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RELIABILITY ASSESSMENT OF PILE BASE RESISTANCE

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Abstract. A reliability of pile resistance is investigated. It is assumed that parameters of soil resistance R_m, that of pile cross-sectional properties A, that of design model error ΔR, that of external actions described via dead loads V_G, variable loads V_Q and that of calculation errors ΔE are statistically distributed values. Statistically distributed data of some above mentioned parameters e.g. are obtained by processing the cone penetration tests (CPT), other taken from the published investigations, as e.g. of external actions. A method for evaluating resistance codified resistance is described. Subsequently the method is realized for axially loaded pile, namely for obtaining the design resistances, the resistance resource, the reliability index.

Proposed method is illustrated for actual case: design of multifunctional complex for sport and relaxation at Dubysos street 10 in Klaipėda, Lithuania. Properties of strata for piles design are given, namely description of soils and their statistically distributed properties, pile resistance calculations, identifying characteristic properties of soil actions. Combinations of partial factors for investigated design approaches are presented. They involve the limit states of collapse and large deformations of pile resistance for axially loaded piles. A bearing capacity of piles' resistances' and reliability index of pile resistance are determined.

Static tests in situ have been carried out for above described case. Processing of the test data yielded the following results: a change of testing pile relative stabilization time versus a change of relative stabilization loading; that of a relative stabilization period for certain loading level. The measured and the calculated characteristic bearing capacities of piles for above mentioned complex for sport and relaxation are presented.

Keywords: pile test, CPT, Eurocode 7, calculation algorithm, reliability index.

1. Introduction

When designing foundations for building one must take into account the fact that soil investigations are not performed at location of each foundation. The foundations are erected in soil layers, might be not investigated to carry loading. At certain depth and location is not known apriori (it might be less or more when compared with the places where the field tests have been performed. The foundations should be designed in the way to avoid limit states or bearing layers. Statistical techniques are employed by many investigators for evaluating load carrying capacity of bearing layer.

The reliability of pile foundation designed on the basis of soil test results depends mainly on the reliability of the calculation method for the pile resistance evaluation and that of on the approach for the spatial variability of the pile resistance evaluation. A lumped parameter lognormal reliability formula in closed form is employed to calculate the reliability index of the pile foundation in case when different sources of uncertainty related the stiffness and monitoring effects of piles installation are met (Baudin 2003), (Low 2002), (Paikowsky, 2002; Medzvieckas *et al.* 2004). A fitting of probability distribution functions to characterize proper distribution in the domain of interest is presented. The performance matrix for the pile foundation is proposed: there are four factors which need to be determined to obtain a reasonable performance matrix of a structure. They are namely: a load frequency for given structure design life, structure performance (i.e., limit states), an importance level of structure and a probability of attaining each limit state under given conditions. Such reliability analysis was employed for two actual pile foundation cases (Honjo 2000). A chaotic nature of the soil makes any deterministic prediction of the pile lengths to be impracticable, thus designers developed an original and ad hoc reliability-based procedure to cope with the problem (Tonon N/A). The spatial variation of soil properties induces the foundation stresses and/or displacements that cannot be predicted when assuming soil homogeneity. A numerical model of a piled raft foundation has been developed to describe how the soil–structure interaction can be influenced by the horizontal soil variability (Niandou 2006; Amšiejus *et al.* 2004). The response surface models are developed using available conventional equations and numerical analysis. Considering the variations in the input soil parameters, the reliability analysis is performed by using the response surface models to obtain an acceptable value of the allowable bearing pressure. The results of the reliability analysis were compared with the results of Monte Carlo simulation and yielded that an application of the response surface method for the probabilistic analysis can considerably reduce the computational efforts and memory resources (Sivakumar Babu 2007).

The distribution of soil resistance parameters (uniformity, other) rises a lot of questions, when it is needed to evaluate it basing on ground tests, leading to various interpretations (Whitman 1984). Different engineers prefer to use various methods of calculation. Some methods of calculation are based on proper evaluation of soil properties (Goble 1999; Amšiejus and Dirgelienė 2007; Žaržojus *et al.* 2007). The factors of safety, to be incorporated in the design equation, highly varies (Kulhawy 1984, 1996; Green and Becker 2001). It is stated, that common employment of practical and theoretical approaches lead to an optimal reasonable design (Committee on Reliability Methods 1995; Kulhawy 1996). It is stated, that improvements could be made in development of reliability based design (RBD) methodologies. Many design codes in our days are based on RBD approach (Kulhawy and Phoon 1996; Amšiejus *et al.* 2009). Some investigators are stating that geotechnical design aims to be more codificated. It could be done via code harmonization in respect of types of materials and wider employment of national annexes (Phoon and Kulhawy 2004). The probabilistic geotechnical design is used for highway bridge design (AASHTO 2002). The geotechnical standards developed starting from working / allowable stress based design (WSD/ASD) ending by Load and Resistance Factor Design (LRFD). The term "LRFD" is used mostly in the United States and conforms an equivalent to "Limit State Design (LSD)". Both LRFD and LSD in some way lead to the partial factors approach being commonly used in Europe. Here different quantities of factors including factored soil parameters is employed. One can find an absence of strong analytical calibration and verification in Eurocode7 (Paikowsky and Stenersen 2000; CEN/TC250 1994). It was proved that LRDF can be used as a simplified reliability based design procedure. In this case it is not necessary to calculate an original global factor of safety. The following conclusions could be made, namely: the calculated reliability is not exact and an algorithm could be developed for probability calculations. LRFD could be used as calculation algorithm for probabilistic design and the algorithm of probabilistic design could be simplified on the basis that calculations posses some freedom in design calculations. The distinction between an accepting of reliability analysis method as a theoretical basis for geotechnical design and calibration of simplified multiple factor design formats is very important (Phoon *et al.* 2003a). The former presents the method of uncertainties and a unified algorithm for risk determination of geotechnical and structural design. The other algorithms that were suggested are : the λ-method (Simpson *et al.* 1981), the worst attainable value method (Bolton 1989), the Taylor series method (Duncan, 2000). Note that one can find no one theoretical method, that could handle in situ problems (met in engineering real practice). When developing the Eurocode 7 a lot of attention was focused on the code harmonization in respect of the geotechnical aspects (Frank 2002; Ovesen 2002; Orr 2002). It is debated that as calculation platform is suitable.

It is important from the practical point of view to keep past practice continuity to obtain the simplified reliability-based design (RBD) equations set. On the other hand, there is no necessity and it may be even low efficiency to keep past practice continuity due to increased fraught facing 3 difficulties due fairly complex problems have been raised (Phoon 2004). It is ascertained that the limitations met by employing the simplified RBD have no significant influence to universality when employing the reliability theory.

The allowable finite element software in concert with comparatively low cost powerful PCs ensures ability to analyze real engineering problems. The latter statement was proved via some cases illustrating that limitations of the implementation (id est LRFD) do not carry over to the underlying reliability framework (Phoon *et al.* 2003a), so there must be put more emphasis on the issue pertaining to the relevance of reliability theory in geotechnical design.

Regarding RBG key elements. It is important to seek the RGB simplified equations to be more employed and developed. The process is increasing, e.g. one can mention many developments on RBD calibration e.g. examples of (Phoon and Kulhawy 2002a, b; Phoon *et al.* 2003c; Phoon and Kulhawy 2004).

2. Reliability valuation of pile resistance

The pile resistance on the basis EC7 has been investigated on the basis of codified reliability by many authors (e.g. Užpolevičius 2006; Phoon, 2008).

The pile resistance R is prescribed via the of soil properties:

$$R = r(D, q_{c,s}, q_{c,b}, t, \gamma, ..., \Delta R) . \tag{2.1}$$

Here:

D is pile diameter;

$q_{c,s}$ is mean cone penetration resistance value of strata layer at pile shaft;

$q_{c,b}$ is mean cone penetration resistance value of strata layer under pile base;

t is height of bearing layer of soil;

γ is unit weight of soil strata above the pile bottom;

ΔR is pile resistance calculation error of calculation method.

The external actions are expressed via set values:

$$E = e(V_G, V_Q, a, ..., \Delta E) . \tag{2.2}$$

where:

V_G is permanent unfavorable actions (e.g. weight of structural parts);

V_Q is variable unfavorable actions (e.g. snow, wind, transport, etc.) ;

a is a set of foundation dimensions;

ΔE is design error of stress calculation model.

The resistance resource Z is described by:

$$Z = R - E = r(D, q_{c,s}, q_{c,b}, t, \gamma, ..., \Delta R) - \\ -e(N_G, N_Q, ..., \Delta E). \tag{2.3}$$

Taking into account that calculation errors in the article's calculations are not evaluated, the above expression can be rewritten by:

$$Z = f_z(N_G, N_Q, D, q_{c,s}, q_{c,b}, t, ..., \gamma) = \\ = f_z(x_1, x_2 ..., x_n). \tag{2.4}$$

Here: x_1, x_2,..., x_n are statistically distributed values of $N_G, N_Q, d, q_{c,s}, q_{c,b}, L_d, ..., \gamma$;

n is a number of statistically distributed values.

It is assumed that the above statistically distributed parameters conform the normal distribution and are described by:

$$x_1 \in N(\mu_{x_1}, \sigma^2_{x_1}), \\ x_2 \in N(\mu_{x_2}, \sigma^2_{x_2}), ..., \quad x_n \in N(\mu_{x_n}, \sigma^2_{x_n}). \tag{2.5}$$

Then by employing the above definitions the resistance resource reliability parameters (1.3) can be expressed by:

$$\mu_z \approx f_z(\mu_{x_1}, \mu_{x_2}, ... \mu_{x_n}). \\ \sigma_z \approx \sqrt{\sum_{i=1}^{n} \left(\frac{\partial \mu_z}{\partial x_i} \sigma_{x_i} \right)^2}. \tag{2.6}$$

Here: μ_z is mean value of the statistically distributed parameters;

σ_z is standard deviation of statistically distributed values.

The resistance resource Z_d , determined by applying design values of arguments, must be equal to zero. Then the equilibrium of the external actions and the resistance is expressed by:

$$Z_d = R_d - E_d = \\ f_{z_d}(x_{1d}, x_{2d}, ..., x_{nd}) = 0. \tag{2.7}$$

Here: R_d and E_d corresponds the design magnitudes of the base resistance and the effect of external actions, respectively;

x_{1d}, x_{2d}, ... , x_{nd} are design magnitudes of resistance, external actions and geometrical dimensions.

The density function of pile resistance resource h(z) and design magnitude of resistance resource Z_d=0 are shown in Fig 1.

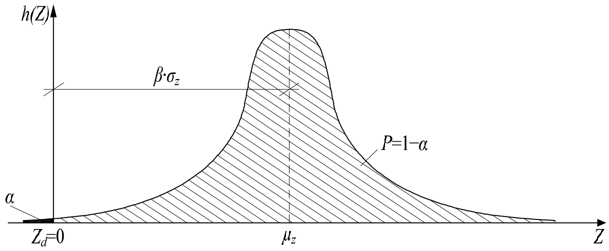

Fig 1. Design magnitude z_d = 0 of codified resistance resource Z and reliability index β; codified reliability $p = \Phi(\beta)$.

The codified magnitude or normalized reliability is described by:

$$P = P(Z \geq z_d) = P(Z > 0) = \\ = P(Z > \mu_z + \beta\sigma_z) = \varphi(\beta) = \\ = \frac{1}{\sqrt{2\pi}} \int_{\beta}^{\infty} e^{-t^2/2} dt. \tag{2.8}$$

Here β is the reliability index calculated by the equation set

$$\begin{cases} \mu_z - z_d = \beta\sigma_z, & (2.9) \\ z_d = 0. & (2.10) \end{cases}$$

Substituting resistance resource Z_d=0 in the above equation set, one obtains

$$\mu_z = \beta\sigma_z. \tag{2.11}$$

Then the reliability index can be expressed by:

$$\beta = \frac{\mu_z}{\sigma_z}. \tag{2.12}$$

One can point out that the codified resistance of pile resistance being determined by the equation set and should be described by:

$$\begin{cases} z_d = f_z(x_{1d}, x_{2d}, ... x_{nd} \,|D) = 0, & (2.13) \\ \beta = \dfrac{f_z(\mu_{x_1}, \mu_{x_2}, ... \mu_{x_7})}{\left[\sum_{i=1}^{n} \left(\dfrac{\partial \mu_z}{\partial x_i} \sigma_{xi} \right)^2 \right]^{0,5}}. & (2.14) \end{cases}$$

The required parameter D ir derived from usual design problem i.e. the geometrical dimension of foundation is obtained via solving the equation (2.13). Then the reliability index β is calculated by equation (2.14). When β is identified, one can evaluate the collapse probability P of pile resistance.

3. Description of soils that takes over pile load

Considered case is of multifunctional complex for sport and relaxation at Dubysos street 10 in Klaipėda, Lithuania. Two bearing layers of strata of piles are investigated. Fig 2 and Fig 3 represent CPTs for the layers per considered depths.

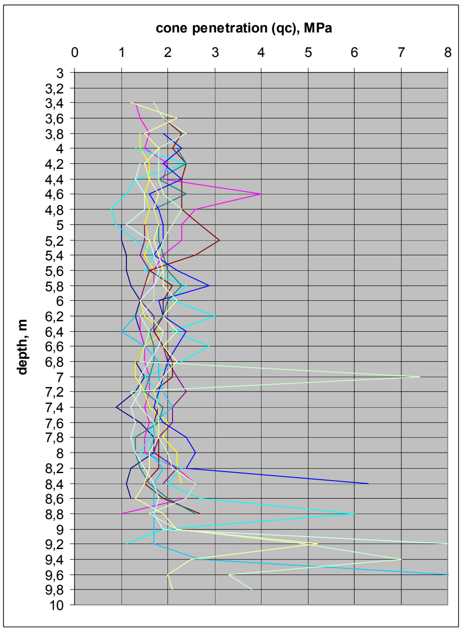

Fig 2. Cone penetration resistance of strata layer, that takes over piles shaft load, via in situ test of clayely sand grey brown, low plasticity

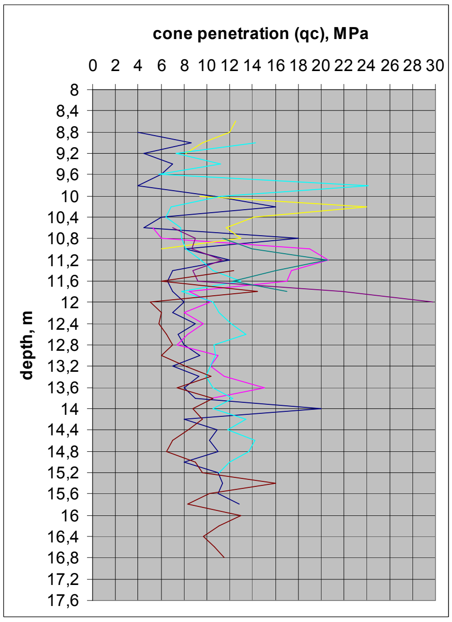

Fig 3. Cone penetration resistance of strata layer, that takes over piles base load, via in situ test of clayely sand grey, very hard**.**

4. Data of piles tests

Three displacement piles of diameter 0.32 m and 8.45÷9.60 m lengths have been tested in situ. The piles were loaded by 130.2 kN steps and reloaded by 217.0 kN steps. The upper load variates from 911 kN to 1041 kN. The time of relative stabilization of the settlement is presented in Table 1.

Table 1. Time of relative settlement stabilization for tested piles having reached upper bound of load level

Load upper bound (in kN)	Relative stabilization for tested pile (in min)		
	Nr. 1	Nr. 2	Nr. 3
130.2	-	-	20
260.4	20	40	40
390.6	60	60	80
520.8	60	80	100
651.0	120	100	120
781.2	200	100	160
911.4	380	320	300

Test data of piles displacements dependences on loads are given in Fig 4.

Test data of piles No 1, No 2 and No 3 are presented in Table 2.

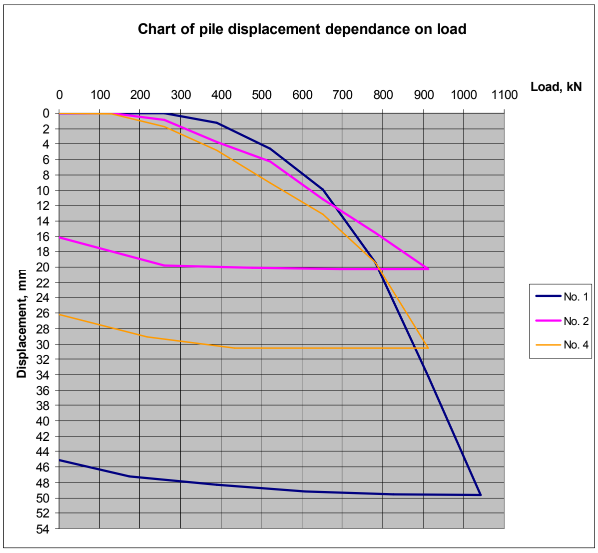

Fig 4. Load versus settlement graphs of tested piles

Table 2. Measured bearing capacity of piles

Pile No	L_m, m	$R_{m;\lambda}$, kN			
		λ= 2	λ= 3	λ= 5	λ= 9,4
1	9.6	563.3	641.9	733.9	874.6
2	8.45	523.5	608.7	793.1	-
3	9.4	439.3	537.7	710.5	904.7

Here λ is a coefficient for evaluating relative settlement of pile:

$$\lambda = {}^{s}\!/\!_{D} \times 100\% \,. \qquad (4.1)$$

where:

s is pile settlement;

D is pile diameter;

$R_{m;\lambda}$ is pile bearing capacity corresponding λ magnitude.

5. Determination of pile bearing capacity

5.1 Determination of pile bearing capacity according test results via static loading

Characteristic magnitude of pile bearing capacity according Eurocode 7 is described by means of formula (2.16):

$$R_{c;k} = Min\left\{\frac{(R_{c;m})_{mean}}{\xi_1};\frac{(R_{c;m})_{min}}{\xi_2}\right\}. \qquad (5.1.1)$$

where ξ_1 and ξ_2 are the factors of correlation corresponding the number of the tested piles. The magnitudes of factors are presented in Table 3.

Table 3. Correlation factor ξ that depends on number of tested locations (ξ values when 2 and 3 strata tests are performed).

Factor	Value	
	n = 2	*n* = 3
ξ_1	1.30	1.20
ξ_2	1.20	1.05

Calculated bearing capacities of tested piles according Eurocode 7 see in Table 4.

Table 4. Calculated bearing capacities of tested piles

Parameters	Calculated bearing capacity, in kN, at normalized settlement s/D			
	s/D = 2	s/D = 3	s/D = 5	s/D = 9.4
$(R_{c;m})_{mean}$	508.7	596.1	745.8	889.65
$(R_{c;m})_{min}$	439.3	537.7	710.5	874.6
$(R_{c;k})_{mean}$	423.9	496.8	621.5	684.3
$(R_{c;k})_{min}$	418.4	512.1	676.7	728.8
$R_{c;k}$	418.4	496.8	621.5	684.3
$R_{c;d\ (A1“+“M1“+“R1)}$*	418.4	496.8	621.5	684.3
$R_{c;d\ (A2“+“M1“+“R4)}$*	321.8	382.1	478.1	526.4
$R_{c;d\ (A1“+“M1“+“R2)}$*	380.3	451.6	565.0	622.1
$R_{c;d\ (A1“+“M2“+“R3)}$*	418.4	496.8	621.5	684.3

* Calculation method is described in section 5.2

5.2 Determination of pile bearing capacity according soil test data

Design scheme of pile resistance is presented in Fig 5.

The dead (V_G) and the variable (V_Q) effects of actions are applied onto the pile. The characteristic magnitudes of the actions were determined having performed analysis of calculation scheme.

The design magnitudes of actions are determined according the code EN 1990: 2002.

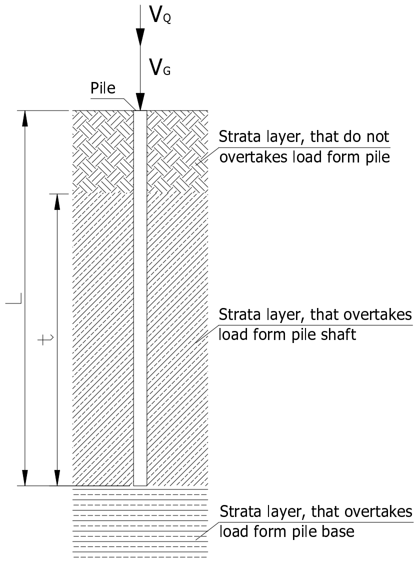

Fig 5. Design scheme of the pile

The design magnitude of an action F_d is obtained directly or via employing a representative magnitude as follows:

$$F_d = \gamma_F \cdot F_{rep}. \qquad (5.2.1)$$

when

$$F_{rep} = \psi \cdot F_k. \qquad (5.2.2)$$

The relative magnitudes of ψ are taken from code EN 1990: 2002.

The partial factors γ_F are selected for permanent and variable unfavorable actions.

The design values of soil properties (X_d) are obtained from the characteristic magnitudes by

$$X_d = X_k / \gamma_M. \qquad (5.2.3)$$

In the case when the variation of geometric dimensions influences significantly to the reliability of the structure, the design magnitudes of the geometric dimensions (a_d) are determined directly or obtained from their nominal magnitudes by employing the formula (see 6.3.4 EN 1990: 2002):

$$a_d = a_{nom} \pm \Delta a. \qquad (5.2.4)$$

Aiming to investigate the actual state versus limit state for the structure, that of restricted deformation of structural element or bearing strata of the structure (see case STR and GEO) one must check the condition:

$$E_d \le R_d. \qquad (5.2.5)$$

The partial factors are applied for the actions (F_{rep}) and their effects (E):

$$E_d = E\{\gamma_F F_{rep}; X_K/\gamma_M; a_d\}. \qquad (5.2.6)$$

or

$$E_d = \gamma_E E\{F_{rep}; X_K/\gamma_M; a_d\}. \qquad (5.2.7)$$

The partial factors are applied for the soil properties (X), the resistances (R) or for the both above mentioned values:

$$R_d = R\left\{\gamma_F F_{rep}; X_K / \gamma_M; a_d\right\}. \qquad (5.2.8)$$

or

$$R_d = R\left\{\gamma_F F_{rep}; X_K / \gamma_M; a_d\right\} / \gamma_R \qquad (5.2.9)$$

The bearing capacity of the pile resistance is calculated by following method. The method compatible for Lithuanian soils which have been proved in engineering practice:

$$R_{c;cal} = p_b + p_s = (9\, {}^{q_{c,b}}\!/_{20} + \gamma_g \times L) \times A_t + u \times \alpha \times \sum_{i=1}^{n} (t \times {}^{q_{c,s,i}}\!/_{20}). \qquad (5.2.10)$$

Here:

$q_{c,b}$ is average cone penetration resistance of strata layer, that takes over piles' shaft load;

γ_g is soil weight force of soil above the pile;

H is height of bounded by ground surface level and pile bottom;

A_t is cross-sectional area of pile;

α is factor equal to 0.9;

$q_{c,s,i}$ is average cone penetration resistance of strata layer, that takes over piles' base load;

u is perimeter of pile cross-sectional area;

t s thickness of soil layer that takes over piles' shaft load.

By applying the conventional design techniques the axially loaded pile resistance limit state versus collapse is checked in respect of the certain design approaches, described by the relevant set of partial factors, namely:

1-st design approach:
1 combination (DA1/1): A1„+“M1„+“R1,
2 combination (DA1/2): A2„+“M1„+“R4,
2-nd design approach:
combination (DA2): A1„+“M1„+“R2,
3 design approach:
combination (DA3): A1“+“M2“+“R3.
(Note „+“ means combination of effects of actions)

Here: A1 and A2 are the sets of partial factors for external actions and their combinations

M1 and M2 are the sets of partial factors for soil properties;

R1 ÷ R4 are the sets of partial factors for employed types of pile resistances.

Partial factors for considered combinations of design approaches are presented in the Table 5, Table 6 and Table 8, respectively. The factors of correlation are presented in Table 7.

The design magnitudes of the soil properties are obtained by combining the characteristic magnitudes and partial factors ξ_3 and ξ_4.

When determining strata bearing capacity of displacement piles, the partial factors for external actions, soil properties and resistances are as given in the Tables 5, 6, 8.

Table 5. Partial factors on permanent and variable unfavorable actions

Set of partial factors for external effects versus considered group	γ_G	γ_Q
A1	1.35	1.5
A2	1.0	1.3

Table 6. Partial factors for soil properties according static penetration tests

Set of partial factors for soil properties versus considered group	γ_{qc}
M1	1,0
M2	1.25*

(* Recommended magnitude (Bond. A, et. al. 2008)).

The characteristic magnitudes for pile base $R_{b;k}$ and shaft resistances $R_{s;k}$ are determined by:

$$R_{c;k} = \left(R_{b;k} + R_{s;k}\right) = \frac{R_{b;cal} + R_{s;cal}}{\xi} = \frac{R_{c;cal}}{\xi} = Min\left\{\frac{\left(R_{c;cal}\right)_{mean}}{\xi_3}; \frac{\left(R_{c;cal}\right)_{min}}{\xi_4}\right\} \qquad (5.2.11)$$

where ξ_3 and ξ_4 are the factors of correlation corresponding the number of bearing soil layer tests. Their magnitudes are presented in Table 7.

Table 7. Correlation factor ξ that depends on number of tested locations (ξ values when 1 strata test is performed).

Value	Magnitude
ξ_3	1.4
ξ_4	1.4

The design magnitudes of the pile resistance are obtained by combining the characteristic magnitudes and the relevant partial factors given in Table 8.

Table 8. Partial factors for displacement piles' resistances evaluation

Set of partial factors of displacement pile strengths versus considered group	γ_b	γ_s	γ_t
R1	1.0	1.0	1.0
R2	1.1	1.1	1.1
R3	1.0	1.0	1.0
R4	1.3	1.3	1.3

The calculated bearing capacities according Eurocode 7 of pile is presented in Table 9.

Table 9. Calculated bearing capacity of pile resistance

Pile No.	R_{ck}, kN	R_{cd}, kN			
		DA1/1	DA1/2	DA2	DA3
1	572.85	534.25	352.53	520.78	458.28
2	687.22	639.33	422.9	624.74	549.77
3	778.57	725.89	479.12	707.79	622.86

6. Evaluation of bearing strata reliability

Evaluating pile resistance reliability is performed by an employing statistically distributed data of values. The data of the arguments of normalized set of equation Z = z (X_1, X_2, X_3, X_4, X_5, X_6, X_7) is given in Table 10. The actions on the pile, was obtained via static calculations of the structural frame. The average standard deviations of the values were taken from the available references. The characteristic diameter of pile is 0.32 m. The average standard deviation magnitude is also taken from available references. The cone penetration resistances of strata layer that takes over piles shaft and base loads and unit weight of soil were obtained from field test data. The average standard deviations were calculated via processing the CPTs results. The deviation of soil layer thickness was identified by processing the field test data according the codified requirements. Standard deviation of soil unit weight is taken from the available references.

Table 10. Magnitudes of normalized set for pile resistance reliability evaluation

Parameters of values	Mean μ_{Xi}	Standard deviation σ_{Xi}	Characteristic magnitude X_{ik}
$X_1 = V_G$ [kN]	430	42.96	500
$X_2 = V_Q$ [kN]	113	22.59	150
$X_3 = D$ [m]	0.336	0.01	0.32
$X_4 = q_{c,b}$ [kPa]	19925	4184.3	13063
$X_5 = q_{c,s}$ [kPa]	1812	507.8	980
$X_6 = t$ [m]	7.45	0.25	7.039
$X_7 = \gamma_s$ [kN/m^3]	22.25	0.156	21.99

$$R_{c,d} = \frac{\frac{\pi}{4}D^2\left(0.45\, {q_{c,b,k}}/{\gamma_M} + \gamma_{pr}L\right)}{\gamma_b} + \frac{0.1413Dt\, {q_{c,s,k}}/{\gamma_M}}{\gamma_s} \tag{6.1}$$

The reliability evaluation of pile resistance bearing resistance is realized via the following steps of the calculations

$$E_d = (F_{Gk} + F_{pk})\gamma_G + N_{Qk}\gamma_Q$$

Then the resistance resource is obtained by:

$$Z_d = R_d - E_d = \frac{\frac{\pi}{4}D^2\left(0.45\, {q_{c,b,k}}/{\gamma_M} + \gamma_{pr}H\right)}{\gamma_b} + \frac{0.1413Dt\, {q_{c,s,k}}/{\gamma_M}}{\gamma_s} - (F_{Gk} + F_{pk})\gamma_G + N_{Qk}\gamma_Q = 0. \tag{6.3}$$

Then the probabilistic parameter of the function is:

$$\mu_z \approx f_z(\mu_{F_G}, \mu_{F_Q}, \mu_D, \mu_{q_{c,b}}, \mu_{q,s}, \mu_t, \mu_{\gamma_{gr}}) \tag{6.4}$$

$$\sigma_z \approx \left[\left(\frac{\delta z}{\delta D}\cdot\sigma_D\right)^2 + \left(\frac{\delta z}{\delta q_{c,b}}\cdot\sigma_{q_{c,b}}\right)^2 + \left(\frac{\delta z}{\delta q_{c,s}}\cdot\sigma_{q_{c,s}}\right)^2 + \left(\frac{\delta z}{\delta t}\cdot\sigma_t\right)^2 + \left(\frac{\delta z}{\delta \gamma_{gr}}\cdot\sigma_{\gamma_{gr}}\right)^2\right]^{0.5} \tag{6.5}$$

The partial derivatives for deviations at point $\boldsymbol{\mu}_X$=($\mu_{X3}, \mu_{X4}, \ldots, \mu_{X7}$) are calculated by:

$$\frac{\delta z}{\delta D} = \frac{\pi}{2}D\left(0.45q_{c,b} + \gamma_{pr}H\right) + 0.1413tq_{c,s}. \tag{6.6}$$

$$\frac{\delta z}{\delta q_{c,b}} = 0.45\frac{\pi}{4}D^2. \tag{6.7}$$

$$\frac{\delta z}{\delta q_{c,s}} = 0.1413Dt. \tag{6.8}$$

$$\frac{\delta z}{\delta t} = 0.1413Dq_{c,s}. \tag{6.9}$$

$$\sigma_z \approx \sqrt{\left(\frac{\pi}{2}D\left(0.45q_{c,b} + \gamma_{pr}H\right) + 0.1413tq_{c,s}\right)^2 + \left(0.45\frac{\pi}{4}D^2\right)^2 + \left(0.1413Dt\right)^2 + \left(0.1413Dq_{c,s}\right)^2 + \left(\frac{\pi}{4}D^2H\right)^2} \tag{6.11}$$

$$\frac{\delta z}{\delta \gamma_{gr}} = \frac{\pi}{4} D^2 H . \qquad (6.10)$$

The reliability index of the pile resistance is calculated according the data presented in Table 10. The pile resistance is checked for three design approaches. The parameters for reliability evaluation are presented in Table 11.

Table 11. Parameter for pile resistance reliability evaluation for designed pile to carry load

Parameters	DA1/1	DA1/2	DA2	DA3
D	0.42	0.48	0.45	0.48
μ_Z	1758.9	2366.3	1994.3	2325
σ_Z	404.81	500.29	443.49	498.4
β	4.345	4.73	4.4967	4.665

When the pile diameter is 0.32 m, the average resistance resource is μ_Z=906.02, the standard deviation σ_Z = 267.93. The above parameters yield a reliability index of β = 3.38.

Conclusions

The calculated and measured bearing capacities of the pile resistance vary within interval of 31 %

The diameter of the pile, designed according Eurocode 7, vary within the bounds 0.42÷0.48 m.

The reliability index of the pile resistance, designed according Eurocode 7, varies within 4.35÷4.73 (the pile resistance collapse probability is $P \approx 7x10^{-6}$).

The reliability index of the pile of diameter 0.32 m, designed according Eurocode 7, is 3.38 (the pile resistance collapse probability is $P \approx 3.6x10^{-4}$)

The determined reliability satisfies a requirement for limit state according the code EN 1990-0.

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