LAB University of Applied Sciences
Technology, Lappeenranta
Double Degree Programme in Civil and Construction Engineering

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Design and calculation process of friction piles

Bachelor's Thesis 2020

Abstract

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Instructors: Kostiantyn Khrameshkin, Lead Design Engineer, Neste Engineering Solutions Oy, Timo Lehtoviita, Lecturer, LAB University of Applied Sciences.

The thesis work was commissioned by Neste Engineering Solutions Oy. The purpose was to study the design of friction pile foundations and create an Excel file with manual for quick and easy calculation, because it includes many factors and checks.

Data for this study were gathered by the instructor from Neste Engineering Solutions Oy, it includes geological section, parameters of pile, CPT results, drawings settlement and diagrams of effective overburden pressure. Design information was collected from the literature. Design information was collected from the literature.

The final result of this work was an Excel file for calculating friction piles with two different types of soil and two different methods of design: empirical calculation and results of site investigations. This file allows to check the designed foundation for compliance with the main criteria.

Keywords: foundation, friction piles, CPT, bearing capacity, settlement

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Symbols

A, B, L – dimensions of pile group;

a, b – dimensions of pile;

A_b – area of the pile base, m^2;

A_g – area of pile group, m^2

$A_{s;i}$ – surface area of embedded length of pile in the i-th layer, m^2;

c_b – undrained shear strength of the soil at base of pile, kPa;

c_u – average undrained shear strength of soil along the shaft, kPa;

D – width or diameter of pile;

D' – width of pile group section, m;

E_g – pile group efficiency;

E_p – modulus of elasticity of the pile material, kN/m^2;

E_s – modulus of elasticity of soil, kN/m^2;

$e_{0(i)}$ – initial void ratio of layer i*;*

FS – factor of safety;

$F_{c;d}$ – design effect of actions, kN;

$\overline{f_c}$ – average penetrometer sleeve friction, kg/cm^2;

$f_{s;i}$ – characteristics of unit shaft resistance in the i-th layer, kPa;

G_{rep} – permanent action, kN

H'_f – height of the fill, m;

I_{wp}, I_{ws} – influence factor;

K – correction factor;

K' – earth pressure coefficient;

K_{Fl} – correlation factor;

K_s – coefficient of lateral earth pressure;

L_n – neutral depth, m;

N_c – bearing capacity factor;

n – number of piles in the group;

n_1, n_2 – column and row count;

P – perimeter of the pile, m;

P_g – perimeter of the pile group, m;

Q_{all} – allowable load-carrying capacity, kPa;

Q_b – ultimate base capacity, kN;

q_b – characteristics of unit base resistance, kPa;

q_c, q_p, etc – cone resistance, kg/cm^2;

Q_n – negative skin friction on a single pile (total downward drag force), kN;

$Q_{n;g}$ – negative skin friction on group piles, kN;

Q_s – ultimate shaft capacity (skin friction), kN;

Q_{rep} – variable action, kN

Q_{ult} – ultimate bearing capacity og single pile, kN;

$Q_{ult;g}$ – ultimate bearing capacity of group piles, kN;

Q_{wp} – load carried at the pile point under working load condition, kN;

q_{wp} – point load per unit area at the pile point, kN/m^2;

Q_{ws} – load carried by skin resistance under working load condition, kN;

$R_{c;d}$ – design compressive resistance, kN;

R_b – base resistance, kN;

$R_{s;i}$ – characteristic pile shaft resistance in the i-th layer, kN;

S – spacing of piles center to center;

S_c – total consolidation settlement, mm;

S_e – total elastic settlement, mm;

S_{e1} – settlement of pile shaft, mm;

S_{e2} – settlement of pile caused by the load at the pile point, mm;

S_{e3} – settlement of pile caused by the load transmitted along the pile shaft, mm;

α – adhesion factor;

α’ – ratio of pile to penetrometer sleeve friction;

β – maximum angular distortion, mm;

δ – angle of friction between the pile and the soil, degrees;

σ'_b – effective overburden pressure at the base of the pile, kPa;

$\overline{\sigma' s}$ – average effective overburden pressure acting along the embedded length of the pile shaft, kPa;

$\sigma'_{0(i)}$ – effective overburden pressure at the middle of each layer, kN/m^2;

$\Delta\sigma'_{(i)}$ – the increase in pressure at the middle of each layer, kN/m^2;

ɣ’ – effective unit weight, kN/m^3;

γ_b, γ_s, γ_{rd}, γ_G, γ_Q – partial factors;

γ_s – unit weight (weight density) of soil, kN/m^3;

γ_{sat} – saturated unit weight (saturated weight density), kN/m^3;

γ_w – unit weight of water (weight density of water),kN/m^3;

μ_s – Poisson's ratio of soil;

ξ – magnitude which depend on the nature of unit friction (skin) resistance distribution along the pile shaft;

ξ_g – group settlement factor;

ϕ – friction angle, degrees;

Δ – maximum differential settlement between any two portions of the foundation, mm;

λ_{GEO} – degree of utilization;

Γ – over design-factor.

1 Introduction

Foundation design is a labor-intensive and complex process, which includes comparison of all kinds of constructive solutions depending on engineering and geological conditions, information on the seismicity, purpose, design and technological features of the structure and conditions of their operation, loads acting on the foundation, conditions of existing developments and the impact of new construction on them, environmental requirements and others.

One of the fundamental design factors is also the design requirements, allowable deformations, safety factors that determine the required margin of safety of the structure in accordance with the requirements of the code and the project.

Nowadays friction piles are less studied and popular in Finnish industrial construction, so it is common to use end-bearing piles, although not in all cases they are more rational technologically and economically.

The thesis work presents recommendations, calculations and comparisons for the design of friction pile foundations using national documents: «RIL 254-2016 Paalutusohje PO-2016».

1.1 Types of bearing pile

Piles are called immersed in the ground or made in the ground vertical or inclined structure. They are designed to transfer the load to the base as the bottom end and friction arising on its side surface when moving. Under the conditions of interaction with the soil piles should be divided into end - bearing and friction piles. This is illustrated in Figure 1.

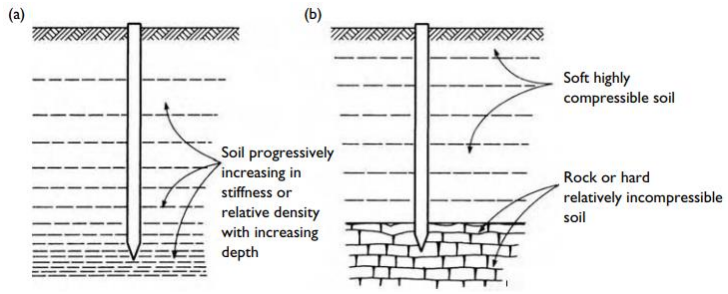


Figure 1. Types of bearing pile (a) friction pile (b) end-bearing-pile (Tomlinson, 2015)

To the end-bearing-piles should include piles all types, based on rocky soils, and driving piles, in addition, – on low-compressible soils. To include a low-compression coarse soil and clay solid and semi-solid consistency at E ≥ 50 MPa. The resistance of the soil, with the exception of negative friction forces on the side surface piles-racks in the calculation of the bearing capacity of soil bases on the compressive the load shall not be taken into account. By friction piles (hanging piles) should be include piles of all types, based on compressible soils and transfer loads on the foundation soil side surface and the lower end. Types of bearing piles are illustrated in Figure 2.

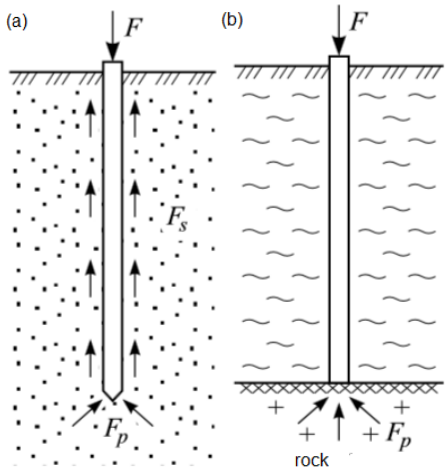


Figure 2. Scheme of load transfer by piles to soil and foundations (a) Friction pile (b) End-bearing-pile (Mangushev, 2016).

2 General theory on design

The design of a pile foundation requires a large collection of data on soils, tests, actions and impacts, reliability class of the building and etc., after that it is possible to calculate the depth of immersion and check the limit states of the pile. The design by calculation is illustrated in Figure 3.

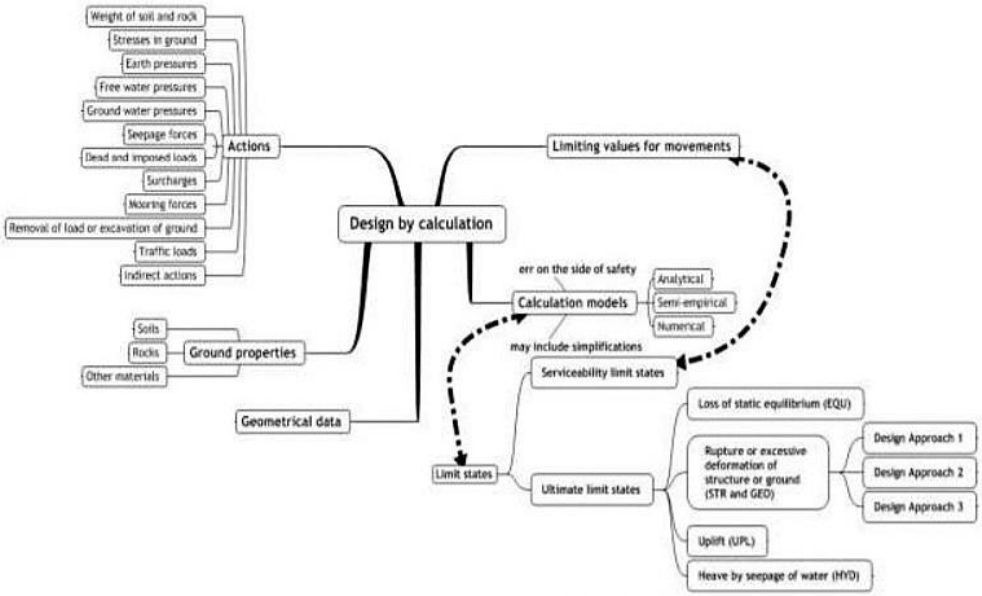


Figure 3. Design by calculation (Bond & Harris, 2008).

2.1 Initial data for design

In general, for the design of pile foundations it is necessary to have the following initial data:

- site conditions with respect to overall stability and ground movements;
- nature and size of the structure and its elements, including any special requirements such as the design life;
- conditions with regard to its surroundings (e.g.: neighbouring structures, traffic, utilities, vegetation, hazardous chemicals);
- ground conditions;
- ground-water conditions;
- regional seismicity;
- influence of the environment (hydrology, surface water, subsidence, seasonal changes of temperature and moisture).

The design should provide solutions that ensure the reliability, durability and cost-effectiveness of structures at all stages of construction and operation.

2.2 Actions on pile foundations

In geotechnical design, the following aspects should be considered for inclusion as actions:

- the weight of soil, rock and water;
- stresses in the ground;
- earth pressures and ground-water pressure*;*
- free water pressures, including wave pressures;
- ground-water pressures;
- seepage forces;
- dead and imposed loads from structures;
- surcharges;
- mooring forces;
- removal of load or excavation of ground;
- traffic loads;
- movements caused by mining or other caving or tunnelling activities;
- swelling and shrinkage caused by vegetation, climate or moisture changes;
- movements due to creeping or sliding or settling ground masses;
- movements due to degradation, dispersion*,* decomposition, self-compaction and solution;
- movements and accelerations caused by earthquakes, explosions, vibrations and dynamic loads;
- temperature effects, including frost action;
- ice loading;
- imposed pre-stress in ground anchors or struts;
- downdrag.

2.3 Ultimate Limit States

The limit States to consider when designing piles are listed below, although the design takes into account only those that are most relevant to a particular situation (EN-1997-1, §7.2(1)P):

- Bearing resistance failure of the pile foundation;

- Insufficient compression resistance of the pile (Fig. 4a);
- Uplift or insufficient tensile resistance of the pile (Fig. 4d);
- Failure in the ground due to transverse loading (Fig. 4f);
- Structural failure of the pile in compression (Fig. 4b), tension (Fig. 4e), bending (Fig. 4g), buckling (Fig. 4c) or shear (Fig. 4h);
- Combined failure in the ground, in the pile foundation and in the structure;
- Excessive settlement, heave or lateral movement;
- Loss of overall stability;
- Unacceptable vibrations.

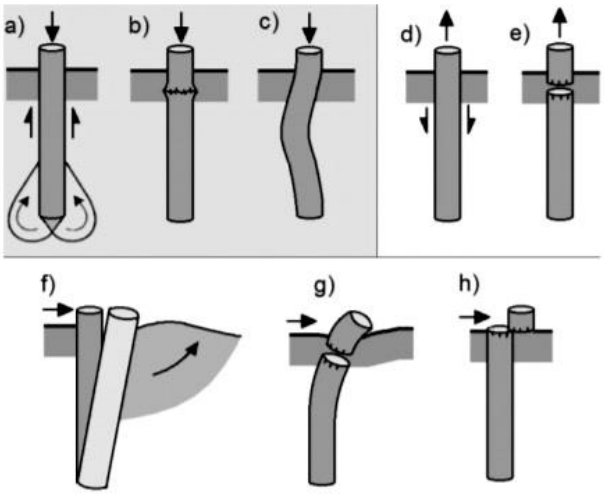


Figure 4. Piles load capacity: (a)–(c) on compression, (d), (e) on tension, (f)–(h) on transverse loading (Wrana, 2015).

2.4 Design Approaches

According to Eurocode 7, The manner in which equations for GEO/STR are applied shall be determined using one of three Design Approaches.

In Finland combination of sets of partial factors:

DA2: A1+M1+R2

Note 1: The partial factors: A (for actions or effects of actions), M (for soil parameters) and R (for resistances).

2.5 Design methods

According to Eurocode 7, The design shall be based on one of the following approaches:

- Static load tests – which have been demonstrated to be consistent with other relevant experience;
- Empirical or analytical calculation – whose validity has been demonstrated by static tests in comparable situations;
- Dynamic tests - whose validity has been demonstrated by static tests in comparable situations;
- Observed performance of a comparable foundation – provided this approach is supported by the results of site investigations and ground testing.

3 Design parameters

3.1 Soil parameters

There are several laboratory methods and in situ tests now available to determine parameters of various soil specimens. The soil properties needed in analysis of foundation are:

1. Undrained shear strength c_u; c_b (s_u), (kPa)

Table 1. Typical values for shear strength

Undrained shear strength		
Hard soil	s_u > 150 kPa	
Stiff soil	s_u = 75 -150 kPa	
Firm soil	s_u = 40 - 75 kPa	
Soft soil	s_u = 20 - 40 kPa	
Very soft soil	s_u < 20 kPa	
Drained shear strength	c' (kPa)	ϕ' (deg)
Sands	0	30 - 45
Clays	0 - 30 kPa	0 - 20

Table 2. Estimated relationship between s_u and I_c

s_u	I_c
15	0,25
25	0,5
100	1,0
200	> 1,0

2. Unit Weight of soil γ, (kN/ m^3)

Typical values for unit weight of soil:

Table 3. Empirical values for γ based on SPT

Relative Density	SPT N Value	γ (kN/$m^{3)}$
Very loose	0 - 4	< 16,0
Loose	5 - 10	15,3 – 20,0
Medium	11 - 30	17,5 – 21,0
Dense	31 - 50	17,5 - 22,5
Very Dense	> 50	> 21,0

3. Unit Weight of water γ_w, (kN/m^3)

γ_w = 9,81 kN/m^3

4. Modulus of elasticity of soil E_s, (kN/m^2)

Table 4. Typical values for modulus of elasticity (after Das, 1994):

Type of soil	Modulus of Elasticity (kN/m^2)
Soft clay	1380 - 3450
Hard clay	5865 - 13800
Loose sand	10350 - 27600
Dense sand	34500 - 69000

5. Poisson's ratio of soil μ_s

Table 5. Typical values for Poisson's ratio (Das, 1994):

Type of soil	Poisson's ratio
Loose sand	0,2 - 0,4
Medium sand	0,25 - 0,4
Dense sand	0,3 - 0,45
Silty sand	0,2 - 0,4
Soft sand	0,15 - 0,25
Medium clay	0,2 - 0,5

3.2 Pile foundation parameters

The general details of the standard precast concrete piles are as follows:

- Standard pile types used in Finland are 250 x 250 mm, 300 x 300 mm and 350 x 350 mm (Fig.5);
- The length of the pile elements available from 3 m to 15 m;
- Depth criteria: $z_a \geq 5$ m; $z_a \geq 1{,}0$ B; $z_a \geq 3{,}0$ D; (Fig.6);
- The usual load from 300 kN to 3000 kN;
- There are also few different kind of reinforcement in the piles, depending on the pile loads. Reinforcement strenght A500HW/ A700HW;
- Concrete strenght 45 to 50 MPa;
- Modulus of elasticity of the pile $21*10^6$ kN/m^2;

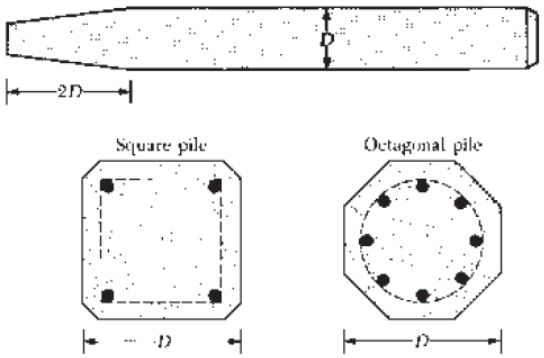


Figure 5. Dimensions (D) of precast piles with ordinary reinforcement (Braja,2017).

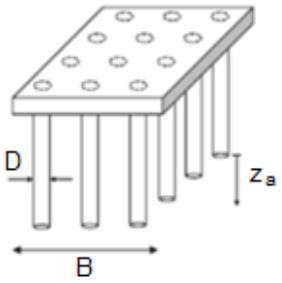


Figure 6. Depth (z_a), Pile Diameter (D), Width of pile group (B) of investigation points for piles and pile groups (Smith, 2014).

4 Algorithm of calculations

Calculation of friction piles consists of determining its main characteristics – pile length, capacity, consolidation and elastic settlement, group piles efficiency, block failure and checking for the limit states under the action of vertical loads - temporary and permanent. All calculations are presented in Excel file.

Calculations:

The determination of pile length is based on equality of the GEO limit state:

GEO limit state

$$F_{c;d} \leq R_{c;d} \qquad (1)$$

Where

$F_{c;d}$ design effect of actions, kN;

$R_{c;d}$ design compressive resistance, kN;

Design effect of actions, kN:

$$F_{c;d} = \gamma_f * F_{rep} \qquad (2)$$

$$F_{rep} = \Psi * F_k \qquad (3)$$

Where

F_{rep} representative value of an action, kN;

F_k characteristic value of an action, kN;

Ψ factor for converting the characteristic value to the representative value;

$$F_{c;d} = \gamma_G * G_{rep} + \gamma_Q * Q_{rep} \qquad (4)$$

G_{rep} permanent action, kN;

Q_{rep} variable action, kN;

$\gamma_f, \gamma_G, \gamma_Q$ partial factors;

Table 6. Partial resistance factors for driven piles

Action	**Symbol**	**Set**	
		A1	*A2*
Permanent:			
Unfavourable			
(Eq.6.10a)		1,35 K_{FI}	
(Eq.6.10b)	$\gamma_{Gkj,sup}$	1,15 K_{FI}	
(Eq.6.10)			1,0 K_{FI}
Favourable			
(Eq.6.10a)		0,9	
(Eq.6.10b)	$\gamma_{Gkj,inf}$	0,9	
(Eq.6.10)			1,0
Variable:			
Unfavourable			
(Eq.6.10b)	γ_Q	1,5 K_{FI}	
(Eq.6.10)			1.3 K_{FI}
Favourable		0	0

Note 1: The case can be expressed as a design equation so that the less favourable of the two following expressions is used as the combination of actions.

$$1{,}15\ K_{FI}\ G_{kj,sup} + 0{,}9\ G_{kj,inf} + 1{,}5\ K_{FI}\ Q_{k,1} + 1{,}5\ K_{FI} \sum_{i>1} \psi_{0,i}\ Q_{k,i} \qquad \text{(Eq.6.10b)}$$

$$1{,}35\ K_{FI}\ G_{kj,sup} + 0{,}9\ G_{kj,inf} \qquad \text{(Eq.6.10a)}$$

K_{FI} depends on the reliability class

Table 7. K_{FI} depending on the reliability class

Reliability Class	K_{FI}	Consequences of failure
RC3	1,1	High
RC2	1,0	Medium
RC1	0,9	Low

Design compressive resistances from ground tests results, kN:

$$R_{c;k} = \frac{\left(R_{b;cal} + R_{s;cal}\right)_{mean}}{\xi_3} \quad and \quad R_{c;k} = \frac{\left(R_{b;cal} + R_{s;cal}\right)_{min}}{\xi_4} \qquad (5)$$

Where

$(R_{b;cal})mean$ the mean calculated base resistance, kN;

$(R_{s;cal})mean$ the mean calculated shaft resistance, kN;

$(R_{b;cal})min$ the minimum calculated base resistance, kN;

$(R_{s;cal})min$ the minimum calculated shaft resistance, kN;

ξ_3, ξ_4 correlation factors;

Table 8 Correlation factors – ground tests results (from NA with Standard SFS-EN1997-1:2004), (n- number of test profiles).

ξ for n =	1	2	3	4	5	7	10
ξ_3	1,85	1,77	1,73	1,69	1,65	1,62	1,60
ξ_4	1,85	1,65	1,60	1,55	1,50	1,45	1,40

The design compressive resistance of the ground may be derived by either:

$$R_{c;d} = \frac{R_{b;d}}{\gamma_b} + \frac{R_{s;d}}{\gamma_s} \qquad (6)$$

Where

γ_b, γ_s partial factors;

Table 9. Partial resistance factors for driven piles

Resistance	Symbol	Set *R2*
Base	γ_b	1,20
Shaft (compression)	γ_s	1,20
Total/combined (compression)	γ_t	1,20
Shaft in tension:		
- short-term loading	$\gamma_{s,t}$	1,35
- long-term loading	$\gamma_{s,t}$	1,50

Base resistance, kN:

$$R_{b;d} = \frac{A_b * q_b}{\gamma_{rd}} \qquad (7)$$

Where

A_b the area of the pile base, m^2;

Table 10. Determination the area of the pile base

Section type	Formula
square	$A_b = a * b$
circular	$A_b = \pi r^2$

γ_{rd} presumed model factor, (γ_{rv});

Table 11. Partial factor sets for EQU, GEO, STR limit states (Elements of soil mechanics).

Parameter		Symbol	EQU	GEO/STR – Partial factor sets A1	A2	M1	M2	R1	R2	R3
Permanent action (G)	Unfavourable	$\gamma_{G;dst}/\gamma_{G;unfav}$	1.1	1.35	1.0					
	Favourable	$\gamma_{G;stb}/\gamma_{G;fav}$	0.9	1.0	1.0					
Variable action (Q)	Unfavourable	γ_Q	1.5	1.5	1.3					
	Favourable	–	–	–	–					
Accidental action (A)	Unfavourable	γ_A	1.0	1.0	1.0					
	Favourable	–	–	–	–					
Coefficient of shearing resistance ($\tan\phi'$)		$\gamma_{\phi'}$	1.25			1.0	1.25			
Effective cohesion (c')		$\gamma_{c'}$	1.25			1.0	1.25			
Undrained shear strength (c_u)		γ_{cu}	1.4			1.0	1.4			
Unconfined compressive strength (q_u)		γ_{qu}	1.4			1.0	1.4			
Weight density (γ)		γ_γ	1.0			1.0	1.0			
Bearing resistance (R_v)		γ_{Rv}						1.0	1.4	1.0
Sliding resistance (R_h)		γ_{Rh}						1.0	1.1	1.0
Earth resistance (R_e)		γ_{Re}						1.0	1.4	1.0

Cohesive soils:

$$q_b = N_c * c_b \qquad (8)$$

Where

q_b characteristics of unit base resistance, kPa;

N_c bearing capacity factor;

Table 12. Determination of bearing capacity factor

Value	Note
9	$for\ \frac{L}{D} \geq 3;\ c_b > 25\ kPa$
6	$for\ c_b \leq 25\ kPa$

c_b undrained shear strength of the soil at base of pile, kPa;

Cohesionless soils:

$$q_b = \sigma_b' * \ N_q \qquad (9)$$

Where

σ_b' the effective overburden pressure at the base of the pile, kPa;

$$\sigma'_b = \Sigma\gamma_{b;i} * L_i = \gamma_{b;1} * L_1 + \gamma_{b;2} * L_2 + \gamma_{b;i} * L_i \qquad (10)$$

$$\gamma_{b;i} = \gamma_{sat;i} - \gamma_w \qquad (11)$$

N_q the bearing capacity coefficient;

N_q is dependent on the ratio L/d (where L = length of embedment of pile, d = diameter or width of pile) and is calculated by bilinear interpolation by the graph. The diagram for determining the bearing capacity factor is illustrated in Figure 7.

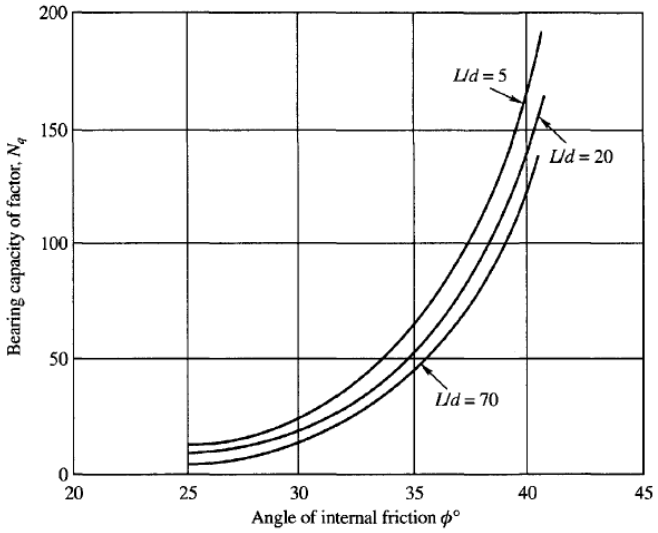


Figure 7. Bearing capacity factor, N_q (Berezantsev).

Characteristic pile shaft resistance, kN:

$$R_{s;k} = \frac{\Sigma A_{s;i} * f_{s;i}}{\gamma_{rd}} \qquad (12)$$

Where

$A_{s;i}$ surface area of embedded length of pile in the i-th layer, m^2;

Table 13. Determination the surface area of embedded length of pile

Section type	Formula
square	$A_{s;i} = 2(a + b) * l_i$
circular	$A_{s;i} = 2\pi r * l_i$

Cohesive soils:

$$f_{s;i} = \Sigma\alpha * c_{u;i} \qquad (13)$$

Where

$f_{s;i}$ characteristics of unit shaft resistance in the i-th layer, kPa;

α adhesion factor;

Table 14. Determination of adhesion factor

Value	Note / Formula
1	$c_u \leq 25\ kPa$
0,5	$c_u \geq 70\ kPa$
-	$1 - (c_u - \frac{25}{90})$

c_u average undrained shear strength of soil along the shaft, kPa;

Cohesionless soils:

$$f_{s;i} = K_s * \overline{\sigma_s{}'} * tan\delta \qquad (14)$$

Where:

K_s coefficient of lateral earth pressure;

Table 15. Determination of the coefficient of lateral pressure of the earth

Pile material	δ	K_s	
		Relative density of soil	
		Loose	Dense
Steel	20 degrees	0,5	1,0
Concrete	0,75ϕ	1,0	2,0
Timber	0,67ϕ	1,5	4,0

$\overline{\sigma_s{}'}$ average effective overburden pressure acting along the embedded length of the pile shaft, kPa;

$$\overline{\sigma_s{}'} = \Sigma\overline{\sigma_{s;i}{}'} \qquad (15)$$

$$\overline{\sigma'_{s;1}} = \frac{\gamma_{s;1}L_1}{2} \qquad (16)$$

$$\overline{\sigma'_{s;2}} = \gamma_{s;1} * L_1 + \frac{\gamma_{s;2}L_2}{2} \qquad (17)$$

$$\overline{\sigma'_{s;3}} = \gamma_{s;1} * L_1 + \gamma_{s;2} * L_2 + \frac{\gamma_{s;3} * L_3}{2} \qquad (18)$$

$$\gamma_{s;i} = \gamma_{sat;i} - \gamma_w \qquad (19)$$

δ angle of friction between the pile and the soil, degrees;

Ultimate bearing capacity, kN:

$$Q_{ult} = Q_b + Q_s \qquad (20)$$

Cohesive soils:

Ultimate base capacity, kN:

$$Q_b = q_b * A_b = N_c * c_b * A_b \qquad (21)$$

Ultimate shaft capacity (skin friction), kN:

α method:

$$Q_s = \Sigma\, f_{s;i} * A_{s;i} = \Sigma\, \alpha * c_{u;i} * A_{s;i} \qquad (22)$$

Hence

$$Q_{ult} = N_c * c_b * A_b + \Sigma\, \alpha * c_{u;i} * A_{s;i} \qquad (23)$$

Cohesionless soils:

Ultimate base capacity, kN:

$$Q_b = q_b * A_b = \sigma'_b * N_q * A_b \qquad (24)$$

Ultimate shaft capacity (skin friction), kN:

$$Q_s = f_{s;i} * A_{s;i} = K_s * \overline{\sigma'_s} * tan\delta * A_{s;i} \qquad (25)$$

Hence

$$Q_{ult} = \sigma'_b * N_q * A_b + K_s * \overline{\sigma'_s} * tan\delta * A_{s;i} \qquad (26)$$

Ultimate bearing capacity based CPT, kN:

$$Q_{ult} = Q_b + Q_s \qquad (27)$$

Vander Veen's Method

The diagram for determining the pile capacity by the Vander Veen method is illustrated in Figure 8.

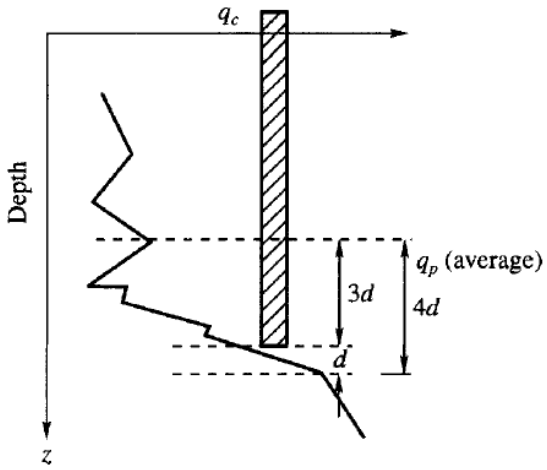


Figure 8. Pile capacity by use of CPT values (Vander Veen).

Cohesionless soils:

Ultimate base capacity, kN:

$$Q_b = q_b * A_b = q_p * A_b \qquad (28)$$

q_p average cone resistance over a depth 4d, kg/cm^2;

Ultimate shaft capacity (skin friction), kN:

$$Q_s = f_s * A_s = \frac{\bar{q_c}}{2} * A_s \qquad (29)$$

$\bar{q_c}$ average cone resistance over the length of the pile shaft under considera-

tion, kg/cm^2;

Schmertmann's Method

All types of soil:

Ultimate base capacity, kN:

$$Q_b = q_b * A_b = q_p * A_b \qquad (30)$$

Where

q_p cone resistance, kg/cm^2;

$$q_p = \frac{(q_{c1} + q_{c2}) + q_{c3}}{2} \qquad (31)$$

q_{c1} average cone resistance below the tip of the pile over a depth which may vary between 0,7d and 4d, where *d* = diameter of pile, kg/cm^2;

q_{c2} minimum cone resistance recorded below the pile tip over the same depth 0,7d to 4d, kg/cm^2;

q_{c3} average of the envelope of minimum cone resistance recorded above the pile tip to a height of 8d, kg/cm^2;

Case 1: When the cone point resistance q_c below the tip of a pile is lower than that at the tip within depth 4d (Fig.9).

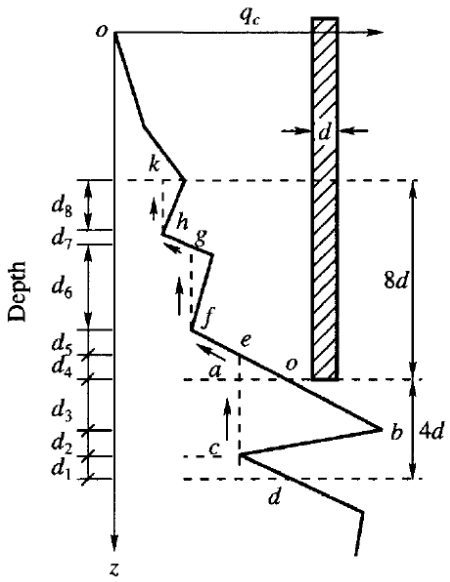


Figure 9. Resistance below pile tip lower than that at pile tip within depth 4d (Schmertmann).

$$q_{c1} = \frac{d_3 * \frac{q_0 + q_b}{2} + d_2 * \frac{q_b + q_d}{2} + d_1 * \frac{q_d + q_e}{2}}{4d} \qquad (32)$$

Where

$q_0, q_b, etc.$ cone resistance to refer to the points on the q_c-profile;

$q_{c2} = q_c$ minimum value below tip within a depth of 4d at point c on the q_c - profile;

$$q_{c3} = \frac{d_4 * q_m + d_5 * \frac{q_m + q_n}{2} + d_6 * q_n + d_7 * \frac{q_g + q_k}{2}}{8d} \qquad (33)$$

$$q_a = q_e;\ q_f = q_g;\ q_h = q_k \qquad (34)$$

Case 2: When the cone resistance *qc* below the pile tip is greater than that at the tip within a depth 4d (Fig.10).

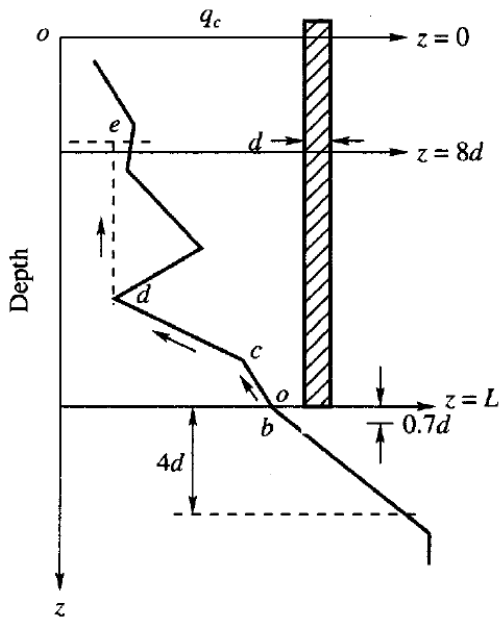


Figure 10. Resistance below pile tip greater than that at pile tip within 0.7d depth (Schmertmann).

$$q_{c1} = \frac{q_0 + q_b}{2} \qquad (35)$$

$q_{c2} = q_o$ minimum value at the pile tip itself;

q_{c3} average of the minimum values along the envelope ocde as before;

Ultimate shaft capacity (skin friction), kN:

Cohesive soils:

$$Q_s = \Sigma\, \alpha' * \overline{f_c} * A_s \qquad (36)$$

α' ratio of pile to penetrometer sleeve friction;

The diagram for determining the ratio of pile to penetrometer sleeve friction is illustrated in Figure 11.

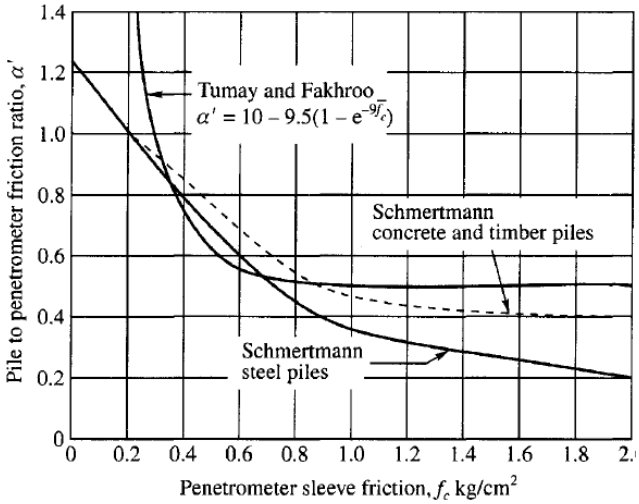


Figure 11. Penetrometer design curves for pile side friction in clay (Schmertmann,1978).

$\overline{f_c}$ average penetrometer sleeve friction, kg/cm^2;

Cohesionless soils:

$$Q_s = K * \left(\frac{1}{2} * \left(\overline{f_c} * A_s\right)\{0 - 8d\} + \left(\overline{f_c} * A_s\right)\{8d - L\}\right) \qquad (37)$$

$\overline{f_s}$ average value of unit pile friction within the depths specified, kg/cm^2;

K correction factor;

The diagram for determining the correction factor is illustrated in Figure 12.

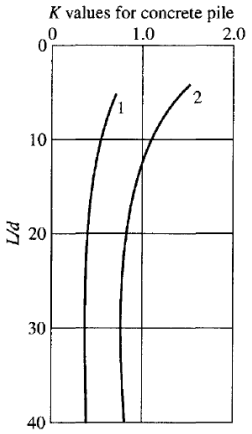


Figure 12. Correction factor from for electrical penetrometer (Schmertmann,1978).

Allowable Pile Capacity, kN:

$$Q_{all} = \frac{Q_{ult}}{FS} \qquad (38)$$

Q_{all} allowable load-carrying capacity for each pile, kPa;

FS factor of safety;

Table 16. Determination the safety factor

Value	Note
2,5	for the total ultimate capacity
1,5	for the shaft capacity
3,0	for the base capacity

Pile Group Efficiency

The group efficiency depends on type of soil, method of installation of piles and spacing of piles. There are some empirical formulas determination of the pile group efficiency.

$$E_g = \frac{Q_{ult;g}}{\Sigma Q_{ult}} \qquad (39)$$

Where

$Q_{ult;g}$ ultimate load bearing capacity of group piles, kN;

ΣQ_{ult} the sum of the ultimate load bearing capacity of piles, kN;

Cohesive soils:

$$Q_{ult;g} = n_1 * n_2 * (N_c * c_b * A_b + \Sigma\, \alpha * c_u * A_s) \qquad (40)$$

$$Q_{ult;g} = c_b * N_c * A * B + \Sigma\, 2 * (A + B) * L * c_u \qquad (41)$$

Compare and choose the smallest of the two values.

Where

A, B, L the dimensions of pile group (Fig. 13);

n_1 column count (Fig. 13);

n_2 row count (Fig.13);

c_b undrained shear strength at the base of the piles, kPa;

c_u undrained shear strength along the sides of the piles, kPa;

N_c bearing capacity factor (usually taken as 9.0);

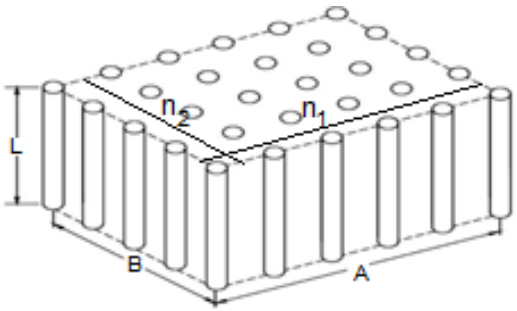


Figure 13. Group of piles (Smith, 2014).

Cohesionless soils:

$$E_g = 1 - \Theta * \frac{(n_1 - 1) * n_2 + (n_2 - 1) * n_1}{90 * n_1 * n_2} \qquad (42)$$

Where

Θ $\tan^{-1}(D/s)$, degrees;

D width or diameter of pile, m;

S spacing of piles center to center,m;

$$Q_{ult;g} = \left[1 - \Theta * \frac{(n_1 - 1) * n_2 + (n_2 - 1) * n_1}{90 * n_1 * n_2}\right] * \Sigma Q_{ult} \qquad (43)$$

$$If\ E_g < 1, then\ Q_{ult;g} = E_g * \Sigma Q_u \qquad (44)$$

$$If\ E_g > 1, then\ Q_{ult;g} = \Sigma Q_{ult} \qquad (45)$$

$$\Sigma Q_{ult} = n * Q_{ult} \qquad (46)$$

$$Q_{all;g} = \frac{Q_{ult;g}}{FS} \qquad (47)$$

Total Consolidation Settlement of pile groups in cohesive soils, m:

One concept of the pile group settlement evaluation was proposed by Terzagi and Peck (1967), which is based using an equivalent base plan area (B) (Z), located at a depth of 1/3 D above the pile toe. The load on a group of piles in this area is then the load transferred to the soil through an equivalent footing. It is assumed that the load extends within the pyramid with lateral slopes of 30 ° degrees and causes a uniform additional vertical stress at lower levels. This is illustrated in Figure 14.

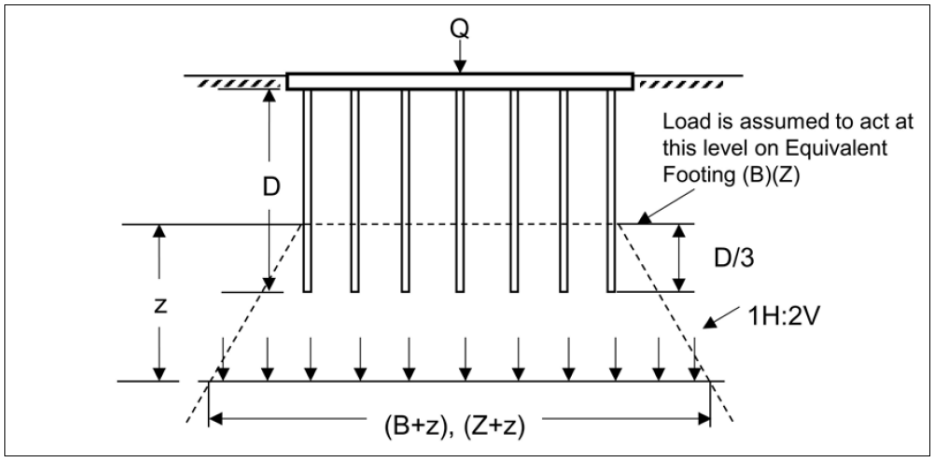


Figure 14. Equivalent footing concept (Terzagi & Peck, 1967).

Figure 15. presents other recommended locations of the equivalent footing for the following load transfer and soil resistance conditions:

- toe resistance piles in hard clay or sand underlain by soft clay,
- piles supported by shaft resistance in clay,
- piles supported by shaft resistance in sand underlain by clay, and
- piles supported by shaft and toe resistance in layered soil profile.

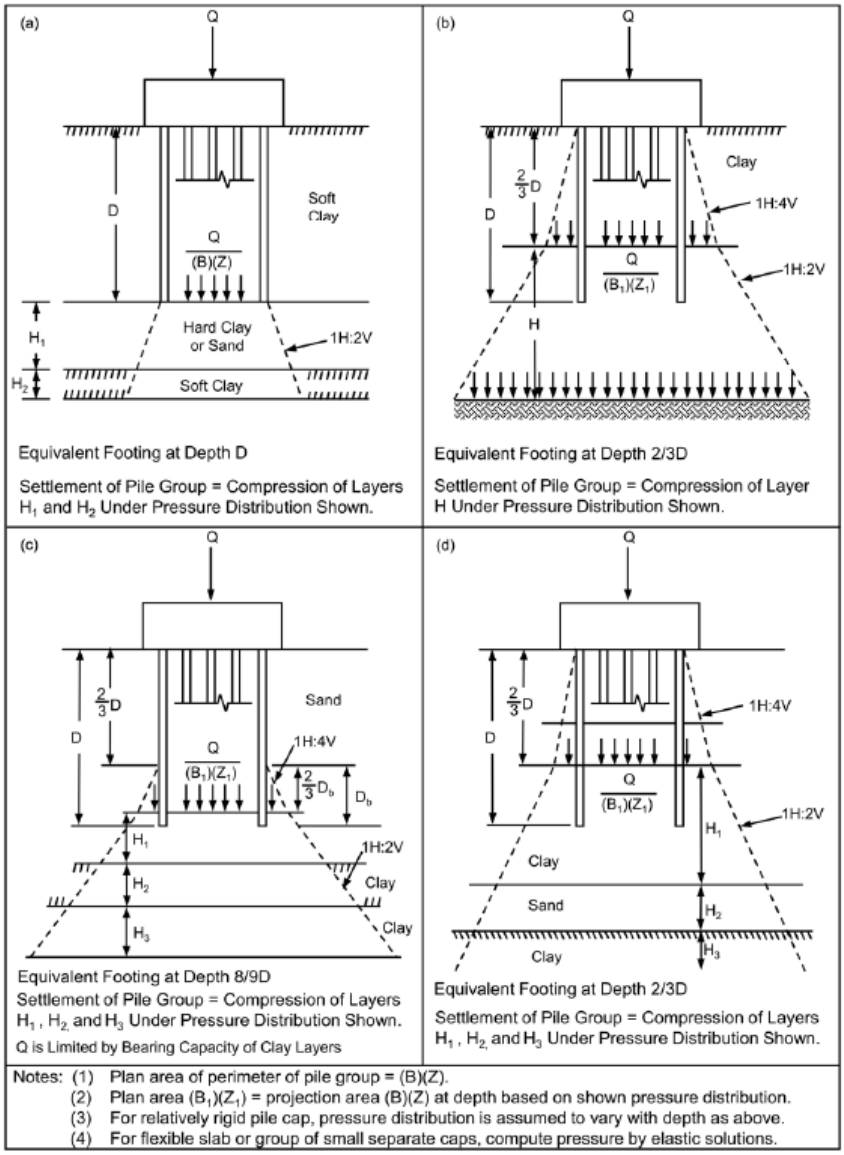


Figure 15. Stress distribution below equivalent footing for a pile group (Cheney & Chassie, 2000).

Consolidation settlement of layer i:

$$\Delta S_{c(i)} = \left[\frac{C_{c(i)} * H_i}{1 + e_{o(i)}}\right] \log\left[\frac{\sigma'_{o(i)} + \Delta\sigma'_{(i)}}{\sigma'_{o(i)}}\right] \qquad (48)$$

Where

$e_{o(i)}$ the initial void ratio of layer i*;*

$\Delta\sigma'_{(i)}$ the increase in pressure at the middle of each layer, kN/m^2;

$$\Delta\sigma'_{(i)} = \frac{Q_g}{(L_g + z_i) * (B_g + z_i)} \qquad (49)$$

$\sigma'_{o(i)}$ the effective overburden pressure at the middle of each layer, kN/m^2;

$$\sigma'_{o(1)} = \frac{\gamma_{s;1} L_1}{2} \qquad (50)$$

$$\overline{\sigma'_{o;2}} = \gamma_{s;1} * L_1 + \frac{\gamma_{s;2} L_2}{2} \qquad (51)$$

$$\gamma_{s;i} = \gamma_{sat;i} - \gamma_w \qquad (52)$$

Total Elastic Settlement of Piles, m:

$$S_e = S_{e1} + S_{e2} + S_{e3} \qquad (53)$$

Settlement of pile shaft, m:

$$S_{e1} = \frac{(Q_{wp} + \xi * Q_{ws}) * L}{A_b * E_p} \qquad (54)$$

Where

Q_{wp} load carried at the pile point under working load condition, kN;

$$Q_{wp} = \frac{Q_b}{FS} \qquad (55)$$

Q_{ws} load carried by skin resistance under working load condition, kN;

$$Q_{ws} = \frac{Q_s}{FS} \qquad (56)$$

ξ magnitude which depend on the nature of unit friction (skin) resistance distribution along the pile shaft (Fig.16);

L length of the pile, m;

E_p modulus of elasticity of the pile material, kN/m^2;

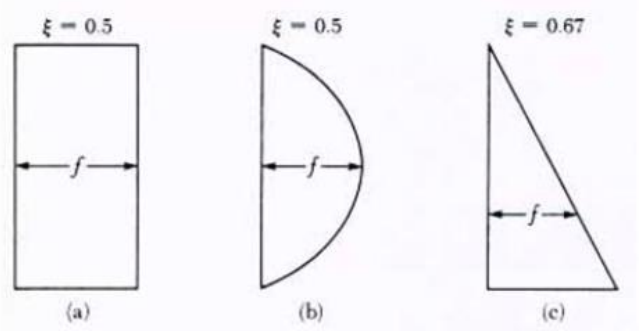


Figure 16. Variations of magnitude (Fundamentals of geotechnical engineering).

Settlement of pile caused by the load at the pile point, m:

$$S_{e2} = \frac{q_{wp} * D}{E_s} * (1 - \mu_s^2) * I_{wp} \qquad (57)$$

Where

q_{wp} point load per unit area at the pile point, kN/m^2;

$$q_{wp} = \frac{Q_{wp}}{A_b} \qquad (58)$$

D width or diameter of the pile, m;

E_s modulus of elasticity of soil at or below the pile point, kN/m^2;

μ_s Poisson's ratio of soil;

I_{wp} influence factor;

The influence factor depends on the shape and the L/B ratio and is shown in the following Table 17.

Table 17. Influence factor I_{wp} (Steinbrenner,1954)

Shape of foundation	Flexible foundation (I_{wp})			Rigid foundations (I_{wp})
	Centre	Corner	Average	
Circle	1.00	0.64	0.85	0.86
Square	1.12	0.56	0.95	0.82
Rectangle				
L/B = 1.5	1.36	0.68	1.20	1.06
L/B = 2	1.52	0.76	1.30	1.20
L/B = 5	2.10	1.05	1.83	1.70
L/B = 10	2.52	1.26	2.25	2.10
L/B = 100	3.38	1.69	2.96	3.40

Settlement of pile caused by the load transmitted along the pile shaft, m:

$$S_{e3} = \left(\frac{Q_{ws}}{P * L}\right) * \frac{D}{E_s} * (1 - \mu_s^2) * I_{ws} \qquad (59)$$

Where

P perimeter of the pile, m;

I_{ws} influence factor;

$$I_{ws} = 2 + 0{,}35\sqrt{\frac{L}{D}} \qquad (60)$$

Total settlement of group piles, m:

$$S_g = \xi_g * S \qquad (61)$$

Where

ξ_g group settlement factor;

$$\xi_g = \sqrt{\frac{D'}{D}} \qquad (62)$$

D' width of pile group section, m;

Check for:

1) GEO limit states

Degree of utilisation:

Bond and Harris (2008) recommend using the ratio of the design effect of actions to the corresponding resistance to verify GEO:

$$_{\mathrm{GEO}} = \frac{\mathrm{F_{c;d}}}{\mathrm{R_{c;d}}} \leq 100\% \qquad (63)$$

Since $_{\mathrm{GEO}}$ < 100%, the GEO limit state requirement is satisfied.

Since $_{GEO}$ > 100%, the design of the pile does not satisfy the GEO limit state requirement.

Over design-factor:

Frank et. al. (2004) define the ratio of the design resistance to the corresponding design effect of actions:

$$\Gamma = \frac{\mathrm{R_{c;d}}}{\mathrm{F_{c;d}}} \qquad (64)$$

Since Γ > 1, the GEO limit state requirement is satisfied.

Since Γ < 1, the design of the pile does not satisfy the GEO limit state requirement.

2) SLS limit states

Settlement of piles and piles group

S_{max} (ρ_{max}) the maximum settlement of any portion of the foundation, mm

Δ the maximum differential settlement between any two portions of the foundation, mm

β= δ/l the maximum angular distortion of buildings with columns, where δ is the differential settlement between the adjacent column footings and l is the column spacing, mm

Deformations are illustrated in Figure 17.

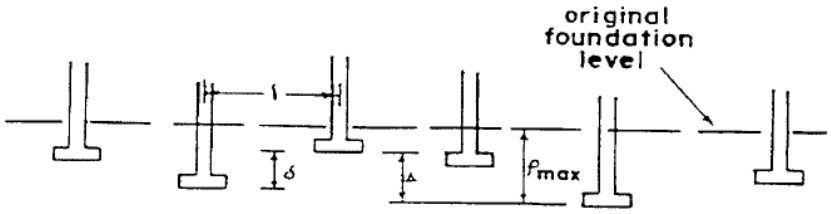


Figure 17. Definitions of maximum settlement (ρ_{max}), maximum differential settlement between any two portions (Δ), maximum angular distortion (β), (An introduction to foundation engeneering).

In Eurocode 7 the maximum allowable settlement for a single pile is 25 mm, for a pile group 60 mm, but usually the limits are determined by the project. The limits of deformations are illustrated in Figure 18,19 and Table 18,19,20.

Table 18. The maximum allowable settlements for buildings and load bearing walls

		Footings	Rafts
Maximum Settlement (mm)	**Clays**	75	75 to 125
	Sands	50	50 to 75
Maximum Differential Settlement (mm)	**Clays**	45	
	Sands	30	

(after Skempton & MacDonald, 1956)

Structure	**Maximum Average Settlement (mm)**
Buildings with plain brick walls on continuous and separate foundations (L = wall length, H = wall height)	
L/H ≥ 2.5	80
L/H ≤ 1.5	100
Buildings with brick walls reinforced with reinforced concrete or reinforced brick	150
Framed buildings	100
Solid reinforced concrete foundations of blast furnaces, smoke stacks, silos, water towers, etc.	300

(after Polshin & Tokar, 1957)

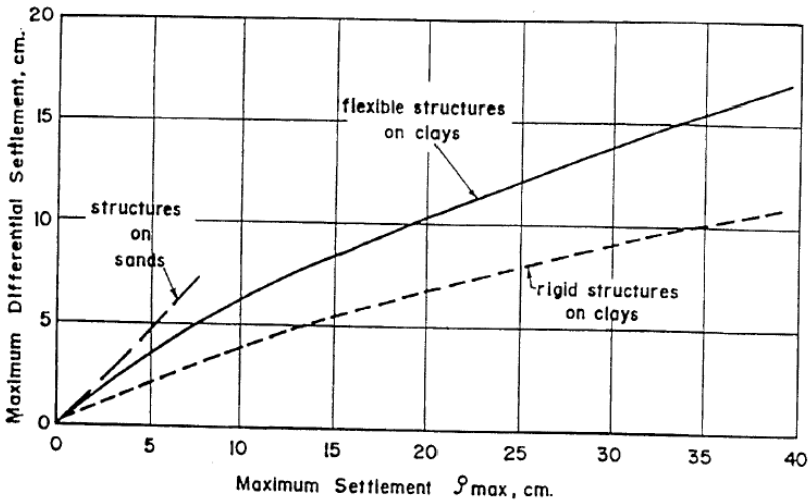


Figure 18. Envelopes of maximum observed differential settlement (Bjerrum, 1962)

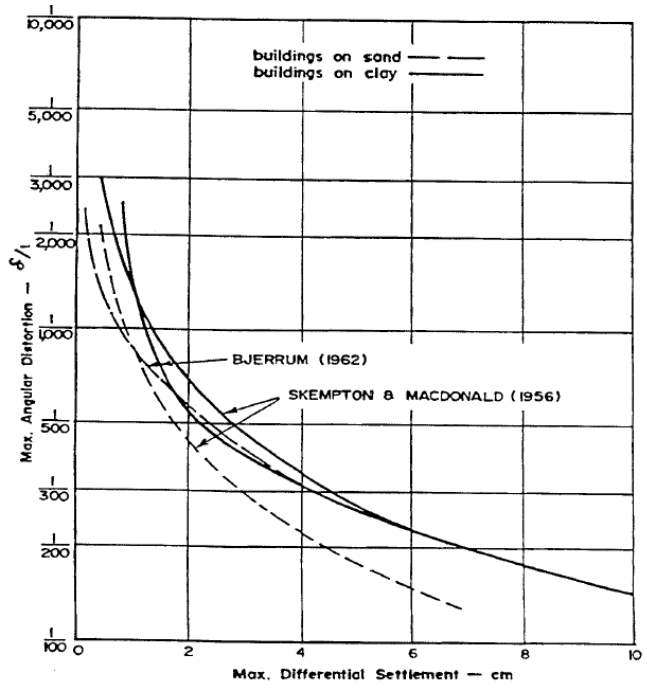


Figure 19. Observed relationship between maximum differential settlement and maximum angular distortion (Skempton and Macdonald (1956), Bjerrum (1962)

Table 19. Limits of angular distortion of buildings (Bjerrum,1963)

Criterion	Angular Distortion (δ/l)
Structural damage to building	1/150
Safe limit for flexible brick walls $\left(H/L < \frac{1}{4}\right)$	1/150
Considerable cracking in panel walls and brick walls	1/150
Difficulties with overhead cranes to be expected	1/300
First cracking in panel walls to be expected	1/300
Safe limit for buildings where cracking is not permissible	1/500
Danger limit for frames with diagonals	1/600
Limit where difficulties with machinery sensitive to settlements to be feared	1/750

Table 20. Maximum allowable angular distortion of buildings and load bearing walls (Polshin & Tokar,1957)

	Sand and Hard Clay	Plastic Clay
Angular distortion for:		
(a) steel and reinforced concrete frame structures	.002	.002
(b) end rows of columns with brick cladding	.007	.001
Relative deflection of plain brick walls for:		
(a) multi storey buildings L/H ≤ 3 L/H ≥ 5 (L = length of deflected part of wall, H = wall height)	 .0003 .0005	 .0004 .0007
(b) one storey mills	.001	.001

4.1 Example of design and calculation friction piles from ground tests result

Engineering and geological section is shown in Figure 20.

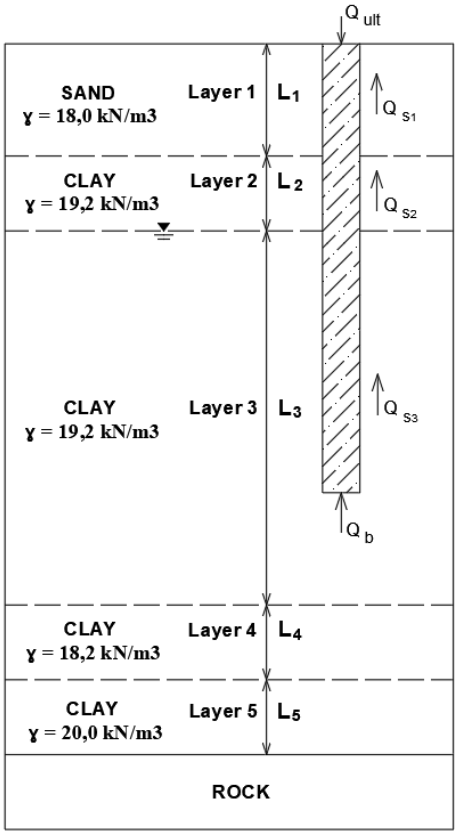


Figure 20. Engineering and geological section (Skutina,2020).

Given data:

- Cross-section precast concrete driven friction pile: a = 0,4 m; b = 0,4 m;
- Soil: layer 1 - sand; layer 2,3,4,5 – clay;
- Embedded length: L_1 = 3 m; L_2 = 2 m;
- Undrained shear strength: (pile 1) c_1 = 55 kPa; c_2 = 102 kPa; c_3 = 105 kPa;

  (pile 2) c_1 = 50 kPa; c_2 = 100 kPa; c_3 = 100 kPa;

  (pile 3) c_1 = 60 kPa; c_2 = 110 kPa; c_3 = 108 kPa;

- Friction Angle: ϕ_1 = 18 degree; ϕ_2 = 19,2 degree; ϕ_3 = 19,2 degree;
- Unit Weight: γ_1 = 17,3 kN/ m^3; γ_1 = 18,1 kN/ m^3; γ_1 = 19,6 kN/ m^3;

Solution:

Design Approach 2. (Axially loaded piles)

Combination 1: A1 “+” M1 “+” R2

Step 1. Design Action (Load) (A1):

$$F_{c;d} = \gamma_G * G_{rep} + \gamma_Q * Q_{rep} = \ 1{,}49 * \ 800 + 0{,}9 * 350 = 1503{,}00\ kN$$

$$\gamma_G = 1{,}35 * K_{FI} = 1{,}35 * 1{,}1 = 1{,}49$$

Note: For transparency in the calculation any difference in the weight of the pile and the displaced overburden load is not included.

Step 2. Material Factors (M1):

$$\gamma_{cu} = 1{,}0$$

So, c_1 = 50 kPa; c_2 = 100 kPa; c_3 = 100 kPa

Note: No modification to adopted soil parameters is required for the design of axially loaded piles.

Step 3. Design Resistance (R1):

$$R_{c;k} = \frac{\left(R_{b;cal} + R_{s;cal}\right)_{mean}}{\xi_3} \quad and \ \ R_{c;k} = \frac{\left(R_{b;cal} + R_{s;cal}\right)_{min}}{\xi_4}$$

The Determination of the required pile length to carry prescribed load. Try 12 m long pile (L_3 =7,0 m) and 30 - number of piles.

Base resistance in cohesive soil (pile 1):

$$R_{b;cal} = A_b * q_b = a * b * N_c * c_b = 0{,}4 * 0{,}4 * 9{,}0 * 105 = 151{,}20\ kN$$

Shaft resistance in cohesive soil (pile 1):

$$R_{s;cal} = \Sigma A_{s;i} * f_{s;i} = \Sigma P * L_i * \alpha * c_{u;i}$$
$$= 2 * (0{,}4 + 0{,}4) * 2 * 0{,}5 * 102 + 2 * (0{,}4 + 0{,}4) * 7 * 0{,}5 * 105$$
$$= 751{,}20\ kN$$

Shaft resistance in cohesionless soil (pile 1):

$$R_{s;cal} = \Sigma A_{s;i} * f_{s;i} = \Sigma P * L_i * K_s * \overline{\sigma_{s;i}'} * tan\delta$$
$$= 2 * (0{,}4 + 0{,}4) * 3{,}0 * 2{,}0 * 27 * tan(0{,}75 * 32) = 82{,}42\ kN$$

$$\overline{\sigma'_{s;1}} = \frac{\gamma_{s;1} L_1}{2} = \frac{(\gamma_{sat;1} - \gamma_w) * L_1}{2} = \frac{(18{,}0 - 0) * 3}{2} = 27{,}0\ kPa$$

Table 21. Ground tests results

№	Parameter	Borehole no.			Mean	Min
		1	2	3		
1	Calculated Base resistance, $R_{b;cal}$	151,20	144,00	155,52	150,24	-
2	Calculated Shaft resistance, $R_{s;cal}$	866,60	835,40	896,20	866,07	-
3	Total resistance, $R_{b;cal}$+$R_{s;cal}$	1017,80	979,40	1051,72	1016,31	979,40
4	Compressive resistance, $R_{c;k}$	-	-	-	-	587,46
5	Design compressive resistance, $R_{c;d}$	-	-	-	-	489,55

Mean calculated total resistance:

$$R_{c;cal}(mean) = \frac{1017{,}80 + 979{,}40 + 1051{,}72}{3} = 1016{,}31 kN$$

Minimum calculated total resistance:

$$R_{c;cal}(min) = 979{,}40\ kN$$

Compressive resistance:

From Table 8, $\xi_3 = 1{,}73$; $\xi_4 = 1{,}6$

$$R_{c;k} = \frac{R_{c;cal}(mean)}{\xi_3} = \frac{1016{,}31}{1{,}73} = 587{,}46\ kN$$

$$R_{c;k} = \frac{R_{c;cal}(min)}{\xi_4} = \frac{979{,}40}{1{,}6} = 612{,}13\ kN$$

The smallest of values obtained from these equations should be used in design, $R_{c;k}$=587,46 kN

Design compressive resistance, $R_{c;d}$

From Table 9, γ_t = 1,2

$$R_{c;d} = \frac{R_{c;k}}{\gamma_t} = \frac{587{,}46}{1{,}2} = 489{,}55\ kN$$

GEO limit state:

$$_{GEO} = \frac{F_{c;d}}{n * R_{c;d}} * 100\% = \frac{1503}{30 * 489{,}55} * 100\% = 10{,}23\%$$

Since $_{GEO}$ < 100 %, the GEO limit state requirement is satisfied.

Conclusion: 30 piles with a length of 12 m and diameter of 400 mm can carry a load of 1150 kN.

The design compressive resistance of the ground may be derived by either:

Partial safety numbers γ_b and γ_s corrective the value of the model factor γ_{rd}. The value to be used for friction piles is at least 1.60 (7.6.2.3(8) NA SFS-EN 1997-1). This method does not take location into account.

From Table 9, γ_b, γ_s = 1,2; γ_{rd} = 1,75

$$R_{b;d} = \frac{R_{b;k}}{\gamma_{rd}} = \frac{150{,}24}{1{,}75} = 85{,}85\ kN$$

$$R_{s;d} = \frac{R_{s;k}}{\gamma_{rd}} = \frac{866{,}07}{1{,}75} = 494{,}90\ kN$$

$$R_{c;d} = \frac{R_{b;d}}{\gamma_b} + \frac{R_{s;d}}{\gamma_s} = \frac{85{,}85 + 494{,}90}{1{,}2} = 483{,}96\ kN$$

GEO limit state:

$$_{GEO} = \frac{F_{c;d}}{n * R_{c;d}} * 100\% = \frac{1503}{30 * 483{,}96} * 100\% = 10{,}35\%$$

Since $_{GEO}$ < 100 %, the GEO limit state requirement is satisfied.

Step 5. Pile Capacity:

Ultimate bearing capacity:

$$Q_{ult} = Q_b + Q_s = 979{,}40\ kN$$

Allowable or working axial load:

$$\mathrm{Q_{all}} = \frac{\mathrm{Q_{ult}}}{\mathrm{FS}} = \frac{979{,}40}{2{,}5} = 391{,}76\ \mathrm{kN}$$

Step 6. Pile Group Capacity

Ultimate bearing Capacity in cohesive soil:

$$Q_{ult;g} = n_1 * n_2 * (A_b * N_c * c_b + \Sigma\, A_s * \ \alpha * c_u) = 6 * 5 * 979{,}40 = 29382\ kN$$

$$Q_{ult;g} = 1{,}3 * c_b * N_c * A * B + \Sigma\, 2 * (A + B) * L * c_u = 1{,}3 * 100 * 9 * 5{,}2 * 3{,}9 + 2 * (5{,}2 + 3{,}9) * 9 * 100 = 23728 + 16380 = 40108\ kN$$

The smallest of values obtained from these equations should be used in design, $Q_{ult;g} = 29382\ kN$

Ultimate bearing Capacity in cohesionless soil:

$$E_g = 1 - \Theta * \frac{(n_1 - 1) * n_2 + (n_2 - 1) * n_1}{90 * n_1 * n_2}$$

$$= 1 - \frac{1}{\tan\left(\frac{0,4}{0,9}\right)} * \frac{(6-1)*5 + (5-1)*6}{90*6*5} = 0,96$$

$E_g < 1, then\ Q_{ult;g} = E_g * \Sigma Q_u$

$\Sigma Q_{ult} = n * Q_{ult} = 30 * 979,40 = 29382\ kN$

$Q_{ult;g} = E_g * \Sigma Q_{ult} = 0,96 * 29382 = 28206,72\ kN$

$Q_{all;g} = \frac{Q_{ult;g}}{FS} = \frac{28206,72}{2,5} = 11282,70\ kN$

Step 7. Foundation Settlement:

Consolidation settlement of pile group is shown in Figure 21.

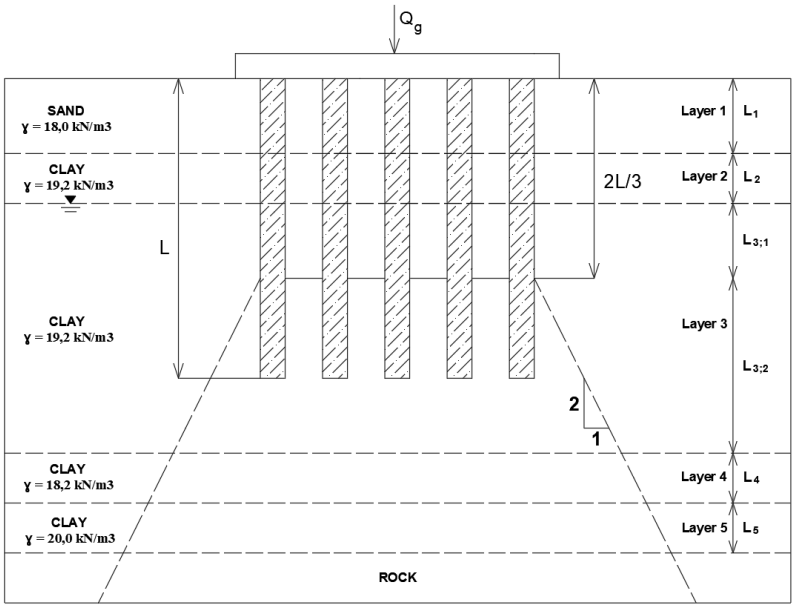


Figure 21. Consolidation settlement of pile group (Skutina,2020)

The calculation of the consolidation of a group of piles can be estimated by assuming an approximate distribution method, commonly known as 2:1.

Because the lengths of the piles are 12 m each, the stress distribution starts at a depth of 8 m below the top of the pile (2L/3).

The effective overburden pressure at the middle of each layer:

For Layer 3

$$\sigma'_{o(3)} = \gamma_{s;1} * L_1 + \gamma_{s;2} * L_2 + (\gamma_{sat;3} - \gamma_{w)} * L_{3;1} + \frac{(\gamma_{sat;3} - \gamma_{w)} * L_{3;2}}{2} =$$

$$= 18 * 3 + 19{,}2 * 2 + (19{,}2 - 9{,}81) * 3 + (19{,}2 - 9{,}81) * 3{,}5 =$$

$$= 153{,}4 \frac{kN}{m3}$$

For Layer 4

$$\sigma'_{o(4)} = \gamma_{s;1} * L_1 + \gamma_{s;2} * L_2 + (\gamma_{sat;3} - \gamma_{w)} * L_3 + \frac{(\gamma_{sat;4} - \gamma_{w)} * L_4}{2}$$

$$= 18 * 3 + 19{,}2 * 2 + (19{,}2 - 9{,}81) * 10 + \frac{(18{,}2 - 9{,}81) * 2}{2}$$

$$= 194{,}69 \frac{kN}{m3}$$

For Layer 5

$$\sigma'_{o(5)} = \gamma_{s;1} * L_1 + \gamma_{s;2} * L_2 + (\gamma_{sat;3} - \gamma_{w)} * L_3 + (\gamma_{sat;4} - \gamma_{w)} * L_4$$

$$+ \frac{(\gamma_{sat;5} - \gamma_{w)} * L_5}{2}$$

$$= 18 * 3 + 19{,}2 * 2 + (19{,}2 - 9{,}81) * 10 + (18{,}2 - 9{,}81) * 2$$

$$+ \frac{(20 - 9{,}81) * 2}{2} = 213{,}27 \frac{kN}{m3}$$

The increase in pressure at the middle of each layer:

For Layer 3

$$\Delta\sigma'_{(3)} = \frac{Q_g}{(L_g + z_3) * (B_g + z_3)} = \frac{2500}{(5{,}2 + 3{,}5) * (3{,}9 + 3{,}5)} = 38{,}83\ kN/m2$$

$$z_3 = \frac{L_{3;2}}{2} = 3{,}5\ m$$

For Layer 4

$$\Delta\sigma'_{(4)} = \frac{2500}{(5{,}2+8)*(3{,}9+8)} = 15{,}92\ kN/m2$$

$$z_4 = L_{3;2} + \frac{L_4}{2} = 8\ m$$

For Layer 5

$$\Delta\sigma'_{(5)} = \frac{2500}{(5{,}2+10)*(3{,}9+10)} = 11{,}83\ kN/m2$$

$$z_5 = L_{3;2} + L_4 + \frac{L_5}{2} = 10\ m$$

Consolidation settlement:

For Layer 3

$$\Delta S_{c(3)} = \left[\frac{C_{c(3)} * H_1}{1+e_{o(3)}}\right]\log\left[\frac{\sigma'_{o(3)} + \Delta\sigma'_{(3)}}{\sigma'_{o(3)}}\right] = \left[\frac{0{,}23*7}{1+0{,}8}\right]\log\left[\frac{153{,}4+38{,}83}{153{,}4}\right] = 0{,}088\ m$$

For Layer 4

$$\Delta S_{c(4)} = \left[\frac{0{,}34*2}{1+1{,}08}\right]\log\left[\frac{194{,}69+15{,}92}{194{,}69}\right] = 0.011\ m$$

For Layer 5

$$\Delta S_{c(5)} = \left[\frac{0{,}2*2}{1+0{,}7}\right]\log\left[\frac{213{,}27+11{,}83}{213{,}27}\right] = 0{,}006\ m$$

Total:

$$\Delta S_c = \Delta S_{c(3)} + \Delta S_{c(4)} + \Delta S_{c(5)} = 0{,}088 + 0{,}011 + 0{,}006 = 0{,}104\ m = 104\ mm$$

4.2 Example of design and calculation friction piles by CPT

Given data:

- Cross-section precast concrete driven friction pile: a = 0,3 m; b = 0,3 m;
- Soil: medium dense sand;

- Embedded length: L = 12 m;

Static cone penetration tests were conducted in this area using an electric conical penetrometer. The qc and fc values obtained from the test were plotted as a function of depth and are shown in Figure 22.

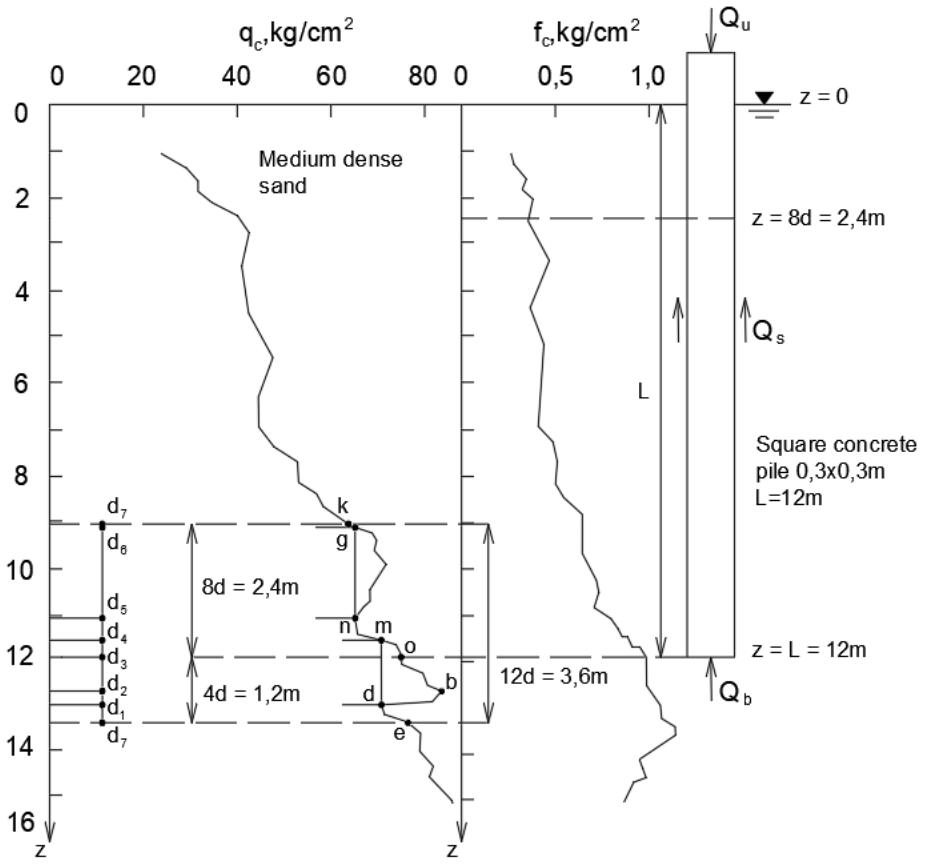


Figure 22. Profile of q_c and f_c values (CPT), (Skutina,2020).

Solution:

Step 1. Cone penetration value:

$$q_{c1} = \frac{d_3 * \frac{q_0 + q_b}{2} + d_2 * \frac{q_b + q_d}{2} + d_1 * \frac{q_d + q_e}{2}}{4d}$$

$$= \frac{0{,}8 * \frac{76 + 85}{2} + 0{,}3 * \frac{85 + 71}{2} + 0{,}4 * \frac{71 + 80}{2}}{4 * 0{,}3} = 98{,}33 \frac{kg}{cm^2} =$$

$$= 9833 \frac{kN}{m^2}$$

$q_{c2} = q_d$ the minimum value below the tip of pile within 4d depth $= 71 \frac{kg}{cm^2}$

$$= 7100 \frac{kN}{m^2}$$

$$q_{c3} = \frac{d_4 * q_m + d_5 * \frac{q_m + q_n}{2} + d_6 * q_n + d_7 * \frac{q_g + q_k}{2}}{8d}$$

$$= \frac{0{,}4 * 71 + 0{,}5 * \frac{71 + 65}{2} + 2{,}0 * 65 + 0{,}1 * \frac{65 + 60}{2}}{8 * 0{,}3} = 82{,}77 \frac{kg}{cm^2}$$

$$= 8277 \frac{kN}{m^2}$$

$$q_p = \frac{(q_{c1} + q_{c2}) + q_{c3}}{2} = \frac{(9833 + 7100) + 8277}{2} = 12605 \frac{kN}{m^2}$$

Step 2. Pile Capacity:

Base capacity:

$$Q_b = A_b * q_b = A_b * q_p = 0{,}3^2 * 12605 = 1134{,}47\ kN$$

Shaft capacity:

$$Q_s = K * \left(\frac{1}{2} * (\overline{f_c} * A_s)\{0 - 8d\} + (\overline{f_c} * A_s)\{8d - L\}\right) =$$

$$= 0{,}8 * \left(\frac{1}{2} * 0{,}34 * 10 * 4 * 0{,}3 * 2{,}4 + 0{,}71 * 10 * 4 * 0{,}3 * 9{,}6\right) * 100$$

$$= 693{,}5\ kN$$

$For\ \frac{L}{D} = \frac{12}{0{,}3} = 40, K = 0{,}8\ for\ concrete\ pile$

Ultimate bearing capacity:

$$Q_{ult} = Q_b + Q_s = 1134{,}45 + 693{,}5 = 1828\ kN$$

Allowable or working axial load:

$$\mathrm{Q_{all}} = \frac{\mathrm{Q_{ult}}}{\mathrm{FS}} = \frac{1828}{2{,}5} = 731{,}19\ \mathrm{kN}$$

Step 3. Total Elastic Settlement:

Settlement of pile shaft:

$$S_{e1} = \frac{(Q_{wp} + \xi * Q_{ws}) * L}{A_b * E_p} = \frac{(370{,}59 + 0{,}67 * 453{,}09) * 12}{0{,}09 * 21 * 10^6} = 0{,}0043\ m$$

$$Q_{wp} = \frac{Q_b}{FS} = \frac{1111{,}78}{3{,}0} = 370{,}59\ kN$$

$$Q_{ws} = \frac{Q_s}{FS} = \frac{679{,}63}{1{,}5} = 453{,}09\ kN$$

Settlement of pile caused by the load at the pile point, m:

$$S_{e2} = \frac{q_{wp} * D}{E_s} * (1 - \mu_s^2) * I_{wp} = \frac{4117{,}7 * 0{,}3}{60000} * (1 - 0{,}3^2) * 0{,}95 = 0{,}018\ m$$

$$q_{wp} = \frac{Q_{wp}}{A_b} = \frac{370{,}59}{0{,}09} = 4117{,}7\ \frac{kN}{m^2}$$

Settlement of pile caused by the load transmitted along the pile shaft:

$$S_{e3} = \left(\frac{Q_{ws}}{P * L}\right) * \frac{D}{E_s} * (1 - \mu_s^2) * I_{ws} = \frac{453{,}09}{2 * (0{,}3 + 0{,}3) * 12} * \frac{0{,}3}{60000} * (1 - 0{,}3^2) * 4{,}21$$
$$= 0{,}0006\ m$$

$$I_{ws} = 2 + 0{,}35\sqrt{\frac{L}{D}} = 2 + 0{,}35\sqrt{\frac{12}{0{,}3}} = 4{,}21$$

Total elastic settlement of single pile:

$$S_e = S_{e1} + S_{e2} + S_{e3} = 0{,}0043 + 0{,}018 + 0{,}0006 = 0{,}023\ m = 23\ mm$$

23 mm < 25 mm, the settlement SLS condition is satisfied.

5 Analysis

The design of the foundation of friction piles is a volumetric algorithm that includes various factors that depend on each other. Let us consider some of them.

The length of the friction piles is determined depending on the design load of the structure and the "weakness" of the soil lying on the site. The larger the first component and the smaller the second, the deeper the piles are installed. In some cases, they are made compound.

Since the strength of the pile material is obviously almost higher than necessary, the calculation of the bearing capacity is usually made only by the strength of the soil.

Despite of the fact that the stability of such piles is achieved by a combined method - from the resistance to the shaft and base, the overall reliability of friction piles is less - they are subject to precipitation under the influence of strong external loads, while the end-bearing piles, due to their support on incompressible soil, are never subjected to settlement.

To increase the overall stability of the foundation, hanging piles are often installed using the pile bush method - 3-6 piles in close proximity to each other. Due to this arrangement, additional compaction of the soil between the piles is achieved and, as a result, the soil resistance to increases to the shaft, and the average load per pile is also reduced.

In such cases where the deformation values still exceed the limit states of STR and GEO, the following methods for reducing the drag force should be used:

- Increase the number of piles

Using more piles reduces the maximum axial compressive force that the pile section carries.

- Increase the structural resistance

Using piles with higher strength or thicker walls results in increased factored structural resistance.

- Reduce soil settlement by preloading

Using preload before the installation of the pile allows to pre-consolidated the soil and in the future to reduces the possible soil settlements.

- Use a friction reducer

Using of a plastic wrap and bitumen coating can reduce friction between the pile and soil.

6 Conclusion

The field of application of pile foundations is residential, industrial and hydraulic construction. The same reinforced concrete structure can work in the ground in two ways - as a friction pile, or as an end-bearing pile. In practice, the difference is in the length of the reinforced concrete pile. End-bearing piles are usually longer. There are cases when their use is not necessary, that is, deep driving into the rock soil is not required, and you can limit the load-bearing capacity of friction piles, saving its length.

This document is based on European standards and is a manual with an Excel file with general instructions and reference literature for the design and calculation of friction piles in various situations. The guide allows to quickly and conveniently predict, analyze, and avoid possible critical situations by defining and freely changing parameters after passing the limit states.

It clearly shows the transition of data from on-site tests and laboratory tests to an analytical form of calculation based on specific examples in automatic mode. For each individual case that differs from the examples, it is necessary to adjust the calculation independently, this depends on the soil layers and their characteristics.

List of references

1. Pile Design and Construction Practice. 6th Edition. Michael Tomlinson, John Woodward
2. Design and Construction of Driven Pile foundations - Volume I, Patrick J. Hannigan, Frank Rausche, Garland E. Likins, Brent R. Robinson, Matthew L. Becker.
3. Elements of soil mechanics. 9th Edition. Ian Smith
4. Fundamentals of geotechnical engineering. 5th Edition. Braja M.Das
5. Principles of Foundation Engineering 7th Edition. Braja M.Das
6. Eurocode 7: Geotechnical design - Part 1: General Rules.
7. Eurocode 7: Geotechnical design. Worked examples. Andrew J. Bond, Bernd Schuppener, Giuseppe Scarpelli, Trevor L.L. Orr
8. National Annex to SFS-EN 1997-1 Eurocode 7: Geotechnical design Part 1-General Rules.
9. Design of pile foundations. Robert N. Hunter, Scheffer Lang, W. N. Carey, JR.
10. Geotechnical Engineering: Principles and Practices of Soil Mechanics and Foundation Engineering (Civil and Environmental Engineering). Chapter 15: Deep Foundations I: Pile Foundations. Murthy V.N.S.
11. Deep foundation: Pile foundation. Marcel Dekker
12. Worked examples- design of pile foundations. Trevor Orr
13. Pile Load Capacity – Calculation methods, Bogumil Wrana
14. Technology and Practice in Geotechnical Engineering. Joseph B.A.
15. Geotechnical manual. Chapter 6. Geotechnical analyses.
16. A designers' simple guide to BS EN 1997.
17. Advanced Foundation Engineering - I, Module 8. NPTEL.
18. Geotechnical handbook. Mangushev R.A.
19. An introduction to foundation engineering.

Appendices

Appendix 1. GEO Limit State

Appendix 2. Ultimate bearing capacity

Appendix 3. Consolidation settlement

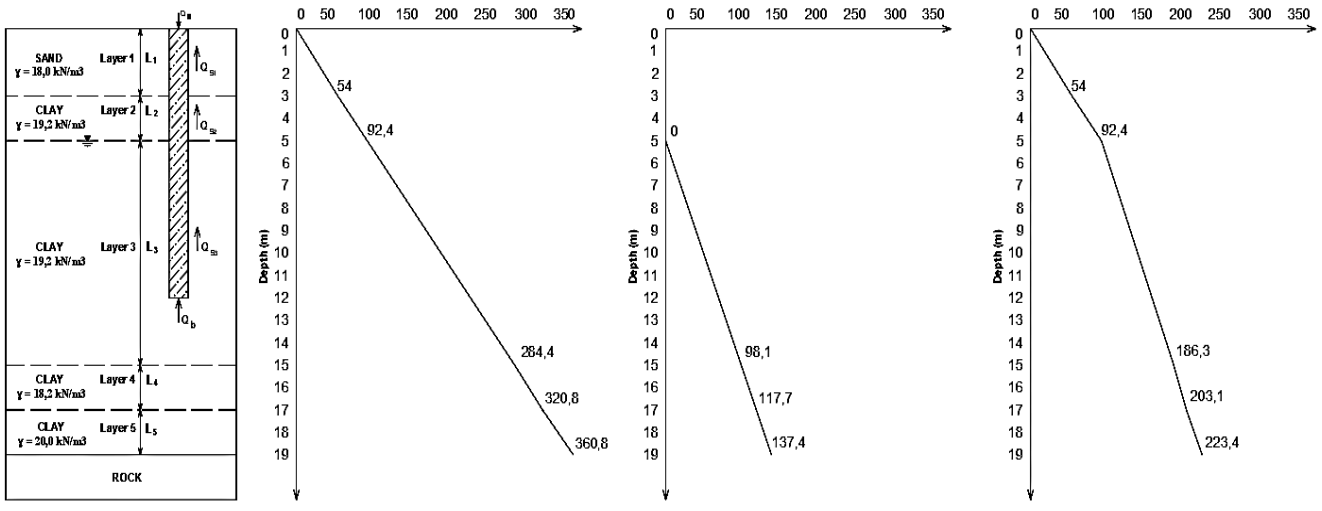
Appendix 4. CPT

Appendix 5. Elastic Settlement

NESTE	Excel file for calculation foundations of friction piles
Author	Date
Skutina Valeriia	22.05.2020

Bearing Capacity

Effective overburden pressure



Pile dimensions

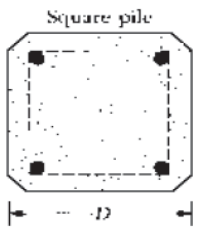
Cross section:

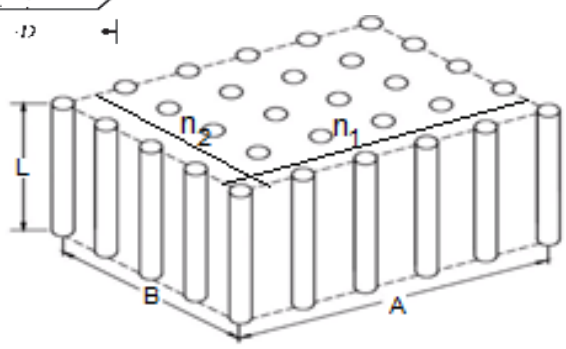
Piles:

a=	0,4	m
b=	0,4	m
or		
r=		
Perimeter:		
P=	1,6	m

Pile group:

A=	5,2	m
B=	3,9	m
P_g=	18,2	m
A_g=	20,28	m^2
number of piles=	30	
column count,n_1=	6	
row count,n_2=	5	





Soil parameters

№	soil	L, m	γ,kN/m^3	$S_{u(2)}$, kPa	e_0	C_c	ϕ, degrees
1	sand	3	18	50	-	-	32
2	clay	2	19,2	100	-	-	-
3	clay	10	19,2	100	0,8	0,23	38
4	clay	2	18,2	75	1,08	0,34	-
5	clay	2	20	125	0,7	0,2	-
6	rock	-	-	-	-	-	-

№	Parameter	Formula	Value	Unit	Note
		GEO limit state			
Design values of actions, $F_{c;d}$		$F_{c;d} = \gamma_G * G_{rep} + \gamma_Q * Q_{rep}$	1503,00	kN	-
1	Partial factor, γ_G	$\gamma_G = 1{,}35 * K_{FI}$	1,49	-	Table 6
2	Partial factor, γ_Q	-	0,9	-	
3	K_{FI}	-	1,1	-	Table 7
4	Permanent action, G_{rep}	-	800,00	kN	Design value
5	Variable action, Q_{rep}	-	350,00	kN	Design value
Design compressive resistances, $R_{c;d}$					
		$R_{c;d} = \frac{R_b}{\gamma_b} + \frac{R_s}{\gamma_s}$	466,38	kN	-
6	Partial factor, γ_b	-	1,2	-	Table 9
7	Partial factor, γ_s				
8	Base resistance, R_b	$R_b = \frac{A_b * q_b}{\gamma_{rd}}$	82,29	kN	-
9	Shaft resistance, R_s	$R_{s;i} = \frac{\Sigma A_{s;i} * f_{s;i}}{\gamma_{rd}}$	477,37	kN	-
10	Model Factor, γ_{rd}	-	1,75	-	> 1,60

11	Area of the pile base, A_b	$A_b = a * b$	0,16	m^2	Square section
		$A_b = \pi r^2$	-	m^2	Circular section
12	Characteristics of unit base resistance, q_b	$q_b = N_c * c_b$	900,00	kPa	Cohesive soils
		$q_b = \sigma_b' * \ N_q$	-	kPa	Cohesionless soils
13	Area of the pile base, A_s	$A_{s;i} = 2(a+b) * l_i$			Square section
		$A_{s;i} = 2\pi r * l_i$			Circular section
14	Characteristics of unit shaft resistance in the i-th layer, f_s	$f_{s;i} = \Sigma\alpha * c_{u;i}$	100,00	kPa	Cohesive soils
		$f_{s;i} = K_s * \overline{\sigma_s'} * tan\delta$	24,04	kPa	Cohesionless soils
15	Bearing capacity factor, N_c	-	9,0	-	Table 12
16	Bearing capacity coefficient, N_q	-	-	-	Figure 7
17	Adhesion factor, α	-	0,50	-	Table 14
18	Coefficient of lateral earth pressure, K_s	-	1	-	loose soil
			2		dense soil
19	Angle of friction between the pile and the soil, δ	$\delta = 0{,}75\varphi'$	-	degrees	Table 15
20	Effective overburden pressure at the base of the pile, σ'_b	$\sigma'_b = \Sigma\gamma_{b;i}L_i = \gamma_{b;1}L_1 + \gamma_{b;2}L_2 + \gamma_{b;i}L_i$	-	kPa	Diagram of effective overburden pressure
21	Average effective overburden pressure acting along the embedded length of the pile shaft, $\sigma'_{s;i}$	$\overline{\sigma'_{s;i}} = \gamma_{s;1}L_1 + \gamma_{s;2}L_2 + \frac{\gamma_{s;i}L_i}{2}$	27,00	kPa	
GEO limit state		$F_{c;d} \leq R_{c;d}$	-		
Length of the pile		-	12,00	m	-

The determination of pile length is based on equality of the GEO limit state :

$$F_{c;d} = n * R_{c;d} \qquad \Lambda_{GEO} = \frac{F_{c;d}}{n * R_{c;d}} * 100\%$$

L3= 7

λGEO= 10,74 ,the GEO limit state requirement is satisfied

Design compressive resistance from ground tests results			
№	1	2	3
Cu_1	55	50	60
Cu_2	102	100	110
Cu_3	105	100	108

ξ_3=	1,73
ξ_4=	1,6

№	Parameter	Borehole no.			Mean	Min
		1	2	3		
1	Base resistance, $R_{b;cal}$	151,20	144,00	155,52	150,24	-
2	Shaft resistance, $R_{s;cal}$	866,60	835,40	896,20	866,07	-
3	Total resistance, Rb;cal+Rs;cal	1017,80	979,40	1051,72	1016,31	979,40
4	Compressive resistance, Rc;k	-	-	-	-	587,46
5	Design compressive resistance, Rc;d	-	-	-	-	489,55

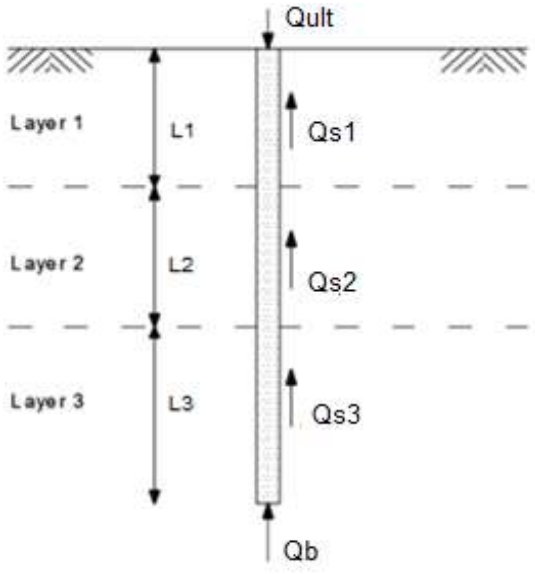
$$\Lambda_{GEO} = \frac{F_{c;d}}{n * R_{c;d}} * 100\% = \quad 10{,}23$$

587,46

612,13

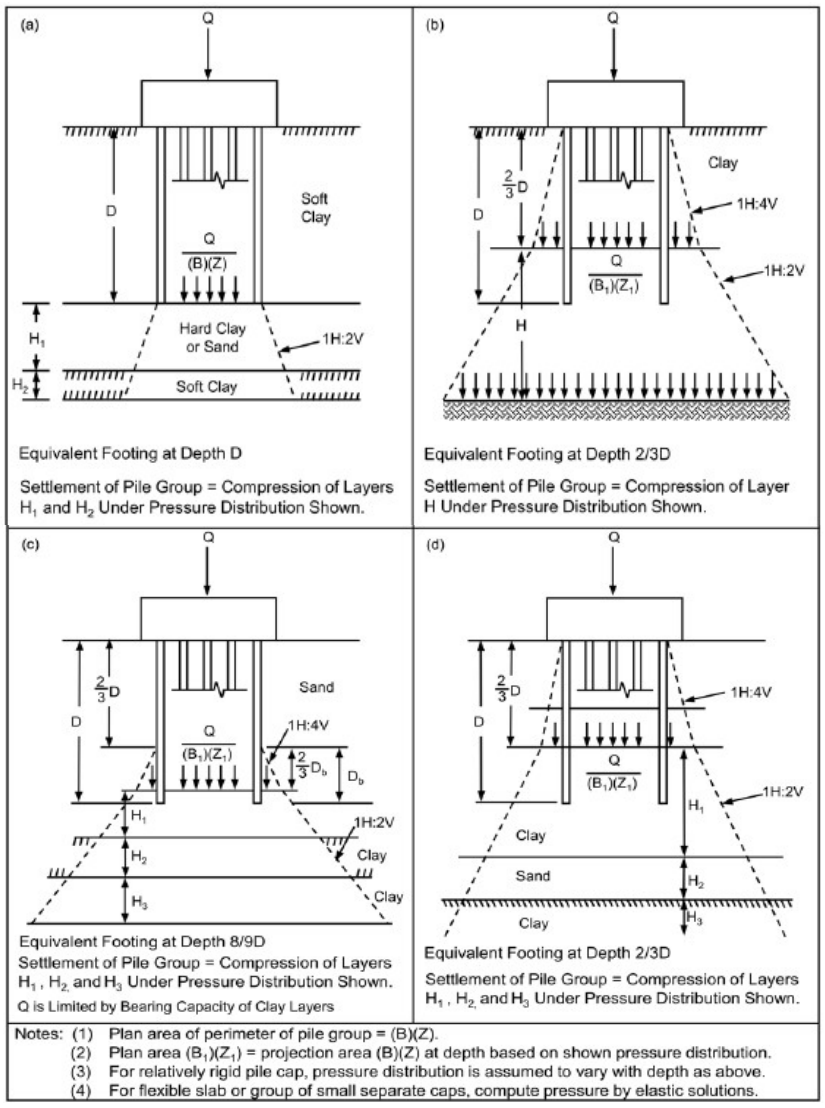
Since Λ_{GEO} < 100 %, the GEO limit state requirement is satisfied.

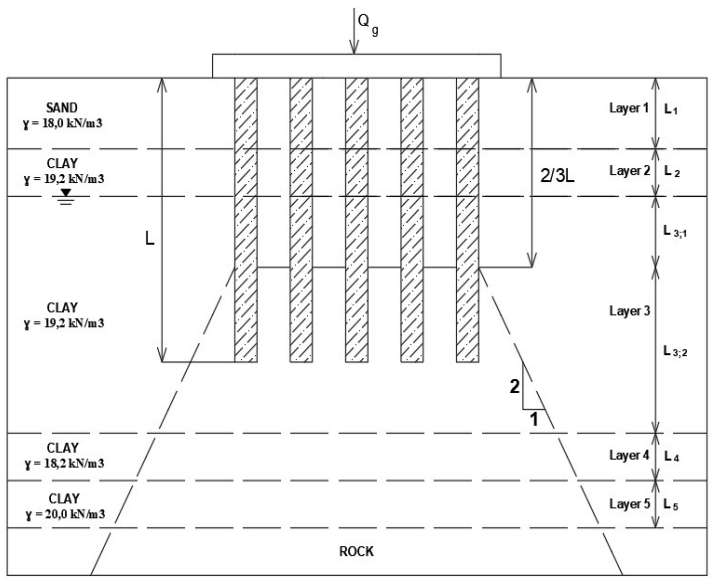
NESTE		Excel file for calculation foundations of friction piles
Author	Date	
Skutina Valeriia		



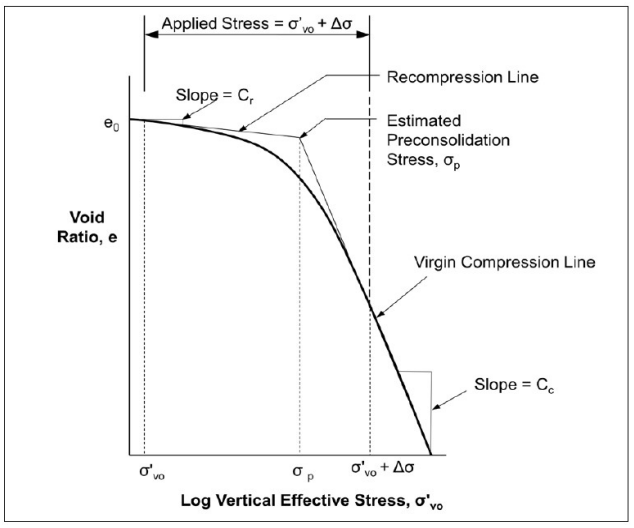
№	Parameter	Formula	Value	Unit	Note
Bearing Capacity of Single Pile					
Ultimate Bearing Capacity of Single Pile, Q_{ult}		$Q_{ult} = Q_b + Q_s$	979,40	kN	-
1	Base capacity, Q_b	$Q_b = q_b * A_b$	144,00	kN	-
2	Shaft capacity, Q_s	$Q_s = f_s * A_s$	835,4033	kN	-
Allowable Pile Capacity, Q_{all}		$Q_{all} = \frac{Q_{ult}}{FS}$	391,76	kN	-
3	Factor of safety, FS	-	1,5	-	for the shaft capacity
			3,0		for the base capacity
			2,5		for the total ultimate capacity
Bearing Capacity of Group Piles					
4	Ultimate Bearing Capacity of Group Piles, Qult;g	$Q_{ult;g} = n_1 * n_2 * (N_c * c_b * A_b + \Sigma\, \alpha * c_u * A_s)$	25920	kN	Cohesive soil (compare and choose the smallest of the two values)
		$Q_{ult;g} = c_b * N_c * A * B + \Sigma\, 2 * (A + B) * L * c_u$	34632		
		$Q_{ult;g} = [1 - \Theta * \frac{(n_1 - 1) * n_2 + (n_2 - 1) * n_1}{90 * n_1 * n_2}] * \Sigma Q_{ult}$	3330,16		Cohesionless soils
5	Θ	$\Theta = \frac{1}{\tan(\frac{D}{S})}$	2,10	-	-
6	Spacing of piles center to center, S	-	0,9	m	-
7	Pile Group Efficiency, Eg	$E_g = \frac{Q_{ult;g}}{\Sigma Q_{ult}}$	1	-	Cohesive soils
		$Q_{ult;g} = E_g * \Sigma Q_{ult}$	0,96	-	Cohesionless soils (If Eg<1,then Q(u;g)=Eg*Σqu; If Eg>1,then Q(u;g)=ΣQu)
8	The sum of the ultimate load bearing capacity of piles, ΣQ_u	$\Sigma Q_{ult} = n * Q_{ult}$	25920	kN	Cohesive soil
			3462,10		Cohesionless soils
Allowable or working axial load of the group piles, Q_{all}		$Q_{all;g} = \frac{Q_{ult;g}}{FS}$	11700,06		-

NESTE	Excel file for calculation foundations of friction piles
Author	Date
Skutina Valeriia	





$L_{1,(m)}$ = 3
$L_{2,(m)}$ = 2
$L_{3;1,(m)}$ = 3
$L_{3;2,(m)}$ = 7
$L_{4,(m)}$ = 2
$L_{5,(m)}$ = 2



№	Parameter	Formula	Value	Unit	Note
	Consolidation Settlement of Group Piles				
	Total Consolidation Settlement of pile group, ΔS_c	$\Delta S_c = \Delta S_{c(3)} + \Delta S_{c(4)} + \Delta S_{c(5)}$	0,104	m	-
1	Consolidation settlement, ΔS_{ci}	$\Delta S_{c(i)} = \left[\frac{C_{c(i)} * H_i}{1 + e_{o(i)}}\right] \log \left[\frac{\sigma'_{o(i)} + \Delta\sigma'_{(i)}}{\sigma'_{o(i)}}\right]$	0,088	m	3 layer
			0,011		4 layer
			0,006		5 layer
3	The effective overburden pressure at the middle, $\sigma'_{o(i)}$	$\sigma'_{o(3)} = \gamma_{s;1} * L_1 + \gamma_{s;2} * L_2 + (\gamma_{sat;3} - \gamma_{gw}) * L_{3;1} + \frac{(\gamma_{sat;3} - \gamma_{gw}) * L_{3;2}}{2}$	153,44	kN/m^2	3 layer
		$\sigma'_{o(4)} = \gamma_{s;1} * L_1 + \gamma_{s;2} * L_2 + (\gamma_{sat;3} - \gamma_{gw}) * L_3 + \frac{(\gamma_{sat;4} - \gamma_{gw}) * L_4}{2}$	194,69		4 layer
		$\sigma'_{o(5)} = \gamma_{s;1} * L_1 + \gamma_{s;2} * L_2 + (\gamma_{sat;3} - \gamma_{gw}) * L_3 + (\gamma_{sat;4} - \gamma_{gw}) * L_4 + \frac{(\gamma_{sat;5} - \gamma_{gw}) * L_5}{2}$	213,27		5 layer
4	The increase in pressure at the middle of each layer, $\Delta\sigma'_{(i)}$	$\Delta\sigma'_{(i)} = \frac{Q_g}{(L_g + z_i) * (B_g + z_i)}$	38,83	kN/m^2	3 layer
			15,92		4 layer
			11,83		5 layer
5	Total load at a depth (2/3)L, Q_g	-	2500	kN	-
6	Distance from z=0 to the middle of the clay layer, i	$z_3 = \frac{L_{3;2}}{2}$	3,5	m	3 layer
		$z_4 = L_{3;2} + \frac{L_4}{2}$	8		4 layer
		$z_5 = L_{3;2} + L_4 + \frac{L_5}{2}$	10		5 layer

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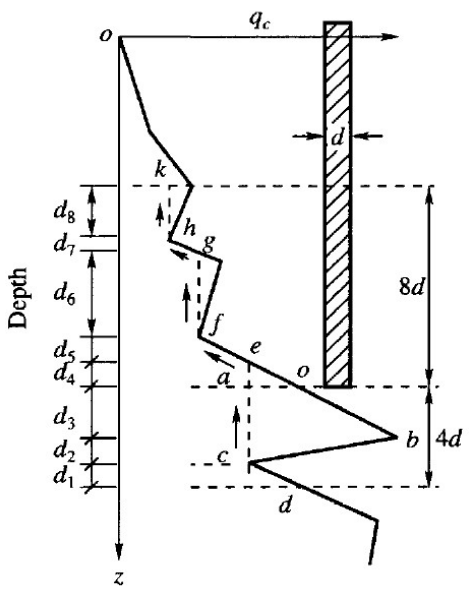
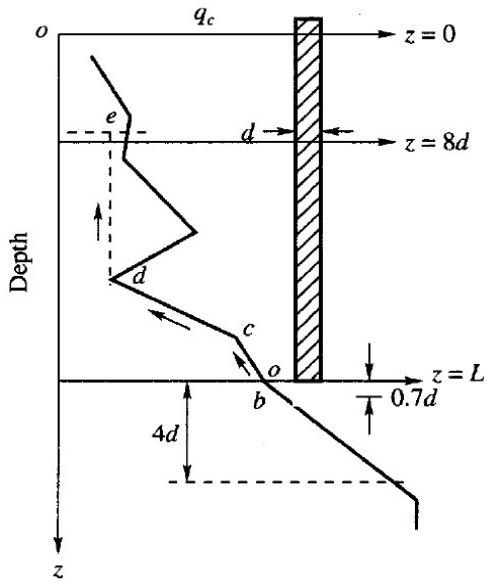
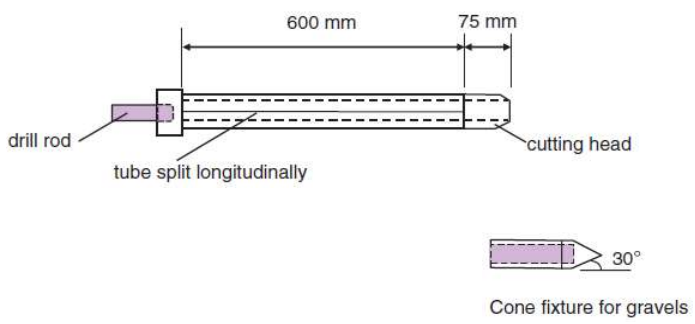
Excel file for calculation foundations of friction piles

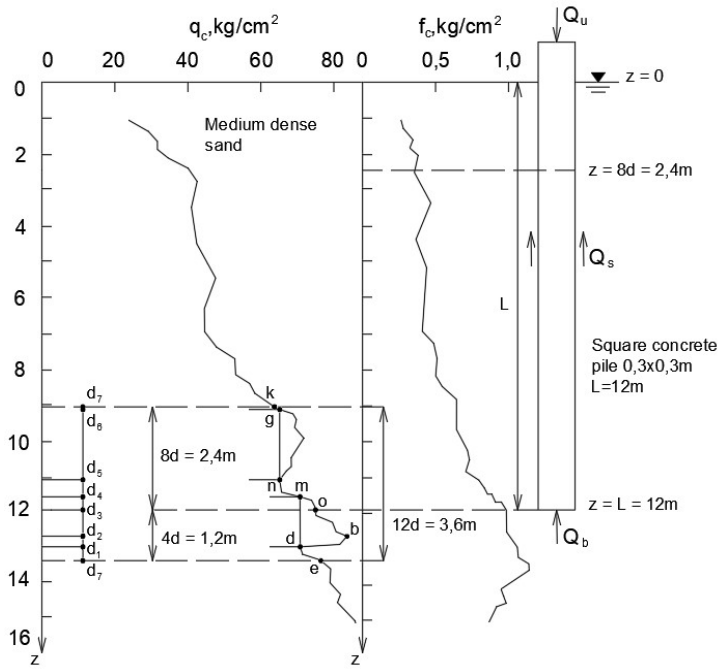
Case 1



Case 2







Pile dimensions			q			d		
			q_0=	76		d_1=	0,4	m
Cross section:			q_b=	85		d_2=	0,3	m
			q_d=	71		d_3=	0,8	m
Piles:			q_e=	80		d_4=	0,4	m
a=	0,3	m	q_m=	71		d_5=	0,5	m
b=	0,3	m	q_n=	65		d_6=	2,0	m
or			q_g=	65		d_7=	0,1	m
r=			q_k=	60				
Perimeter:								
P=	1,2	m	8d=	2,4	m			
Length			4d=	1,2	m			
L=	12	m	L-8d=	9,6	m			

№	Parameter	Formula	Value	Unit	Note
CPT					
Design values of actions, $F_{c;d}$		$F_{c;d} = \gamma_G * G_{rep} + \gamma_Q * Q_{rep}$	1503,0	kN	-
1	Partial factor, γ_G	-	1,49	-	Table 7
2	Partial factor, γ_Q	-	0,9	-	
3	K_{FI}	-	1,1	-	
4	Representative value of an action, G_{rep}	-	800,00	kN	Design value
5	Representative value of an action, Q_{rep}	-	350,00	kN	Design value
Ultimate Bearing Capacity of Single Pile, Qult		$Q_{ult} = Q_b + Q_s$	1791,41	kN	-
6	Base capacity, Q_b	$Q_b = A_b * q_p$	1111,78		-
7	Cone penetration value, q_p	$q_p = \frac{(q_{c1} + q_{c2}) + q_{c3}}{2}$	1260,52	t/m^2 (metric)	

8	Average cone resistance below the tip of the pile over a depth which may vary between 0,7d and 4d, q_{c1}	$q_{c1} = \frac{d_3 * \frac{q_0 + q_b}{2} + d_2 * \frac{q_b + q_d}{2} + d_1 * \frac{q_d + q_e}{2}}{4d}$	983,33	t/m2 (metric)	Case 1: the cone point resistance qc below the tip of a pile is lower than that at the tip within depth 4d (diagram)
		$q_{c1} = \frac{q_0 + q_b}{2}$	-		Case 2: the cone resistance qc below the pile tip is greater than that at the tip within a depth 4d
9	Minimum cone resistance recorded below the pile tip over the same depth 0,7d to 4d, q_{c2}	-	710		Case 1: minimum value below tip within a depth of 4d at point c, (diagram)
			-		Case 2: minimum value at the pile tip itself
10	Average of the envelope of minimum cone resistance recorded above the pile tip to a height of 8d, q_{c3}	$q_{c3} = \frac{d_4 * q_m + d_5 * \frac{q_m + q_n}{2} + d_6 * q_n + d_7 * \frac{q_g + q_k}{2}}{8d}$	827,71		Case 1:
		-	-		Case 2: average of the minimum values along the envelope ocde as before
11	Shaft capacity, Q_s	$Q_s = \Sigma\, \alpha' * \bar{f_c} * A_s$	-	kN	Cohesive soils
		$Q_s = K * \left(\frac{1}{2} * (\bar{f_c} * A_s)\{0 - 8d\} + (\bar{f_c} * A_s)\{8d - L\}\right)$	679,63		Cohesionless soils
12	Ratio of pile to penetrometer sleeve friction, α'	-	-	-	Figure 24
13	Average sleeve friction, $\bar{f_c}$	-	0,34	kg/cm^2	Figure 35 (from 0-8d)
			0,71		Figure 35 (from 8d-L)
14	Correction factor, K	-	0,8	-	Figure 25
Allowable Pile Capacity, Q_{all}		$Q_{all} = \frac{Q_{ult}}{FS}$	716,565	kN	-

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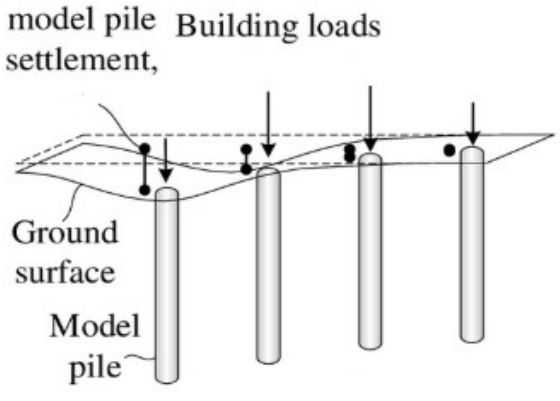

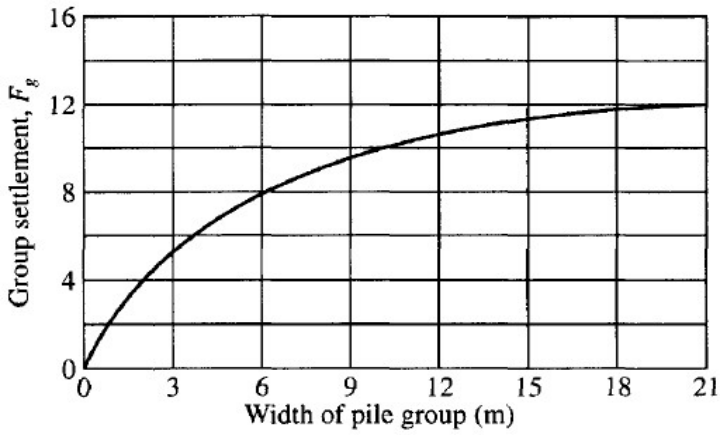

L=	12	m
a=b=	0,3	m
D'=	5	m
Qall=	716,57	kN
Qs=	679,63	kN
Qb=	1111,78	kN
Ep=	$21*10^6$	kN/m^2
Es=	60000	kN/m^2
µs=	0,3	

№	Parameter	Formula	Value	Unit	Note
Elastic Settlement					
Total Elastic Settlement of Pile, S		$S = S_1 + S_2 + S_3$	0,023	m	-
1	Elastic settlement of pile, S_1	$S_1 = \frac{(Q_{wp} + \xi * Q_{ws}) * L}{A_b * E_p}$	0,0043	m	-
2	Settlement of pile caused by the load at the pile tip, S_2	$S_2 = \frac{q_{wp} * D}{E_s} * (1 - \mu_s^2) * I_{wp}$	0,018	m	-
3	Settlement of pile caused by the load transmitted along the pile shaft, S_3	$S_3 = (\frac{Q_{ws}}{P * L}) * \frac{D}{E_s} * (1 - \mu_s^2) * I_{ws}$	0,0006	m	-
4	Load carried at the pile point under working load condition, Q_{wp}	$Q_{wp} = \frac{Q_b}{FS}$	370,59	kN	FS = 3,0
5	Load carried by frictional (skin) resistance under working load condition, Q_{ws}	$Q_{ws} = \frac{Q_s}{FS}$	453,09	kN	FS = 1,5
6	Module of elasticity of the pile material, E_p	-	21000000	kN/m2	-
7	The magnitude which depend on the nature of unit friction (skin) resistance distribution along the pile shaft, ξ	-	0,67	-	Figure 28
8	Point load per unit area at the pile point,q_{wp}	$q_{wp} = \frac{Q_{wp}}{A_b}$	4117,701	kN/m^2	-
9	Width or diameter of pile, D	-	0,4	m	-
10	Module of elasticity of soil at or below the pile point, E_s	-	60000	kN/m^2	-
11	Poisson's ratio of soil, μ_s	-	0,3	-	Table 6
12	Influence factor, I_{wp}	-	0,95	-	Table 18
13	Influence factor, I_{ws}	$I_{ws} = 2 + 0{,}35\sqrt{\frac{L}{D}}$	4,21	-	-
Total Settlement of Group Piles, Sg		$S_g = \xi_g * S$	0,09	m	-

14	Group settlement factor	$\xi_g = \sqrt{\frac{D'}{D}}$	4,08	-	-
Serviceability Limit State Design		$E_c \leq C_d$	-		

22,7 < 25 mm, the settlement SLS condition is satisfied.