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Bearing capacity analysis of shallow foundations from CPT data

Analysis de la capacité à porter de fondation superficielles en utilisant des résultats CPT

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ABSTRACT
Application of cone penetration test, CPT has been increased in foundation engineering due to supplying continuous and accurate soil profile in recent years. A static analytical model based on general shear failure mechanism of logarithm spiral has been developed for calculating directly bearing capacity of footings, q_{ult} from cone resistance q_c. The transform of failure mehanism from shallow to deep, foundation dimension and data processing has been considered in the proposed method. Six current CPT direct methods for determining bearing capacity of footings have been investigated. The proposed method and six others were compared to the measured capacity ranging from 1.7 to15 kg/cm^2 of 21 footings with range of diameter from 0.3 to 3m which compiled in a database. The statistical and cumulative probability approaches for validation of methods indicate optimistic results for proposed method.

RESUME
Dans les années dernières, l'application de l'essai de l'infiltration de cône, à cause de mesurage continu du profil de sol, a développé dans la branche de génie de fondation. On a présenté un modèle d'analyse afin de déterminer la capacité à porter la charge des fondations superficielles en utilisant des résultats CPT, en considération de mécanisme de la rupture de section de spirale logarithmique. On a fait attention à l'effet du déplacement de mécanisme superficiel à profond, les dimensions de fondation et la manière de traitement des données dans une méthode présentée et proposée. On a étudie six méthodes directes pour déterminer la capacité à porter le charge en exerçant des résultats CPT. La méthode proposée et six méthodes trouvées avec des données ont été rassemblées dans une banque d'information contenant 21 cas des résultats CPT accompagnant des résultats de l'essai à faire la charge des fondations superficielles en dimension 0.3–3 mètres dans le plan et avec la capacité à porter le charge définitif et final de sol sous-fondation comme suit 1.7 – 15 kg, tout a été estimé et comparé. La vérification des fautes relatives et la probabilité de rassemblage résultés ayant de bons résultats optimistes pour la nouvelle méthode lesquelles peuvent être considérables aux dessins géotechniques

1 INTRODUCTION

One of the main step for safe and economic design of foundations is ultimate bearing capacity determination. The maximum load that can be applied to subgrade soil from foundation with no occurance of shear failure and limiting settlement in anallowable upperbond to avoid serviseability damages of superstructure. Four approaches currently are used to determine the bearing capacity of shallow and deep foundations; static analysis, in-situ testing methods, full-scale loading tests and using presumed values recommended by codes and handbooks. Among these approaches, theoretical solution (static analysis) is more common and applied first. The other aproaches are realized as supplementary of static analysis.

In-Situ testings has shown an increase in recent years for geotechnical engineering. This is due to rapid development of in-situ testing instruments, improved understanding of soil behavior, and subsequent realization of some limitations and inadequacies of conventional laboratory testing.

The Cone Penetration Test, CPT, incontrast to other common in-situ tests is simple, fast, relatively economical, and it supplies continuous records with depth. The results are interpretable on both empirical and analytical basis, and a variety of sensors can be incorporated with cone penetrometer. Evaluating bearing capacity from CPT data is one of the earliest applications of this sounding and includes two main approaches: direct and indirect methods.

Direct CPT methods apply the measured values of cone bearing for toe resistance with some modifications regarding scale effects (influence of foundation width to the cone diameter ratio). Indirect CPT methods employ friction angle and undrained shear strength values estimated from CPT data based on bearing capacity and/or cavity expansion theories. The analogy of cone penetrometer and pile has been caused that research work in foundation engineering is focused on application of CPT data for deep foundations. In this paper, different methods of CPT data for determining bearing capacity of shallow foundations will be investigated. By utilizing a database including full scale loading test results of footings and adjacent CPT soundings, the capability of predictive methods has been compared.

2 DIRECT METHODS FOR DETERMINING BEARING CAPACITY OF FOOTINGS FROM CPT DATA

Direct CPT methods which initiated theoretically or empirically, relates the ultimate bearing capacity of soils, q_{ult} to the cone point resistanc, q_c with some modification factors. Among different methods, the followings are commonly used by geotechnical engineers:

Schmertmann (1978) proposed bearing capacity factors based on Terzaghi basic formula for non-cohesive soils from CPT data as follows:

$$q_{ult} = \overline{q}N_q + 0.5\gamma BN_\gamma \tag{1}$$

$$N_q = N_\gamma = 1.25\overline{q}_c \quad \text{and} \quad \overline{q_c} = \sqrt{q_{c1} \times q_{c2}} \tag{2}$$

where
q_{ult} = ultimate bearing capacity of footing
$\overline{q}$ = overburden stress at foundation base = γD_f

γ = effective density of soil around footing

B & D_f = width and depth of footing

N_q, N_γ = non-dimensional bearing capacity factors

q_{c1} = arithmetic average of q_c values in an interval between footing base and 0.5B beneath footing base .

q_{c2} = arithmetic average of q_c values in an interval between 0.5B to 1.5B beneath footing base.

Meyerhof (1976) suggested a direct method for estimating q_{ult} from cone resistance as follows:

$$q_{ult} = \overline{q}_c\left(\frac{B}{12.2}\right)\left(1+\frac{D_f}{B}\right) \quad (3)$$

$\overline{q}_c$ = arithmetic average of qc values in a zone including footing base and 1.5B beneath footing.

Factor of safety at least 3 is recommended by Meyerhof to obtain the allowable bearing pressure.

Owkati (1970) proposed separated equations for ultimate bearing capacity of sands as follows:

$$q_{ult} = 28 - 0.0052(300 - \overline{q}_c)^{1.5} \qquad \text{strip footings} \quad (4)$$

$$q_{ult} = 48 - 0.009(300 - \overline{q}_c)^{1.5} \qquad \text{square footings} \quad (5)$$

$\overline{q}_c$ = such as deffined by meyerhof, in terms of kg/cm^2.

CFEM (1992), Canadian Foundation Engineering Manual, recommended an equations for evaluation of allowable bearing capacity using:

$$q_a = 0.10\,\overline{q}_c \quad (6)$$

Also, based on CFEM (1992) the safety facter of 3 has been suggested. Hence, the ultimate bearing capacity is

$$q_{ult} = 0.30\overline{q}_c \quad (7)$$

Eslaamizaad and Robertson (1996), according to Meyerhof method, (Eq. 3) by using some case histories and CPT soundings close to foundation locations in conesionless soils proposed a relationship between q_{ult} and q_c as follow:

$$q_{ult} = k\overline{q}_c \quad (8)$$

where k is a correlation facter, and is a function of B/D_f , shape of footing and sand density.

Tand et al. (1995) employed a few full scale load tests and CPT data and suggested the ultimate bearing capacity of shallow footings on lightly cemented medium dense sand by following equation:

$$q_{ult} = R_k q_c + \sigma_{v0} \quad (9)$$

where R_k ranges from 0.14 to 0.2, depending on the footing shape and depth and σ_{v0} is total stress at the footing base.

3 NEW DIRECT CPT METHOD

A direct CPT method (Eslami and Gholami, 2002) has been developed based on an analytical model for determining ultimate bearing capacity, q_{ult} of shallow foundations from CPT cone resistance, q_c.

Research by De Beer, (1963), Meyerhof , (1976), and Eslami & Fellenius, (1997) indicated that a type of logarithmic spiral failure zone and attribuled rupture surfaces to reach penetrometer body need a penetration depth of at least 10 times of penetrometer diameter to fully mobilize the ultimate unit toe resistance. In other words, penetration depth of 10 footing or cone diameter is required to transform shallow mechanism to deep mechanism of rupture surface . In the former, the rupture surface extent to the ground level but for the latter the rupture surface reaches to the penetrometer or pile body. The mechanism of transforming shallow to deep mechanism of rupture surface is illustruted in Figure 1.

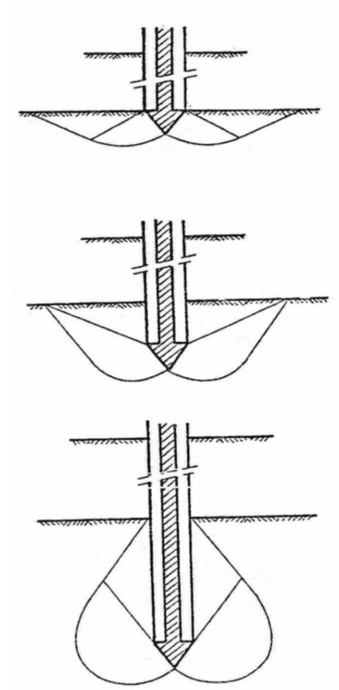

Figure 1. Transfrom of shear failure surface from shallow to deep mechanism (Nottingham, 1975)

Regarding to the basic bearing capacity formula, the length and depth of rupture surface have a major role in mobilized foundation bearing capacity. Therefore, the q_{ult} to q_c in direct approach can be correlated as follows:

$$q_{ult} = \overline{\alpha} \times \overline{q}_{c,g} \text{ and } \overline{\alpha} = (\alpha_1 + \alpha_2)/2 \quad (10)$$

$$\alpha_1 = \frac{L_\theta}{L_\circ} \text{ and } \alpha_2 = \frac{A_\theta}{A_\circ} \quad (11)$$

$\overline{q}_{c,g}$ = geometric average of, q_c values from footing base to 2B beneath footing.

α_1 = modification transforming length ratio

α_2 = modification transforming area (depth) ratio

$\overline{\alpha}$ = avearge of length and area modification factor

$L_\circ, L_\theta, A_\circ$, and A_θ have been shown in Figure 2.

According to Fig. 2, by assumption of log spiral general shear failure, the radius of rupture zone, r, can be calculated as:

$$r = r_0 e^{\theta \operatorname{tg}\phi} = \frac{B}{2}\operatorname{tg}(\pi/4 + \phi/2) \times \frac{1}{\cos(\pi/4 - \phi/2)} e^{\theta \operatorname{tg}\phi} \quad (12)$$

where r is the radius of the logarithmic spiral; r_0 is the radius of the logarithmic spiral for $\theta = 0$, θ is the angle between a radius and r_0, as shown in Fig. 2; and ϕ is the angle between the radius and the normal at that point on the spiral (assumed equal to the friction angle of the soil).

A simplified correlation for estimating ϕ angle from q_c and effective stress level has been suggested:

$$\phi = (\log(\frac{\overline{q_c}}{\gamma' z}) + 0.5095)/0.0915 \quad (13)$$

Which obtained from Eq. 1 by q_c replacing instead of q_{ult}, and using standard cone penetrometer geometry.

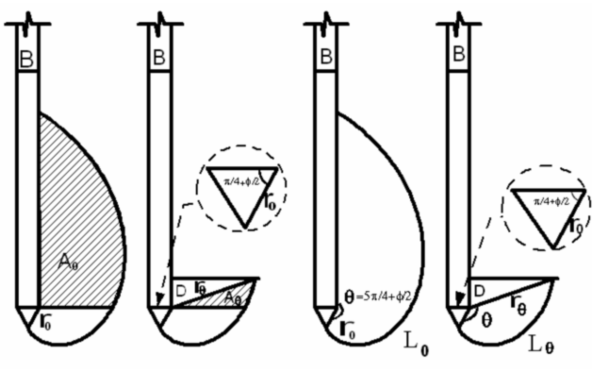

Figure 2. Rupture surfaces regarded in the propsed method

Regarding Fig. 2, the depth of embedment is deraived as:

$$D = y = r_0 e^{\theta \mathrm{tg}\phi} \times \sin(\theta - \frac{\pi}{2} - (\frac{\pi}{4} - \frac{\phi}{2})) \quad (14)$$

By integrating the curve length and using m as:

$$m = \frac{B}{2} \mathrm{tg}(\pi/4 + \phi/2) \times \frac{1}{\cos(\pi/4 - \phi/2)} \quad (15)$$

The relative depth, the ration of foundation depth to foundation width is dervied as following equation :

$$\frac{D_f}{B} = \frac{\mathrm{tg}(\pi/4 + \phi/2) e^{\theta \mathrm{tg}\phi}}{2\cos(\pi/4 - \phi/2)} \times \sin(\theta - 3\pi/4 + \phi/2) \quad (16)$$

Therefore, the length of rupture surface become

$$L_\theta = \frac{m\sqrt{1 + \mathrm{tg}^2\phi}}{\mathrm{tg}\phi} \times (e^{\theta \mathrm{tg}\phi} - 1) \quad (17)$$

As a result, the ratio of rupture surface length in shallow to deep conditions become

$$\alpha_1 = \frac{L_\circ}{L_{\theta=(\pi+\pi/4-\phi/2)}} = \frac{e^{\theta \mathrm{tg}\phi} - 1}{e^{(5\pi/4-\phi/2)} - 1} \quad (18)$$

In addition to length of rupture surface effect, the influence of surcharge around penetrometer (depth) can be considered as area (A) of surcharge and calculated:

$$A_\theta = m^2 / 4\mathrm{tg}\phi(e^{2\theta \mathrm{tg}\phi} - 1)$$
(19)

$$\alpha_2 = \frac{A_\circ}{A_{\theta} = (\pi + \pi/4 - \phi/2)} = \frac{e^{2\theta \mathrm{tg}\phi} - e^{2(3\pi/4-\phi/2)}}{e^{2(5\pi/4-\phi/2)} - e^{2(3\pi/4-\phi/2)}} \quad (20)$$

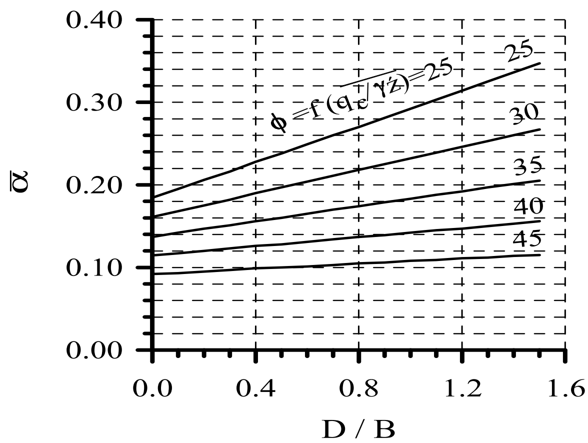

Figure 3. Combined length and area correlation factor

Based on Eq. 10, the variation of $\overline{\alpha}$, as averge of α_1 and α_2 is illustrated in Fig. 3 as a function of ϕ and D/B values.

Finally, summary of proposed method in step by step procedure is as follows:

The zone between foundation base to 2B under base can be divided in sublayers. The values of $\overline{q}_{c.g}$ and $(\overline{q_c/\gamma' z})_g$ (geometric average) in this interval is calculated.

The average ϕ angle is obtained by using $(\overline{q_c/\gamma' z})_g$ (geometric average) from Eq. 13

Based on D/B and ϕ , from Fig. 3, $\overline{\alpha}$ can be obtained

Having $\overline{\alpha}$, the q_{ult} is calculated as: $q_{ult} = \overline{\alpha} \times \overline{q}_{c,g}$

4 EVALUATION OF DIRECT CPT METHODS

A database has been compiled from five sites including 21 full scale footings and/or plate load tests accompanying of CPT soundatings close to foundation locations. Following, the brief summary of site specifications will be reviewed:

Site No.I, located in Texas, USA and was reported by Briaud and Gibbens (1994). Five square spread footings with B ranges from 1 to 3m and D_f from 0.7 to 0.89m were constructed on uniform sand (SP). The measured q_{ult} was about 15 kg/cm^2 and the q_c values varies from 40 -110 kg/cm^2.

Site No.II, reported by Amar (1979) four square 1m surface footing was loaded on silt (ML) soil. The measured q_{ult} ranges from 3 to 3.75 kg/cm^2 and the q_c values was in the range of 17 to 28 kg/cm^2.

Site No.III, is located in Texas, USA (Tand et al., 1994). Four circular footing with diameter of 1.75 to 2m were located in depth of 2.16m to 2.35m. The subsoil is silty sand (SM&SP). The q_{ult} values varied from 9.4 to 13.5 kg/cm^2.

Site No.IV, is located in Tabriz, Northwest of IRAN (Saniee, 1993). Three load tests were perfromed on 0.6m width surface plates. The site is fromed of silty and clayey sands in q_c which the measured q_{ult}values ranges from 12.6 to 12.8 kg/cm^2 and the q_c values varies from 30 to 100 kg/cm^2.

Site No.V, according to report by Consoli et al., 1998, five plate load tests has been done on 0.3 to 0.6m diameter plates. The subsoil of footing is mixture of clay and sand. The q_{ult} obtained from PLT was about 1.7 kg/cm^2 and the q_c values was in the range of 5-20 kg/cm^2.

Utlilizing 21 footing case histories, the predicted bearing capacity compared to the measuned from test results. For validation of outputs the statistical and cumulative probability approaches has been used to evaluate the acuracy of results according to following equations.

$$E_r = \frac{q_{ult,Cal.} - q_{ult,Mes}}{q_{ult,Mes}} \quad (21)$$

$$P = \frac{i}{n+1} \quad (22)$$

where
E_r= relative error
$q_{ult,Cal}$ and $q_{ult,Mes}$ = calculated and measured bearing capacity
P=cumulative probability
n &i = number of total case and case number

Since the negative and positive values from Eq. 21 may neutralize each other , the absolute values has been regarded for comparison. The average absolute error for six current methods is 51% with average standard deviation, SD 36%.

Wheras for new method the absolute error is 17% with SD=15%.

Comparison of predictive methods with cumulative probability apprach is presented in Figure 4.

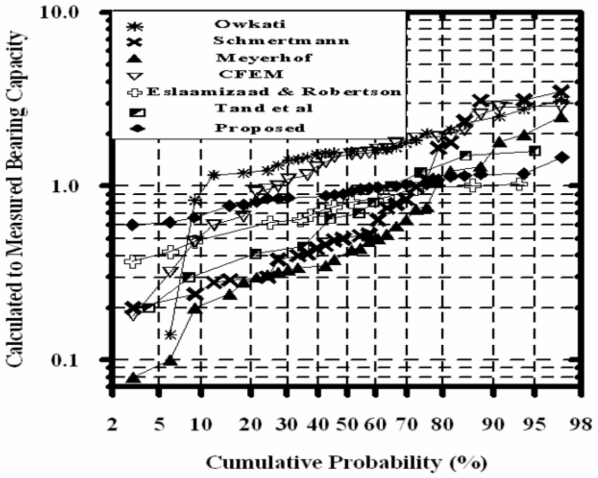

Figure 4. Comparison of methods by Cumulative probability

The results of comparison indicateds the $q_{ult,Cal} / q_{ult,Mes}$ at probability, p= 50% is closer to unity for proposed method than others. It means the new methed predicts foundation capacity with less overestimation or underestimation than other methds. Besides, the slope of line through points for current CPT methods exhibit higher dispersion than the new method. Therefore, the results are closer for the new method to log – normal distribution.

5 CONCLUSIONS

For determining bearing capacity of shallow foundations from CPT data six direct methods and a new one has been presented and compared. The new method is based on analytical model for relating deep failure surface of cone point to shallow rupture surface beneath footings. The ultimate bearing capacity of foundations, q_{ult} is equaled to cone point resistanc, q_c by a correlation facter in direct approach. The correlation facter is a function of ϕ angle and footing relative depth, in which ϕ angle can be obtained in each depth from effective stress level and q_c values.

A database has been compiled including 21 footings including load test results of q_{ult} and CPT sounding which performed close to foundation locations.

Validation of methods for foundation capacity prediction, by absolute error and probabilty approaches shows less scalter and more accuracy for new method than current methods. Because of the promising resuls, the proposed methed can be considered as an approach in geotechnical design.

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