SUGGESTION ABOUT DETERMINATION OF THE BEARING CAPACITY OF PILES ON THE BASIS OF CPT SOUNDING TESTS

JÓZSEF PUSZTAI

About the authors

Pusztai József
Budapest University of Technology and Economics,
Department of Geotechnics
Műegyetem rkp. 3. K. mf 1., 1521 Budapest, Hungary
E-mail: pusztai@eik.bme.hu

Abstract

The Cone Penetration Test (CPT) is a well-recognized tool for the calculation of the ultimate bearing capacity of piles. Within the Hungarian physiographic territory, the CPT and Static Pile Load Tests of the bored (Continuous Flight Auger - CFA, protective tube) and driven (Franki) piles installed in different soils (gravel, sand and clay) were compared to determine the ultimate bearing capacity of piles by using new formulae.

Keywords

bearing capacity of piles, cone penetration test

1 BACKGROUND

Both international and Hungarian professional literature [1-8] deal intensively with the topic of the load bearing capacity of piles, determined on the basis of *in situ* exploration methods. This is a result of the rapid and extended proliferation of new exploration technologies which reveal more information about the underground condition on the spot (CPT and CPTu) than traditional boring methods. Having gained in this way substantive additional knowledge about the soil through new parameters, engineers try to develop appropriate formulae or equations that enable more efficient design and construction of structures. This also means that more reliable predictions can be made about the bearing capacity of a pile already in the design stage.

The relevant professional literature arrived to the unanimous conclusion that presently the most informative method for the determination of bearing capacities of piles in granular soils is the CPT (Cone Penetration Test) probing technology, because it differentiates between the cone resistance (q_c) and the local sleeve friction (f_s). The equipment produces a diagram describing separately these two resistances as a function of depth. An example is shown in Fig. 1.

In the Netherlands the design code [3] comprises the rules derived via innumerable cone tests and experiments for capacity calculations.

The load bearing capacity of the pile is determined from the cone resistance (q_c) of the CPT test. This is because the cone resistance values are more sensitive to variation in soil density than the sleeve friction, and the identification of the soil type from the ratio of q_c to f_s is not always clear-cut.

Consequently, in the traditional manner, the ultimate bearing capacity of a single pile (Q_u) is calculated as the sum of the ultimate resistance of the base (Q_b) and the ultimate resistance of the shaft (Q_s) capacities:

$$Q_u = Q_b + Q_s = A_b \cdot \overline{q_c} + U \cdot L \cdot \overline{\tau_s} \qquad (1)$$

where:

A_b = nominal plan area of the base of the pile [m^2]
U = length of the pile's periphery [m]
L = length of the pile [m]
$\overline{q_c}$ = average cone resistance in the zone of the pile toe [MPa]
$\overline{\tau_s}$ = average ultimate skin friction along the pile shaft [MPa].

Based on experience, Meigh [3] suggested using the following correlation between pile skin friction and cone resistances (Table 1).

The values given in Table 1 refer to piles that are exposed to static loads. Meigh [3] proposes to take the ultimate skin friction to 0,12 MPa at the utmost.

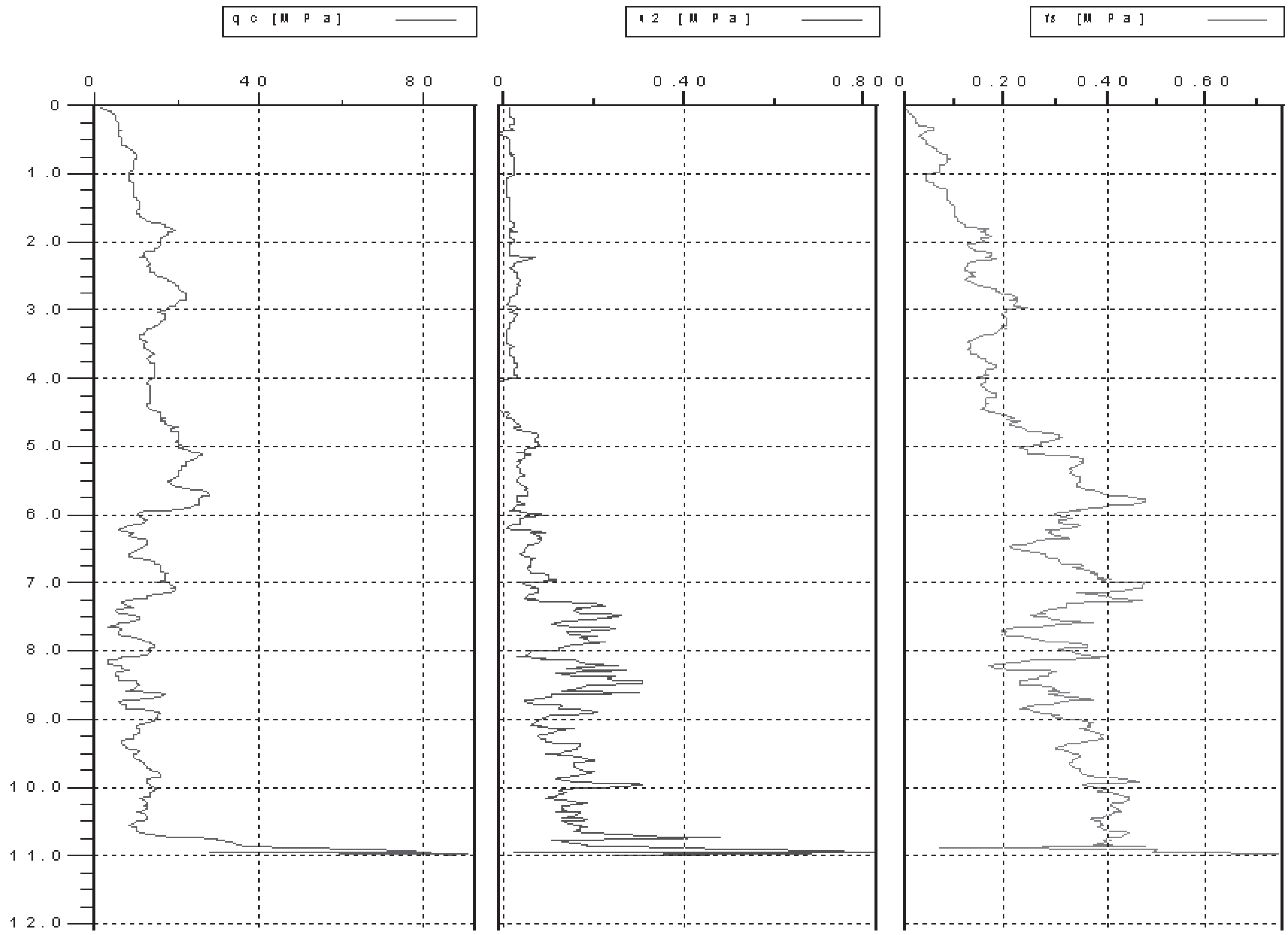

Figure 1. Measurement results of a Cone Penetration Test.

Table 1. Correlation between skin friction and cone resistance.

Pile type	Ultimate unit skin friction (τ_s)
Timber	0,012 q_c
Precast concrete	0,012 q_c
Steel displacement	0,012 q_c
Open ended steel tube + H-section	0,008 q_c
Open ended steel tube driven into fine to medium sand	0,0033 q_c

The average resistance against the progress of the cone, or penetration ($\overline{q_c}$), can be derived using the formula:

$$\overline{q_c} = \frac{\overline{q_{c-1}} + \overline{q_{c-2}}}{2} \qquad (2)$$

In the Netherlands, in accordance with the advice of Meigh [3], the method generally applied is the one in which the average cone resistance ($\overline{q_{c-1}}$) is determined to the depth of four times the pile diameter (4D) below the toe, and the average cone resistance ($\overline{q_{c-2}}$) to the depth of eight times the pile diameter (8D) above the pile toe.

Regarding the 4D – 8D method, it is important to note that:

- minor peak depressions have to be ignored in the calculation; supposedly, they do not refer to thin weak strata, and
- the q_c > 30 Mpa values will be also ignored in this interval.

Obviously, there are also methods other than the 4D – 8D method; in use, however, they only differ in the calculated depth below the pile toe (for example by taking 2D, instead of the 4D suggested above).

Te Kamp [9] preferred to suggest the safety factors presented in Table 2, for the calculation of limiting capacity in the Netherlands, when the 4D – 8D method is used**:**

Table 2. Factor of safety for piles.

Pile type	Factor of safety
Timber	1,7
Precast concrete, straight shaft	2,0
Precast concrete, enlarged shaft	2,5

Because of the disturbance and loosening of the soil via the boring tool, the Codes advise not to use cone resistance values when the skin resistance of bored piles is calculated.

The relationship established for Dutch soil conditions is not necessarily applicable to cohesionless soils everywhere. The yielding and rupture of the soil caused by pushing a cone into the ground are different from those resulting from driving a pile by hammer followed by static loading. The work of Vesic [10] has shown the importance of the state of preconsolidation and mineralogy of the soil grains in any correlation of in-situ conditions with pile resistance. By coincidence, static cone resistance in the Netherlands (and Belgium) was found to be equal to pile base resistance. Elsewhere, Gregersen et al. [11] found the pile base resistance to be only one half of the cone resistance for loose medium to coarse sands in Norway, and Gruteman et al. [12] reported that a factor of 0.75 was applied to the cone resistance to obtain the ultimate base resistance of piles in silty sands in Russia.

2 IN-SITU TESTS

The analysis of in-situ test data can result in better design parameter estimates which will affect the ultimate bearing capacity Q_u of piles. A comparison of the Static Pile Load Test and CPT measurements in Hungarian soils was undertaken to better define mechanisms affecting Q_u and to create formulae that are appropriate for Hungarian soils and that also consider construction methods.

In this sense, the author selected the results of domestically performed Static Pile Load Tests where the results of the CPT tests were also available. Altogether, data from seven CFA tests, three tests with protective tubes, and 26 Franki piles were gathered (Table 3).

3 SUGGESTED FORMULAE TO CALCULATE THE ULTIMATE BEARING CAPACITY OF PILES

The derived formula with the values in Fig. 2 relates to the failure load of a single pile. In deriving the formulae, the customary static basis has been used as a starting point, whereby the ultimate bearing capacity of a single pile (Q_u) is the sum of the ultimate resistance of the base (Q_b) and the ultimate resistance of the shaft (Q_s) capacities:

$$Q_u = Q_s + Q_b \qquad (3)$$

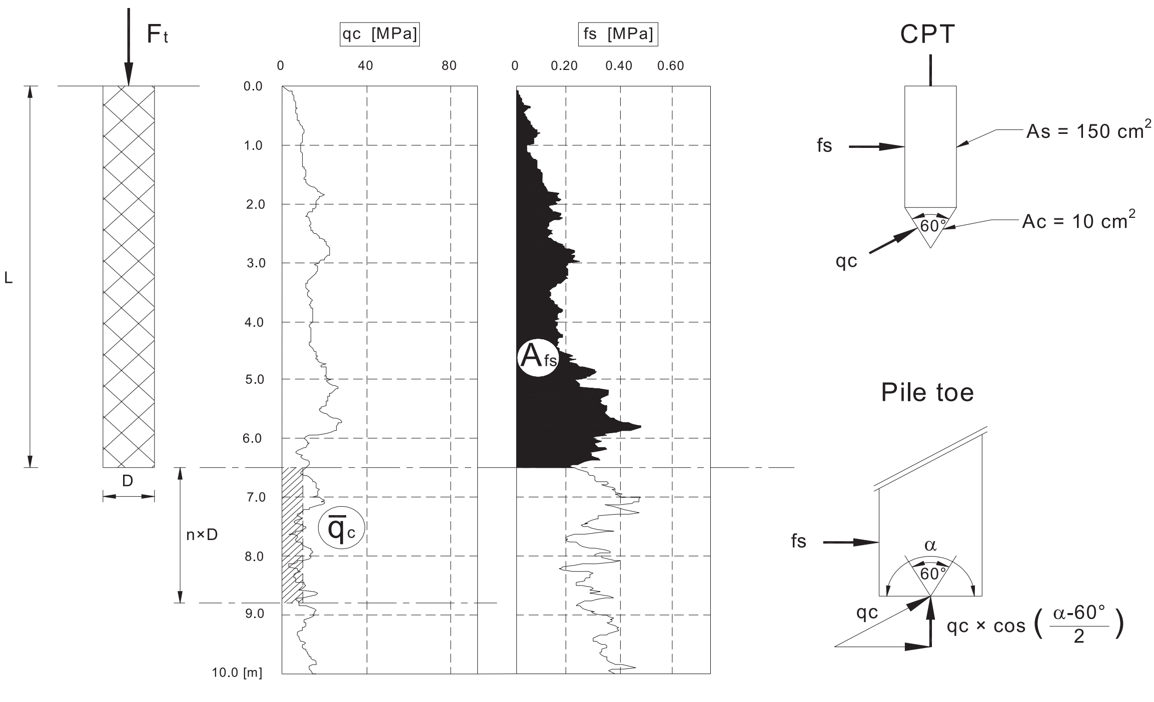

Figure 2. Concept for the estimation of the failure load of a single pile.

The first part of the formula (Q_s) depends on the total surface area of the shaft, the earth pressure acting thereon, the interactive forces between the surrounding soil and the shaft, and the technology of fabrication. These make:

$$Q_s = \beta_s \cdot U \cdot L \cdot \overline{\tau_s} = \beta_s \cdot U \cdot A_{fs} \qquad (4)$$

where:

U = length of the pile's periphery [m]
L = length of the pile [m]
A_{fs} = area of the plotted f_s curve from the CPT probe test (explained in *Fig. 2*) [MPa×m]
β_s = empiric factor with view on the applied piling technology; it expresses the shaft resistance [-]
$\overline{\tau_s}$ = average ultimate skin friction along the pile shaft [MPa].

The second part of the formula (Q_b) depends on the extension of the surface area where the pile toe rests, the specific resistance of the soil in the zone of the pile base, and on the applied piling technology. These make:

$$Q_b = \beta_b \cdot A_b \cdot \overline{q_c} \cdot \cos\left(\frac{\alpha - 60^\circ}{2}\right) \qquad (5)$$

where:

A_b = nominal plan area of the base of the pile [m^2]
β_b = empiric factor with view on the applied piling technology; it expresses the base resistance [-]

$\overline{q_c}$ = average value of the cone resistance below the pile toe (explained in Fig. 1) [MPa].
[It has been observed that the depth (n×D) below the pile toe is strongly influenced by the applied piling technology, which has to be accounted for when the average q_c value is derived].

4 ANALYSIS OF THE ULTIMATE CAPACITY OF PILES USING THE STATIC PILE LOAD TEST

The pile load tests were performed according to the standard loading procedure described in the Hungarian Standards, MSZ 15005-1:1989 and MI 04.190:1984.

All pile load tests have been carried out until the failure load was reached.

5 RESULTS OF THE COMPARISON OF THE MEASURED (STATIC PILE LOAD TEST) AND CALCULATED (CPT) ULTIMATE BEARING CAPACITIES OF PILES

The results of the recommended CPT method used to estimate the ultimate bearing capacity of the selected piles discussed in the in-situ tests section were compared with the results of the Static pile Load Tests. The findings can be seen in Table 3.

Table 3. Comparison of calculated and measured bearing capacities of piles.

Location (ap = motorway)	Pile type	Length [m]	Diameter [m]	Soil below the toe	From CPT using the formulae				Static Pile Load Tests	Differ-ence
					β_s [1]	β_b [1]	n [1]	$Q_{u, calculated}$ [kN]	$Q_{u, measured}$ [kN]	[%]
M3 ap./B 2	Franki	7,00	0,60	Gravel	1,40	1,70	3	**3 694**	**3 650**	-1%
M3 ap./B 3		5,00	0,60	Gravel				**3 176**	**3 750**	18%
M3 ap./H 29		7,00	0,60	Gravel				**2 275**	**2 350**	3%
M30 ap./1		9,50	0,60	Gravel				**4 974**	**4 550**	-9%
M30 ap./4		7,00	0,60	Sand				**5 011**	**4 375**	-13%
M3 ap./H 30		4,00	0,60	Sand				**3 243**	**3 720**	15%
M3 ap./B 9		6,50	0,60	Clay	2,40	2,70	3	**3 820**	**4 350**	14%
M3 ap./B 6		7,00	0,60	Clay				**2 190**	**2 050**	-6%
M3 ap./B 7		9,00	0,60	Clay				**3 637**	**2 980**	-18%
M3 ap./B 11		9,00	0,60	Clay				**2 809**	**3 240**	15%
M3 ap./B 13	Protective tube	23,00	1,00	Clay	0,45	0,05	1	**2 867**	**2 600**	-9%
M3 ap./B 14		17,80	1,00	Clay				**2 116**	**2 100**	-1%
M3 ap./H 32		20,60	1,00	Clay				**2 890**	**3 250**	12%
M3 ap./HB 44	CFA	15,50	0,80	Sand	0,75	0,75	2	**1 740**	**1 780**	2%
M3 ap./HB 46		14,50	0,80	Sand				**3 225**	**2 760**	-14%
M3 ap./HB 47		13,50	0,80	Sand				**2 665**	**3 050**	14%
M3 ap./H 35		14,60	0,80	Sand				**3 125**	**3 050**	-2%
M3 ap./HB 42		15,80	0,80	Fine sand				**2 160**	**1 927**	-11%
M30 ap./2		13,80	0,80	Clay				**4 023**	**3 900**	-3%

Based on the results of the performed calculations, the regression coefficient (r) for each piling technology is as follows:

For D = 60 cm diameter Franki piles: r = 0.87,

For D = 100 cm diameter piles bored in protective tubes: r = 0.84,

For D = 80 cm diameter piles bored with CFA technology: r = 0.94.

On the basis of piling technologies and pre-calculations the assumptions used and conclusions are as follows:

- In the course of the calculations the bulb diameter for the Franki piles was assumed to be equal to the trunk diameter; so the expansion of the bulb is included in the factor β_b.
- To account for the densification of the soil in the case of driven D = 60 cm diameter Franki piles in granular soils, it is recommended to use the values $\beta_s = 1.40$ and $\beta_b = 1.70$ (higher than for the $K_o = 1\text{-}\sin\varphi$ equilibrium pressure), as well as 3D zone-depth, in the calculations.
- For D = 60 cm diameter Franki piles in cohesive soils, it is recommended to use $\beta_s = 2.40$, $\beta_b = 2.70$, and 3D zone-depth.
  The values $\beta_s = 2.40$ and $\beta_b = 2.70$ are just one unit higher (because of the pore-water pressure) than in the case of granular soils.
- In the case of D = 100 cm diameter piles bored in protective tubes – presumably due to the accumulated pulverised sediment at the bottom of the hole – it is recommended to use $\beta_b = 0.05$ for the base resistance and $\beta_s = 0.45$ for the shaft resistance (lower than $K_o = 1\text{-}\sin\varphi$), as well as 1D depth-zone.
- For D = 80 cm diameter piles bored with the CFA technology, it is recommended to use $\beta_b = \beta_s = 0.75$ and 2D depth-zone.

6 SUMMARY

This study presents the evaluation of a new method in predicting the ultimate bearing capacity of different piles (Franki piles, piles bored in protective tubes and piles bored with the CFA technology) driven into different soils in Hungary.

Thirty six pile load test reports – with CPT soundings adjacent to the test pile – were collected. The prediction of pile capacity was performed for each pile; however, the statistical analysis and the evaluation of the suggested prediction method were based on the results of the nineteen piles (presented in this paper) that plunged (failed) during pile load tests.

An evaluation scheme was executed to evaluate the CPT values based on the ability to predict the measured ultimate bearing capacity. Different values (β_s, β_b and n×D) were suggested for different piling technologies for the evaluation scheme.

Based on the results of this study, the suggested formulae using the results of the CPT testing are given to predict the ultimate load bearing capacity of the piles.

While one may not expect that any calculation – carried out based on the result of either the CPT test or any other probing test – will lead in all cases straight to the determination of the exact ultimate bearing capacity of piles derived by using static loading test results, the performed study proves that more accurate approaches can be found to replace traditional static formulae in the design stage.

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