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USE OF CONE PENETRATION TEST IN PILE DESIGN

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Abstract

Modern methods of pile design often make extensive use of 'in situ' test data. Different pile capacity prediction methods were used to evaluate the axial capacity of 13 full-scale test piles. The pile load tests were performed on CFA piles in various soil conditions. The predicted behaviours of the piles are discussed and compared with the results of the pile load tests.

Keywords: cone penetration test, piezocone, pile capacity.

1. Introduction

The prediction of pile capacity is complicated by the large variety of soil types and installation procedures. Many methods have been proposed to predict the axial capacity of single piles. These methods can be divided in three main groups:

1. Full scale pile load test: This test exactly describes the piles behaviour with a load-settlement curve. At the moment this is the best method to predict the capacity of a single pile. The disadvantages of this method are: the costs of such a test are high, and it is rarely feasible in the stage of planning.
2. Calculation methods based on results of laboratory tests: The low accuracy of these methods makes their economical use difficult.
3. Calculation methods based on results of 'in-situ' tests: Among the numerous in-situ devices, the cone penetration test (CPT) and the piezocone (CPTu) represent the most versatile tools for geotechnical design. One of the earliest applications of this device was to predict a pile's axial capacity. As a model pile it is pushed into the ground and measurements are made of the resistance to penetration of the cone. Using this test the pile capacity can be predicted time- and cost-efficiently even in the stage of planning. Nevertheless the accuracy of this method will not achieve that of full scale tests, but the reliability of predictions based on CPT is improving.

2. Test Site

Most of the tests were performed at the construction of highway M3 in Hungary, but other tests were also performed at different sites in Hungary. The tested piles were CFA (continuous flight auger) piles. The length of the piles ranged from 6.20 m to 22.00 m, the diameter ranged from 0.60 m to 1.00 m. The pile load tests were performed in different soil conditions.

3. Interpretation of Pile Load Tests

The ultimate bearing capacity of the tested piles, determined by the method described in the Hungarian Code (MI-04-190), ranged from 830 kN to 3900 kN. The pile axial capacity consists of two components: end bearing load (base resistance) and side friction load (shaft resistance). At the pile load tests only total capacity of the piles is measured, so it was necessary to calculate the base resistance and the shaft resistance of the piles separately, based on the results of the performed tests. These values were determined based on detailed analysis of the test results and soil conditions.

4. Prediction of Pile Capacity Using CPT Data

The maximum bearing resistance of each single pile was predicted using the following methods:

- DIN 4014 (German Standard) method
- Bustamante and Giasenelli (1982) method (LCPC method)
- EUROCODE 7-3 method (process recommended by EC 7-3)
- ERTC3 method (De Cock, F. – Legrand C., 1997)

In all cases the pile capacity is derived from the known formula:

$$F_{\max} = F_{\max,\ \text{base}} + F_{\max,\ \text{shaft}},$$

where: $F_{\max}$ is the maximum bearing resistance of the pile;
$F_{\max,\ \text{base}}$ is the maximum base resistance;
$F_{\max,\ \text{shaft}}$ is the maximum shaft resistance;

DIN 4014 Method

The German Standard provides different methods for cohesive and non-cohesive soils. In case of non-cohesive soils the unit base and shaft resistance of a bored pile can be calculated using the following tables (*Table 1*, *Table 2*):

Table 1. Calculation of unit base resistance (DIN 4014 Non-cohesive soils)

Average cone tip resistance (q_c) [MPa]	Unit base resistance [MPa]
10	2.0
15	3.0
20	3.5
25	4.0

Table 2. Calculation of unit shaft resistance (DIN 4014 Non-cohesive soils)

Average cone tip resistance (q_c) [MPa]	Unit shaft resistance [MPa]
0	0.00
5	0.04
10	0.08
15	0.12

For the calculation of unit base resistance the average cone tip resistance is determined between the pile tip and depth of three times the pile diameter under the tip.

In case of cohesive soils the unit base resistance and the unit skin friction can be determined with an indirect method, using the following tables (*Table 3*, *Table 4*):

Table 3. Calculation of unit base resistance (DIN 4014 Non-cohesive soils)

Average undrained shear strength (c_u) [MPa]	Unit base resistance [MPa]
0.1	0.80
0.2	1.50

Similarly to the non-cohesive case average c_u is determined between the pile tip and depth of three times the pile diameter under the tip.

So the unit base and shaft resistance are calculated using the undrained shear strength (c_u) of cohesive soils. There are numerous methods to predict c_u based on CPT values, but the method used for this purpose has a great influence on the estimated pile capacity. So the final predicted pile capacity may be different using different methods for undrained shear strength prediction.

Table 4. Calculation of unit shaft resistance (DIN 4014 Non-cohesive soils)

Average undrained shear strength (c_u) [MPa]	Unit shaft resistance [MPa]
0.025	0.025
0.1	0.04
0.2	0.06

Bustamante and Giasenelli (1982) *Method*

In this case the unit base and shaft resistances of a pile are calculated directly from the cone resistance (q_c) using the following equation:

$$p_{\text{max, base}} = k_c \cdot q_{c,avg}$$
$$p_{\text{max, shaft}} = \frac{q_{c,z}}{\alpha},$$

where:

- $q_{c,avg}$ is the average value of $q_{c,z}$ between the depth of 1.5 D above and 1.5 D below the pile tip.
- $q_{c,z}$ is q_c at depth z
- k_c, α factors depending on pile- and soil type and measured cone tip resistance (q_c) (see in *Table 5*.)

Table 5. k_c and α factors (Bustamante and Giasenelli; 1982)

	q_c [kPa]	k_c	α	max p_{shaft} [kPa]
soft clay	$q_c < 1000$	0.4	30	15
moderately compact clay	$1000 < q_c < 5000$	0.35	40	80
compact to stiff clay, compact silt	$q_c > 5000$	0.45	60	80
silt and loose sand	$q_c < 5000$	0.4	60	35
moderately compact sand and gravel	$5000 < q_c < 12000$	0.4	100	120
compact to very compact sand and gravel	$q_c > 12000$	0.3	150	150

EUROCODE-7-3 method

In this method the maximum unit base and shaft resistance can be derived form the following equations:

$$p_{\text{max, base}} = \alpha_p \cdot 0.5 \cdot \left(\frac{q_{c,I,\text{ mean}} + q_{c,II,\text{ mean}}}{2} + q_{c,III,\text{ mean}} \right)$$
$$p_{\text{max, shaft}} = \alpha_s \cdot q_{c,z}$$

where:

$q_{c,I,\text{ mean}}$ is the mean of the $q_{c,I}$ values over the depth running from the pile base level to a level (critical depth) which is at least 0.7 times and at most 4 times the pile base diameter deeper. (Critical depth: where the calculated value of $p_{\text{max, base}}$ becomes a minimum)

$$q_{c,I,\text{ mean}} = \frac{1}{d_{\text{crit}}} \cdot \int_0^{d_{\text{crit}}} q_{c,I}\,\mathrm{d}z$$
$$0.7 \cdot D \leq d_{\text{crit}} \leq 4 \cdot D$$

$q_{c,II,\text{ mean}}$ is the mean of the lowest $q_{c,II}$ values over the depth going upwards from the critical depth to the pile base

$$q_{c,II,\text{ mean}} = \frac{1}{d_{\text{crit}}} \cdot \int_{d_{\text{crit}}}^{0} q_{c,II}\,\mathrm{d}z$$

$q_{c,III,\text{ mean}}$ is the mean value of the $q_{c,III}$ values over a depth interval running from the pile base level to a level of 8 times the pile base diameter higher. This procedure starts with the lowest $q_{c,II}$ value used for computation of $q_{c,II,\text{ mean}}$.

$$q_{c,III,\text{ mean}} = \frac{1}{8 \cdot D} \cdot \int_0^{-8D} q_{c,III}\,\mathrm{d}z$$

$q_{c,z}$ is q_c at depth z
α_p is the pile class factor
α_s is a factor depending on the pile class and soil conditions (*Table*6).

ERTC3 Method

This method has the same process for calculation of unit base resistance as the EC 7-3 method, but the determination of the shaft resistance is modified. It can be calculated with the same process, but α_s values are different (*Table* 7).

Table 6. Friction coefficient, α_s (EUROCODE 7-3)

Soil type	relative depth z/D	α_s
fine to coarse sand		0.006
very coarse sand		0.0045
gravel		0.003
clay/silt ($q_c \leq 1$ MPa)	$5 < z/D < 20$	0.025
clay/silt ($q_c \geq 1$ MPa)	$z/D \geq 20$	0.055
clay/silt ($q_c > 1$ MPa)	not applicable	0.035
Peat	not applicable	0

Table 7. Friction coefficient, α_s (ERTC 3)

Non-cohesive	Gravel	0.003
	Sandy gravel	0.0045
	Fine sand	0.006
	Sandy silt	0.008
	Silt	0.01
Cohesive	$q_c >$ 2500 kPa	0.015
	1500 kPa $< q_c <$ 2500 kPa	0.025
	1000 kPa $< q_c <$ 1500 kPa	0.035
	500 kPa $< q_c <$ 1000 kPa	0.045
	$q_c <$ 500 kPa	0.055

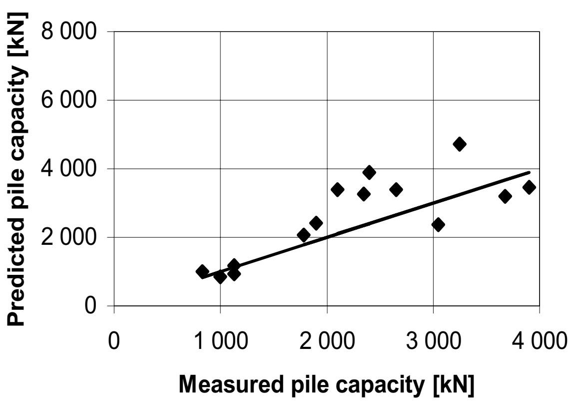

Fig. 1. DIN 4014 method

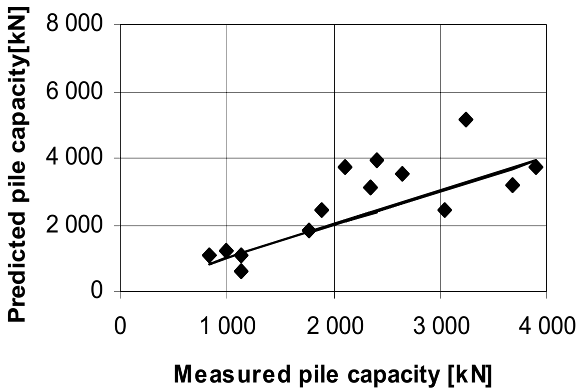

Fig. 2. LCPC method

5. Reliability of the Prediction Methods

In the following figures the results of the pile capacity predictions are shown. In each figure the predicted pile capacity values are plotted against the results of pile load tests (measured pile capacity). The continuous lines represent the perfect prediction ‘zone’.

Fig. 1 shows the results of the DIN 4014 method. As it was mentioned earlier it is an indirect method in case of cohesive soils, so the prediction method of undrained shear strength (c_u) has a serious effect on the calculated pile capacity. So it is obvious that the reliability of this method is varying with various c_u prediction methods.

LCPC (BUSTAMANTE and GIASENELLI, 1982) method provides a direct calculation process using solely the measured cone tip resistance (q_c) values. The results of this method are shown in *Fig. 2*. By comparing the two figures we can say that this process gives similarly accurate pile capacity values.

Fig. 3 shows the results of the EUROCODE 7-3 method. Among the investigated methods this one has the most complicated process for estimation of unit base resistance of a pile. This calculation process results in very accurate results, but the prediction of unit skin friction is less accurate in some cases. As it can be seen in *Fig. 3* in these particular cases the pile capacities are extremely overestimated. These piles were of large diameter, long piles in stiff clays of high plasticity and in each case the skin friction of the piles were overestimated.

The ERTC 3 method has the same process for determination of unit base resistance, but the calculation process of unit skin friction is modified. In *Fig. 4* the results of this prediction method are shown. We can see that this method gives reliable results even in cases in which the pile capacities were extremely overestimated by EUROCODE 7-3 method.

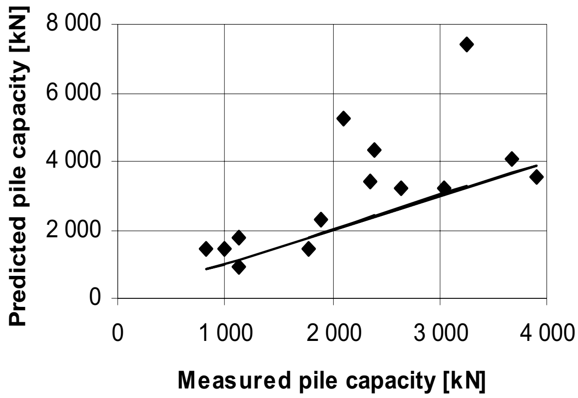

Fig. 3. EUROCODE 7-3 method

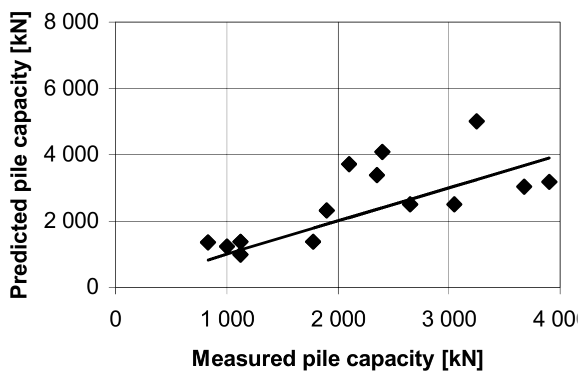

Fig. 4. ERTC 3 method

6. Summary and Conclusion

Four pile capacity methods were evaluated using CPT data for thirteen full scale axial pile load tests. The test piles were CFA piles with various geometry and were measured in various soil conditions. DIN 4014, LCPC, EUROCODE 7-3 and ERTC 3 methods were used for pile capacity predictions.

Although the number of performed tests is not enough for statistical analysis, the results of performed tests are summarized in the following table (*Table*8):

In case of the EUROCODE 7-3 method the standard deviation and average estimated value are higher than the respective values of other methods. Nevertheless these differences are caused by the overestimation of pile capacities of some piles. Without these results the EUROCODE 7-3 method gives similarly reliable results for pile capacity.

Table 8. Results of performed tests

	DIN 4014	LCPC	EC 7-3	ERTC 3
avg. value	116.0 %	119.1 %	142.3 %	120.1 %
std. deviation	29.3 %	36.5 %	51.7 %	35.3 %

It is necessary to note that the pile capacities were generally overestimated. To decide whether it is caused by the differing soil conditions or by differences in the pile installation process requires further studies. Nevertheless it is clear that these methods represent a useful tool in geotechnical design.

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