sigma_3)$, was obtained by plotting the $(\sigma_1/\sigma_3)$ ratio used for the tested specimens against $\sigma_3$ and then taking R values from the plot when $\sigma_3$ is equal to the effective overburden pressure $\sigma_{vo}$. The minor principal stress $\sigma_3$ at which soil specimens fail during testing was assumed to be the effective overburden pressure corresponding to the sample depth.

The cone factor $N_c$ was computed as the ratio of $q_c$ to $S_u$, and then a statistical analysis was made to correlate the two parameters for the soil types tested. The analysis yielded a linear relationship with a good matching of the $q_c$ and $S_u$ data ($R^2 = 0.81$) as given below:

$$q_c = 34.9\, S_u + 0.16 \qquad (15)$$

$N_c$ values of 32.5 and 39.5 were obtained for the soils located above and below ground water table respectively which represents the unsaturated and saturated moisture conditions. For practical purposes, a single $N_c$ value of 35 was assumed for all soil types irrespective of their moisture condition.

A research study involving a large data size (187 samples) was carried out in 2004 [32] to investigate the effects of soil type and stress history on the $q_c - S_u$ relationship. The soils tested were divided into clay soils (CL and CH types) and silt soils (ML and MH types). The $S_u$ and $q_c$ values were computed for each soil group and the cone factor $N_c$ was determined from data analysis as given in Table (3). Generally, rather poor relationships were found between $S_u$ and $q_c$ when the data pertaining to combined clay and silt soils were analyzed. However some relationship trends were established upon considering each soil type separately with $N_c$ values ranging from 34 for the CL and CH clays to 37.2 for the MH silt soils. As may be noted in Table (3), an exceptionally high $N_c$ value of 61.5 was obtained for the low plastic silt soil but the correlation between $S_u$ and $q_c$ data was very poor.

It worth mentioning that the $N_c$ values obtained for the CL, CH and MH soil groups compare favorably to those reported in a previous study [2].

The effect of stress history expressed in terms of the over-consolidation ratio (OCR) on the $q_c - S_u$ relationship was broadly investigated in this study. The soil types were grouped according to their OCR into normally or slightly consolidated (OCR < 2), moderately over-consolidated (2< OCR < 6) and heavily over-consolidated (OCR ≥ 6). Generally, the study showed that the $q_c - S_u$ relationship of a given soil may be significantly affected by stress history such that a lower $N_c$ corresponds to a higher OCR and vice versa, but no conclusive findings were drawn in this respect.

**Table 3: Cone factor $N_c$ obtained for Sudanese silt and clay soils**

| **Soil main group** | **Silts** | | | **Clays** | | | **All soils** |
|---|---|---|---|---|---|---|---|
| **Soil subgroup** | ML | MH | ML+ MH | CL | CH | CL + CH | |
| **Data size** | 37 | 36 | 73 | 32 | 79 | 113 | 187 |
| **Cone factor $N_c$** | 61.5 | 37.2 | 51.5 | 34.0 | 34.5 | 34.4 | 38.7 |
| **$R^2$ Coefficient** | 0.18 | 0.52 | 0.32 | 0.35 | 0.31 | 0.10 | 0.07 |

A comprehensive study was undertaken in 2017 on fine grained soils to investigate the effects of soil type, moisture condition and stress history on the $q_c - S_u$ relationship [33]. Database pertaining to low to high plastic clays and silts collected from many study sites in different states were considered for analysis. Figure 6 shows the plasticity properties of the soils types considered.

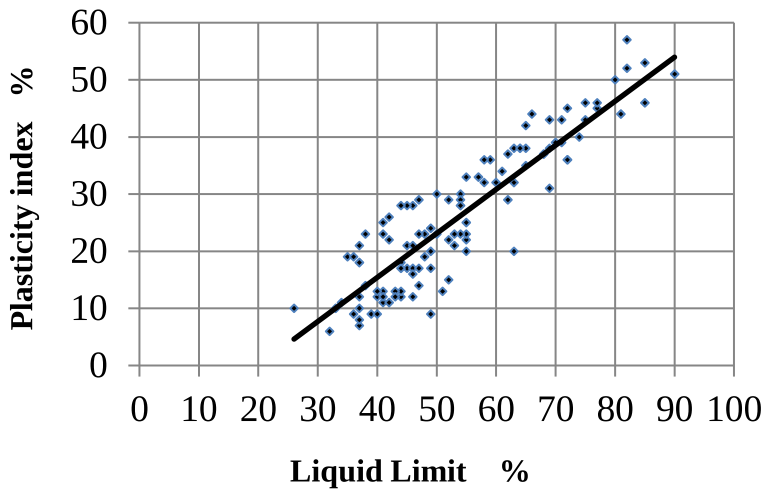


**Figure 6: Plasticity properties of studied soil types**

The undrained shear strength test results indicated that most of the soil samples were of firm to very stiff consistency but few of them were soft or hard. The cone factor, denoted in this study by $N_k$ was found to vary significantly over a range of 35.1 to 55.6 for all soil types with average values of 37.5 and 44.1 for the clay and silt soils respectively. The average $N_k$ value obtained for the clay soils is generally in close agreement with that previously reported for the CH and MH soils [2]. However, the $N_k$ values were also compared to the values published in literature and found to be much higher than those reported in several countries for saturated soft to firm cohesive soils [14, 15, 16, 18]. $N_k$ values lower than those obtained in this study were reported in few previous studies for hard and over-consolidated clay and silt soils [6, 18].

Furthermore, the study revealed that for a given soil the $N_k$ is not a simple constant but depends on several factors such as moisture condition and soil stiffness prevailing in the field and therefore, the relationship between $S_u$ and $q_c$ will be influenced by these factors. The database was analyzed to investigate the effects of soil type, moisture condition and stress history on the $q_c - S_u$ relationship. It was noted that a reliable correlation cannot be developed for soils of different types and pre-consolidation histories and this makes establishing a general and sound data correlation difficult. Moreover, the $N_k$ value may differ significantly even for the same soil depending on its initial moisture condition i.e. whether it is saturated or unsaturated.

In an attempt to obtain a sound relationship the values of $q_c$ and $S_u$ were modified to the net cone resistance $q_{cn}\,(= q_c - \sigma'_v)/\sigma'_v$ and undrained shear strength $S_{un}(= S_u/\sigma'_v)$ both normalized to effective overburden pressure. A rigorous analysis was then performed taking into account the over-consolidation ratio (OCR) effect to develop a suitable mathematical form for modeling the $q_{cn}$ and $S_{un}$ relationship. The best relationship was established for the clay and silt soils when $q_{cn}$ was plotted against the ($S_{un}$OCR) product as shown in Figure 7 and Figure 8 respectively.

The trends depicted in the Figure 7 and Figure 8 indicate that there is a linear relationship between the two terms as expressed in the following formulae:

$$Clay\ soils: \quad \frac{(q_{c-}\sigma_{vo})}{\sigma_{vo}} = 6.0\left(\frac{S_u OCR}{\sigma_{vo}}\right) + 20.7 \tag{16}$$

$$Silt\ soils: \quad \frac{(q_{c-}\sigma_{vo})}{\sigma_{vo}} = 13.9\left(\frac{S_u OCR}{\sigma_{vo}}\right) + 8.1 \tag{17}$$

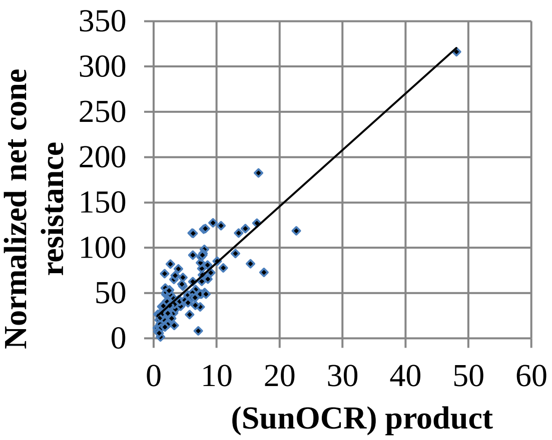


**Figure 7 Relationship between $q_{cn}$ and ($S_{un}$ OCR) product for clay soils**

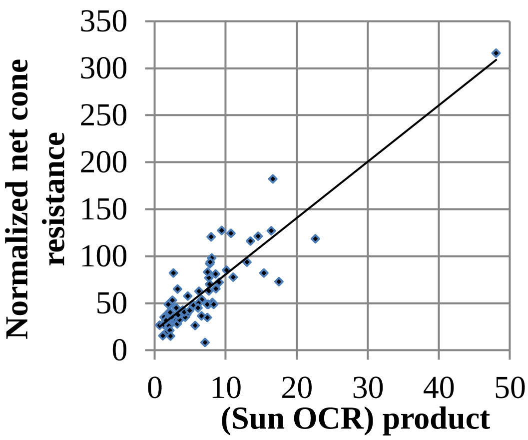


**Figure 8 Relationship between $q_{cn}$ and ($S_{un}$ OCR) product for silt soils**

The goodness of the above relationships is reflected by the fairly high $R^2$ coefficients values (0.79 and 0.81) achieved from regression analysis for the clay and silt soils soil. A similar relationship but of slightly lower correlation ($R^2 = 0.74$) was obtained from analysis as expressed by Equation 18 for the data plotted in Figure 9 pertaining to both soil types.

$$All\ soils: \quad \frac{(q_{c-}\sigma_{vo})}{\sigma_{vo}} = 6.23\left(\frac{S_u OCR}{\sigma_{vo}}\right) + 20.94 \tag{18}$$

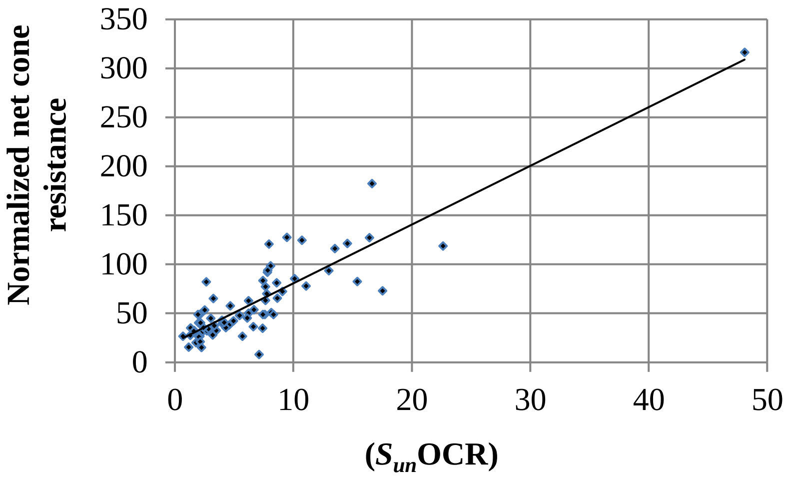


**Figure 9: Relationship between $q_{cn}$ and ($S_{un}$ OCR) product for all soils**

In fact, very poor relationships were initially obtained when the analysis was done before normalization of the $q_c$ and $S_u$ values to the effective overburden pressure $\sigma'_v$. Thus, the normalization of $q_{cn}$ and $S_u$ to $\sigma'_v$ has led to a better matching of the two parameters as indicated by the high $R^2$ values.

### 3.4 Soil compressibility

A research was undertaken in 1994 [34] to estimate the compressibility characteristics of Sudanese cohesive soils from CPT data. Many samples collected from sites in Khartoum, Northern and southern Kurdofan and White Nile states were considered in the study program. Oedometer tests were performed on undisturbed samples of clayey, silty and sandy soils to determine the main soil compressibility parameters i.e. the compression index ($C_c$), coefficient of volume compressibility ($m_v$) and constrained modulus ($E_s$).

The study results did not reveal a meaningful relationship between $q_c$ and $C_c$ when different soil types were considered in the analysis. Accordingly, the samples were sorted into plastic fine grained soils (clays and silts) and non-plastic (sands) soils to develop better $q_c - C_c$ correlation as shown in Figure 10 for silty and sandy soils and Figure 11 for clayey soils.

The data pertaining to each soil group was analyzed and the following relationships were developed for the plastic and non-plastic soils:

Plastic soils: $$C_c = 0.001{q_c}^2 - 0.03q_c + 0.38 \qquad (19)$$

Non-plastic soils: $$C_c = 0.002{q_c}^2 - 0.05q_c + 0.47 \qquad (20)$$

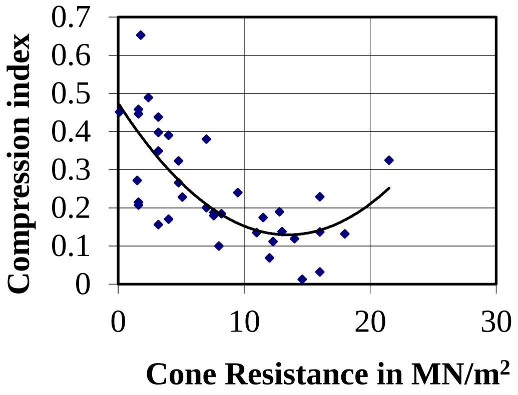


**Figure 10 Relationship between $C_c$ and $q_c$ for silty and sandy soils**

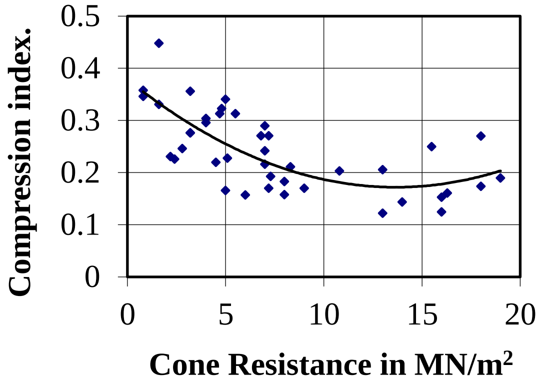


**Figure 11 Relationship between $C_c$ and $q_c$ for clay soils**

However, a significant scatter was observed upon plotting $C_c$ and $q_c$ against each other indicating that the matching of the data was not so good ($R^2$ = 0.53). It was deemed necessary to include a parameter accounting for soil variability in order to improve the reliability of developed relationships. The plasticity index, PI, and fines content (the soil fraction with grain sizes finer than 75µm) FC, were found to be the best soil type indicative parameters for the plastic and non-plastic soil groups respectively and were therefore used in analysis. The following two equations which incorporate PI and FC indices were developed for expressing the $q_c - C_c$ relationships for the plastic and the non-plastic soils data plotted in Figure 12 and Figure 13 respectively. The $q_c$ is expressed in MN/m$^2$ whereas the PI and FC are in percent in the above equations and figures.

Plastic soils: $$C_c = 1/PI\,(0.007{q_c}^2 + 0.28q_c + 2.19) \qquad (21)$$

Non-plastic soils: $$C_c = 1/FC\,(0.25{q_c}^2 - 6.67q_c + 48.2) \qquad (22)$$

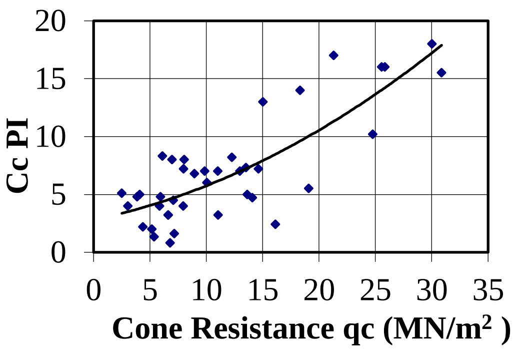


**Figure 12: Relationship between ($C_cPI$) product and $q_c$ for clay soils**

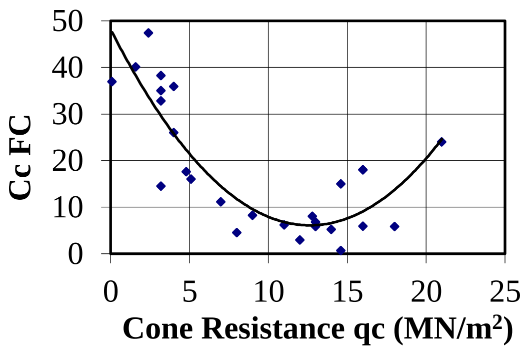


**Figure 13: Relationship between ($C_cFC$) product and $q_c$ for silty and sandy soils.**

Furthermore, the study investigated the relationship between CPT data and the coefficient of volume compressibility ($m_v$) and constrained modulus ($E_s$). As was previously indicated, if for a given soil the compressibility constant $\alpha$ which defines the relationship between $q_c$ and the compressibility constant C as given in Equation 6 is known then the values of $m_v$ and $E_s$ may be estimated from cone resistance $q_c$. A pressure increment from 100 to 200kN/m$^2$ was assumed for computing $\alpha$ value in each oedometer test.

To establish a good correlation of CPT data and soil compressibility the constant $\alpha$ was first related to the friction ratio $R_f$ using the same data plotted in Figures 14 and 15 to obtain Equations. 23 and 24 for the plastic and non-plastic soils, respectively.

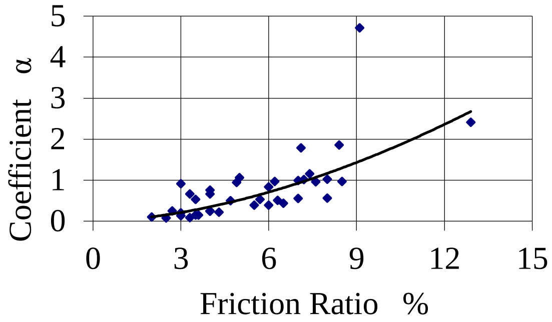


**Figure 14: Relationship between $\alpha$ and $R_f$ for plastic soils**

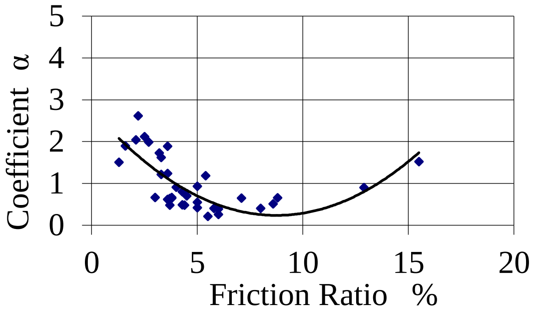


**Figure 15: Relationship between $\alpha$ and $R_f$ for nonplastic soils**

$$\alpha = 0.032{R_f}^{1.74} \qquad (R^2 = 0.61) \qquad (23)$$

$$\alpha = 0.032{R_f}^2 - 5.8R_f + 2.77 \qquad (R^2 = 0.56) \qquad (24)$$

To estimate the compressibility parameters $E_s$ or $m_v$ from CPT data, the constant $\alpha$ is first computed from the above two equations and the values of $\alpha$ and $q_c$ are then substituted in Equation 8 or Equation 9 for the soil type being considered.

## 3.5 Prediction of over-consolidation ratio of cohesive soils from CPT results

A research was undertaken on 125 undisturbed cohesive soils to investigate the relationship between stress history evaluated in terms of the overconsolidation ratio, OCR and cone resistance $q_c$ [35]. The OCR was plotted against the corresponding $q_c$ for all samples but no relationship was observed because the scatter of data points was significant. However, when the soils were categorized according to the plasticity index, PI and friction ratio $R_f$ as parameters accounting for soil variability and the $q_c$ values were normalized to the effective overburden pressure ($\sigma'_v$) certain relationship trends were revealed as shown in Figure 16 and Figure 17.

Two sets of equations were derived to express the best fit regression lines for the OCR-$q_{cn}$ relationships for the soils types tested based on the assigned **PI** and $R_f$ ranges as given in Tables (4) and (5) respectively.

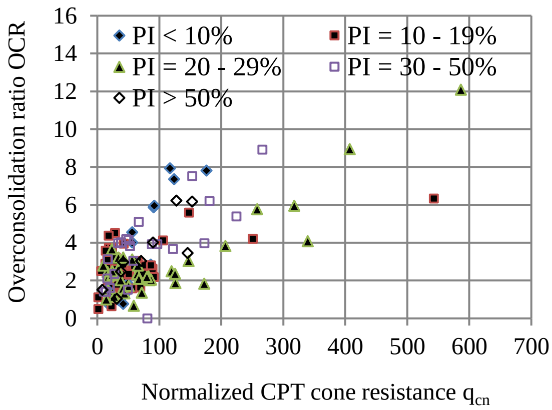


**Figure16: Relationship between OCR and $q_{cn}$ for soils of different PI ranges**

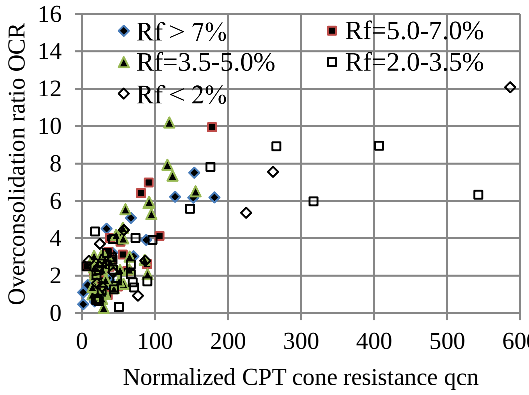


**Figure 17 Relationship between OCR and $q_{cn}$ for soils of different $R_f$ ranges**

**Table 4: Correlation relationships for OCR and $q_{cn}$ (in kPa) for soils of different of PI ranges**

| PI ( % ) | Data size | Best fit line equation | $R^2$ |
|---|---|---|---|
| < 10 | 29 | $OCR = 0.046q_{cn} + 0.67$ | 0.79 |
| 10 - 19 | 44 | $OCR = 0.011q_{cn} + 2.12$ | 072 |
| 20 - 29 | 38 | $OCR = 0.015q_{cn} + 1.32$ | 0.77 |
| 30 - 50 | 22 | $OCR = 0.021q_{cn} + 2.12$ | 0.66 |
| > 50 | 9 | $OCR = 0.028q_{cn} + 1.19$ | 0.71 |

**Table 5: Correlation relationships for OCR and $q_{cn}$ (in kPa) for soils of different $R_f$ ranges**

| $R_f$ (%) | Data size | Best fit line equation | $R^2$ |
|---|---|---|---|
| > 7.0 | 20 | $OCR = 0.034q_{cn} + 1.23$ | 0.79 |
| 5 - 7 | 21 | $OCR = 0.047q_{cn} + 0.73$ | 0.71 |
| 3.5 - 5 | 40 | $OCR = 0.049q_{cn} + 0.56$ | 0.65 |
| 2 - 3.5 | 27 | $OCR = 0.013q_{cn} + 2.10$ | 0.74 |
| < 2.0 | 16 | $OCR = 0.018q_{cn} + 1.41$ | 0.85 |

The fairly high $R^2$ values revealed indicate a good matching of correlated soil data. The $q_{cn}$-OCR relationship model based on the $R_f$ is preferred than that on PI, because $R_f$ can be readily obtained from CPT results i.e. no laboratory testing is required and also the better degree of data correlation in sandy soils.

### 3.6 Application of CPT for Assessment of Swelling Potential and Consistency of Clay Soils

The CPT method was used in a study published in 1988 to classify the swelling potential of Sudanese clay soils of low to very high degree of expansiveness [36]. The swelling potential of the soils were divided according to known schemes of expansive soil classification into four categories; including the low, moderate, high and very high degrees of swelling potential. The CPT cone resistance $q_c$ and skin friction $f_s$ of all soils types were computed and plotted as shown in Figure 18 using different symbols for soils of different expansiveness degrees. Four different zones were identified in Figure 18 to enable the direct classification of the soil swelling potential from CPT data only.

In addition, the cone resistance $q_c$ was correlated to the clay relative consistency, $C_r$ [= (LL - w)/PI] and a linear relationship was found between $q_c$ (kg/cm$^2$) and $C_r$ as illustrated in Figure 18 and expressed by Equation 25.

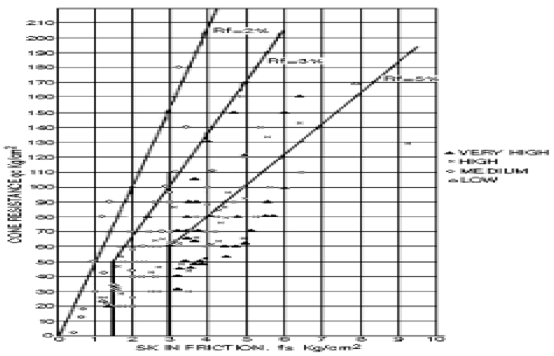


**Figure 18: Classification of soil swelling potential from CPT data**

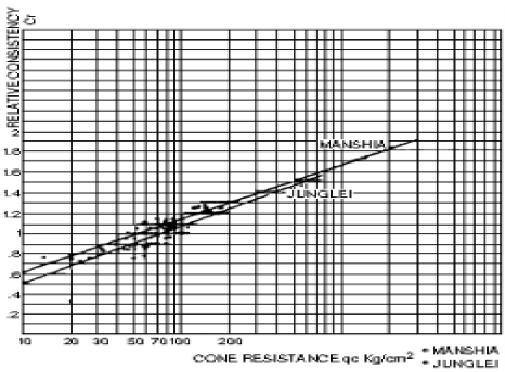


**Figure 19: Variation of soil relative consistency with cone resistance**

$$C_r = 0.6 \log q_c - 0.11 \qquad (25)$$

From Equation 25 and the best fit lines in Figure 19 useful guidelines were proposed as given in Table (6) for estimating $C_r$ values for clay soils from CPT cone resistance, $q_c$.

**Table 6: Relationship between CPT cone resistance and consistency of expansive soils**

| $q_c$ (kg/cm$^2$) | < 4 | 4 - 10 | 10 - 27 | 27 - 70 | 70 - 185 | 185 - 480 | > 480 |
|---|---|---|---|---|---|---|---|
| Consistency index $C_r$ | < 0.25 | 0.25-0.50 | 0.50 - 0.75 | 0.75 - 1.0 | 1.0-1.25 | 1.25-1.50 | > 1.50 |
| Clay soil description | Extremely soft | Very soft | Soft | Medium stiff | Stiff | Very stiff | Hard |

# 4 Concluding remarks

Several research studies have been undertaken in Sudan to investigate the value of the CPT method which has proved to be useful and reliable in the evaluation of soil behavior in many countries. Great efforts were made in the various studies to determine and collect database pertaining to soils of different types and conditions. The experience gained on the CPT application for Sudanese soils is highlighted and the main findings and conclusions drawn from the reviewed studies are outlined hereunder.

Generally, the experience gained proved that the CPT method can be successfully used to evaluate the geotechnical characteristics of Sudanese soils. The test is relatively fast, reliable, economical and has sound theoretical basis for data interpretation.

Reliable relationships were developed between the CPT cone resistance ($q_c$) and the SPT $N$ value. Further modifications based on the friction ratio $R_f$ were introduced in 2002 in the form of $q_c - R_f - N$ charts and in 2017 wherein $q_c$ was related to the $N/R_f$ ratio to improve the reliability of the CPT-SPT developed correlation and make it applicable to different soil types.

The undrained shear strength $S_u$ of alluvial clayey and silty soils was correlated to cone resistance $q_c$ through a factor $N_c$ of 35. Subsequent studies showed that the $q_c - S_u$ relationship is dependent on soil type and over-consolidation ratio. In a recent study, the cone factor $N_k$ was redefined as a ratio of normalized net cone resistance ($q_{cn}$) to undrained strength ($S_{un}$) and was found to vary from 37.5 and 44.1 for clay and silt soils, respectively. It was revealed that $N_k$ is strongly dependent on stress history, moisture condition and soil stiffness and a statistical $q_{cn} - S_{un}$ model considering these effects was developed.

The compressibility characteristics of some Sudanese cohesive soils were empirically related to the CPT cone resistance $q_c$. The reliability of developed correlation was improved by considering soil type effect reflected in terms of plasticity index PI, fines content FC and friction ratio $R_f$ soil parameters.

Sound relationships were revealed between normalized cone resistance $q_{cn}$ and overconsolidation ratio OCR based on database pertaining to soils grouped according to assigned PI and $R_f$ ranges.

The CPT method has proved to be useful in the classification of swelling potential of typical Sudanese expansive soils and the estimation of the relative consistency index, $C_r$ of clay soils.

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