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Determining soil shear strength parameters from CPT and CPTu data

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Abstract. Determining the soil shear strength parameters, i.e. cohesion (C), internal friction angle (ϕ) is done by means of laboratory tests and the in situ testing data. The cone penetration tests, CPT and CPTU, are not only quick and economical, but also repeatable and show continuous records of soil parameters with depth. The common approaches for shearing strength parameters determination from CPT data are on the basis of bearing capacity and cavity expansion theories. In this study, different methods of soil shear strength parameters determination from CPT and CPTu results, q_c, f_s and u, were reviewed and investigated. A new method is proposed for C, ϕ prediction on the basis of all quantities, q_c, u and f_s, from CPTu considering bearing capacity mechanism of failure. One advantage of this method is improved accuracy in the case of erroneous data by using all three outputs of CPTu. The proposed, current and experimental test results of an information bank including 32 CPT and CPTu results were assessed in five sites. The comparison of predicted and measured C and ϕ angle values indicates good consistency and low scatter for the proposed method. This can be led to more accurate and applicable continuous soil parameters in optimized geotechnical design.

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1. Introduction

There are many drawbacks in application of laboratory tests in geotechnical practices such as limitations in sample size, disparity of results and problems associated with sample preparation, transportation and maintenance. Therefore, in situ testing has become more common and affordable in geotechnical engineering. On the other hand, in situ tests have shortcomings such as lack of control on stress paths, drainage conditions and involvement of field complexities. Thus, each of these procedures has their own merits and disadvantages, and their results can be used in geotechnical engineering as a means of complementary considerations [1].

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The CPTu test is a popular repeatable in situ test that obtains a continuous vertical profile of soil, and can be used in soft to medium deposits. This test enables the capability of providing continuous profile of q_c (cone tip resistance), f_s (sleeve friction) and u (pore pressure parameter) in every inch of the subsoil depths [2]. Soil profile and geotechnical characteristics of soil layers are also well determined by means of this information [3]. Different researchers such as Muromachi 1972 [4], Robertson and Campanella 1988 [5], and Chen and Juang 1996 [6] have studied the determination of shear strength parameters from CPT data. However, only S_u has been determined in fine grained soils, or ϕ in non-cohesive soils.

This treatment tends to determine soil strength parameters, using all available CPTu test output data including q_c, f_s and u. To this end, two main theories have been implemented for the estimation of

C and ϕ; bearing capacity [7] and cavity expansion [8] approaches.

The available bearing capacities are in base and sleeve of the penetrometer, and failure parameters are in tip q_c and shaft f_s. These parameters are correlated to the values of soil shearing strength parameters. Obviously, lab tests for determining C and ϕ angles must be proportionate to the shearing mechanism in penetrometer sleeve and tip parts as well.

2. Review of methods for determining shear strength parameters from CPT and CPTu

Researchers presented different methods in order to determine the effective shear strength parameters in fine and coarse grained soils, which have been briefly reviewed in the following.

Muromachi (1972) [4] assumed the slip surface as a logarithmic spiral during the cone penetration, and proposed the following equation for non-cohesive soils:

$$q_c = 3/2p_0 \cos\varphi.\left(e^{2\pi \tan\varphi} - 1\right), \tag{1}$$

where p_0 is effective surcharge stress.

Trial and error is required to determine the angle of internal friction. This equation estimates internal friction angle to the nearest degree.

Meyerhof (1974) [9] presented the following equation for internal friction angle in cohesionless soils:

$$\phi = \tan^{-1}\left(\frac{q_c}{0.5N_q}\right), \tag{2}$$

where q_c is measured cone resistance, and N_q is bearing capacity factor.

Schmertmann (1978) [10] studied sandy soils behavior and suggested a correlation between ϕ and relative density, that is (see Figure 1):

$$\phi = 28^\circ + 0.15D_r, \tag{3}$$

where D_r is relative density.

Mitchell and Durgunoglu (1983) [11] investigated the relation between ϕ and q_c from CPT, regarding bearing capacity failure. They proposed a relation among ϕ, q_c and effective overburden stress as illustrated in Figure 2, based on bearing capacity theory. The basic equation of bearing capacity can be expressed as follows:

$$q_{\text{ult}} = CN_c + \bar{q}N_q + 0.5\gamma BN_\gamma, \tag{4}$$

where:

B = Penetrometer diameter (35.7 mm);

N_c, N_q, N_γ = Bearing capacity factors;

q_{ult} = Ultimate bearing capacity;

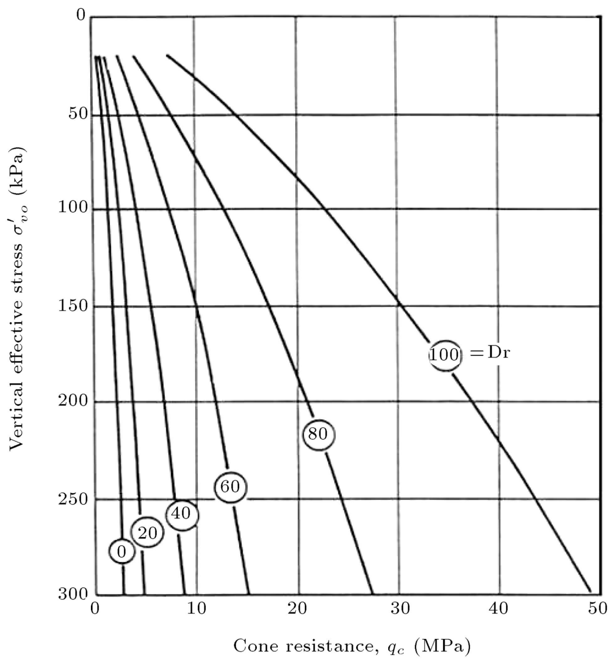

Figure 1. Cone tip resistance changes with vertical effective stress graphic [10].

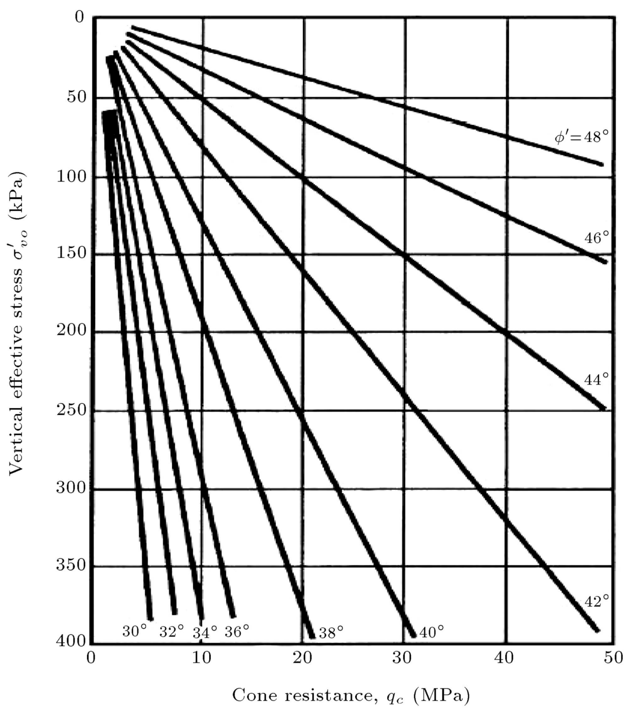

Figure 2. Friction angle changes and cone tip resistance [11].

$\bar{q}$ = Effective stress in depth z.

Since Mitchell and Durgunoglu's study [11] on granular soils, the cohesion term has been neglected, q_{ult} is then equal to q_c, which causes the soil to fail as the penetrometer moves down. Bearing capacity factors are dependent on friction angle, therefore, ϕ can be written as a function of q_c.

Robertson et al. [5,12-14] focused on sandy soils in drained conditions, and presented Eq. (5) to determine the internal friction angle as follows:

$$\phi = \tan^{-1}\left[0.1 + 0.38\log\left(\frac{q_c}{\sigma'_{v0}}\right)\right], \tag{5}$$

where σ'_{vo} is effective vertical stress (effective overburden stress).

Also, Figure 3 presents the different recommended methods for determination of N_q, as a function of ϕ angle, which is applied commonly in pile design.

Senneset et al. (1988) [15] stipulated that in coarse grained soils, pore pressure is negligible during cone penetration. They presented the following correlation based on the cone tip resistance:

$$q_c = \left[(N_q - 1)\left(\sigma'_{v0} + \frac{c'}{\tan\phi'}\right)\right] + \sigma_{v0}, \tag{6}$$

where N_q is bearing capacity factor, ϕ' and c' are effective shearing strength parameters, and σ_{vo} is total vertical stress (total overburden stress).

While the penetrometer penetrates coarse grained soils, the soil structure distorts, and without soil cohesion, the above equation is corrected as the form of Eq. (7):

$$N_q = \left(\frac{q_c - \sigma_{v0}}{\sigma'_{v0}}\right) + 1. \tag{7}$$

By calculating the quantity of N_q in Eq. (7), friction angle value can be obtained by Figure 4.

Kulhawy and Mayne (2003) [16] considered the bearing capacity theory and investigated 24 types of sands, and proposed:

$$\phi = 17.6 + 11\log\left(\frac{q_c}{\sqrt{100\sigma'_v}}\right). \tag{8}$$

The general equation indicating the logical relation between cone tip resistance, q_c, and shearing stress in undrained conditions, using basic bearing capacity equations, can be expressed as:

$$q_{\text{ult}} = S_u N_c + \bar{q}. \tag{9}$$

By substituting q_{ult} with q_c and $\bar{q}$ by σ_{v0}, and realizing N_k instead of N_c, S_u can be determined as follows:

$$S_u = \frac{q_c - \sigma_{v0}}{N_k}, \tag{10}$$

where:

$S_u =$ Undrained shear strength;

$N_k =$ A coefficient; $10 < N_k < 15$ for N_c (Normaly Consolidated) soils.

For expanding the relation between q_c and S_u, it is necessary to determine N_k. Thus, values of S_u and

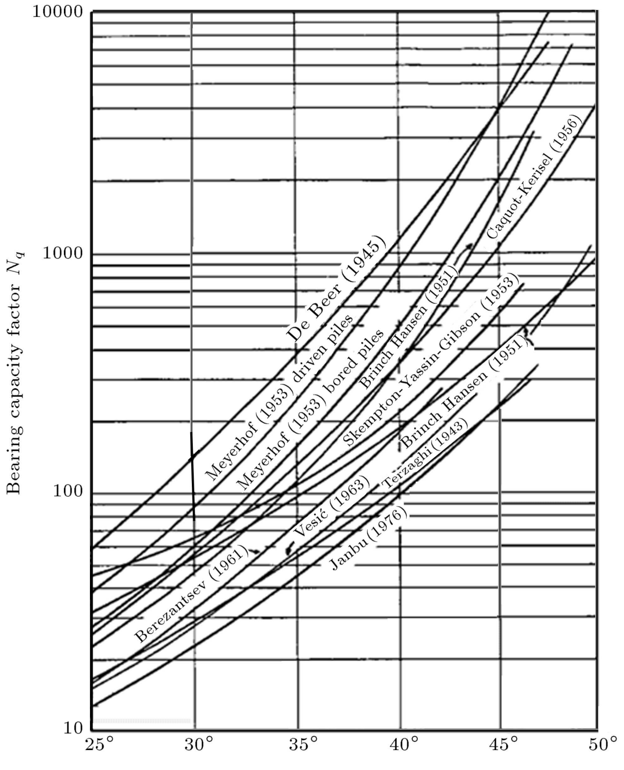

Figure 3. Recommended methods for N_q by ϕ [8].

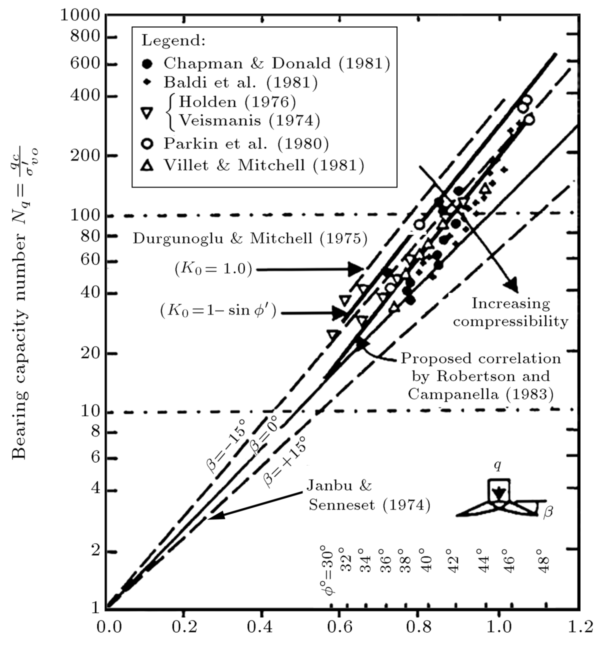

Figure 4. N_q changes on the basis of ϕ and angle β [15].

q_c obtained from in situ tests such as CPT and VST (vane shear test) can be effective. Whereas, the CPT results, dependenent on VST, are the disadvantages of this method.

Senneset et al. (1982) [17] suggested the following relation for soils with low permeability, i.e. clays and silts, which produce excess pore pressure.

$$q_n = (N_q - 1).(\sigma'_{v0} + a) - N_u \Delta u_T, \tag{11}$$

where:

$q_n = q_t - \sigma'_{vo}$ = net resistance of cone tip;

Δu_T = additional pore pressure in cone tip;

N_u = bearing capacity factor, $N_u = 6 \tan \phi'(1 + \tan \phi')$;

a = ratio between shoulder area (cone base) unaffected by the pore water pressure to total shoulder area.

The foregoing statements imply that available predictive relations are mostly based on ϕ or S_u, and focus on bearing capacity theory. Furthermore, there is almost no study in the literature, considering combination of ϕ and C, concurrently.

3. Analytical modeling for failure mechanism around penetrometer tip

In shallow foundations, bearing capacity is affected by soil nature and expansion of failure zones. Expansion of failure surface depends on three important factors; dimensions, relative foundation depth and soil geotechnical parameters. Meyerhof (1983) [18] postulated failure mechanism as spiral logarithms, using the loading test results. Hence, the assumed logarithmic spiral curve is a function of the internal friction angle (soil type) and the penetrometer area. The necessary relative depth for shear stress mobilization in total failure surface is the critical depth. In other words, the minimum penetration is equal to critical depth for preparing the maximum resistance for penetrometer base. For low penetration depths, in proporation to the critical value, the linear reduction of resistance in penetrometer-base resistance was considered. De Beer (1963) [19] indicated that displacement piles placed in bearing layers have an equal unit resistance of the penetrometer tip. Similarity between penetrometer and pile performances caused some researchers, such as Meyerhof (1974) [9], to assign spiral logarithm to the mechanism of the penetrometer base; so this curve is the function of soil type, diameter and penetration depth of a penetrometer. This mode of failure is total shearing. Failure surface reaches the penetrometer's sleeve. As a result, for failure surface, logarithm curve radius is determined by Eq. (12):

$$r = r_0 e^{\theta \tan(\phi)}, \tag{12}$$

where:

θ : Angle between radii in each point of failure surface;

r : Logarithmic spiral radius;

ϕ : Angle between failure zone's radius and normal line to logarithmic spiral (assumed to be equal to penetrometer diameter);

r_0 : Logarithmic spiral radius for $\theta = 0$ (r_0 is equal to penetrometer's diameter).

Eslami and Fellenius (1997) [20] stated that to obtain the height of failure zone on the penetrometer, the angle of θ is equal to 180 degrees; subsequently, Eq. (12) changes to:

$$r_c = b e^{\pi \tan(\phi)}. \tag{13}$$

The distance for the deepest point of the failure surface, y, which is under the tip of penetrometer, is calculated by the following equations:

$$y = r \cos\theta = b e^{\theta \tan\theta} \cos\theta, \tag{14}$$

$$\frac{dy}{d\theta} = 0 \Rightarrow \theta = \phi. \tag{15}$$

The failure surface calculated by Eq. (15) for internal friction angles of 25 to 40 is illustrated in Figure 5(a). Overall, a penetrometer penetrates soil layers with internal friction angle of 25 to 35 degrees much easier. In some types of soil, the height of failure zone is in $4b$ to $9b$, and the failure depth is between $1.1b$ to $1.5b$ and the maximum horizontal width in failure zone is $2b$ to $5b$ (b is penetrometer's diameter).

For $\phi = 30$ and $\theta = 180$, r_c is equal to $6b$ and the total height of the failure zone is about $7.5b$.

This research is based on simultaneous combination and analysis of bearing capacity theory and shearing stress equation while failure. Furthermore, in bearing capacity theory, effective bearing capacity (q_E= "effective" cone resistance) is used instead of total bearing capacity (q_t), and shearing stress is attributed to sleeve resistance (f_s), so all outputs of CPT and CPTu (q_c, f_s and u) are used.

4. Proposed approach

The method under consideration is predicated on the hypothesis that soil shear strength parameters, C and ϕ, can be calculated by the information resulted from CPT and CPTu, q_t ($q_E = q_t - u_2$), f_s and u, as:

$$\begin{cases} q_{\text{ult}} = q_E = CN_c + \bar{q}N_q + 0.5\gamma BN_\gamma \\ f_s = C + \sigma'_{hc} \tan\delta \end{cases} \tag{16}$$

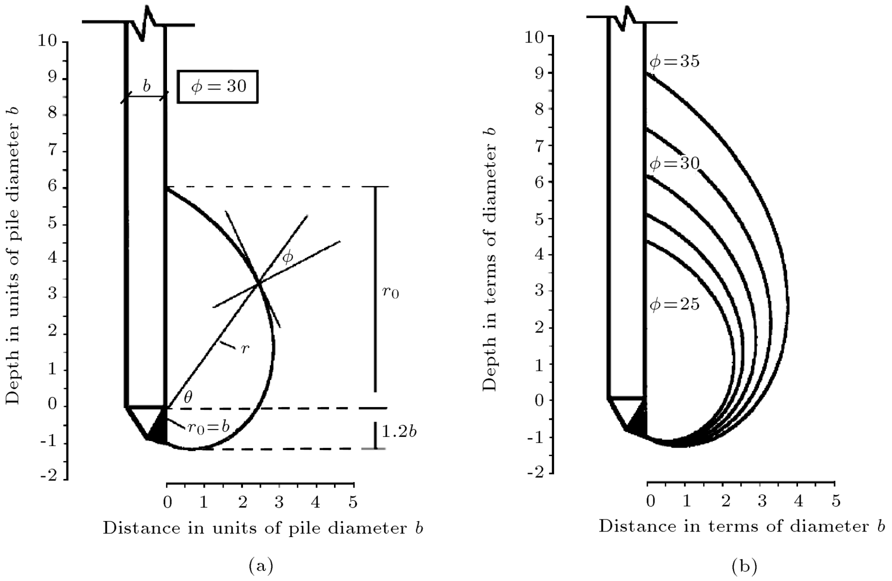

Figure 5. (a) Logarithmic spiral failure mode around the cone tip. (b) Failure surface for different ϕ angles [20].

For calculation of drained shear strength parameters, q_E can be used instead of q_c. Also, the last part of Eq. (16) is negligible due to small size of cone diameter, i.e. 35.7 mm, and based on Jamiolkowski and Robertson's suggestion (1988) [21]:

$$\frac{\sigma'_{hc}}{\sigma'_{h0}} = \frac{K_{\text{CPT}}}{K_0} = 0.000789 \left[\frac{q_c - \sigma_{\text{mean}}}{\sigma'_{\text{mean}}}\right]^{1.44}, \tag{17}$$

where σ_{mean} and σ'_{mean} are the vertical total and effective stresses, respectively.

The lateral stress increases by increasing the relative density. Usually, in calculation, it is assumed that the lateral stress value is equal to resistant horizontal stress by acceptable accuracy as follows:

$$\sigma'_{hc} = \left(\frac{1+\sin\phi}{1-\sin\phi}\right)\sigma'_{v0}\tan\left(\frac{\pi}{4}+\frac{\phi}{2}\right), \tag{18}$$

$$\sigma'_{h0} = k_0.\sigma'_{v0} = (1-\sin\phi)\sigma'_{v0},$$

$$\sigma'_{v0} = q, \qquad K_0 = 1-\sin\phi, \tag{19}$$

$$\sigma_{\text{mean}} = \left(\frac{\sigma_v + 2\sigma_h}{3}\right) = \left(\frac{\sigma_v(1+2k_0)}{3}\right), \tag{20}$$

$$\sigma'_{\text{mean}} = \frac{\sigma'_v + 2\sigma'_h}{3} = \frac{\sigma'_v(1+2k_0)}{3}. \tag{21}$$

δ is the friction angle between soil and penetrometer, about 0.3 to 0.7ϕ for sand, and increases by increase in soil relative density. The $\delta(\delta = (2/3)\Phi)$ values are between (0.3 to 0.6)ϕ. High δ values refer to stiff preconsolidated clays, while low δ values are for normally consolidated and soft clays.

q_E and q_c are very different in clay and fine gained soils, because of the presence of significant pore pressure. Senneset and Janbu (1985) [22] demonstrated that drainage is impossible in these soils, owing to the small time of penetrometer penetration. Thus, CPTu tests in clay are related to undrained conditions and, in turn, undrained bearing capacity. According to Senneset and Janbu (1985) [22], $N_u\Delta u_t$ should be subtracted from bearing capacity if effective shear strength parameters have been used. N_u is empirically chosen and depends on the friction angle.

Thus, the relation between bearing capacity and shear stress is determined as follow:

$$\begin{cases} q_E = CN_c + \bar{q}N_q + 0.5\gamma BN_\gamma - N_u\Delta u \\ f_s = C + \sigma'_{hc}\tan\delta \end{cases} \tag{22}$$

Based on the theory of bearing capacity, stiffness parameters and Possion's ratio should be considered for determining of the soil shear strength parameters as follows:

$$\sigma'_{hc} = \sigma_{h0} + [Su(1+\ln I_r)], \qquad q_E = q_t - u_2, \tag{23}$$

where I_r is rigidity index.

The relations between the above parameters can be summarized as follows:

$$I_{rr} = \frac{I_r}{1} + I_r\Delta, \tag{24}$$

where:

I_{rr} : reduced rigidity index;

Δ : is volume strain.

where:

$$\Delta = 0.005(1-\phi_{\text{rel}})\left(\frac{q_0}{P_a}\right), \tag{25}$$

and:

$$\phi_{\text{rel}} = \frac{\phi - 25}{45 - 25}, \tag{26}$$

where ϕ_{rel} is relative friction angle.

$$I_r = \frac{E_d}{2(1 + 2\vartheta_d)\bar{q}_0} \tan\theta, \tag{27}$$

where:

E_d : Drained elastic modulus;

ϑ_d : Drained Possion's ratio, $\vartheta_d = 0.1 + 0.3\phi_{\text{rel}}$;

$\bar{q}_0$: Effective vertical stress;

θ : The angle between the radius and r_0 in log spiral failure surface.

Studies on N_u reveal that this factor can be estimated from the following relation [22]:

$$N_u = 6\tan\phi'(1 + \tan\phi'). \tag{28}$$

Therefore, in two equations, based on q_c, f_s and u data, all soils are attributed for C and ϕ calculations:

$$\begin{cases} f_s = C + \sigma'_{hc}.\tan\left(\frac{2}{3}\phi\right) \\ q_E = N_c.C + \bar{q}.N_q + 0.5\gamma B N_\gamma - N_u \Delta U \end{cases} \tag{29}$$

$$N_q = \tan^2\left(45 + \frac{\phi}{2}\right) e^{\pi \tan\phi},$$

$$N_c = (N_q - 1)\cot\phi, \qquad N_\gamma = 2(N_q + 1)\tan\phi. \tag{30}$$

By replacing N_q, N_γ and N_c, based on ϕ angle, the followings can be derived:

$$N_q.C.\cot\phi + \bar{q}N_q + \gamma B N_q \tan\phi + \gamma B \tan\phi$$
$$= N_u \Delta U + q_E, \tag{31}$$

$$\tan^2\left(45 + \frac{\phi}{2}\right) e^{\pi\tan\phi}[C.\tan\phi + \bar{q} + \gamma B\tan\phi]$$
$$+ \gamma B \tan\phi = N_u\Delta U + q_E, \tag{32}$$

$$\left[(C + \gamma B)\tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) e^{\pi\tan\phi} + \gamma B\right]\tan\phi$$
$$+ \bar{q}\times\tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) e^{\pi\tan\phi} = N_u\Delta U - q_E. \tag{33}$$

This set of nonlinear equations cannot be solved by using regular methods, such as Newton-Roffson or Taylor expansion. So, it is necessary to use commonly used software.

In this case, Maple [23], version 14, was chosen to solve the problem.

$$C + 0.000789(1 - \sin\phi)\sigma'_{v_0}\tan\left(\frac{2}{3}\phi\right)$$
$$\left[\frac{q_c - \left(\frac{\sigma_{v_0} - 2\sigma_{h_0}}{3}\right)}{\left(\frac{\sigma'_{v_0} - 2\sigma'_{h_0}}{3}\right)}\right]^{1.44} = f_s, \tag{34a}$$

$$\left(\tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) e^{\pi\tan\phi} - 1\right) C\cot\phi$$
$$+ \bar{q}.\tan^2\left(\frac{\pi}{4} + \frac{\pi}{2}\right) e^{\pi\tan\phi}$$
$$+ \gamma B\left[\tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) e^{\pi\tan\phi} + 1\right]\tan\phi$$
$$= q_E + N_u\Delta U. \tag{34b}$$

5. Geotechnical records

A database of CPT and CPTu results were compiled and analyzed to verify accuracy of the proposed approach in comparison to other existing relations for C and ϕ predictions. Laboratory and in situ test results from the following case histories are gathered:

Site No. 1: Evanston, IL site [24];

Site No. 2: Vancouver, BC site [25];

Site No. 3: Savannah site [26];

Site No. 4: Babolsar located in Southern Caspian Sea, North of Iran [27];

Site No. 5: Fereidoonkenar Harbour located in Southern Caspian Sea, North of Iran [28].

Site No. 1 is located in North Western University, Evanston, Illinois, USA [24]. CPTu data were obtained in a soil profile consisting of 7 m of sand, deposited on normally consolidated silty clay. The piezometer was attached to the cone face (u_1) and not behind the shoulder (u_2= pore pressure measured at cone shoulder). The method of converting the pore pressure measurement to the u_2-value presented by Finno (1989) [24] has been accepted here. CPTu data in Evanston site is shown in Figure 6.

Site No. 2 is along the shore of Fraser River, Vancouver, British Columbia, Canada. There is a 20 meters thick mixed soil profile of deltaic deposits of clay, silt and sand. In Figure 7, CPTu test results are illustrated.

Site No. 3, Savannah in the U.S. soil deposits, consists of clay till to the depth of 12 m, a thin layer (2 m) of silty sand, and then the clay layer extended to

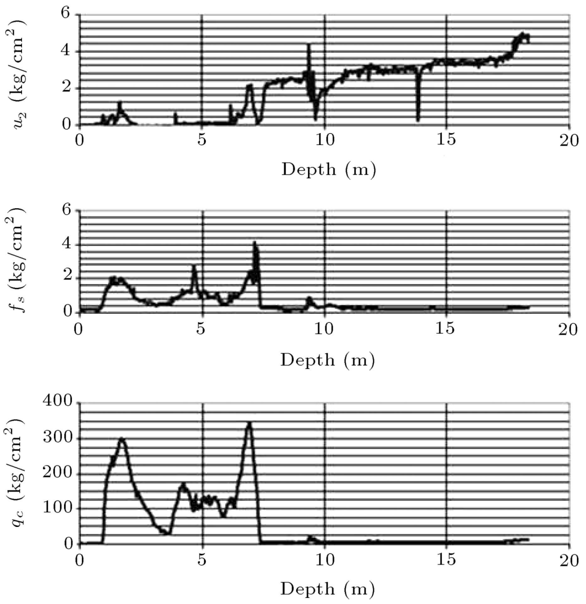

Figure 6. CPTu records in Evanston site [24].

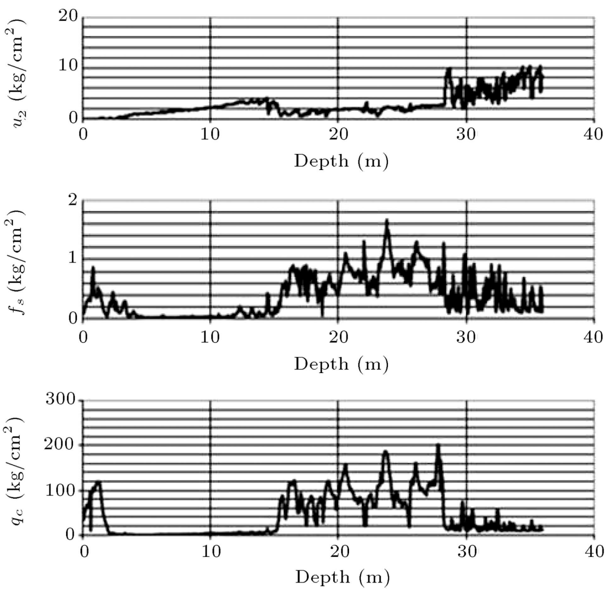

Figure 7. CPTu records in Vancouver site [25].

the depth of 17 m. Deeper soil layer is sandy silt and clayey sand, as shown in Figure 8.

Site No. 4, Babolsar, is located in Southern Caspian Sea Shore, North of Iran. Investigations show that the first two meters of soil is clay and to depth of 26 m is the mixture of sand and silty sand deposits. There are thin layers of clay and silt in the depths of 10 and 16 m. CPTu test results in Babolsar site are shown in Figure 9.

Site No. 5, Fereidoonkenar Harbour, is located in Southern Caspian Sea, North of Iran. Studies indicate that there is a thin layer of clay and silt in depths of 8 m. Other layers include a range of coarse grained to silty sand deposits, as presented in Figure 10.

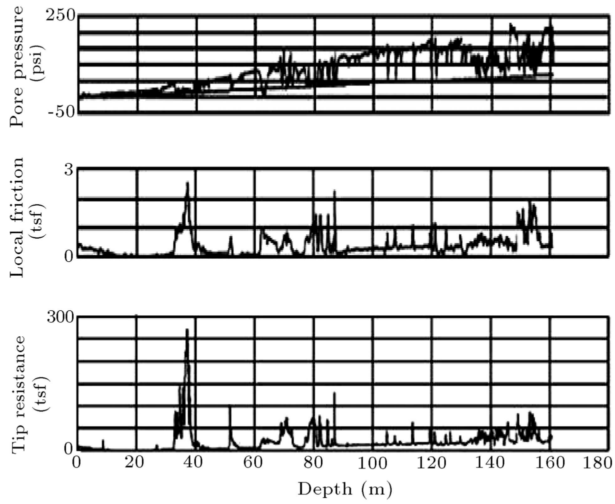

Figure 8. CPTu records in Savannah site [26].

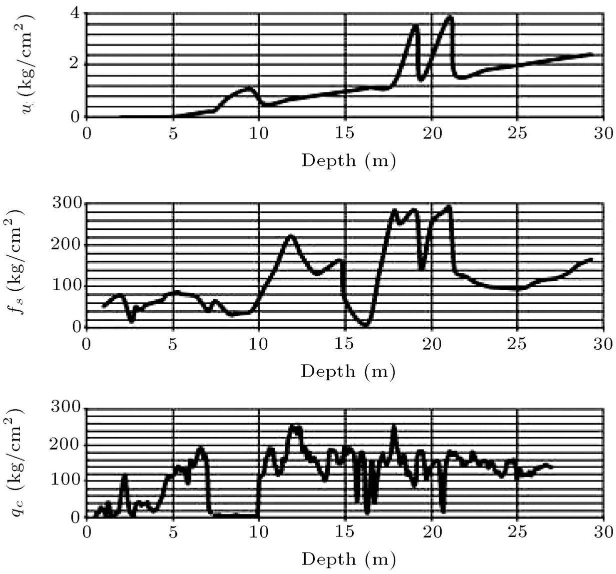

Figure 9. CPTu records in Babolsar Caspian sea site, Iran [27].

6. Validation of methods and discussion

Table 1 summarizes the results of C and ϕ parameters from CPT and CPTu data for 5 sites and 32 actual tests for the proposed approach and the correlations suggested by the followings:

- Muromachi, 1972 [4];
- Meyerhof, 1974 [9];
- Schmertmann, 1978 [10];
- Mitchell and Durgunoglu, 1983 [11];
- Robertson and Campanella, 1988 [5];
- Kulhawy and Mayne, 2003 [16].

Experimental equations to estimate shear strength parameters from CPT data are somehow conservative,

Table 1. Results of different methods for determining of effective shearing strength parameters for the proposed sites.

No.	Description	Soil internal friction angle limits ϕ (deg)								Cohesion limits C' (kg/cm^2)	
		Ref. [5]	Ref. [4]	Ref. [16]	Ref. [10]	Ref. [11]	Ref. [9]	Lab. test results	Proposed method	Lab. test results	Proposed method
1	I, Evanston	50.8	44.2	47.1	43	-	42	42	44.5	1.60	1.68
2	I, Evanston	37.4	33.87	35.7	34.75	36.7	33	31.5	33	0.4	0.47
3	I, Evanston	46.2	44	45	43	45.7	43.7	43	45	1.2	1.26
4	I, Evanston	-	-	-	-	-	-	-	26	0.1	0.12
5	I, Evanston	-	-	-	-	-	-	-	14	0.4	0.44
6	I, Evanston	-	-	-	-	-	-	-	25.5	0.07	0.08
7	I, Evanston	-	-	-	-	-	-	-	23.3	0.02	0.02
8	II, Vancouver	-	-	-	-	-	-	9-13	11.5	0.09	0.13
9	II, Vancouver	-	-	-	-	-	-	17	18	0.43	0.88
10	II, Vancouver	-	-	-	-	-	-	11	14	0.21-0.46	0.57
11	II, Vancouver	-	-	-	-	-	-	17	18	2.34	2.71
12	II, Vancouver	36.7	32	38	36.1	37.6	37.5	24-27	26	2.10	2.37
13	III, Savannah	53	44.5	46.15	43	42.6	37.8	42	43	0.27	0.32
14	III, Savannah	47.7	43.2	42	43	47	36	38	40	0.17	0.2
15	III, Savannah	-	-	-	-	-	-	-	10	0.15	0.18
16	III, Savannah	-	-	-	-	-	-	-	7	0.15	0.18
17	III, Savannah	40.6	39.8	38.3	38.5	40.5	36.7	39.5	40.5	0.32	0.35
18	III, Savannah	39	34.5	37.2	37	38.7	34.7	29	30	0.8	0.89
19	III, Savannah	44.2	43.1	42.2	41.5	44	38.5	39	41	1.34	1.41
20	III, Savannah	44.2	42	42.6	41.8	43.7	39	39	40.5	0.8	0.85
21	IV, Babolsar	-	-	-	-	-	-	-	25	0.4	0.43
22	IV, Babolsar	40.43	32.17	39.46	38.05	41.2	37	31	33	1.04	1.12
23	IV, Babolsar	-	-	-	-	-	-	-	12	0.1	0.12
24	IV, Babolsar	-	-	-	-	-	-	-	34	2.28	2.36
25	IV, Babolsar	38.47	38.8	39.5	37.75	40.5	36	34	35	1.8	1.84
26	IV, Babolsar	35.5	34.77	36.76	34	35	32	32	33	1.4	1.42
27	V, Fereidoonkenar	43.4	34.1	39.67	40	44	39	32	34	0.7	0.75
28	V, Fereidoonkenar	32.5	33.58	33.37	31	33	-	31	32	0.35	0.38
29	V, Fereidoonkenar	40.2	37.3	41	40.3	40.8	38.5	34	35	0.9	0.92
30	V, Fereidoonkenar	-	-	-	-	-	-	-	23	3.3	3.34
31	V, Fereidoonkenar	36.4	35.65	38.72	33.6	37.6	34	32	33	0.6	0.69
32	V, Fereidoonkenar	37.7	36.72	40	38	38.1	35	31	32.5	1.5	1.56

and values from these methods are generally lower than those obtained from the recently developed methods, such as Robertson and Campanella (1988) [5] and Chen and Juang (1996) [6].

Test series 4 to 7, 15, 16, 21, 23, 24 and 30, as indicated in Table 1, are presented for clayey soils, while others evaluate sands. Investigation of sandy soils implies that the values generated for angle of internal friction, using the proposed method, is slightly lower than Robertson and Campanella (1988) [5], Mitchell and Durgunoglu (1983) [11], Kulhawy and Mayne (2003) [16], Muromachi (1972) [4] and Schmertmann (1978) [10] methods, while they match the Meyerhof (1974) [9] suggestion.

Also, comparison of laboratory results and methods, based on CPT and CPTu data, manifests that all methods including this study predict higher values of soil parameters. A reason for such differences can

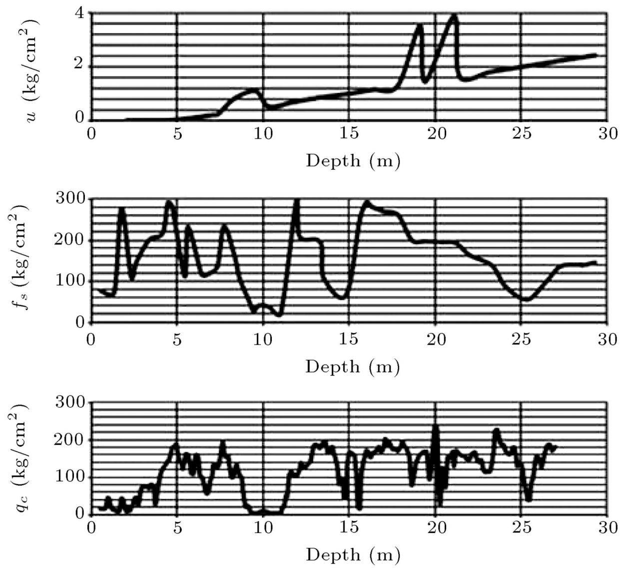

Figure 10. CPTu records in Fereidoonkenar Caspian sea site, Iran [28].

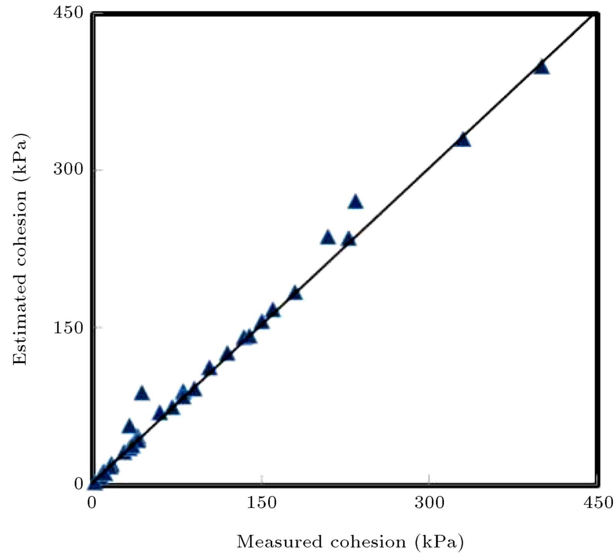

Figure 11. Comparison between the measured cohesion in laboratory and estimated cohesion by the proposed method.

be related to the failure rate of soil mass. Owing to the higher rate of soil disruption for CPT in comparison to experimental results, strength parameters obtain higher values for penetrometer. This can be considered in two ways. First, a few relations have been developed using mechanical cone data, which involve somewhat lower accuracy than electrical one. Also, some equations are empirical, and mainly offer conservative values. Second, in other methods, for the failure strength pattern, only internal friction was considered, and the cohesion is neglected in such cases. Therefore, the values generated for angle of friction, using current methods, are mainly higher. Besides, they give no explanation about the cohesion factor of soils.

A significant feature of the method proposed herein is that it produces drained shear strength parameters, while according to the literature, no extended research has been conducted on this subject, and no equation has ever been proposed. In addition, Robertson and Campanella (1988) [5], Mitchell and Durgunoglu (1983) [11], Kulhawy and Mayne (2003) [16], Muromachi (1972) [4], Schmertmann (1978) [10] and Meyerhof (1974) [9] methods are not applicable on clayey or mixed soils, whereas the proposed method is applicable in this area.

In this study, all of the outputs of CPTu tests have been used, therefore, the possibility of the wrong records is minimized. Current methods are just on the basis of one output, q_c, while the errors may sometimes be in the value of q_c. The outputs of prediction for C parameter and laboratory test results are compared graphically in Figure 11. Also, the results of the proposed method relatively converge on the bisector line, as illustrated in this figure, which implies that the proposed method is able to predict strength parameters of clayey soils, while current methods do not suggest any relations for determining drained shear strength parameters in clayey soils.

Comparisons between the proposed and currently used relations are shown in Figure 12 for calculated ϕ angle. Determining the value of internal friction angle by means of proposed and suggested relations indicates that Meyerhof (1974) [9] and the proposed method are almost predicted in the same values, while the others such as Robertson and Campanella (1988) [5], Mitchell and Durgunoglu (1983) [11], Kulhawy and Mayne (2003) [16], Muromachi (1972) [9] and Schmertmann (1978) [10] involve the trend of overestimation.

7. Conclusions

Most of the current methods for determining the soil shear strength parameters from CPT and CPTu data are presented and discussed. These methods can be applied in sands, silts and are only able to estimate the friction angle of soil, regardless of their cohesion.

In the proposed method, both shear strength parameters can be determined simultaneously from all CPT or CPTu data. According to the fundamental equations for determining the bearing capacity and using the CPTu data, two sets of equations and unknown parameters for all soils can be derived for those inputs which are q_c, f_s and u. For solving these sets of nonlinear equations, Maple software was used.

Laboratory test results, the proposed new approach and existing correlations for prediction of C and ϕ angle parameters with practical results obtained from five sites were implemented in 32 case records from

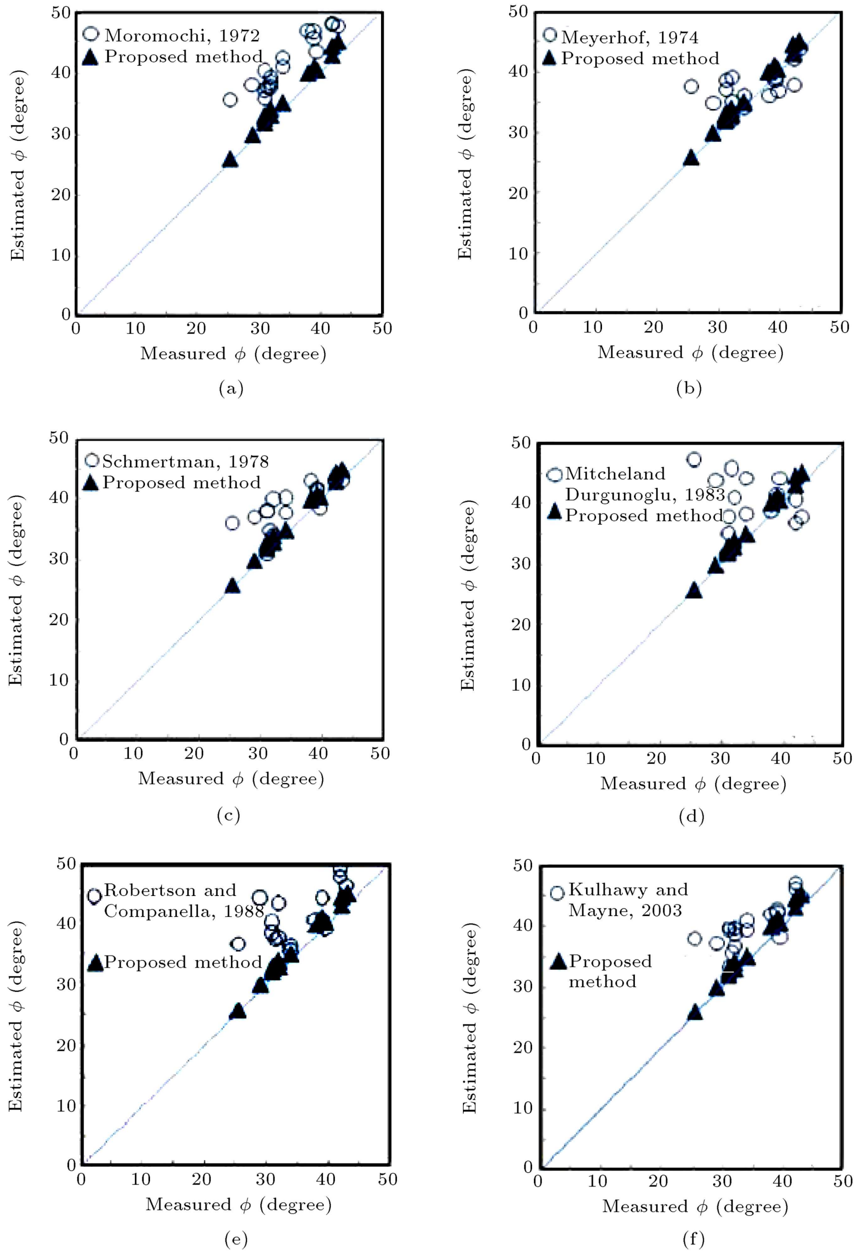

Figure 12. Comparison between estimated and measured values for friction angle.

the same sites. The results indicate that the proposed method and existing relations fit well together. This new method led to improved accuracy by eliminating incorrect registration records due to the simultaneous application of three quantity output including q_c, f_s and u from CPT or CPTu data.

The predicted C and ϕ values by the proposed approach in comparison to the measured results from laboratory tests show more consistency than the currently used equations suggested by researchers such as Robertson and Campanella (1988) [5], Mitchell and Durgunoglu (1983) [11], Kulhawy and Mayne (2003) [16], Muromachi (1972) [4], Schmertmann (1978) [10] and Meyerhof (1974) [9]. In fact, in these methods, cohesive parameters have not been considered in bearing capacity equation. Hence, it causes the failure loads be attributed to second part of equation, that is the function of internal friction angle. Therefore, the internal friction angle, which is obtained by current methods, is almost higher than the measured ones. Moreover, these methods do not consider the mixed soils in which cohesion parameter is partly important as a component of shearing resistance. This shortcoming is covered and compensated by the proposed method. Due to good consistency and less scatter for the soil parameters from the proposed approach, it can be considered in optimized geotechnical design and practice.

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