Analytical approach for determining soil shear strength parameters from CPT and CPTu data

Approche analytique pour déterminer la résistance au cisaillement d'un sol à partir d'essai CPT et CPTu

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ABSTRACT: The common approaches for soil strength parameters determination from CPT data are on the basis of bearing capacity and cavity expansion theories. A new method is proposed for C, ϕ prediction using all quantities, q_c, u and f_s from CPTu considering bearing capacity mechanism of failure at cone tip and direct shear failure along penetrometer sleeve. One advantage of this method is improvement the accuracy in the case of erroneous data by using all three output of CPTu. Laboratory test results, the two sets of nonlinear equations by the proposed approach and existing correlations of C and ϕ angle parameters have been compared applying on a data base compiled from four sources. It has been considered that the internal friction angle which is obtained by current methods is almost relatively higher than the measured values. Also, the comparison indicates good consistency and low scatter for the proposed method.

RÉSUMÉ: Les approches communes pour les paramètres de résistance des sols, déterminés par CPT, sont basées sur la capacité de cisaillement et les théories d'expansion des cavités. Une nouvelle méthode a été proposée pour C, ϕ et utilise toutes les quantités de prévision, q_c, u et f_s de CPTu, en considérant la capacité de cisaillement et le mécanisme de failure dans type paramide et failure cisaillement direct, le long du pénétromètre manchon (sleeve). Une des avantages de cette méthode est d'améliorer exactitude dans le cas des donnés fausse utilisation, tout les trois sortie de CPTu. Les résultats des essais du laboratoire, les deux combinaisons des équations non linéaires, l'approche proposée et les corrélations existantes de c et l'angle de ϕ est comparée appliquée sur quatre bases de données. On considère que la friction interne obtenue par la méthode courante est toujours relativement plus grande que la valeur mesurée, aussi la comparaison montre la bonne consistance et le bas scatter pour la méthode proposée.

KEYWORDS: Soil shear strength, Cohesion and friction parameters, CPT and CPTu data, Bearing capacity theory

1 INTRODUCTION

Geotechnical investigation by CPTu provide continuous vertical profile of cone tip resistance (qc), sleeve friction (f_s) and pore water pressure (u_2) in every inch of the subsoil depth (Lunne et. al, 1997). The CPTu test is used in soft to medium deposits, and not applicable in cemented sand, hard clay and gravelly strata. The penetrometer is a useful tool to identify of thin layers where the traditional sampling procedures cannot be employed. Also, using the CPTu test may distinct the liquefiable or collapsible soil layers around 50 mm thickness in depth (Tavenas and Leroueil, 1987), (Eslami and Fellenius, 2004).

In alluvial soils containing gas, determining undrained shear strength by traditional sampling procedures and using UU triaxial tests may lead to conservative results. In granular soils, determining the friction angle (ϕ) as one of the major soil strength parameters by using direct shear or triaxial tests involves uncertainties due to sampling difficulties, confining pressure simulation and limitations of size effects (Mitchell and Durgunoglu, 1983). The main advantage of CPTu versus other in situ test procedures is the relatively elimination of undisturbed sampling, performance in real condition regarding stress level and geological aspects. Furthermore, by using the continuous data in one inch interval of depth, shear strength parameters (C,ϕ), can be obtained which have significant role in geotechnical designs.

2 SHEAR STRENGTH PARAMETERS BY CPTU DATA

Two main theories have been implemented for the estimation of shear strength parameters by using CPT and CPTu results; bearing capacity (Janbu and Senneset, 1974), (Durgunoglu, 1975) and cavity expansion (Vesic, 1972) approaches. The methods which are based on bearing capacity theories; for penetrometer penetration mechanism, it is assumed that cone tip resistance (q_c) is equivalent with ultimate load of a deep circular foundation in subsoil and leads the soil mass to be failed. Whereas, failure assumption in cavity expansion theory is based on required pressure for forming of deep hole in an elastic-plastic environment which is fitted with the pressure needed for creation and cavity expansion in the same volume under identical conditions. So far, Muromachi, 1972, Schmertmann, 1978, Mitchell and durgunoglu, 1983, Robertson and Campanella, 1988, Kulhawy and Mayne, 1990 have studied on determination of shear strength parameters from CPT and CPTu data which solely have presented S_u in fine grained or ϕ in granular soils.

3 ANALYTICAL MODEL FOR C AND ϕ BY CPTu DATA

By applying two basic equations on determination of the deep foundation bearing capacity, one for tip and other for penetrometer sleeve, using the effective bearing capacity instead of total stress approach and extension of the relationships, a dual equation system with two unknowns, can be achieved as below under static loading conditions.

$$\begin{cases} CN_C + qN_q + 0.5\,\gamma BN_\gamma = q_E = q_t - u_2 \\ C + \sigma'_{hc}\tan\left(\frac{2}{3}\phi\right) = f_s \end{cases} \tag{1}$$

Considering deep bearing capacity factors proposed by (Junbu, 1974 base failure model) and applying the analytical Eslami and Fellenius, (1997) model based on CPTu results, the relations can be summarized as follows:

$$N_q = (\tan\phi + \sqrt{1+\tan^2\phi})^2.\exp(2\xi\tan\phi) = \frac{(\sin\phi + 1)^2}{\cos\phi}.\exp(2\xi\tan\phi) \quad (2)$$
$$N_c = \left(N_q - 1\right)\cot\phi\ ,\quad N_\gamma = 2\left(N_q + 1\right)\tan\phi$$

Eq. 3 is expressed according to empirical results for ξ at $\frac{q_t}{p_a}$. Also, N_q can be achieved from Eq. 4 which is shown as below:

$$\xi = 3.05 * 10^{-3}\left(\frac{q_t}{p_a}\right) + 1.2 \quad (3)$$

$$N_q = \frac{(\sin\phi + 1)^2}{\cos\phi}.\exp[(6.1 * 10^{-3}(\frac{q_t}{p_a}) + 2.4)\tan\phi] \quad (4)$$

Jamiolkowski and Robertson, 1988 presented a correlation for σ'_{hc} as function of σ_{h0} and mean in situ stresses:

$$\frac{\sigma'_{hc}}{\sigma'_{h0}} = 7.89 * 10^{-4}[\frac{q_t - \sigma_{mean}}{\sigma'_{mean}}]^{1.44} \quad (5)$$

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:

$$\sigma'_{hc} = \left(\frac{1+\sin\phi}{1-\sin\phi}\right)\sigma'_{v0}\ \tan\left(\frac{\pi}{4}+\frac{\phi}{2}\right) \quad (6)$$

$$\sigma'_{h0} = k_0.\sigma'_{v0} = (1-\sin\phi)\sigma'_{v0} \quad ,\sigma'_{v0} = q \quad (7)$$

$$\sigma_{mean} = \frac{\sigma_v + 2\sigma_h}{3} = \frac{\sigma_v(1+2k_0)}{3} \quad (8)$$

$$\sigma'_{meam} = \frac{\sigma'_v + 2\sigma'_h}{3} = \frac{\sigma'_v(1+2k_0)}{3} \quad (9)$$

By substitution Eqs. 2 to 9 in two basic Eq.1 can be achieved two sets of equation.10 as follow:

$$\begin{cases} u_2 + \gamma B\tan\phi + qN_q + \gamma BN_q\tan^2\phi + C\left(\frac{N_q - 1}{\tan\phi}\right) = q_t \\ C + 7.89 * 10^{-4}(1-\sin\phi)\,\sigma'_{v0}\tan(\frac{2}{3}\phi)[\frac{q_t - (\frac{\sigma_v - 2\sigma_h}{3})}{(\frac{\sigma'_v - 2\sigma'_h}{3})}]^{1.44} = \end{cases} \quad (10)$$

4 EXPERIMENTAL RECORDS FOR EVALUATION

Geotechnical properties and information including experimental results from the data base of four sites have been compiled. These records are containing 25 series of CPT and CPTu data and shear strength parameters measured by laboratory tests which are used for evaluating developed model. The site specifications are briefly reviewed as follows:

Site No. 1, Narenjestan tourism complex, (Mandro Co., 2012); site is located in southern bank of Caspian Sea in Mazandaran Province, Iran. According to borehole operations results, observation and field tests from ground level silty sand with medium dense deposits is located to the depth of 7.5 m. Following the depth of 7.5 m the firm silt layer with high plasticity exist with thickness of 2 m. From depth of 9.5 m down to end of boring poorly graded, silty sand and sand are located with of dense condition and classified as an SM, SP.

Site No. 2, Narges Hotel complex, (Sham-e Co., 2012); is located in southern Caspian Sea Shore in the suburb of Sari city in Iran. The observation of three boreholes by rotary drilling indicate that the superficial soil layer consists of alluvial gray sea sand with some silt which exists to the depth of 11m. According to USCS this layer is classified as SP, SM or SP-SM. Between depth of 10m to 14m fine clay and silt layer are located in dirty green color with the thickness of 1m to 4m which is classified as CL. The bottom layer is containing fine sea alluvial sand which is observed in depth of 11m to 30m and is classified as SM. Also, the ground water level is located below 3m of ground surface. For determining soil shear strength parameters of filed soil stratification, direct shear, triaxial and uniaxial tests are accomplished on samples. Also, according to SPT records in subsurface depths around 10m, the N values are ranged from 22 to 35, which represent medium to dense relative density for upper layer. The N values in depth of 10m to 14m and 14m to 30m vary from 12 to 25 and 22 to 45, respectively, and classified as dense to high dense coarse grained deposit. The CPTu profile in Sari Narges Hotel site is shown in Fig. 1.

Site No. 3, East Changi, (Choa et al. 2004); site is a recovery site which is located in eastern costal of Changi Airport in Singapore. From geotechnical investigations, it is observed that the geomaterial is a kind of soft to medium clay.

Site No. 4, University of Texas which is known as A&M Site, (Briaud and Gibbens, 1994). It is one of the international site of study in geotechnical basis and is located in Texas Province, USA. Soil deposits are formed of silty sand.

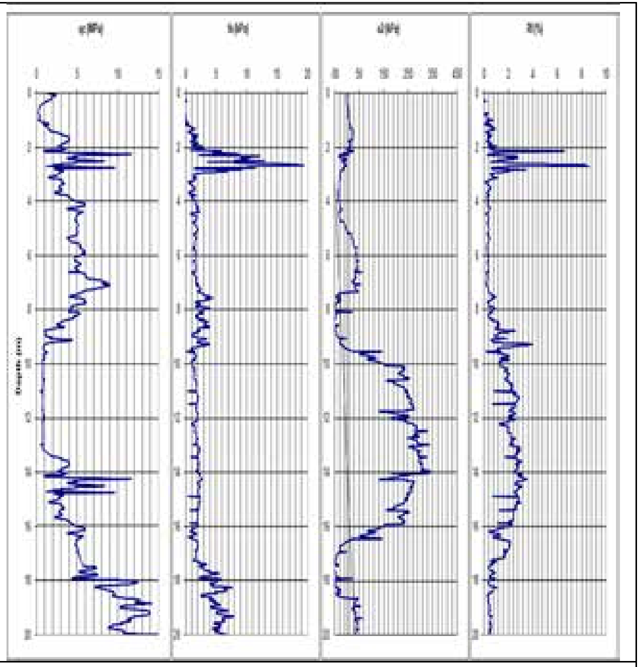

Figure 1. CPTu profiles in Narges Hotel Complex, (Sham-e Co., 2012)

The accumulated results of analytical procedure in 25 cases and also laboratory test results are presented in Table 1.

Table 1. Shear strength parameters according to proposed method and laboratory test results for 25 measure cases

				C (kPa)		ϕ	
Site No.	Soil type	q_E (MPa)	f_s (kPa)	Lab test	proposed	Lab test	Proposed
I	SM	30	22	4	3.7	30	32
I	SP	13	65	4.5	5	30	31
I	SM	11	50	4	3.8	31	32
I	MH	13	40	50	49	4	6
I	SP	5	55	4	5	33	31
I	SM	22	110	4	3.5	33	31
I	SM	40	150	4	4.5	35	36
I	SM	28	140	4	5	35	37
I	SM	30	135	6	5.2	34	36
I	SM	18	60	6	8.4	38	37
п	SM	6	75	3	2.5	32	31
п	SM	5	30	0	1	32	32
п	SM	7	80	6	6.6	31	32

II	CL	4	70	29	30	2	2.5	
II	SM	4	90	58	57	2	2.7	
III	CL	2	14	30	29	12	12	
III	ML	2	55	28	29	14	15	
III	SM	2	27	5	6.1	16	16	
III	CH	2	56	57	56	5	6	
III	CL	2	78	35	36	8	9	
IV	SM	1	6	0	1	33	34	
IV	SM	7	30	0	1	36	38	
IV	SM	8	60	0	1.5	32	33	
IV	SM	6	38	1	1.5	-	8	
IV	SM	9	75	9	8.4	-	10	

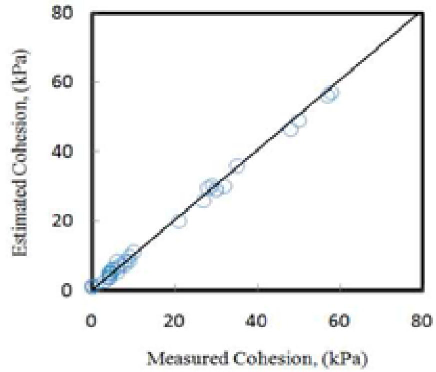

Fig. 2. Comparison between the measured cohesion in laboratory and estimated cohesion by proposed method

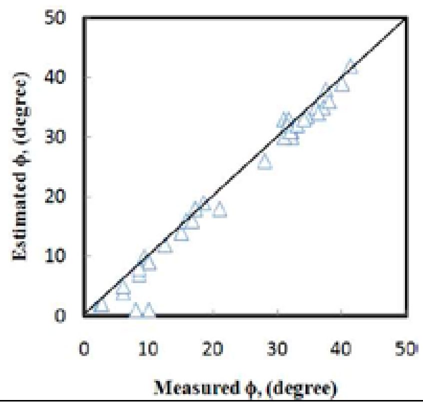

Fig. 3. Comparison between estimated and measured values for friction angle

5 VALIDATION OF RESULTS AND DISCUSIONS

The accomplished geotechnical study in each site has been upon borehole excavations. The samples dependent on soil stratification and from different depths are taken as distributed and undistributed specimens. For determining the shear strength parameters, direct shear, uniaxial and triaxial tests are done on samples in laboratory. Meanwhile, because of high quality of sampling in triaxial test and logicality of the test results in laboratory, it can be more adequate. Four practical cases include CPT and CPTu test results associated with laboratory test results and SPT records are used for evaluating the proposed analytical relations.

The measurement results by laboratory tests and also, prediction by using analytical procedure, are presented in Table 1. Evaluation of results expressed the fact that the suggested procedures not only can spontaneously predict and determine both shear strength parameters but also it contain acceptable and reasonable results. Fig. 2 is associated to evaluation and comparison between laboratory results and suggested analytical model for determining the cohesion parameter. The measured and predicted C values show good agreement which denotes the capability of analytical approach. Also, Fig. 3 shows the comparison between measured values and analytical procedure results for internal friction angle within the range of study in four sites. As for the laboratory results which are achieved from drained triaxial test and suggested analytical model, it is observed that the proposed analytical procedures based on CPT and CPTu in cases with cohesion and internal friction angle, almost has identical to laboratory results.

The laboratory results are compared with different presented procedures by researchers are shown in Fig. 4a to 4f. According to graphs, the achieved friction angle values by other procedures are always greater than the suggested analytical procedure values and laboratory results. Meanwhile, it is observed that the friction angle values from Meyerhof, (1974) results are closer to bisector line indicating close agreement between the predicted and measured values. Moreover, the presented analytical procedure and laboratory results have more coincidence and are closer to actual values. While, the values obtained from current methods, are more than the experimental results and analytical method.

The current procedures do not contain any recommendation for soil cohesion and it is one of the advantages for the proposed procedure. Also, it is not depending only one of the test outputs rather, the entire CPT and CPTu outputs such as q_c, f_s and u are used in equations, hence the error creation reaches to minimum value in inaccurate records, because of the simultaneous employment of each three output quantities, the other advantages in the presented analytical procedure contrary to traditional procedures. Furthermore, the shear strength parameters derived from actual subsurface failure mechanisms condition in cone tip and sleeve has been realized reasonably in proposed relations.

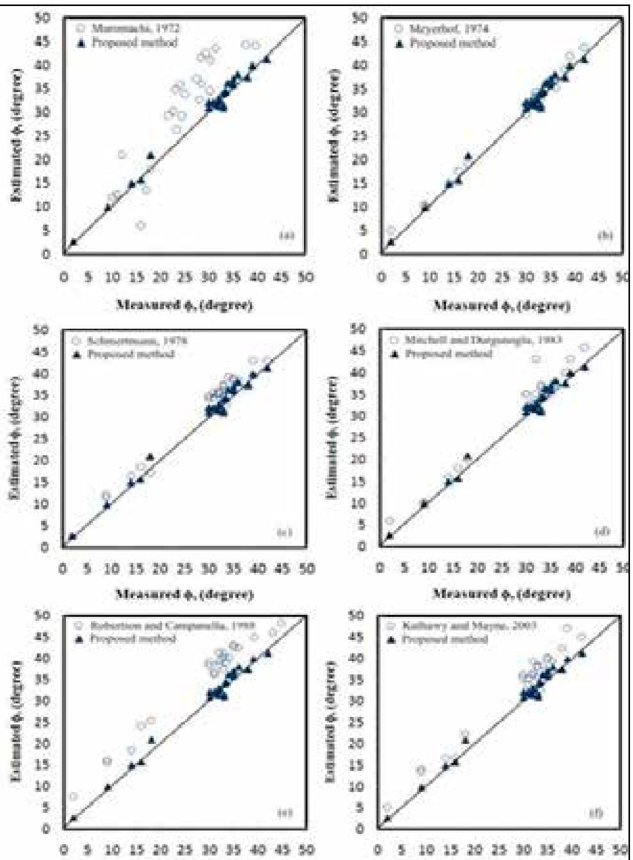

Fig. 4. Comparison between estimated and measured values for friction angle

6 CONCLUSIONS

Geotechnical study by CPT or CPTu can determine continuous vertical profile of cone tip resistance (q_c), sleeve friction (f_s) and pore water pressure (u) in every inch of the subsoil depth. Hence, the shear strength parameters can be well determined which have major role in geotechnical design.

In this study two main theories have been implemented for the estimation of shear strength parameters by using CPT i. e., bearing capacity in cone tip and direct mode of shear failure in along penetrometer jacket. So far, different researchers have studied on determination of shear strength parameters from CPT and CPTu data which solely have presented S_u in fine grained or ϕ angle in granular soils. The entire of CPTu data, q_c, f_s and u are used to calculate C and ϕ, via bearing capacity theory and shear stress relation at failure condition. By combining these relations and applying the proposed analytical Eslami and Fellenius, (1997) model based on CPTu results and direct shear failure along cone sleeve, the drained shear strength parameters values include cohesion and internal friction angle can be derived simultaneously.

In proposed procedure the error creation reaches to minimum value through inaccurate records, because of the simultaneous use of each three output quantities. The existence methods for determining the internal friction angle are rely on only one of the test outputs (depending only to q_c) while the inaccurate records creates more error in shear strength parameters. But, three parameters q_c, f_s and u are dependent on friction angle in presented procedure and lead to prorate the error cases. The current procedures do not contain any recommendation for soil cohesion and it is one of the advantages in the proposed procedure. The presented procedure differs from common procedure results by increasing fine grains in soil. Comparison with 25 data sets of C and ϕ from laboratory tests and predicted by the proposed method indicate good agreement and consistency.

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