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ASSESSMENT OF DIFFERENT METHODS FOR DESIGNING BORED PILES

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Abstract. This work concerns the design of bored piles on the site of Elektrenai Power Plant in Lithuania. Equipment for supporting bored pile foundation in the power plant consists of a gas turbine, a steam turbine and a generator. The foundation not only endures high loads but also sustains a strong dynamic impact due to the vibration of equipment in the power plant under working conditions. A solution to the pilling problem was adopted for the following reasons: i) the insufficient capacity of soil to support acute stresses; ii) high requirements of slab settlements and bearing capacity with regard to the main equipment used in the power plant.

The objective of this work is to assess the methods used for estimating immediate settlement and carrying capacity of the pile considering cone penetration tests conducted in Elektrenai power plant. For settlement estimation, four methods, including Bowles (Bowles 1997) and Schmertmann methods (Schmertmann 1978) as well as the methods described in EN 1997-2 and NEN 6743 (NEN 6743:1991/A1:1997) were employed. A carrying capacity of the pile was evaluated with the help of direct methods that utilize data on the cone penetration test (CPT). The following direct methods such as the Schmertmann method (Schmertmann 1978), the de Ruiter and Beringen method (de Ruiter, Beringen 1979), the Bustamante method (LCPC) (Bustamante, Gianeeselli 1982) and the methods described in EN 1997-2 and NEN 6743 (NEN 6743:1991/A1:1997) were applied.

Pilling foundation was evaluated performing immediate settlement analysis and included the examination of soil data received from cone and dynamic penetration tests, boreholes and laboratory tests. Soil properties were estimated taking into account investigation into the site of Elektrenai power plant and a soil exploration program developed according to Lithuanian standards. Pile settlement analysis showed that settlement value made 13.6 mm (pile toe settlement) and the settlement value of an elastic deformation of the pile from vertical compressive loads was 2.3 mm, for the most conservative situation. For such structure, foundation settlement should not exceed 16 mm. Elektrenai power plant has high reliability requirements, and therefore the pile having the diameter of 800 mm with a pile length of 27 m was adopted to endure overall loads.

Keywords: design of bored piles, foundation for the gas and steam turbine, cone penetration test, pile bearing capacity, pile settlement analysis.

1. Introduction

Deep foundations such as bored piles are relatively long and generally slender structural foundation members that transmit superstructure loads to deep soil layers. In geotechnical engineering, deep foundations usually serve as the foundations when soil conditions are not suitable for the use of shallow foundations. The behaviour of the pile depends on many different factors, including pile characteristics, soil conditions and properties, installation method and loading conditions. Pile foundations have to be proportioned both to interface with soil at a safe stress level and to limit settlements to an acceptable amount.

The prediction of pile load carrying capacity can be achieved using different methods such as analysis methods (static and dynamic analysis) and pile load tests. In this work, pile load carrying capacity was evaluated applying the static analysis method based on soil properties acquired from laboratory tests and static analysis utilizing the results of in situ tests, e.g. a cone penetration test (CPT). Bored piles are the most common type of nondisplacement piles the load carrying capacity of which consists of two components: shaft tangential resistance and base compressive resistance. The shaft resistance of the piles, in most cases, is fully mobilized before maximum base resistance is reached (Frank 2006). After the full mobilization of shaft resistance, any increment of axial load is transferred fully to the base. As shaft resistance is mobilized early in the loading process, the determination of base resistance is the key element in pile design (Lee, Salgado 1999).

The prediction for pile settlement can be achieved as a sum of pile heel settlement and an elastic deformation of the pile. Settlement analysis plays an important role in building foundation, even though only few modern buildings collapse from excessive settlements, which is uncommon for partial collapse or localized failure in a structural member to occur (Kempfert, Gebreselassie 2006). Excessive settlement and differential movement can cause distortion and cracking in structures (Salgado *et al.* 2007) especiallyfor rotary machines which are particularly sensitive to bearing misalignment. In other words, a suitable method of state-of-the-art design under regional conditions may greatly reduce the risk factor of settlement problems without unduly raising foundation costs.

The objective of this work consists of four parts:

1. To assess the load carrying capacity of the pile considering the basic condition for an ultimate limit stage (EN 1997-2).
2. To assess the load carrying capacity of the pile using direct methods utilizing CPT data in Elektrenai power plant. The load carrying capacity of the pile was evaluated employing the Schmertmann method (Schmertmann 1978), the de Ruiter and Beringen method (de Ruiter, Beringen 1979), the Bustamante method (LCPC) (Bustamante, Gianeeselli 1982) and the methods described in NEN 6743 (NEN 6743:1991/A1:1997).
3. To assess immediate settlement employing the analytical Bowles method (Bowles 1997).
4. To assess immediate settlement employing four methods, including Schmertmann (Schmertmann 1978), EN 1997-2, Bustamante (LCPC) and the methods described in NEN 6743 (NEN 6743:1991/A1:1997).

In this work, deep pile foundation was designed to sustain loads from gas and steam turbine equipment on the site of Elektrenai power plant in Lithuania. When designing pile foundation, the required length of the pile was estimated based on the load from superstructure, allowable stress in the material of the pile and in situ soil properties that were estimated taking into account site investigation and soil exploration program according to Lithuanian regulations. Investigation data were based on cone penetration and dynamic penetration tests, boreholes, excavations as well as on soil and laboratory investigations. Based on the obtained data, four geological layers were generalized and applied for designing pile foundation.

Pile settlement analysis disclosed that the total settlement value was 16 mm, including 2 mm settlements of an elastic deformation of the pile from vertical compressive loads. For such structure, foundation settlement should not be more than <2%D where D is the diameter of the pile. The introduced settlement criteria were taken according to equipment settlement guidelines (General Electric Design Basis Document, Volume 1, 2008), which means that the pile should be working within the limit of the mobilization of its shaft resistance. It was found that for the pile of the diameter of 800 mm, 27 m was necessary length sufficient to handle overall load.

2. Piled Foundation Considerations

Piled foundation was chosen due to two different reasons:

- The capacity of soil to undergo great stresses, i.e. the bearing capacity of soils represents the ability of soil to safely carry pressure placed on soil by piles without undergoing shear failure with accompanying large settlements.
- Special requirements for the main equipment for settlement bearing capacity. The equipment consisted of a gas turbine, a steam turbine and a generator. The generator is coupled to the gas turbine through rigid coupling and is connected to the steam turbine by flexible coupling. The equipment induced high loads, which, in turn, induced great stresses on the foundation. The combined unit of the gas turbine, the generator and steam are found on a single slab foundation. It has to provide adequate resistance and comportment to all static and dynamic equipment conditions.

As the main purpose of the foundation is to receive loads from equipment and to transmit them to the piles, so that settlement and dynamic criteria should be satisfied. According to the analyses of stresses induced by loads, gas and steam turbine equipment required deep pile foundation.

For designing deep pile foundation, the required length of the pile was estimated from the load of power plant equipment, stresses in the pile material, and the soil properties. It was based on the following steps (Gabrielaitis, Papinigis 2010):

1. Soil properties were determined in light of site investigation and the soil exploration program according to Lithuanian regulations.
2. Superstructure loads were obtained from the manufacturer of the gas and steam turbine (General Electric) and included a design verification load of 2500 kN and a service working load of 2239 kN.
3. The bored cast-in-place piles having the diameter of 800 mm were adopted and rested on the very dense sandy bed. Based on data from the previous two steps, the estimation of pile length was performed along with pile carrying capacity and settlements.

These steps are described in the following sections.

3. Physical and Mechanical Properties of Soil

Soil properties were determined from site investigation and the soil exploration program on the site of Elektrenai power plant, Lithuania. Geological investigation involved boreholes (BH), cone and dynamic penetration tests (PT) and trial pits (TP). Totally, 8 boreholes of the depth of 30 m and other 45m-deep were drilled. Soil samples were taken from trial pits in order to determine granulometric composition, plasticity and Proctor density. 21 tests on the cone penetration (CPT) of the depth of up to 25–35 m were carried out. At 4 points below 15 m, precise measurements of pore pressure have been carried out (CPTu). 16 dynamic penetration (DPSH) tests were performed in the depth of up to 15 m. XIII engineering geological layers (EGL) were determined in the research area and based on investigation data about CPT, DPSH of boreholes, excavations, soil as well as on laboratory investigations.

The surface of the investigation site was levelled and the major part of the area was replaced with manmade soil (tplIV) consisting of silty sand (SU, SUo), low plasticity clay (TL), intermediate plasticity clay (TM), silty clay (TU) and gravel sand (GU). The thickness of the manmade soil layer ranges from 0.5 m to 2.20 m with the altitudes ranging from 96.0 m to 97.9 m. The depth of limnoglacial sediments ranges from 13.20 m to 15.80 m. The altitudes of the layer sole ranges from 82.14 m to 84.93 m of the altitude. Below, silty sand (SU, SUo) was present to 67.7 m of the altitude.

In view of investigation into engineering geological layers, four geological layers were generalized:

1. medium to firm clay sediment, TU, TL, TM (the depth of the layer is up to 15 m from the surface);
2. medium to coarse silty sand, dense (the depth of the layer is up to 19 m from the surface);
3. medium to coarse silty sand, medium dense (the depth of the layer is up to 25 m from the surface);
4. medium to coarse silty sand, very dense (the depth of the layer is up to 30 m from the surface).

These four layers were used for designing and calculating piling foundation (Gabrielaitis, Papinigis 2010).

4. Assessment of the Carrying Capacity of the Bored Pile

4.1. Pile Carrying Capacity Employing the Ultimate Limit Stage (EN 1997-2)

Pile carrying capacity was evaluated taking into account the basic condition for the ultimate limit stage, which is:

$$F_{c,d} \leq R_{c,d}, \tag{1}$$

where $F_{c,d}$ is the design load of the ultimate limit stage normal to the foundation and $R_{c,d}$ is the design bearing resistance of the foundation against loads normal to it. $F_{c,d}$ includes the weight of the foundation and of any backfill material placed on the top of it. Earth pressure on the structural elements above the foundation level is geotechnical action and is included in $F_{c,d}$ where relevant.

The basic inequality $F_{c,d} \leq R_{c,d}$ has to be checked for recommended partial safety factors in persistent and transient situations (Eurocode 7). In our case, the value of $F_{c,d}$ was calculated and accepted to be equal to 2500 kN.

The value of $R_{c,d}$ may be calculated using analytical or semi-empirical methods. The concept of a separate evaluation of shaft friction and base resistance forms the basis of all 'static' calculations of pile carrying capacity. The basic equation is

$$R_{c,d} = R_{b,d} + R_{s,d}\,, \tag{2}$$

where $R_{c,d}$ represents the total load carried at the pile head, which is the summation of base and shaft resistance which in turn, is the multiplication of base and shaft areas, A_b and A_s, by the respective unit of the characteristic value of resistances $q_{b,k}$ and $q_{si,k}$ (Tomlinson 2001):

$$R_{c,d} = A_b \cdot q_{b,k} + \sum_{i=1}^{n} A_{si} \cdot q_{si,k}\,, \tag{3}$$

where i is the soil layer index; n is number of layers crossed by the pile.

Design compressive resistance $R_{c,d}$ is estimated from Equation 4:

$$R_{c,d} = \frac{N_q \cdot \sigma'_v \cdot A_b}{\xi \cdot \gamma_b} + \frac{\sum_{i=1}^{n} K_s \cdot \sigma'_{vo} \cdot \tan\delta' \cdot A_{si}}{\xi \cdot \gamma_s}. \tag{4}$$

The first term on the right of the equation (4) represents base resistance divided by partial safety factors (ξ, γ). Base resistance is described by bearing capacity factor N_q and overburden earth pressure σ'_v. Bearing capacity factor N_q is related to the peak angle of the shearing resistance ϕ' of soil and the slenderness ration (L/D) of the pile. The values of the effective angle of shearing resistance ϕ' is required to obtain factor N_q (Peck *et al.* 1974). In our case, ϕ' is derived from SPT results obtained from the DPSH test and is described in Table 1. Herein, to apply DPSH data in Equation 4, N_{20} DPSH data were converted to N_{30} SPT values (Spagnoli 2007) where N is blow count recorded in a standard penetration test. Although SPT is not considered as a refined and completely reliable method of investigation, N values give useful information with regard to the consistency of cohesive soils and the relative density of cohesionless soils. The accepted values of shearing resistance ϕ' for the active zone are presented in Table 1. Overburden earth pressure σ'_v, is given in Table 2.

The second term on the right of Equation 4 represents shaft ultimate resistance divided by partial safety factors (ξ, γ). Shaft ultimate resistance $R_{s,d}$ is described by the coefficient of horizontal earth pressure K_s, the average of effective overburden earth pressure over the depth of soil layer σ'_{vo} and the value of δ' which is the characteristic or average value of the angle of friction between the pile and soil. The angle of friction δ' between the pile surface and soil is related to the average effective angle of shearing resistance ϕ' over the length of the pile shaft (Tomlinson 2001). The coefficient of horizontal earth pressure K_s is not constant over the depth of the pile shaft and depends on the relative density of soil, the state of soil consolidation and the volume displacement (L/D) of soil by the pile. The situation was estimated considering geological investigation and is presented in Table 3. The obtained values are depicted in Table 3. The estimation of shaft ultimate resistance *layer by layer* is introduced in Table 4.

Table 1. Simplified subsoil structure

Layer	Level (m)	Lithology	γ_s (kN/m^3)	N_{20}DPSH	N_{30}SPT	ϕ'
1	98–83	Clayey deposit, medium to firm consistency	19.5	–	–	10°(*)
2	83–79	Medium to coarse silty sands, dense	26.0	25–30	45–54	32°
3	79–73	Medium to coarse silty sands, medium dense	26.0	18–22	32–40	30°
4	73–69↓	Medium to coarse silty sans, dense to very dense	26.0	26–50	47	34°

Note: (*) obtained from direct shear testing

Table 2. Overburden earth pressure at the pile toe

Layer	Level (m)	Lithology	γ' (kN/m^3)	Thickness (m)	σ' at level bottom (kPa)
1a	98–95	Clayey deposit, medium to firm consistency ↑GWL	19.5	3	58.5
1b	95–83	Clayey deposit, medium to firm consistency ↓GWL	5.2	12	120.9
2	83–79	Medium to coarse slightly silty sands, dense	12.2	4	169.7
3	79–73	Medium to coarse slightly silty sands, medium dense	11.2	6	236.9
4	73–69	Medium to coarse slightly silty sans, dense to very dense	12.2	4	285.7

Table 3. The coefficient of horizontal soil stress (K_s) (K_0 – the coefficient of earth pressure at rest, $K_0 = 1 - \sin\phi'$)

Layer	Level (m)	Lithology	ϕ'	K_0	$K_s = 0.85\,K_0$
1	98–83	Clayey deposit, medium to firm consistency	10°	–	–
2	83–79	Medium to coarse slightly silty sands, dense	32°	0.47	0.40
3	79–73	Medium to coarse slightly silty sands, medium dense	30°	0.50	0.43
4	73–69	Medium to coarse slightly silty sans, dense to very dense	34°	0.44	0.37

Table 4. Shaft ultimate resistance $R_{s,k}$

Layer	Level (m)	ϕ'	K_s	Average σ'_{vo} at each level (kPa)	A_s (m^2)	R_s (kN)
1	98–83	10°	–	–	–	
2	83–79	32°	0.40	145.3	10.05	191
3	79–73	30°	0.43	203.3	15.08	381
4	73–69	34°	0.37	261.3	10.05	328
					$\sum R_{s,k}$	900

Equation 4 includes partial resistance factors (ξ and γ) derived from Eurocode 7 in Table 5 (Frank 2006).

Shaft ultimate resistance $R_{s,k}$ is presented in Table 4.

Safety factors are summarized in Table 5.

Table 5. Safety factors applied in Eq. 4

Resistance	ξ(for $n = 1$)	γ (for $R = 4$)	Applied ξ, γ
Base	1.4	1.6	2.24
Shaft	1.4	1.3	1.82

For the worst site-wide situation, shaft friction resistance $R_{s,d}$ was estimated to be equal to 495 kN and base resistance $R_{b,d}$ was estimated to be equal to 4488 kN. The sum of shaft and base resistances $R_{c,d}$ was equal to 4983 kN. This value satisfied Equation 1 where the sum of shaft and base resistances $R_{c,d}$ should be larger than (or equal to) design axial compression load on the single pile at ultimate limit state $F_{c,d}$.

4.2. Pile Carrying Capacity Employing CPT Data

Direct CPT methods evaluate pile toe resistance (q_t) to measured cone tip resistance (q_c) by averaging cone tip resistance over the assumed influence zone. Pile shaft resistance (f) is either evaluated taking into account measured sleeve friction (f_s) or measured cone tip resistance (q_c).

The **Schmertmann method** (Schmertmann 1978) determines maximum pressure at pile toe $p_{max,toe}$ applying Equation 5:

$$p_{max,toe} = \alpha_p \cdot q_t. \tag{5}$$

Maximum pressure in the pile toe in Equation 5 depends on equivalent average cone tip resistance $q_{c,eq}$ and pile toe coefficient α_p, which identifies the type of the pile. Its values are calculated based on the values of cone tip resistance q_c. For bored piles, α_p is equal to 0.5 (ENV 1997-2).

Modified equivalent average cone tip resistance $q_{c,eq}$ is determined by Equation 6:

$$q_t = \left(\frac{q_{c,1} + q_{c,2}}{2}\right), \tag{6}$$

where $q_{c,1}$ is the minimum of the average cone tip resistances of the zones ranging from 0.7D *to* 4D below the pile tip (where D is pile diameter) and $q_{c,2}$ is the average of minimum cone tip resistances over distance 8D above the pile tip. Schmertmann suggested an upper limit of 15 MPa for pile toe bearing capacity.

According to the Schmertmann's method, maximum pile shaft resistance ($p_{max,shaft}$) is calculated using Equation 7:

$$p_{max,shaft} = K\left[\sum_{y=0}^{8D} \frac{y}{8D} f^*{}_s A_s + \sum_{y=8D}^{L} f_s^* A_s\right], \tag{7}$$

where K – the correction factor for sand equal to the ratio of unit pile shaft resistance and unit penetrometer sleeve local friction; y – depth at which side resistance is calculated; L – pile length; D – embedded pile length; f_s^* – the mean value of penetrometer sleeve local friction f_s in the interval given by bracket subscript; A_s – the area of pile shaft surface in the given interval.

Maximum pile bearing capacity, which is a sum of q_t and f magnitudes, according to the Schmertmann method, was estimated to be equal to 6085 kN.

The **de Ruiter and Beringen method** (de Ruiter, Beringen 1979) is based on experience gained in the North Sea. This method is also known as the European method and uses different procedures for clay and sand.

In sand, pile toe resistance (q_t) is calculated similarly to the Schmertmann method.

Pile shaft resistance (f) for each soil layer along the pile shaft is determined by Equation 8:

$$f = min\begin{cases} f_s \\ \dfrac{q_c(\text{side})\ \text{compression}}{300} \\ 120kPa \end{cases}. \quad (8)$$

Maximum pile bearing capacity, according to the de Ruiter and Beringen method, was estimated to be equal to 5457 kN.

The **Bustamante method (LCPC)** (Bustamante, Gianeeselli 1982) is based on the analysis of 197 pile load tests with a variety of pile types and soil conditions. In this method, both pile toe resistance (q_t) and shaft resistance (f) of the pile are obtained from cone tip resistance (q_c). Sleeve friction (f_s) is not used. Pile toe resistance is determined with reference to Equation 9:

$$q_t = k_b \cdot q_{c,eq}, \quad (9)$$

where k_b is an empirical bearing capacity factor that varies from 0.15 to 0.60 depending on the soil type and pile installation procedure. For bored piles, the value of k_b is 0.15.

Pile shaft resistance (f) in each soil layer is estimated from the equivalent cone tip resistance ($q_{c,eq}$) of the soil layer, soil type, pile type and installation procedure (Bustamante, Gianeeselli 1982).

Maximum pile bearing capacity, according to Bustamante and Gianeselli method, was estimated to be equal to 3535 kN.

4.3. Analysis of Pile Bearing Capacity Employing Program Pile CPT (Geo5)

The paper is also aimed at evaluating software Pile CPT (Geo5) in order to verify its capabilities for practical local usage. Program Pile CPT serves to verify the bearing capacity and settlement of a single pile. Pile bearing capacity analysis was carried out according to such standards and approaches as EN 1997-2, NEN 6743, LCPC (Bustamante) and Schmertmann. For all methods, essential input parameters are dimensionless coefficients adjusting the magnitude of bearing capacity and shaft friction respectively. During analysis, a partial factor in base resistance and that in shaft resistance were accepted according to Table 5. Pile toe and shaft friction coefficients are automatically calculated based on the pile type and surrounding soils.

The results of all pile bearing capacity analysis are estimated applying different methods and displayed in Table 6. The pile length of 27 meters was used only for calculation using the ultimate limit stage method (EN 1997-1). In all other methods, due to the depth of CPT influence zone, pile length was accepted to be equal to 20 meters.

Table 6. A comparison of pile bearing capacity results obtained using different methods

Method	Pile length, m	Pile bearing capacity, kN	Pile bearing capacity, kN Pile CPT (Geo5) software
Ultimate limit stage (EN 1997-2)	27	4983	4303.48
Schmertmann	20	6085	3721.41
de Ruiter and Beringen	20	5457	–
Bustamante method (LCPC)	20	3535	6080.89
NEN 6743	20	–	6050.49

5. Assessment of Immediate Settlement of the Bored Pile

5.1. Assessment of Pile Immediate Settlement Employing the Analytical Method

Total settlement can be assessed (Bowles 1997) as the sum of axial and point settlement for conservative end-bearing behaviour considering a low or negligible contribution to shaft resistance:

$$H_p = \frac{P \cdot L}{A \cdot E_p} + q \cdot D \cdot \frac{1-\mu^2}{E_s} \cdot mI_s \cdot I_F \cdot F_1. \quad (10)$$

The first term (before the sum sign) on the right of the equation (10) describes average pile axial settlement for pile length L, average cross-section area A and an elastic modulus of pile E_p. Length L is estimated to be 67% and 100% of the total pile length, taking 100% at clayey part and 75% at embedment sand. It is equal to 16.8 m. Elastic pile modulus E_p is determined according to cylinder compressive strength f_{ck} (for f_{ck} = 30 MPa, E_p = 32.000 MPa). Maximum applied load at pile head P is equal to the service working load of P = 2239 kN.

The second term in the equation (10) describes point settlement that depends on pile load q representing pile bearing pressure at the point. It is equal to input load divided by A_p, i.e. 4450 kPa. The stress-strain modulus of soil below pile point E_s is obtained from dense and very dense sands with $N_{20} > 30 \rightarrow N_{30} > 50$ and equals $E_s > 100$ MPa. The Poisson ratio of sand soil μ equals 0.3 while shape factor mI_s equals 1.0. Embedment factor I_F has the value of 0.50 because the ratio of pile length L and diameter D ratio is higher than 5. Reduction factor F_1 was set to 0.75, since point bearing and considering some skin resistance.

According to the equation (10) the total value of settlement H_p was estimated to be equal to 16 mm. This value could be considered as maximum, is obtained from the conservative side and based on the end-bearing behaviour of the pile.

Pile settlement analysis showed that the total expected maximum settlement value was 16 mm. It includes 2 mm settlement of pile deformation from vertical compressive loads. For such structure, foundation settlement should not be more than 2% of the pile diameter. For the pile of 800 mm, foundation settlement should not be more than 16 mm. Calculation shows that for the pile of the diameter of 800 mm, necessary length makes 27 m, which is sufficient to endure overall load.

5.2. Analysis of Pile Immediate Settlement Employing Program Pile CPT (Geo5)

In program Pile CPT (Geo5), the magnitude of pile head settlement w_d is calculated with reference to Equation 11:

$$w_d = w_{toe,d} + w_{el,d}, \tag{11}$$

where $w_{toe,d}$ is pile toe settlement due to acting forces consisting of two components: pile toe settlement due to force acting at the toe ($w_{toe,d,1}$) and pile toe settlement due to force acting on the shaft ($w_{toe,d,2}$); $w_{el,d}$ is pile settlement due to elastic compression. The magnitudes of settlements $w_{toe,d,1}$ and $w_{toe,d,2}$ are determined from built-in graphs according to the NEN 6743 standard, which allows to determine:

1. pile settlement due to toe vertical force (pile settlement in the percentage of the equivalent pile diameter plotted as a function of toe vertical force given in the percentage of maximum toe resistance q_t);
2. pile settlement due to shaft force (pile settlement in mm is plotted as a function of shaft force given in the percentage of maximum shaft resistance f).

Pile settlement analysis was carried out following standards and approaches EN 1997-2, NEN 6743, LCPC (Bustamante) and Schmertmann. The results of the made comparison were obtained employing different methods and are presented in Table 7.

The settlement analysis of program Pile CPT is performed according to the suggested standard NEN 6743 where pile toe and pile shaft forces are given and partial factors for base resistance and shaft resistance were involved in calculations. These factors were also accepted in line with Table 5.

Table 7. A comparison of pile settlement results obtained using different methods

Method	Pile length, m	Immediate settlement, mm
Bowles	27	15.9
Schmertmann	20	13.5
EN 1997-2	20	10
Bustamante method (LCPC)	20	6.8
NEN 6743 Limit state 1B	20	8
NEN 6743 Limit state 2	20	7.4

6. Conclusions

To evaluate the results of pile carrying capacity in consideration of the basic condition for the ultimate limit stage (EN 1997-2):

1. Schmertmann, LCPC (Bustamante) and de Ruiter and Beringen methods were used without applying partial factors (for more details, see Section 4.2);
2. Schmertmann, LCPC (Bustamante), de Ruiter and Beringen, EN 1997-2 and NEN 6743 methods were used in program Pile CPT (Geo5) applying partial factors (for more details, see Section 4.3).

The obtained results presented in Table 6 demonstrate reliable calculations carried out employing different methods. If calculations using Pile CPT program

(Geo5) pointed to the magnitudes of partial factors were equal to 1, the result of pile bearing capacity would be similar to the performed calculations applying the analytical method described in Section 4.1. For example, the results of the Schmertmann method shows a 5955.90 kN value of pile bearing capacity if partial factors are eliminated in Pile CPT program.

Pile settlement analysis was performed employing five most widely used standards and approaches: EN 1997-2, NEN 6743, LCPC (Bustamante), de Ruiter and Beringen, Schmertmann and Bowles. Settlement values are similar to all applied methods. The largest value of bored pile settlement was obtained employing the analytical Bowles method. The analysis of pile settlement employing different methods for using Pile CPT Program (Geo5) shows reliable results. Therefore, it can be confirmed that NEN 6743, Bowles and Schmertmann methods can be used for assessing the settlements of bored piles in our region.

Nomenclature

Notation:

A pile cross section area (m^2);
A_b pile base area (m^2);
A_{si} pile shaft area (m^2);
D pile diameter (m);
E_p elastic modulus of the pile (MPa);
E_s stress-strain modulus of the soil (MPa);
F_1 reduction factor (–);
I_F embedment factor (–);
K_s coefficient of horizontal soil stress;
K correction factor;
k_b empirical bearing capacity factor;
L pile length (m);
mI_s shape factor (–);
N_q bearing capacity factor (–);
P maximum applied load at pile head (kN);
$p_{max,toe}$ maximum pressure at pile toe (kPa);
$p_{max,shaft}$ maximum pile shaft friction (kPa);
q_t pile toe resistance (kPa);
$q_{c,eg}$ equivalent average cone tip resistance (kPa);
$q_{c,1}$, $q_{c,2}$ mean values of cone tip resistances (kPa);
s pile shape coefficient (–);
$F_{c,d}$ design axial compression load on a single pile at the ULS (kN);
f pile shaft resistance (kPa);
$R_{c,d}$ design value of compressive ground resistance of a single pile at ULS (kN).
w_d pile head settlement (mm);
$w_{toe,d}$ pile toe settlement due to acting forces (mm);
$w_{el,d}$ pile settlement due to elastic compression (mm).

Greek symbols:

α_p pile type factor (–);
α_s haft friction coefficient (–);
β expanded pile toe coefficient (–);
μ poisson ratio for soil;
σ'_{vok} effective overburden earth pressure (kPa);
φ' shearing resistance (°).

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SKIRTINGŲ METODŲ VERTINIMAS PROJEKTUOJANT GRĘŽTINIUS PAMATUS

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Santrauka. Straipsnyje nagrinėjami įvairūs skaičiavimo metodai, skirti gręžtiniams pamatams projektuoti. Lietuvos elektrinės teritorijoje projektuojant kombinuoto ciklo dujų turbinos bloko pamatus, dėl ypatingų statinio reikalavimų polių nuosėdžiams bei laikomajai gebai buvo parinkti gręžtiniai poliai. Pagrindinis darbo tikslas – įvertinti gręžtinių polių nuosėdžių ir laikomosios gebos skačiavimus pagal statinio zondavimo duomenis bei analitinius sprendimus, taikant skirtingus skaičiavimo metodus. Polio nuosėdžiams ir laikomajai gebai vertinti buvo taikyti šie metodai bei standartai: Bowles, Schmertmann, de Ruiter ir Beringen, Bustamante (LCPC), EN 1997-2 ir NEN 6743 standartų metodika. Remiantis statinio ir dinaminio zondavimo bei laboratorinių tyrinių duomenimis, suminio nuosėdžio reikšmė buvo gauta lygi 16 mm, iš jų 2 mm sudaro nuo vertikaliosios apkrovos gniuždomo polio deformacija. Tokio tipo statinių pagrindo nuosėdis negali būti didesnis nei 2 % polio skersmens. Atlikti skaičiavimai parodė, kad 800 mm skersmens ir 27 m ilgio polio visiškai pakanka statinėms ir dinaminėms apkrovoms atlaikyti.

Reikšminiai žodžiai: statinis zondavimas, gręžtinių polių laikomoji galia, polių nuosėdžių skaičiavimai, dujų turbinos, gilieji pamatai.

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