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DESIGN OF DEEP FOUNDATIONS ON BORED PILES

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Abstract. The paper describes the design of pile foundation on the site of the Elektrenai power plant, Lithuania. The foundation is aimed to support equipment of the power plant consisting of the gas turbine, the steam turbine and the generator. Besides high loads, the equipment had a strong dynamic impact on the foundation due to its working conditions and vibration. The pilling solution was adopted due to different reasons: i) the capacity of the soil to support great stresses over it; ii) the special requirements of the main equipment about settlements, movements and stresses. Pilling foundation was evaluated through immediate settlement analysis, which was carried out employing four most widely used methods. It included analysis of the soil data from cone and dynamic penetration tests, boreholes and laboratory tests. Soil properties were estimated from site investigation and soil exploration program according to Lithuanian standards. Pile settlement analysis showed that settlement value was 14 mm (pile toe settlement), and settlement value of elastic deformation of pile from vertical compressive loads was 3 mm. For such structure, foundation settlement should not be more than 16 mm (i.e., no more than 2 % of pile diameter). It was estimated that for pile of diameter 800 mm, pile length of 24 m was sufficient to endure overall loads.

Keywords: deep foundations, bored piles, foundation for gas and steam turbine, pile settlement analysis, cone penetration test.

1. Introduction

Foundation has to be proportioned both to interface with the soil at a safe stress level and to limit settlements to an acceptable amount. Settlement analysis plays an important role in building foundation, even though only few modern buildings collapse from excessive settlements, it is not uncommon for a partial collapse or a localized failure in a structural member to occur (Kempfert and Gebreselassie 2006). Excessive settlement and differential movement can cause distortion and cracking in structures (Salgado *et al*. 2007). In other words, current state-of-the-art design methods may greatly reduce the risk factor of settlement problems. A major factor that greatly complicates foundation design is that the soil parameters have to be obtained on construction site prior the project calculation. Great care should be exercised in determining the soil properties at the site for the depth of possible interest so that one can as accurately as possible determine whether a pile foundation is needed and, if so, that neither an excessive number nor lengths are specified. In this work, pile foundation was used to control settlement at marginal soil site and care was taken to utilize the existing ground so that a necessary pile length and minimum settlements are ensured.

The scope of this work was to design the pile foundation that will be needed for gas and steam turbine equipment on the site of the Elektrenai power plant, Lithuania. In the design of a pile foundation, the required pile length was estimated based on the load from the superstructure, allowable stress in the pile material, and the in situ soil properties. Soil properties were estimated from site investigation and soil exploration program according to Lithuanian regulations. Investigation data were based on cone penetration and dynamic penetration tests, boreholes, excavations and soil as well as laboratory investigations. From these data, four geological layers were generalized that were applied in design of pile foundation.

Pilling foundation was evaluated through immediate settlement analysis, which was carried out employing four most widely used methods. The results showed negligible difference in estimation of immediate settlements; however they presented significant difference in estimation of required pile length and pile capacity. The results that were the same in majority of methods were accepted in this work. Pile settlement analysis estimated that total settlement value was 17 mm, including 3 mm settlements of elastic deformation of pile from vertical compressive loads. For such structure, foundation settlement should not be more than <2 %D (where D is a diameter of the pile). Such settlement criteria was taken according to equipment settlement guidelines (General Electric Design Basis Document, Volume 1, 2008) and it means pile should be working within the limit of mobilization of its shaft resistance. It was estimated that for pile of diameter 800 mm, necessary length of 27 m was sufficient to handle overall loads.

The most reliable means of determining the actual pile capacity is pile-load tests. It helps to evaluate pile performance and determine whether the piles are adequately designed and placed. Therefore, the performance of load test for determining the actual pile capacity is a topic of future research.

2. Pilling foundation consideration

Pilling foundation was chosen due to two different reasons:

- The capacity of the soil to support great stresses over it. In other words, bearing capacity of soils represents the ability of soil to safely carry the pressure placed on the soil from pile without undergoing a shear failure with accompanying large settlements.
- The special requirements of the main equipments about settlements, movements and stresses. The equipment consisted of the gas turbine, the steam turbine and the generator. The generator is coupled to the gas turbine through a rigid coupling and is connected to the steam turbine by a flexible coupling. The equipment induced high loads, which, in turn, induced great stresses on the foundation. The combined unit of gas turbine, generator and steam are founded with a unique foundation. It has to provide the adequate resistance and comportment for all the static and dynamic equipment conditions.

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

For a design deep pile foundation, the required pile length (for a given pile diameter) was estimated from the load from the superstructure, allowable stress in the pile material, and the in situ soil properties. It was based on the following steps:

1. Soil propertied were determined from site investigation and soil exploration program according to Lithuanian regulations.
2. Superstructure loads were obtained from the manufacturer of gas and steam tribune (General Electric). It included design verification load of 2500 kN and service working load of 2239 kN.
3. The bored cast-in-place piles were adopted of diameter 800 mm that rested on the sandy bed. Based on the data from previous two steps, estimation of pile length was performed along the pile capacity and settlements.
4. The next step is the design of pile-groups, which settlements are larger than that obtained for a single pile. However this research evaluates a single pile, which can be considered as a first step in design of pile foundation. Settlement of a single pile is a prerequisite for estimation of pile-group settlements (either from empirical and theoretical approach). Estimation of pile-group settlement is a topic of further investigations long with designing the group geometry that satisfy a given problem.

These steps (except the last one) are described in the following sections.

3. Physical and mechanical properties of the soil

Soil propertied were determined from site investigation and soil exploration program on 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 were drilled. Soil samples were taken from trial pits in order to determine granulometric composition, plasticity and Proctor density. 21 tests of cone penetration (CPT) of the depth of up to 15 m were carried out. At 4 points below 15 m precise measurement of pore pressure have been carried out (CPTu). There were 16 dynamic penetration (DPSH) tests performed in the depth of up to 15 m. XIII engineering geological layers (EGL) were determined in investigation area based on investigation data of CPT and DPSH of boreholes, excavations and soil as well as laboratory investigations.

Surface of investigation site was levelled and the major part of 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 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 the 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 altitude. Below that, the silty sand (SU, SUo) was present to 67.7 m of altitude.

From the investigation of engineering geological layers, four geological layers were generalized:

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

These four layers were used in the design and calculations of piling foundation. Description of these layers is presented in Fig 1.

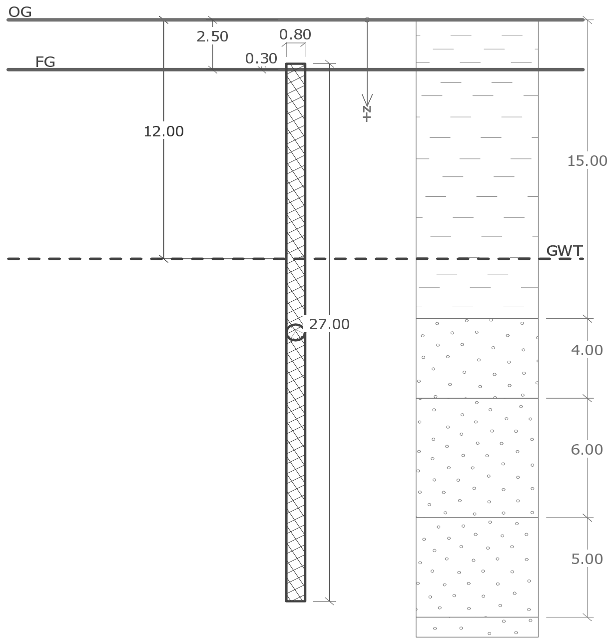

Fig 1. Geotechnical profile of the site where gas turbine is planned

4 Evaluation of bearing capacity of bored pile

Bearing capacity was evaluated through the basic condition for ultimate limit stage. The basic condition for ultimate limit state being:

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

where $F_{c,d}$ is ultimate limit state design load 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 top of it. Earth pressures on structural elements above the foundation level are geotechnical actions and are also included in $F_{c,d}$ where relevant.

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

The value of $R_{c,d}$ may be calculated using analytical or semi-empirical models. The concept of the 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 resistances. The base and shaft resistances, in turn, are the multiplication of base and shaft areas, A_b and A_s, by the respective unit of characteristic value of the 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 a soil layer index, and the summation is over the number n of layers crossed by the pile.

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

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

Equation (4) includes partial resistance factors (ξ ,γ), estimated from the Eurocode 7 and presented in Table 5 (Frank 2006). Simplified subsoil structure (worst situation site-wide) is presented in Table 1.

Average of effective overburden earth pressure at pile toe is described in Table 2.

Coefficient of horizontal earth pressure is depicted in Table 3.

Table 1. Simplified subsoil structure

Layer	Levels (m)	Lithology	γ_d(kN/m^3)	N_{20}DPSH	N_{30}SPT	ϕ'
1	98-83	Clayey deposit, medium to firm consistency	19.5	-	-	$10^{\circ(*)}$
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	73-68↓	Medium to coarse silty sans, dense to very dense.	26.0	26 - 50	47	34°

(*)obtained from direct shear testing

Table 2. Overburden earth pressure at pile toe

Layer	Levels (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

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

It should be noted that applied safety factors are slightly higher than required by Eurocode 7 (Frank 2006) for the worst combination at bored piles, and using one pile test (n = 1). The safety factors are summarized in Table 5.

Table 3. Coefficient of horizontal soil stress (K_s) (K_0 – coefficient of earth pressure at rest, K_0=1-*sin* ϕ'):

Layer	Levels (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	Levels (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
					$\Sigma R_{s,k}$	900

Table 5. Safety factors applied in eq.4

Resistance	ξ(for n=1)	γ (for R=4)	$\xi\ \gamma$	Applied
Base	1.4	1.6	2.24	3
Shaft	1.4	1.3	1.82	2

For the worst site-wide situation, the sum of shaft and base resistances $R_{c,d}$ was equal to 3801 kN. This value satisfied equation 1, where the sum of shaft and base resistances $R_{c,d}$ should be larger than (or equal to) a design axial compression load on single pile at the ultimate limit state $F_{c,d}$.

5. Pile settlement analysis

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

$$H_p = \frac{P \cdot L}{A \cdot E_p} + q \cdot D \cdot \frac{1-\mu^2}{E_s} \cdot mI_s \cdot I_F \cdot F_1 \tag{5}$$

The first term (before the sum sign) on right-hand side of the equation (5) described the average pile axial settlement for pile length, L, average cross-section area, A, and an elastic modulus of the 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 23.5 m. Elastic pile modulus, E_p, is determined according to the cylinder compressive strength f_{ck} (for f_c = 30 MPa, E_p = 32.000 MPa). Maximum applied load at pile head, P, is equal to service working load of P = 2239 kN.

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

According to equation (5) the total value of settlement, H_p, was estimated to be equal to 17 mm. This value could be considered as maximum, obtained from the conservative side, based on end-bearing behavior of the pile.

Pile settlement analysis showed that total settlement value was 17 mm. It includes 3 mm settlements of pile deformation from vertical compressive loads. For such structure, foundation settlement should not be more than 2 % of pile diameter. For the pile of 800 mm diameter, the foundation settlement should not be more than 16 mm. The calculation shows, that for pile of diameter 800 mm, the necessary length was 27 m. Such length is sufficient enough to endure overall loads.

6 Comparison between different methods

To evaluate obtained results, pile settlement analysis was performed employing three other, most widely used, standards and approaches. Approaches of Schmertmann (Schmertmann 1986), vertical bearing capacity– Spring method and CPT (ENV 1997-3) standard were applied for estimation immediate settlement, required pile length and pile capacity. The results are presented in Table 6.

As can be seen, the settlement values are nearly the same irrespective of applied method. However, the situation regarding values of estimated pile length and pile capacity is different. Bowles method (Bowles 1997) presents the longest necessary pile length, i.e., 27 m, while other methods indicate the necessary pile length to be about 24 m. Even larger differences are revealed, when comparing values of pile capacity obtained by different methods. Schmertmann method (Schmertmann 1986) indicates the largest value of pile capacity, which is 2.8 times larger than value of pile capacity obtained by Bowles method (Bowles 1997).

Table 6. Comparison between results obtained by different methods for loading *P=2500 kN*.

Methods	Pile length, m	Immediate settle-ment, mm	Overall pile capacity, kN
Bowles (Bowles, 1997)	27	17.0	3801
Schmertmann, (Schmertmann, 1986)	23.5	14.3	10959
CPT (ENV 1997-3)	23.5	9.0	9604
Vertical bearing capacity– Spring method	23.5	14.0	8139

The Schmertmann and CPT (ENV 1997-3) methods differ from other methods in determination of the toe and shaft bearing capacities. The determination of total pile bearing capacity, and calculation of settlement is then performed in the same way as presented in previous sections.

In Schmertmann method, the maximum bearing capacity of a single pile based on the values of tip resistance q_c of the i^{th} static penetration test is given by:

$$F_{\max,i} = F_{\max,toe,i} + F_{\max,shaft,i} \tag{6}$$

Equation (6) is described by maximum toe, $F_{max,toe,i}$, and shaft, $F_{max,shaft,i}$, resistances from i^{th} CPT test.

The maximum pressure at pile toe is determined as follows:

$$p_{\max,toe} = \alpha_p \cdot q_{c,eq} \tag{7}$$

The maximum pressure at pile in equation (7) depends on equivalent average cone tip resistance, $q_{c,eq}$, and pile toe coefficient, α_p, which identifies the type of pile. Its values are calculated based on the values of cone tip resistance q_c. For bored piles with steel casing α_p is equal 0.5 (ENV 1997-3).

The maximum shaft friction is given by:

$$p_{\max,shaft} = \alpha_s \cdot q_{upr} \tag{8}$$

The maximum shaft friction in equation (8) is described by shaft friction coefficient α_s and filtered cone tip resistance, q_{upr}. The later depends on minimum value of mean of the cone tip resistance. The shaft friction coefficient, α_s, depends on the depth to equivalent diameter ratio (see Schmertmann 1978 for more details).

In CPT (ENV 1997-3) method, the maximum bearing capacity of a single pile is based on equation (6). The maximum pressure at pile toe is then determined from the corresponding penetration test according to Eurocode 7: Geotechnical design– Part 2:

$$p_{\max,toe} = 0.5 \cdot \alpha_p \cdot \beta \cdot s \cdot \left(\frac{q_{c,I,m} + q_{c,II,m}}{2} + q_{c,III,m} \right) \quad (9)$$

In equation (9), pile toe coefficient, α_p, identifies the type of pile, while pile shape coefficient, *s*, represents the influence of a rectangular pile. Expanded pile toe coefficient, β, describes the influence of an expanded pile shank (toe). Subscript *m* indicates mean values of cone tip resistances $q_{c,I}$, $q_{c,II}$, and $q_{c,III}$, estimated according to (ENV 1997-3).

The maximum shaft friction is given by:

$$p_{\max,shaft} = \alpha_s \cdot q_{c,z,a} \quad (10)$$

The maximum shaft friction in equation (10) is described by cone tip resistance, $q_{c,z,a}$, at dept *h*, and shaft friction coefficient, α_s, which depends on the type of piles. This coefficient is considered when calculating the maximum shaft friction. For bored piles with steel casing α_s is equal 0.005 (ENV 1997-3).

Analysis of vertical bearing capacity using the Spring method (described by Wong and Teh 1995) provides the limit loading curve and distributions of forces and displacements developed along the pile. The limit loading curve describes the variation of vertical load as a function of the pile settlement. The method employed following soil parameters: angle of internal friction, cohesion, unit weight and deformation modulus of a given soil. The solution procedure is based on a semi-analytical approach. The pile is represented by standard beam elements. The response of surrounding soil follows from the solution of layered subsoil as a generalization of the Winkler-Pasternak model (described by Anjos *et al*. 2006). The elastic rigid plastic response in shear is assumed along the pile-soil interface in view of the Mohr-Coulomb failure criterion (described by Anjos *et al*. 2006). The normal stress acting on the pile is determined from the geostatic stress and soil (concrete mixture) pressure at rest.

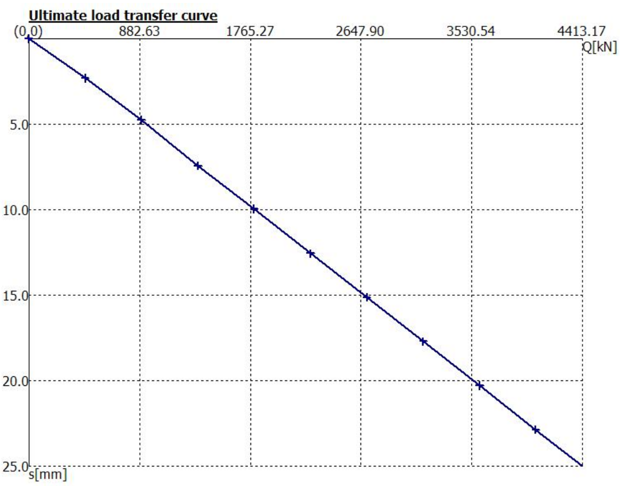

Fig 2. Ultimate load transfer curve (vertical bearing capacity– Spring method)

As a result, the analysis provides the limit loading curve, which describes the variation of vertical load, *P*, as a function of the pile settlement, *s*. Fig 2 shows a shape of the limit loading curve for a given problem. The maximal value of settlement is equal to 25 mm.

From the results presented in Table 6, it was concluded that for pile with 8x∅20, C30/37 longitudinal reinforcement the minimum required pile length of 24 m is sufficient enough to sustain overall load of 2500 kN. Although, the results showed negligible difference in estimation of immediate settlements, they presented substantial variations in estimation of required pile length and pile capacity. The results that were the same in at least two methods were accepted in this work. Thus, the minimum required pile length of 24 m obtained by three method Schmertmann (Schmertmann 1986), vertical bearing capacity– Spring method and CPT (ENV 1997-3) was considered to be the most reliable result.

Conclusions

The piling foundation was designed to support gas and steam turbine equipment on the site of the Elektrenai power plant, Lithuania. Four different methods (Bowles 1997, Schmertmann 1986, vertical bearing capacity– Spring method and CPT (ENV 1997-3) were applied in this study to determine immediate settlement, required pile length and pile capacity. They showed negligible difference in estimation of immediate settlements and presented signification variations in estimation of required pile length and pile capacity. The results that were the same in the majority of methods were accepted in this work.

For such structure, foundation settlement should not be more than 16 mm (i.e., no more than 2 % of pile diameter). It was estimated that for pile of diameter 800 mm, necessary length of 24 m was sufficient to endure overall loads.

A major factor that greatly complicates foundation design was marginal soil condition and the special requirements of the main equipments about settlements, movements and stresses. Therefore, load test for determining the actual pile capacity and settlement, and evaluation of time dependent effect, is a topic of future research.

Nomenclature

Notation

- A area of pile cross section (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 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;
- 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;
$p_{max,shaft}$ maximum shaft friction;
q bearing pressure at point (kPa);
$q_{c,eq}$ equivalent average cone tip resistance;
q_{upr} filtered cone tip resistance;
$q_{c,z,a}$ cone tip resistance at dept *h;*
$q_{c,I,m}$, $q_{c,II,m}$, $q_{c,III,m}$ mean values of cone tip resistances;
s pile shape coefficient (-);
$F_{c,d}$ design axial compression load on a single pile at the ULS (kN);
$R_{c,d}$ design value of compressive ground resistance of a single pile at ULS (kN).

Greek symbols:
α_p pile type factor (-);
α_s shaft friction coefficient (-);
β expanded pile toe coefficient (-);
μ Poisson ratio for soil;
σ'_{vok} effective overburden earth pressure (kPa);
φ' shearing resistance (º).

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