Pile capacity estimated from CPT data – Six methods compared

Estimation de la force portante des pieux à partir des données de pénétration statique – Comparaison entre six méthodes d'approche

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ABSTRACT: Five methods to determine pile axial capacity from mechanical cone penetrometer data and one method using piezocone data are presented and discussed. The methods are applied to 24 case histories combining CPTu data and capacities obtained in a static loading test. The results are favorable to the CPTu method, which shows better agreement with the capacity determined in a static loading test and less scatter than the CPT methods. Average error of the CPTu-method was 7 %, while the average error of the CPT-methods ranged from 19 % through 36 %.

RESUME: Cinq methodes pour determiner la capacite axiale de pieux a partir de donnees de penetrometre a cone mecanique et une methode qui utilise les donnes de piezocone sont presentees et discutees. Les methodes sont aplliques a 24 cas reel combinant des donnes de CPTu et des capacites obtenues a partir d'essais de chargement statique. Les resultats sont favorables pour la methode de CPTu, qui presente un meilleur accord avec la capcite determinee a partir de chargement statique et moins de variabilite que les methodes de CPT. L'erreur moyenne de la methode CPTU etailt de 7 %, cependant, l'erreur moyenne des methodes de CPT variaient entre 19 % et 36 %.

1. INTRODUCTION

Very early on in the development of the CPT, because of the similarity between a cone penetrometer and a pile, cone resistance and sleeve friction data were used for estimating pile capacity. Two main approaches for application of cone data to pile design evolved: indirect and direct methods.

Indirect CPT methods employ soil parameters, such as friction angle and undrained shear strength estimated from the cone data as based on bearing capacity and/or cavity expansion theories, which introduces significant uncertainties. The indirect methods disregard horizontal stress, apply strip-footing bearing capacity theory, and neglect soil compressibility and strain softening. The authors consider these methods less suitable for use in engineering practice and will not further refer to them.

Direct CPT methods more or less equal the cone resistance with the pile unit resistances. Some methods may use the cone sleeve friction in determining unit shaft resistance. Several methods modify the resistance values to consider the difference in diameter between the pile and the cone. The influence of mean effective stress, soil compressibility, and rigidity affect the pile and the cone in equal measure, which eliminates the need to supplement the field data with laboratory testing and to calculate intermediate values, such as K_s and N_q.

To relate cone resistance to the pile unit toe resistance, an arithmetic average of the CPT data over an "influence zone" is used. Before finding the average, the data are filtered and smoothened to eliminate extremes, that is, peaks and troughs are excluded from the records.

2. CURRENT CPT DIRECT METHODS

The following direct methods are considered in this study:

1. Schmertmann and Nottingham
2. deRuiter and Beringen (European)
3. French (LCPC)
4. Meyerhof
5. Tumay and Fakhroo
6. Eslami and Fellenius

The **Schmertmann and Nottingham** method is based on a summary of the work on model and full-scale piles presented by Nottingham (1975) and Schmertmann (1978). The unit toe resistance, r_t, in sand and in clay is taken as equal to the average of the cone resistance. The q_c-average is determined in an influence zone extending from 6b through 8b above the pile toe (b is the pile diameter) and 0.7b through 4b below, as outlined in Fig. 1. The extent of the zone depends on the trend of the q_c-values and follows recommendations by Begemann (1963), who based the zone extent on an assumed a logarithmic-spiral failure pattern for the pile toe. Detailed filtering and minimum-path rules apply for selecting the average cone resistance. An upper limit of 15 MPa is imposed for the unit toe resistance.

The unit shaft resistance, r_s, may be determined from the sleeve friction as expressed by Eq. 1.

$$r_s = K f_s \tag{1}$$

where r_s = unit shaft resistance
K = a dimensionless coefficient
f_s = sleeve friction

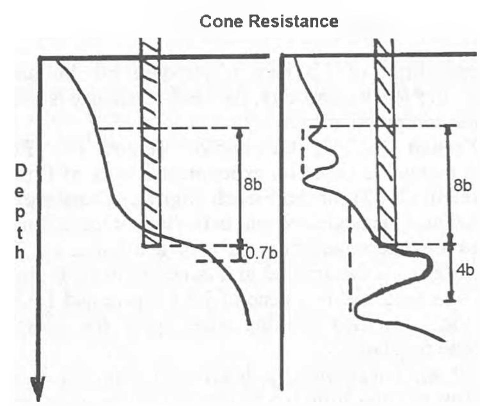

Fig. 1 Schmertmann and Nottingham rules for the influence zone and average q_c

The K-coefficient depends on pile shape and material, cone type, and embedment ratio. In sand, the K-coefficient ranges from 0.8 through 2.0, and, in clay, it ranges from 0.2 through 1.25. Within an embedment depth of eight pile diameters, the unit shaft resistance is linearly interpolated from zero at the ground surface to the value of Eq. 1.

Alternatively, in sand, but not in clay, the shaft resistance may be determined from the cone resistance as given in Eq. 2.

$$r_s = C\,q_c \tag{2}$$

where r_s = unit shaft resistance
C = a dimensionless coefficient; a function of pile type ranging from 0.8 % through 1.8 %
q_c = cone resistance (total; uncorrected for pore pressure)

An upper limit of 120 KPa is imposed on the unit shaft resistance, r_s, regardless whether it is determined from Eq. 1 or Eq. 2. For tension capacity, the shaft resistance is reduced to 70 % of that determined by Eqs. 1 or 2.

The **deRuiter and Beringen** (1979) method (also called the European method) is based on experience gained from offshore construction in the North Sea. For unit toe resistance of a pile in sand, the method is the same as the Schmertmann and Nottingham method. In clay, the unit toe resistance is determined from total stress analysis applied according to conventional bearing capacity theory as indicated in Eqs. 3 and 4.

$$r_t = N_c\,S_u \tag{3}$$

$$S_u = q_c/N_K \tag{4}$$

where r_t = unit toe resistance
N_c = conventional bearing capacity factor
S_u = undrained shear strength
q_c = cone resistance (total; uncorrected for pore pressure)
N_K = a dimensionless coefficient, ranging from 15 through 20, reflecting local experience

An upper limit of 15 MPa is imposed for the unit toe resistance which is further governed by the overconsolidation ratio, OCR, of the soil.

The unit shaft resistance in sand is determined by either Eq. 1 with K = 1 or Eq. 2 with C = 0.3 %. In clay, the unit shaft resistance may also be determined from the undrained shear strength, S_u, (obtained by Eq. 4), as given in Eq. 5.

$$r_s = \alpha\,S_u \tag{5}$$

where r_s = unit shaft resistance
α = adhesion factor equal to 1.0 and 0.5 for normally consolidated and overconsolidated clays, respectively
S_u = undrained shear strength

An upper limit of 120 KPa is imposed on the unit shaft resistance. For tension capacity, the shaft resistance is reduced to 75 % of the compression capacity.

The **French** (LCPC, Laboratoire Central des Ponts et Chausees) method is based on experimental work of Bustamante and Gianeselli (1982) for the French Highway Department. The sleeve friction, f_s, is neglected and both the unit toe and unit shaft resistances are determined from the cone resistance, q_c. The unit toe resistance, r_s, is determined in a range of 40 % through 55 % of the q_c, averaged within a zone of 1.5 b above and 1.5 b below the pile toe. Detailed filtering rules apply for selecting the average cone resistance.

The unit shaft resistance is determined from Eq. 2 with the C-coefficient ranging from 0.5 % through 3.0 %, as governed by magnitude of the cone resistance, type of soil, and type of pile. Upper limits of the unit shaft resistance are imposed, ranging from 15 KPa through 120 KPa depending on soil type, pile type, and pile installation method.

The **Meyerhof** method (1956; 1976; 1983) is based on theoretical and experimental studies of deep foundations in sand. For unit toe resistance, the influence of scale effect of piles and shallow penetration in dense sand strata is considered by applying two modification factors, C_1 and C_2, to the q_c average. The unit toe resistance in sand is given by Eq. 6. The unit toe resistance of a bored pile is reduced to 30 % of that determined from Eq. 6.

$$r_t = q_{ca}\,C_1\,C_2 \tag{6}$$

where r_t = unit toe resistance
q_{ca} = arithmetic average of q_c in a zone ranging from "1b" below through "4b" above pile toe
C_1 = $[(b + 0.5)/2b]^n$; modification factor for scale effect when $b > 0.5$, otherwise $C_1 = 1$
C_2 = $D_b/10b$; modification for penetration into dense strata, when $D_b < 10b$, otherwise $C_2 = 1$
n = an exponent equal to 1 for loose sand, 2 for medium dense sand, 3 for dense sand
b = pile diameter
D_b = embedment of pile in dense sand strata

The unit shaft resistance is determined from either Eq. 1 with K = 1, or Eq. 2 with C = 0.5 %. For bored piles, reduction factors of 70 % and 50 %, respectively, are applied to the calculated values of shaft resistance.

The **Tumay and Fakhroo** method is based on an experimental study in clay soils in Louisiana. The unit toe resistance is determined by the same way as in the Schmertmann and Nottingham method. The unit shaft resistance is determined according to Eq. 2 with the K-coefficient determined according to Eq. 7, where the coefficient is no longer dimensionless but as given in Eq. 7.

$$K = 0.5 + 9.5\,e^{-90fs} \tag{7}$$

where f_s = sleeve friction in MPa

For a sleeve friction ranging from 10 KPa through 50 KPa, Eq. 7 results in a K-coefficient ranging from about a value of 4.5 through 0.6.

The **Eslami and Fellenius** method (Eslami and Fellenius, 1995; 1996, and Eslami, 1996) is based on the piezocone, CPTu. In contrast to the five other methods, no filtering of the data or minimum path is used. Instead, the influence of odd "peaks and troughs" is reduced by mean of employing the geometric average of the cone point resistance as opposed to the arithmetic average used by the five CPT methods. Furthermore, the cone resistance is transferred to the effective cone resistance, q_E, by subtracting the measured pore pressure, u_2, from the measured total cone resistance. The thus determined *effective geometric average* is set equal to the pile unit toe resistance determined over an influence zone extending from 4 b below through 8 b above the pile toe when a pile is installed through a weak soil into a dense soil, and from 4 b below through 2 b above the pile toe when a pile is installed through a dense soil into a weak soil.

$$r_t = q_E \tag{8}$$

where r_t = unit toe resistance
q_E = geometric average of the cone point resistance after adjustment to effective stress

Also the pile unit shaft resistance is referenced to the average effective cone point resistance with the cone point resistance value modified according to soil type. The soil type is classified from a combination of sleeve friction and effective cone resistance in a Begemann type diagram (Begemann, 1965) according to the chart shown in Fig. 2, as developed by Eslami (1996) and Eslami and Fellenius (1996).

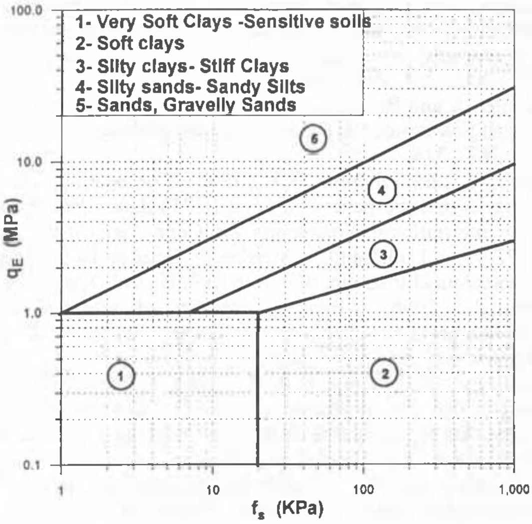

Fig. 2 CPTu classification chart

$$r_s = C_s q_E \quad (9)$$

where r_s = unit shaft resistance
C_s = shaft correlation coefficient, which is a function of soil type determined from the soil classification chart presented in Table 1.
q_E = geometric average of the cone point resistance adjusted to effective stress

TABLE 1. Shaft correlation coefficient, C_s

Soil Type	C_s
1. Very soft clay and soft sensitive soil	8.0 %
2. Soft clay	5.0 %
3. Stiff clay and mixture of clay and silt	2.5 %
4. Mixture of silt and sand	1.0 %
5. Sand and gravely sand	0.4 %

3. COMMENTS ON THE METHODS

When using either of the five first methods, difficulties arise in applying some of the recommendations of the methods. For example:

1. The CPT methods were developed more than a decade ago, before the piezocone came in general use, therefore, they do not consider the more accurate measurements achievable with the piezocone (Campanella and Robertson, 1988).
2. Although the recommendations are specified to soil type (clay and sand; very cursorily characterized), the methods do not include a means for identifying the soil type from CPT data. Instead, the soil profile governing the coefficients relies on information from conventional boring and sampling, and laboratory testing, which may not be fully relevant to the CPT data.
3. All the five CPT methods include random smoothing and filtering of the CPT data to eliminate extreme values. This results in considerable operator-subjective influence of the results.
4. The cone resistance is not corrected for the pore pressure on the cone shoulder and, therefore, the data behind the methods include errors—smaller in sand, larger in clay.
5. The CPT methods employ total stress values, whereas effective stress governs the long-term behavior of piles.
6. All of the CPT methods are locally developed, that is, they are based on limited types of piles and soils and may not be relevant outside the local area.
7. The upper limit of 15 MPa, which is imposed on the unit toe resistance in the Schmertmann and Nottingham, European, and Tumay and Fakhroo methods, is not reasonable in very dense sands where values of pile unit toe resistance higher than 15 MPa frequently occur. Excepting Meyerhof method, all CPT methods impose an upper limit also to the unit shaft resistance, which cannot be justified because values of pile unit shaft resistance higher than the recommended limits occur frequently.
8. All CPT methods involve a judgment in selecting the coefficient to apply to the average cone resistance used in determining the unit toe resistance.
9. In the European method, the overconsolidation ratio, OCR is used to relate q_c to r_t. However, while the OCR is normally known in clay, it is rarely known for sand.
10. In the European method, considerable uncertainty results when converting cone data to undrained shear strength, S_u, and, then, in using S_u to estimate the pile toe capacity. S_u is not a unique parameter and depends significantly on the type of test used, strain rate, and the orientation of the failure plane. Furthermore, drained soil characteristics govern long-term pile capacity also in cohesive soils. The use of undrained strength characteristics for long-term capacity is therefore not justified. (Nor is it really a direct CPT method).
11. In the French method, the length of the influence zone is too limited. (The influence zone is the zone above and below the pile toe in which the cone resistance is averaged). Particularly if the soil strength decreases below the pile toe, the soil average must include the conditions over a depth larger than 1.5b distance below the pile toe.
12. The French method makes no use of sleeve friction, which disregards an important aspect of the CPT results and soil characterization.

The CPTu method (Eslami and Fellenius method) avoids all of the mentioned difficulties.

4. CASE RECORDS

The six methods have been applied to twenty-four pile case histories and the results are compared. The cases comprise full-scale pile loading tests and CPT soundings performed close to the pile locations. Fourteen of the cases include compression static loading tests and ten of the cases include tension tests. The soil profiles range from sites with soft and stiff clay, medium to dense sand, and mixture of clay-silt-sand. The embedment lengths for the pile case histories range from 7 m through 34 m and the pile diameters range from 200 mm through 625 mm. The pile capacities range from 80 KN through 5,700 KN. But for four of the sites, where the site consists of sand, the cone test data were obtained by the piezocone.

5. RESULTS

The cone test methods were applied to the cone data to determine the total capacity of the piles and the results were compared to the capacities determined in the static loading tests. (The Tumay and Fakhroo method is pertinent to 6 cases and the Meyerhof method to 17 cases). The comparison of the methods is best made in terms of the *relative difference* between the tested and calculated pile capacities, determined as the difference between the calculated and that found in the static loading test divided by the static loading test value. The relative difference is positive or negative reflecting when the calculated value is respectively larger or smaller than the capacity found in the static loading test.

The relative difference of the estimated pile capacity for the six methods varies between the cases. A strict average of the errors does not provide meaningful results. The *absolute error average* is more representative. Fig. 3 presents the absolute average errors resulting from the study.

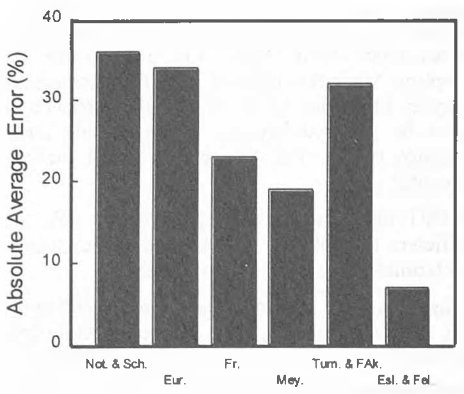

Fig. 3 Absolute average error

For total capacity, the Schmertmann and Nottingham, the European, and the Tumay and Fakhroo method show the high absolute error average of about 30 %, while the French and the Meyerhof method give the absolute error of 23 % and 19 %, respectively. In contrast, the Eslami and Fellenius method, gives an absolute error average of only 7 %, which indicates a very good agreement between the calculated and measured capacities. More important, the agreement is consistent for all 24 cases as evidenced by a standard deviation of 6 %, as opposed to standard deviation values ranging from 12 % through 22 % for the other five methods.

6. CONCLUSIONS

Five direct CPT methods and one direct CPTu method for determining pile capacity are presented and discussed.

For the CPT methods, the main factors causing significant error in pile capacity estimation are that the methods: apply subjective smoothing of the CPT data, employ undrained shear strength, S_u, impose limits on pile unit toe and shaft resistances, apply broad correlation factors to separate tension and com,pression as well as steel and concrete, disregard soil sensitivity, dilatency effects, and effective stress, among others. These disadvantages are avoided in the CPTu method.

The methods are applied to 24 case histories reporting tests on piles of different sizes, types, and lengths installed in different types of soil, where cone test data and results of static loading tests are available. The results of a comparison of the calculated pile capacities to the measured are very favorable to the CPTu method, which shows better agreement with the capacity determined in a static loading test and less scatter than the CPT methods. The CPTu method is simple, easy to apply, and independent of all operator subjective influence. Therefore, it is notably suitable for use in engineering practice.

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