Toe Bearing Capacity of Piles from Cone Penetration Test (CPT) Data

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SYNOPSIS A direct CPT method is proposed for determining the toe capacity of a single displacement pile. The method makes use of a simple mathematical rule for determining the average cone resistance, q_c, adjusting it to the effective stress and relating it to the unit toe resistance of a pile, r_t. Case histories comprising CPT data, soil characteristics, and results from full-scale pile loading tests are referenced to the methods. The case histories involve sites with soft clay and sand, sand interbedded with thin silt and clay layers, and medium to dense sand. The pile embedment lengths range from 9 m through 31 m. The pile capacities range from 300 KN through 5,800 KN with measured toe resistances ranging from 62 KN through 4,050 KN. Pile toe capacities calculated by the proposed method are compared to toe capacities calculated by four other direct methods currently employed in North American practice. The proposed method gives values that are more consistent and closer to the measured than the current methods.

INTRODUCTION

Determining axial capacity of piles is a challenge under the best of circumstances. The engineering practice has developed several methods to overcome the uncertainty in the analysis and design. However, due to simplifying assumptions regarding soil stratigraphy, distribution of shaft resistance along a pile, and soil-pile structure interaction, the methods provide qualitative results rather than truly quantitative values directly useful in the pile design. In recent years, the Cone Penetration Test (CPT) has become the preferred in-situ test for pile design and analysis. This is because the CPT is simple, fast, relatively economical, and provides continuous records with depth that are interpretable on both empirical and analytical bases.

CURRENT METHODS

Two main approaches are used for the application of CPT data to pile design: indirect methods and direct methods. Indirect methods will not be discussed here. Direct CPT methods apply cone bearing for unit toe resistance and sleeve friction for unit shaft resistance by the analogy of the cone penetrometer as a model pile. The following four CPT direct methods for pile capacity estimation are used in current North American practice.

- The Schmertmann and Nottingham method (1975; 1978)
- The DeRuiter and Beringen method (1979)
- The LCPC method (Bustamante and Gianeselli, 1982)
- The Eurocode method (1993)

The methods address both shaft and toe resistances. However, this presentation is limited to a study of the pile toe resistance calculated from the CPT cone resistance.

The Schmertmann method is based on a summary of the work on model and full-scale piles presented by Nottingham (1975). The unit toe resistance of a pile, r_t, in sand and clay is taken as equal to the average of the cone resistance, q_c. The average q_c value is determined from the graphical representation of the CPT measurements in a zone defined by the failure pattern for the pile toe, ranging from 8b above (b is pile diameter) and from 0.7b through 4b below the pile toe (the actual value depends on the trend of q_c-values), as was originally proposed by Begemann (1963), who imposed an upper limit of 15 MPa on the toe resistance. The unit toe resistance is further governed by the overconsolidation ratio, OCR.

The DeRuiter and Beringen method (also called the European method) is based on experience gained in the North Sea by Fugro Consultants International. This method is very similar to the Schmertmann method. Indeed, for unit toe resistance of a pile in sand, the method is the same as the Schmertmann method. In clay, the unit toe resistance is determined from the undrained shear strength, S_u, as follows.

$$r_t = N_c S_u \tag{1}$$

$$S_u = q_c / N_k \tag{2}$$

where N_c is the conventional bearing capacity factor and N_k is a non-dimensional cone factor ranging from 15 through 20.

The LCPC (Laboratoire Central des Ponts et Chausees) method (also called the French method) developed by Bustamante and Gianeselli (1982) is a result of experimental work by the French Highway Department. The experimental database for this method is based on the results of a large number of full-scale pile loading tests. The average q_c, is determined within a zone of 1.5b above and 1.5b below the pile toe and the unit toe resistance of a pile is determined as a percentage of the q_c-value ranging from 40 % through 55 %, as governed by cone resistance magnitude, soil, and pile types.

The Eurocode (Frank, 1994) is combination of the general rules for geotechnical design in the "Eurocode 7-Part 1" and the code of the French Highway Administration. The method is very similar to LCPC method. The difference is that the unit toe resistance is determined as a range of 50 % through 55 % of average q_c.

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

1. All methods include random smoothing of the data, that is, elimination of peaks and troughs, which subjects the results to considerable subjective operator influence.
2. In the Schmertmann and European methods, 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.
3. 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.

4. In the French and Eurocode methods, the extent of the zone above and below the pile toe in which the cone resistance is averaged, appears to be too limited. As considered in the Schmertmann method, 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.
5. The upper limit of 15 MPa, which is imposed on the unit toe resistance in the Schmertmann and European methods, is not reasonable in very dense sand where values of r_t higher than 15 MPa frequently occur.
6. All methods involve a judgment in selecting the coefficient to apply to the average cone resistance to arrive at the unit toe resistance.
7. The measured cone resistances are total stress values whereas effective stress governs the pile capacity.
8. Considerable uncertainty exists due to the effects of installation, strain softening, fissured clay, resistance degradation, sensitivity, dynamic pore pressures, and shallow penetrations of cone and pile into dense sand strata.

CONE RESISTANCE AVERAGE

All four current methods employ a graphic approach to relate cone resistance to the unit toe resistance of pile, where the q_c-values are first filtered by excluding peaks and troughs. The mean of the smoothened curve is taken to be the average q_c to use. Filtering and smoothing the cone data is necessary, because were a true mean produced from the data, the high and low values would have a disproportionate influence on the average. The filtering approach was developed when the CPT data were obtained in diagrams only and it brings about a considerable judgment leeway in the average cone resistance (eyeballing uncertainty). However, the subjective filtering is now not necessary, because current tests produce quantified results in the form of tables of data, easily accessible for determining the average by direct computer manipulation, making the graphic methods redundant.

The arithmetic average is defined as:

$$q_{ca} = 1/n\ (q_{c1} + q_{c2} + ... + q_{cn}) \qquad (3)$$

Having the CPT data in the computer, the average q_c according to Eq. 3 can be obtained automatically. However, without first excluding peaks and troughs, this average is only useful in homogenous soils and soils providing uniform values. Filtering is necessary in most cases and, if done manually, it offsets the advantage of the computer. A filter effect can be achieved directly, however, by instead calculating the geometric average of the q_c-values, defined as:

$$q_{cg} = (q_{c1}\ q_{c2} ...\ q_{cn})^{1/n} \qquad (4)$$

The bias in the arithmetic mean arises from the influence of the absolute magnitude instead of ratios of variations (Kennedy et al., 1986). For example, the arithmetic average of the numbers 0.5 and 2.0 is 1.25, and the geometric average is 1.00. If the numbers are 0.33 and 3.00 instead, the mean becomes 1.65 while the geometric average is still 1.00. If a set of data is made up of the numbers 0.33, 0.50, 2.00, and 3.00, the arithmetic and geometric averages are 1.46 and 1.00, respectively.

Assume that a set of values is as follows: 5, 5, 2, 5, 25, 5, 6, 1, 6, 6, 30, and 6, where the dominant values lie between 5 and 6. The arithmetic and geometric averages are 8.50 and 5.71, respectively. The result shows that the geometric average is closer to the dominant values, as opposed to the arithmetic average which is not representative for the dominant range.

The natural variability of many sand deposits produces q_c profiles with many sharp peaks and troughs. Therefore, by taking the geometric average of q_c-values in a zone at the vicinity of pile toe, a filtered representative value that is unaffected by operator's judgment and, therefore, repeatable, is obtained.

PROPOSED METHOD

A direct CPT method is proposed that includes determining the geometric average of all q_c-values at the vicinity of pile toe. For now, the zone at the vicinity of pile toe is taken to be the same as used by the Schmertmann and the European methods.

Pile capacity is governed by effective stresses in the soil, not total. Rather than obtaining the unit pile toe resistance as an arbitrary percentage of the average total cone resistance, the proposed method determines the toe resistance as the cone resistance minus the pore pressure measured by means of the piezocone. The proposed method, therefore, requires the CPT sounding to be made with the piezocone. However, in sand, normally, the pore pressures can be assumed to be essentially unchanged due to the cone penetration and older types of CPT equipment are still useful.

Thus, the unit toe resistance of a pile is as follows:

$$r_t = (q_{e1} \cdot q_{e2} \ldots q_{en})^{1/n} \qquad (5)$$

where

q_e = q_t - u
q_t = total cone resistance = q_c + (1-a)u
u = pore pressure, usually u_2
a = net area ratio of a cone
n = number of values in the considered zone

(The most useful location of the piezometer is behind the cone. The pore pressure measured at this location is called u_2).

CASE RECORDS

Six case histories are included in this study to reference the methods. The cases comprise full-scale pile loading tests where the pile toe capacity were determined, and include CPT soundings performed close to the piles.

UBC Research Site: A 324-mm, 31 m pipe pile was tested at the Lulu Island in Fraser River Delta, British Columbia. The soils consist of about 15 m of organic silty clays underlain by a 15 m thick medium sand deposit followed by 60 m normally consolidated clayey silt containing thin sand layers (Robertson and Campanella, 1988).

Northwestern University: A 450-mm, 15 m pipe pile was tested in conjunction with the 1989 ASCE Foundation Engineering Congress held at the Northwestern University, Evanston. The pile was installed through 7 m dense sand stratum overlying a soft clay layer (Finno, 1989; Fellenius, 1991).

Hunter's Point, San Francisco: A 273-mm, 9 m pipe pile was tested in conjunction with a pile Prediction Symposium organized by

FHWA, 1986. The soil at the site consists of about 2 m miscellaneous fill underlain by 11 m hydraulic sand fill followed by a clay layer (Fellenius, 1986; O'Neill 1988).

Baghdad University: Two 285-mm square concrete piles with embedment lengths of 11 m and 15 m were tested at Baghdad University Complex in 1984. The soil at the site consists of uniform sand (Altaee et al., 1992 and 1993).

Port of Los Angeles: A static pile loading test was performed on a 600-mm octagonal concrete piles with embedment length of 26 m at the Port of Los Angeles in 1985. The soil at site consists of an upper 15 m thick sand layer underlain by a 6 m thick fine-grained soil layer followed by dense sand (CH2M Hill, 1987).

Table 1 summarizes the case information, presenting pile data, soil type at pile toe, and measured pile and toe capacities, as well as the unit toe resistance is calculated from the toe capacity and cross sectional area of pile. The N_t-values are back-calculated from the toe capacity using the following equation:

$$R_t = r_t\, A_t = N_t\, \sigma'_{v=D}\, A_t \qquad (6)$$

where

R_t = pile toe capacity
r_t = unit toe resistance
A_t = cross sectional area of pile
$\sigma'_{v=D}$ = vertical effective stress at pile toe
D = pile embedment depth
N_t = bearing capacity factor

The back-calculated N_t-values, obtained for the different piles, lie within normally observed ranges of 3 through 30 for clay and 30 through 150 for sand (Fellenius, 1995).

Fig. 1 illustrates the CPT results including cone resistance, q_c, sleeve friction, f_s, and measured pore pressure. The groundwater table and the pile toe depths are indicated. At the UBC site, the pore pressures were measured behind the cone (u_2), whereas at NWU site, it was measured at the face of cone (u_3). For other cases, no pore pressures were measured. However, because the soils at these sites consist of sand, the pore pressures were considered to correspond to the distance to the groundwater table.

RESULTS

Table 2 presents the calculated average total cone resistances, q_c, in the zone near the pile toe and the corresponding unit pile toe resistances, r_t.

Fig. 2 shows a graphical comparison of results in terms of total pile toe resistance as determined by the four current methods and the proposed method and compared to the measured values. The figure has been separated into two diagrams showing three cases with piles having a measured toe resistance smaller than 400 KN and three cases with piles having a measured toe resistance larger than 400 KN.

Table 3 presents a compilation of the relative error in the calculations of the pile toe capacity by the methods. The relative error is determined as the difference between the calculated and measured values divided by the measured values. A negative value indicates an underestimation of the pile toe capacity.

The relative errors of the estimated pile toe capacity for the Schmertmann and European methods are smaller than those for the French and the Eurocode methods. How-

TABLE 1. Pile data, measured capacities, and soil conditions at pile toe

No.	Site	D m	b mm	A_t m^2	Shape, mtrl.	Soil at pile toe	R_u KN	R_t KN	r_t KPa	N_t
1	Univ. of B.C. (UBC5)	31.0	320	0.082	P, S	Clayey silt	1,100	180	2,195	9
2	N. W. Univ. (NWU)	15.3	450	0.159	P, S	Soft clay	1,020	62	390	3
3	Hunter Point	9.1	270	0.059	P, S	Sand	490	335	5,678	70
4	(FHWA)	11.0	285	0.081	Sq., C	Sand	1,000	360	4,444	30
5	Baghdad (BGHD1)	15.0	285	0.081	Sq., C	Sand	1,600	480	5,926	30
6	Baghdad (BGHD2)	25.8	610	0.308	Oct., C	Dense sand	5,785	4,050	13,149	60
	Port of L. A. (POLA)									

b=Pile diameter, P=Pipe, Sq.=Square, Oct.= Octagonal, S= Steel, C= Concrete

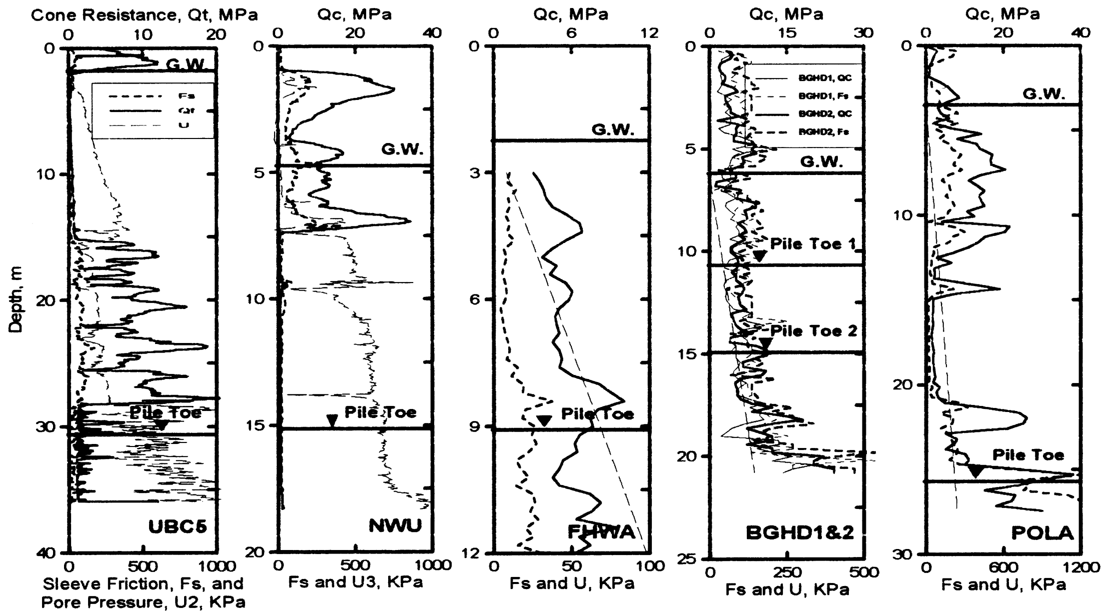

Fig. 1 CPT soundings from the sites. Notice, all cases use different depth and stress scales

TABLE 2. Average q_c and pile unit toe resistance from CPT methods

Method	UBC5		NWU		FHWA		BGH1		BGH2		POLA	
	q_{cavg} KPa	r_t KPa	q_{cavg}	r_t	q_{cavg}	r_t	q_{cavg}	r_t	q_{cavg}	r_t	q_{cavg}	r_t
Schmertm	1,765	1,765	580	580	4,850	4,850	3,000	3,000	6,500	6,500	15,000	15,000
European	1,260	1,260	360	360	4,850	4,850	3,000	3,000	6,500	6,500	15,000	15,000
French	2,030	1,015	600	300	7,200	3,600	3,940	1,970	9,240	4,260	25,300	23,500
Eurocode	2,030	1,117	600	339	7,200	3,600	3,940	1,970	9,240	4,620	25,300	12,650
Proposed	**1,900**	**1,900**	**375**	**375**	**6,200**	**6,200**	**4,070**	**4,070**	**6,470**	**6,470**	**14,380**	**14,380**

TABLE 3. Relative error (%) in capacity as determined by different methods

Methods	UBC5	NWU	FHWA	BGHD1	BGHD2	POLA	Average Errror	Standard Deviation
Schmertmann	-21	48	-15	-33	9	14	23	27
European	-43	-8	-15	-33	9	14	20	20
French	-53	-22	-37	-55	-22	-23	35	14
Eurocode	-48	-16	-37	-55	-22	-4	30	18
Proposed	**-12**	**-6**	**8**	**-8**	**9**	**9**	**9**	**8**

ever, the average error of pile toe capacity estimation for the four current methods is relatively high (20 % through 35 % with a Standard Deviation of 14 % through 27 %). In contrast, the proposed method shows a good agreement with the measured values (9% average error with a Standard Deviation of 8%), and more important, the agreement is consistent for all the six cases.

CONCLUSIONS

The proposed direct CPT method for determining the pile toe resistance from the cone resistance has been tested on six piles of different size and lengths installed in different type soils and compared to the results of four current methods. The proposed method is independent of operator judgment in filtering data and in choosing correlation factors, which affect all current methods. The results of the comparison are very favorable to the proposed method.

The study is a part of an ongoing research and it is the intent to develop the method further, including a review of the extent of the zone above and below the pile toe. The method will also include a study of the calculation of pile shaft resistance (early results were excluded from this presentation due to space limitations).

The authors would very much appreciate receiving case history data to add to the data base.

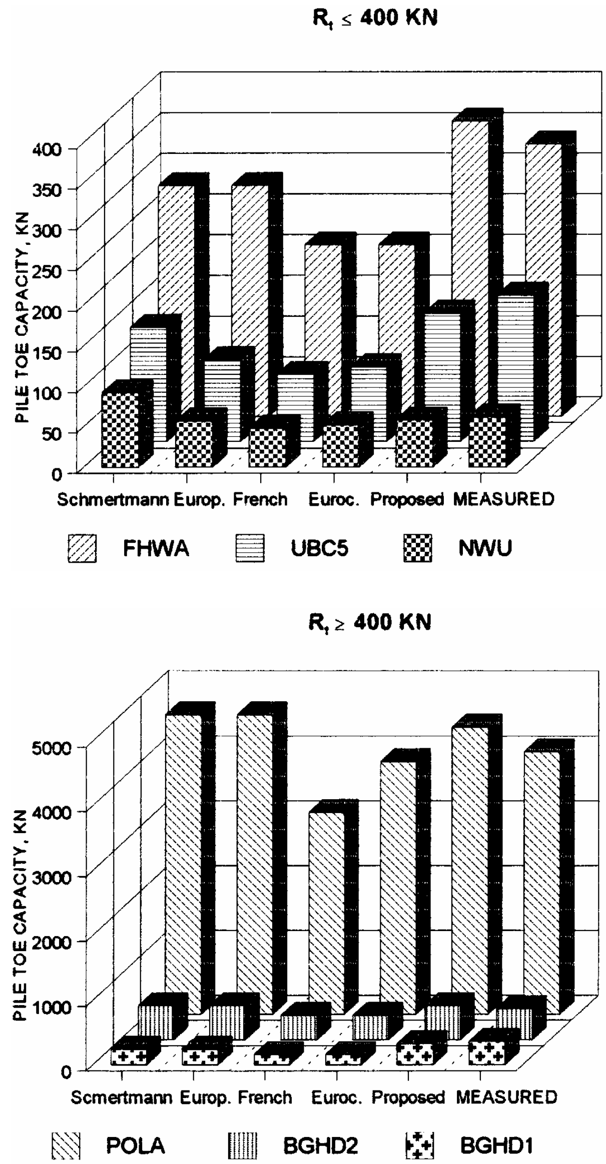

Fig. 2 Comparison between calculated and measured pile toe capacities

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