Analysis of the accuracy of CPT and leading parameters methods to determination of the pile bearing capacity

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Abstract. The paper presents a comparison of various methods used to calculate the bearing capacity of columns and the results of static loads. In the first part of the article, the methodology for the calculation of the load bearing capacities based on CPT static probes and using leading parameters are presented. Next, the investment and the hydrogeological conditions in which the columns were made are described and then subjected to test loads. The results include a summary of the dependence of the length of the columns on their load bearing capacity calculated with three methods for two different CPT sounds. A comparison of the results of calculated settlement of the column head depending on the amount of force applied to them and the results from static loads is also provided.

1. Introduction
The current rules for geotechnical design in Poland are included in Eurocode 7 (Parts 1 and 2) [1, 2]. Pile’s axial or lateral capacity can be derived on the basis of static load testing (SLT) [3-5], dynamic load testing (DLT) [6], pile driving reports [7, 8] or calculations based on numerous soil parameters derived from field investigation. Some methods enable computing separately pile base and shaft resistance [9, 10]. For the last above mentioned case of pile foundation design, the rules are limited to the introduction of certain calculation proposals and the presentation of partial factor sets. This means that the designer is not obliged to use one particular method but is allowed to make a free choice as regards its selection. This type of approach allows the use of calculation methods based on either leading parameters (method according to [14]), CPT static sounding (LCPC method [11], method according to EC7 [2]), analytical solutions [13] or the finite element method [14]. All of these methods are based on different input parameters and have been developed by different researchers in different regions of the world. Therefore, in order to verify the calculations and to build experience in order to continuously improve the calculation methods, it is necessary to verify the computations by applying static test loads of piles. The purpose of this article is to present selected calculation methods applied in Poland and to compare their results with the results of static test loads of piles performed on one of the projects located in central Poland.

2. Calculation methods
2.1. Method based on leading parameters [14]
The basic method of calculating the pile bearing capacity used in Poland has been described in [14]. It is focused on base resistances and shaft resistances fixed in tables determined in an experimental way.
depending on the type of soil and leading parameters (density index $I_D$ and liquidity index $I_L$). It was assumed that these resistances occur at depths greater than a certain critical depth $h_c$ or $h_t$ (figure 1) and for the initial diameter of the pile $D_0=0.40$ m. The impact of the pile diameter is taken into account by changing the critical depth according to formula (1).

$$h_{ci} = h_c \frac{D_i}{D_0}$$

(1)

It should also be noted that the limit resistance occurs when the head of pile reaches a displacement of 5% of its diameter. Application of proper technological ($S_p$, $S_s$), correction ($m_1$, $m_2$) and material ($\gamma_m$) coefficient allows to take into account the following aspects: applied technology, group work of piles or occurrence of weak soil.

The appearance of weak soils, understood as easily compressible soils which may have an negative impact on piles, is taken into account by introducing the "negative friction" and defining the level of interpolation $h_i$, i.e. the ordinate from which resistances will be calculated (figure 2).

Figure 1. Base resistance ($q$) and shaft resistance ($t$) depending on the depth and diameter of the drilled pile

Figure 2. Diagrams for interpretation of base resistance ($q$) and shaft resistance ($t$) for stratified soils
The design pile bearing capacity $N_t$ is described by the formula (2) as a sum of base $N_p$ and shaft $N_s$ components.

$$N_t = N_p + N_s = m_s \left( A_p \cdot \gamma_m \cdot q \cdot S_p + \sum_{i=1}^{n} A_{si} \cdot \gamma_m \cdot t_i \cdot S_{si} \right)$$

(2)

where:
- $q$ – unit base bearing capacity
- $t_i$ – unit shaft bearing capacity in the $i$-th layer
- $A_p$ – area of pile base
- $A_{si}$ – area of pile shaft in the $i$-th layer

2.2. CPTu based methods

The Cone Penetration Test (CPTu) consists of pushing a penetration rod finished with a special measurement cone into the ground [12-14]. During the test the localization of cone, resistance under the cone tip $q_c$, local skin friction on the sleeve $f_s$ and pore pressures $u_1$, $u_2$, $u_3$ are recorded. As a consequence of analogies to the work of a pile the CPTu soundings may be easily used to estimate pile bearing capacity.

As in the previous method, the maximum characteristic compressive resistance is the sum of the base bearing capacity $R_{b,k}$ and shaft bearing capacity $R_{s,k}$ according to formula (3). It differs in the fact that the unit resistance is determined directly from the sounding results not from leading parameters.

$$R_{c,k} = R_{b,k} + R_{s,k} = A_p \cdot q_{b,k} + \sum_{i=1}^{n} A_{si} \cdot q_{s,k,i}$$

(3)

where:
- $q_{b,k}$ – characteristic unit base bearing capacity
- $q_{s,k,i}$ – characteristic unit shaft bearing capacity in the $i$-th layer

The methods differ mainly in the empirical coefficients used for the determination of the unit resistance limits and the method of determining the areas $l_1$ and $l_2$ used for the calculation of the average resistance at the pile base (figure 3).

Figure 3. Impact zone designation scheme for Bustamante and EC7 method

The calculation method provided by Bustamante, also called LCPC, and the method presented in Eurocode 7 Annex D.7 [2] are applicable to most types of piles and soils. While Bustamante [12] precisely determines the values $l_1$ and $l_2$, which determine how the limit resistance at the base of a pile is averaged, these areas are not given deterministically for the method proposed in Eurocode.
There is also a difference in the way of determining the characteristic unit bearing capacities by using the coefficients $\psi_1$, $\psi_2$, $\psi_3$ for LCPC and $\alpha_p$, $\alpha_s$, $\beta_s$ for EC7 depending on the technology and type of soil.

2.2.1. **LCPC method** [12]

To determine the characteristic base and shaft bearing capacity according to Bustamante method formulas (4) to (8) may be used.

\[ q_{b,k} = \psi_1 \cdot \bar{q}_c \]  
\[ q_{s,k,i} = \frac{\bar{q}_{c;i}}{\psi_2} \quad \text{or} \quad q_{s,k,i} = \frac{f_{si}}{\psi_3} \]  
\[ q_c = \frac{1}{l_1 + l_2} \cdot \int_{h-l_1}^{h+l_2} q_c(h) dh \]  
\[ q_{c;i} = \frac{1}{\Delta h} \cdot \int_{h-i}^{h} q_c(h) dh \]  
\[ f_{si} = \frac{1}{\Delta h} \int_{h-i}^{h} f_s(h) dh \]

where: $\psi_1$, $\psi_2$, $\psi_3$ – bearing capacity coefficients
\[ \bar{q}_c \] – averaged resistance under the cone tip between $l_1$ and $l_2$
\[ \bar{q}_{c;i} \] – averaged resistance under the cone tip in the $i$-th layer of thickness $\Delta h$
\[ f_{si} \] – averaged sleeve resistance in the $i$-th layer of thickness $\Delta h$

2.2.2. **Method according to EN 1997-2:2007 Annex D.7** [2]

In order to comply with the Eurocode designations the following labels according to formulas (9) and (10) have been adopted.

\[ q_{b,k} \equiv p_{\text{max,base}} \]  
\[ q_{s,k,i} \equiv p_{\text{max,shaft}} \]

The maximum base and shaft resistance according to the method described in Annex D.7 to Eurocode 7 may be derived from the equations (11) and (12).

\[ p_{\text{max,base}} = \min \left[ 0.5 \cdot \alpha_p \cdot \beta_s \left( \frac{q_{c,I,\text{mean}} + q_{c,II,\text{mean}} + q_{c,III,\text{mean}}}{2} \right); 15 \text{MPa} \right] \]  
\[ p_{\text{max,shaft}} = \alpha_s \cdot q_{c,z,a} \]

where: $\alpha_p$, $\alpha_s$, $\beta_s$ – bearing capacity coefficients
\[ q_{c,I,\text{mean}} \], $q_{c,II,\text{mean}}$, $q_{c,III,\text{mean}}$ – averaged resistance under the cone tip in the I, II and III zone according to figure 3.
\[ q_{c,z,a} \] – the value of resistance under the cone tip on the depth of $z$.

3. **Investment description**

The analyzed object is located in central Poland. Due to its location in the city centre the area of the investment has been undergoing strong transformation over the centuries. The parcels covered by the project used to be used as farm and residential buildings, but were deconstructed.
In the vicinity of the designed building there is a multi-family residential development. The surface layer of the ground in the whole area of the investment is anthropogenic embankment deposited up to the depth of 3.9–5.0 m below the ground level. In some places there are horizontal inclusions of organic soils (mud, silts and humus-sands) of different thickness. Ground water stabilizes at a depth of 4.8–6.6 m below the ground level. A five-storey residential building with a utility function and an underground garage is designed. The building has been placed on a foundation plate about 3.5 m below the ground level.

Due to the occurrence of weak and compressible soil layers it is not possible to place the building directly on the ground. In order to fulfil the ultimate and serviceability limit states it is necessary to make the ground stronger. Vertical concrete inclusions in the form of Full Displacement Piles of 410 mm diameter (FDP 410) have been designed. The columns are designed to transfer the loads carried by the building and minimize settlement.

4. Results of the analysis

Figure 4. presents the results of the calculation of pile bearing capacity in the form of the dependence of limit force on length of the pile. The computations were carried out using the methods mentioned before for two CPT probes (LCPC, EN-1997) and their corresponding interpretations in the range of leading parameters (PN-83/B). The CPT based methods computation were performed using GEO5 Pile CPT software.

![Figure 4. CPT1 and CPT2 sounds (left), diagram of the pile bearing capacity dependence on length of the pile (right)](image)

The results of the calculations for the CPT1 probe are in the range of 300–500 kN at the depth of 3.5 m and 550–1000 kN at the depth of 10.0 m. For CPT2 the limits are 380–450 kN for a 3.5 m column and 800–1200 kN for a 10.0 m column. In the range 3.5–5.5 m the behaviour of the individual curves does not differ. For the depth of 5.5–6.0 m in case of CPT2 there is a noticeable increase in load capacity according to PN-83/B, which does not occur in CPT based methods. It is worth to note that the method according to EN-1997 starts to detect a much weaker layer visible on CPT2 at the depth of 7.5 m already at 6.0 m of the column length which the LCPC method does not observe until the pile length of 7.0–7.5 m.
The leading parameter method has no mechanism to detect weakening or strengthening of the load-bearing capacity as a result of moving from one layer to another. The obtained graphs are of a rapid change character, while the methods according to CPT probing give results that pass smoothly between individual soil layers. In case of detecting a stronger layer of small thickness as for CPT1 at the depth of 8.0–9.0 m in LCPC method and in the PN-83/B method an increase of bearing capacity is visible. The method proposed in EN-1997 does not record an increase in bearing capacity due to a larger range of averaging zones.

Two static load tests [11] on 7.65 m and 7.10 m long columns were carried out. These tests involved gradual static loading of the columns, i.e. after each increase of the load it was necessary to wait for the displacement of the column head to be stabilized. The results of the tests are presented in the form of strain-force graphs compared to the calculation results of the CPT based methods in figure 5 and figure 6. Computation of displacement for PN-83/B was not carried out due to the high uncertainty of the results associated with this method.

![Figure 5](image1.png)

**Figure 5.** Results of test load 1 and calculations for 7.65 m length column

![Figure 6](image2.png)

**Figure 6.** Results of test load 2 and calculations for 7.10 m length column

The tests resulted in the following load bearing capacity values for a displacement of 5% and 10% of the column diameter respectively (1) for a 7.65 m column: 750 kN and 820 kN (figure 5); (2) for a 7.1 m column: 675 kN and 750 kN (figure 6).

Load bearing capacities obtained with method based on leading parameters should correspond to the test results received for displacement equal to 5% of the column diameter. The values of limit forces for 7.1 m and 7.65 m length of columns are respectively for (1) CPT1: 815 N and 770 kN, (2) CPT2: 450 kN and 650 kN. As it may be seen, the results of computation in the first case differ from the test no more than 10% but in the second case underestimation of load bearing capacity may have reached even 50%.
5. Conclusions
The accuracy of recognition of the ground is a crucial element in the design of the foundation of an object. The above example correctly illustrates the influence of the substrate layer detected during CPT probing on the calculation results of the various methods. When designing the soil improvement, it should be taken into account how the obtained results may correspond to the actual load capacities received from the test loads. A fact worth remembering in the above analysis is the range of influence zones taken into account in the calculation of the pile base bearing capacity. The most risky method in this respect seems to be the PN-83/B, which is not able to show the effect of horizontal inclusions on the column bearing capacity until its base is located in this layer. This implies the necessity for the designer to be more careful, i.e. to keep the appropriate lengths of fixing in the strata and distance from the bottom of the layer in order to actually provide the calculated load-bearing capacity. On the other hand, the method indicated in EN 1997, when considering impact zones with large thicknesses, is able to ignore the positive influence of inclusions of stronger layers. In the case under consideration, this method seems to give highly conservative results, while the LCPC method gives a slightly higher results in comparison to the tests.

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