Formulary, VGTF01 - May 15, 2019

1 Basic concepts

1.2 Vertical stresses

$$\sigma_z = \int \gamma(z)\ dz \tag{1.1}$$

$$\sigma_z = \sum_{i=1}^{n} g \cdot \rho_i \cdot z_i = \sum_{i=1}^{n} \gamma_i \cdot z_i \tag{1.2}$$

where; σ_z = The vertical stress in the soil given in [kPa]
γ = The unit weight of the soil, in [kN/m^3]
z = Depth in [m]

1.3 Effective stress

$$\sigma'_z = \sigma_z - u \tag{1.3}$$

where; σ'_z = The effective vertical stress given in [kPa]
σ_z = The vertical stress given in [kPa]
u = The pore water pressure given in [kPa]

1.4 Pore water pressure

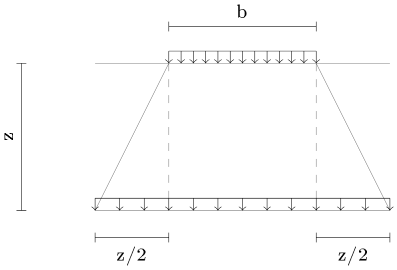
In hydrostatic conditions

$$u = \gamma_w \cdot z_w \tag{1.4}$$

where; u = Pore water pressure given in [kPa]
γ_w = The unit weight of water, in [kN/m^3]
z_w = Depth from groundwater table [m]

1.5 2:1 method

For the two-dimensional case:



$$\Delta\sigma = \frac{b \cdot q}{b + z} \tag{1.5}$$

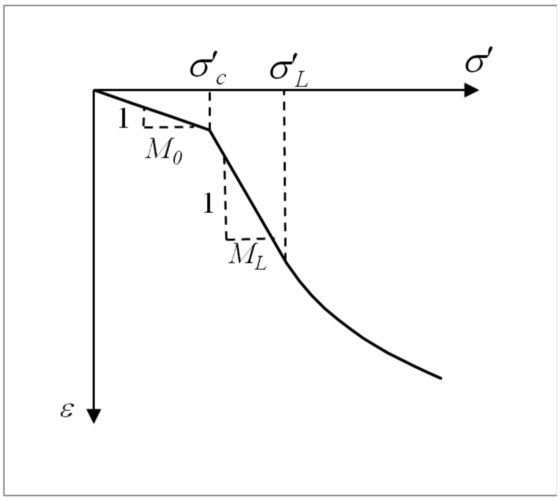
where; $\Delta\sigma$ = Additional stress given in [kPa]
q = Surface load given in [kPa]
b = Width of the load given in [m]
z = Depth in [m]

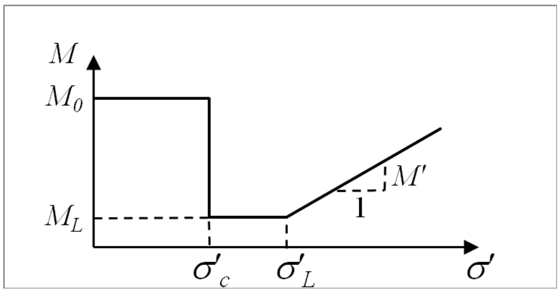
For the case with limited extension in two directions, i.e. rectangular load:

$$\Delta\sigma = \frac{b \cdot l \cdot q}{(b+z)(l+z)} \tag{1.6}$$

where; $\Delta\sigma$ = Additional stress given in [kPa]
q = Surface load given in [kPa]
b = Width of the load given in [m]
l = Length of the load given in [m]
z = Depth in [m]

1.6 Settlement calculations





$$\delta = \int_0^h \frac{\Delta\sigma(z)}{M(z)}\, dz \tag{1.7}$$

where; δ = Deformation given in [m]
$M(z)$ = Compression modulus [kPa]
$\Delta\sigma$ = Additional stress in [kPa]
z = Depth in [m]

Settlement equation for constant compressive modulus in the whole stress range:

$$\delta = \sum_i \frac{\Delta\sigma_i}{M_i} \cdot h_i \tag{1.8}$$

where; δ = Deformation given in [m]
M = Compression modulus [kPa]
$\Delta\sigma$ = Additional stress in [kPa]
h = Thickness in [m]

Bi-linear compressive modulus for the stress range at hand:

$$\delta = \sum_i \left(\frac{\sigma_c' - \sigma_0'}{M_0} + \frac{\sigma_0' + \Delta\sigma - \sigma_c'}{M_L} \right) \cdot h_i \tag{1.9}$$

where; δ = Deformation given in [m]
M_0 = Compression modulus where the stress is less than the pre consolidation pressure [kPa]
M_L = Compression modulus where the stress is greater than the pre consolidation pressure [kPa]
$\Delta\sigma$ = Additional stress in [kPa]
σ_c' = The pre consolidation pressure in [kPa]
σ_0' = Stress level in the soil in [kPa]
h = Thickness in [m]

For the case with rather high stress levels the stress-strain curve is no longer linear. The compressive modulus can then be evaluated as, M', by plotting the modulus as a function of stress. For high stress levels the settlement can then be evaluated as:

$$\delta = \sum_i \left(\frac{\sigma_c' - \sigma_0'}{M_0} + \frac{\sigma_L - \sigma_c'}{M_L} + \frac{1}{M'} \ln \left(1 + (\sigma_0' + \Delta\sigma - \sigma_L') \cdot \frac{M'}{M_L} \right) \right) \cdot h_i \tag{1.10}$$

where; δ = Deformation given in [m]
M_0 = Compression modulus where the stress is less than the pre consolidation pressure [kPa]
M_L = Compression modulus where the stress is greater than the pre consolidation pressure [kPa]
M' = Increase rate of compression modulus in high stress ranges
$\Delta\sigma$ = Additional stress in [kPa]
σ_c' = The pre consolidation pressure in [kPa]
σ_0' = Initial stress level in the soil in [kPa]
σ_L' = Stress level in the soil at which hardening behaviour starts [kPa]
h = Thickness in [m]

2 Eurocode

2.2 Ultimatte Limite States GEO - STR

Design Approach 1 consisting in two combinations: in Combination 1 partial factors are applied to the actions while soil strength parameters are not factored; in Combination 2 partial factors are applied to the ground strength parameters while permanent actions are not factored and a smaller partial factor than in Combination 1 is applied to variable actions

Design Approach 2 factors are applied to actions (or effects of actions) and to resistances simultaneously

Design Approach 3 factors are applied separately to structural and geotechnical actions, and to material properties simultaneously

Types of geotechnical structures	Design Approach
Piles, design through calculation	DA 2
Piles, design through trial loading	DA 2
Piles, design from constructive load-bearing capacity	DA 3
Retaining structures	DA 3
Slopes and embankments *	DA 3
Slabs	DA 3

* does not apply to natural slopes.

2.3 Reliability management

Partial factors for consequences classes

- CC1: $\gamma_d = 0.83$,
- CC2: $\gamma_d = 0.91$,
- CC3: $\gamma_d = 1.0$.

2.4 Design values of actions

The combination of actions for STR and GEO limit states is the less favourable of the two following expressions:

$$E_d = \gamma_d(\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1}\psi_{0,1}Q_{k,1} + \sum_{i\geq 1} \gamma_{Q,i}\psi_{0,i}Q_{k,i}) \tag{2.1}$$

$$E_d = \gamma_d(\sum_{j\geq 1} \xi\gamma_{G,j} G_{k,j} + \gamma_{Q,1}Q_{k,1} + \sum_{i\geq 1} \gamma_{Q,i}\psi_{0,i}Q_{k,i}) \tag{2.2}$$

where; G_k = Characteristic permanent load
Q_k = Characteristic variable load
ψ, γ_G and γ_Q = Partial factors according to the tables below

Design Approach 1: Applying in separate calculations design values from Table 2 and Table 1 to the geotechnical actions as well as the other actions on/from the structure;

Design Approach 2: Applying design values from Table 1 to the geotechnical actions as well as the other actions on/from the structure;

Design Approach 3: Applying design values from Table 2 to the geotechnical actions and, simultaneously, applying partial factors from Table 1 to the other actions on/from the structure,

Persistent and transient design situation	Permanent actions		Leading variable action	Accompanying variable actions	
	Unfavourable ($G_{kj,sup}$)	Favourable ($G_{kj,inf}$)	Action	Main	Others
Eq. 2.1	$\gamma_{G,j} = 1.35$ $\xi = 1.0$	$\gamma_{G,j} = 1.0$ $\xi = 1.0$		$\gamma_{Q,1} = 1.5^*$	$\gamma_{Q,i} = 1.5^*$
Eq. 2.2	$\gamma_{G,j} = 1.35$ $\xi = 0.89$	$\gamma_{G,j} = 1.0$ $\xi = 1.0$	$\gamma_{Q,1} = 1.5^*$		$\gamma_{Q,i} = 1.5^*$
* ATTENTION: γ_Q is always zero when the action is favourable					

Table 1: Design values of actions (STR/GEO) (Set B)

Persistent and transient design situation	Permanent actions		Leading variable action	Accompanying variable actions	
	Unfavourable ($G_{kj,sup}$)	Favourable ($G_{kj,inf}$)	Action	Main	Others
	$\gamma_{G,j} = 1.1$ $\xi = 1.0$	$\gamma_{G,j} = 1.0$ $\xi = 1.0$	$\gamma_{Q,1} = 1.4^*$		$\gamma_{Q,i} = 1.4^*$
* ATTENTION: γ_Q is always zero when the action is favourable					

Table 2: Design values of actions (STR/GEO) (Set C)

2.5 Design values of geotechnical parameters

$$X_d = X_k/\gamma_M \tag{2.3}$$

where; X_d = Design value of parameter
X_k = Characteristic value of parameter
γ_M = Partial factors according to the table below

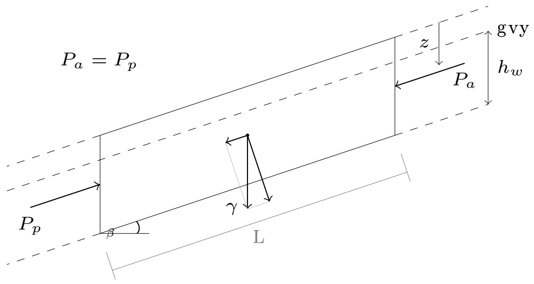
Soil parameter	Symbol	Set	
		M1	M2
Angle of shearing resistance*	$\gamma_{\varphi'}$	1.0	1.3
Effective cohesion	$\gamma_{c'}$	1.0	1.3
Undrained shear strength	γ_{cu}	1.0	1,5
Unconfined strength	γ_{qu}	1.0	1.5
Weight density	γ_{γ}	1.0	1.0

*This factor is applied to $\tan\varphi'$

Table 3: Partial factors for soil parameters(γ_M)

3 Slope stability

3.1 Plane slip surfaces



$$E_d = \gamma_d(\gamma_G\,\gamma\,h + \gamma_Q\,q)\,\Delta l\cos\beta\sin\beta \tag{3.1}$$

$$R_d = \tau_d = \frac{c}{\gamma_c} + \sigma'_z\cos^2\beta\,\frac{\tan\phi}{\gamma_\phi} \tag{3.2}$$

with:

$$\sigma'_z = \gamma_G(\gamma\,h - \gamma_w\,h_w) + \gamma_Q\,q \tag{3.3}$$

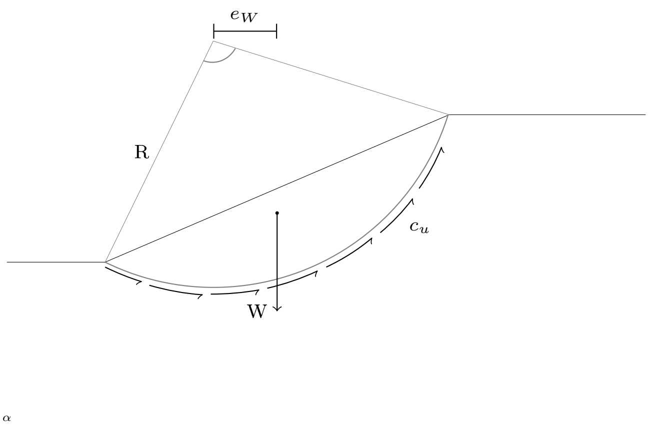
Safety factor:

$$F_{c\phi} = \frac{c}{\gamma\cdot z\cdot\sin\beta\cdot\cos\beta} + \frac{\gamma\cdot z - \gamma_w\cdot h_w}{\gamma\cdot z}\cdot\frac{\tan\phi}{\tan\beta} \tag{3.4}$$

where;	c	= Cohesion.
	β	= Angle of the slope.
	γ	= Unit weight of the soil.
	γ_w	= Unit weight of water.
	z	= Depth of soil layer.
	h_w	= Depth from ground water table.
	ϕ	= Angle of internal friction.
	γ_d, γ_G, γ_Q, γ_c and γ_ϕ	= Partial coefficients according to the tables in Section 2.

3.2 Circular slip surfaces

3.2.1 General method



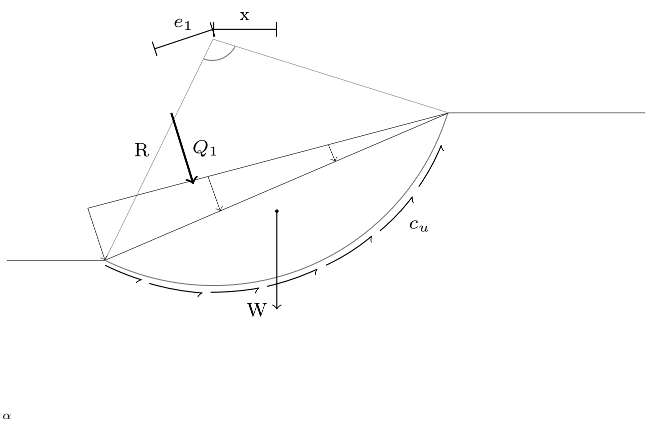
$$E_d = \gamma_d(\gamma_G\, W_k\, e_W) \tag{3.5}$$

$$R_d = \frac{c_u}{\gamma_{cu}} R\, l = \frac{c_u}{\gamma_{cu}} R^2\, \alpha \tag{3.6}$$

$$F_c = \frac{c_u \cdot R^2 \cdot \alpha}{W \cdot e_W} \tag{3.7}$$

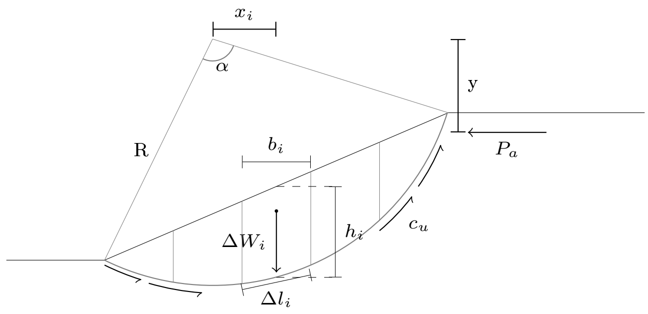
where;	c_u	= Undrained cohesion.
	W	= Soil weight.
	e_W	= Lever to centre of gravity.
	γ_d, γ_G and γ_{cu}	= Partial coefficients according to the tables in section 2.

3.2.2 Liquid analogy



$$W \cdot e_W = Q_1 \cdot e_1 \tag{3.8}$$

3.2.3 Method of slices



$$W \cdot e_W = \sum(\Delta W_i \cdot x_i) \tag{3.9}$$

$$R_d = \sum(\frac{c_u}{\gamma_{cu}} \Delta l_i) \tag{3.10}$$

where; c_u = Undrained cohesion.
ΔW_i = $b_i \cdot h_i \cdot \gamma$
x_i = Lever to centre of gravity of soil sector.
Δl_i = Length of slip surface of soil sector.

4 Foundations

4.1 Spread foundations

4.1.1 Design actions

Loads from the structure
unfavorable loads

use the most unfavorable of the next two equations

$$E_d = \gamma_d \cdot 1.35 \cdot G_{kj,sup} + \gamma_d \cdot 1.5 \cdot \psi_{0,1} \cdot Q_{k,1} + \gamma_d \cdot 1.5 \cdot \psi_{0,i} \cdot Q_{k,i} \tag{4.1}$$

$$E_d = \gamma_d \cdot 0.89 \cdot 1.35 \cdot G_{kj,sup} + \gamma_d \cdot 1.5 \cdot Q_{k,1} + \gamma_d \cdot 1.5 \cdot \psi_{0,i} \cdot Q_{k,i} \tag{4.2}$$

favorable loads

$$E_d = G_{kj,inf} \tag{4.3}$$

Geotechnical loads
unfavorable loads

$$E_d = \gamma_d \cdot 1.10 \cdot G_{kj,sup} + \gamma_d \cdot 1.4 \cdot Q_{k,1} + \gamma_d \cdot 1.4 \cdot \psi_{0,i} \cdot Q_{k,i} \tag{4.4}$$

favorable loads

$$E_d = G_{kj,inf} \tag{4.5}$$

4.1.2 Bearing resistance

$$R_d = q_b A'; \tag{4.6}$$

where; q_b = Bearing capacity.
A' = Effective area of the slab.

4.1.3 General bearing capacity equation

Bearing capacity:

$$q_{bd} = c_d \cdot N_{cd} \cdot s_c \cdot d_c \cdot i_c + q \cdot N_{qd} \cdot s_q \cdot d_q \cdot i_q + 0.5 \cdot \gamma \cdot B \cdot N_{\gamma d} \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \tag{4.7}$$

where c_d is the shear strength design value and N_{cd}, N_{qd} and $N_{\gamma d}$ are the design values for the bearing capacity factors calculated using ϕ'_d.

$$\phi'_d = \arctan\left(\frac{\tan\phi'}{\gamma_\phi}\right) \tag{4.8}$$

$$c_d = \frac{c}{\gamma_c} \tag{4.9}$$

Bearing capacity factors, N$_c$, N$_q$ and N$_\gamma$:

For $\phi = 0$, (undrained conditions)

$$N_c = \pi + 2 \tag{4.10}$$

$$N_q = 1 \tag{4.11}$$

$$N_\gamma = 0 \tag{4.12}$$

Table 4: Values of the bearing capacity factors N_c , N_q och N_γ for different values of the friction angle ϕ'.

ϕ'	N_c	N_q	N_γ
16	11.6	4.34	1.42
17	12.3	4.77	1.70
18	13.1	5.26	2.02
19	13.9	5.80	2.40
20	14.8	6.40	2.84
21	15.8	7.07	3.36
22	16.9	7.82	3.96
23	18.0	8.66	4.67
24	19.3	9.60	5.51
25	20.7	10.7	6.48
26	22.3	11.9	7.64
27	23.9	13.2	8.99
28	25.8	14.7	10.6
29	27.9	16.4	12.5
30	30.1	18.4	14.7

ϕ'	N_c	N_q	N_γ
31	32.7	20.6	17.4
32	35.5	23.2	20.6
33	38.6	26.1	24.4
34	42.2	29.4	29.0
35	46.1	33.3	34.5
36	50.6	37.8	41.1
37	55.6	42.9	49.1
38	61.4	48.9	58.9
39	67.9	56.0	70.9
40	75.3	64.2	85.6
41	83.9	73.9	104
42	93.7	85.4	126
43	105	99.0	154
44	118	115	190
45	134	135	234

Empirical correction factors:

Effect of eccentric load:

$$e = e_h \cdot \frac{H}{V} \tag{4.13}$$

Effective Area:

$$A' = b_{ef} \cdot l_{ef} \tag{4.14}$$

where; $b_{ef} = b - 2 \cdot e_b$
$l_{ef} = l - 2 \cdot e_l$
b = The width of the footing.
l = The length of the footing.

Shape of footing:

$$s_c = 1 + 0.2\,\frac{b_{ef}}{l_{ef}}; \qquad \text{if } \phi' = 0 \tag{4.15}$$

$$s_c = 1 + 0.2\,\frac{N_q\, b_{ef}}{N_c\, l_{ef}}; \qquad \text{if } \phi' \neq 0 \tag{4.16}$$

$$s_q = 1 + \tan\phi'\,\frac{b_{ef}}{l_{ef}} \tag{4.17}$$

$$s_\gamma = 1 - 0.4\,\frac{b_{ef}}{l_{ef}} \tag{4.18}$$

Foundation depth:

$$d_c = 1 + 0.35\,\frac{d}{b_{ef}}; \qquad d_c \leq 1.7 \tag{4.19}$$

$$d_q = 1 + 0.35\,\frac{d}{b_{ef}}; \qquad d_q \leq 1.7 \tag{4.20}$$

$$d_\gamma = 1 \tag{4.21}$$

where; d = The foundation depth.

Inclined load:

$$i_c = 1 - \frac{mH}{b_{ef}\,l_{ef}\,c_u\,N_c}; \qquad i_q = 1; \qquad \text{if } \phi' = 0 \tag{4.22}$$

$$i_c = i_q - \frac{1 - i_q}{N_c\,\tan\phi'}; \qquad \text{if } \phi' \neq 0 \tag{4.23}$$

$$i_q = \left(1 - \frac{H}{V + \frac{b_{ef}\,l_{ef}\,c_d}{\tan\phi'}}\right)^m \tag{4.24}$$

$$i_\gamma = \left(1 - \frac{H}{V + \frac{b_{ef}\,l_{ef}\,c_d}{\tan\phi'}}\right)^{m+1} \tag{4.25}$$

where; V = Vertical load.
H = Horizontal load.

When the horizontal load component acts in transverse direction of the foundation the parameter m is given by

$$m = m_b = \frac{2\,l_{ef} + b_{ef}}{l_{ef} + b_{ef}} \tag{4.26}$$

and when acts in longitudinal direction m is given by

$$m = m_l = \frac{2\,b_{ef} + l_{ef}}{b_{ef} + l_{ef}} \tag{4.27}$$

When the horizontal load component acts in a direction that forms the angle θ with the longitudinal direction of the foundation m is given by

$$m = m_l \cos^2\theta + m_b \sin^2\theta \tag{4.28}$$

4.2 Pile foundations

4.2.1 Characteristic value of the geotechnical bearing capacity

$$R_{c,k} = R_{b,k} + R_{s,k} = q_{b,k} \cdot A_b + R_{s,k} \cdot A_s \tag{4.29}$$

where; $R_{c,k}$ = Total resistance.
$R_{b,k}$ = End resistance.
$R_{s,k}$ = Shaft resistance.
$q_{b,k}$ = The nominal compressive strength of the soil at the toe at ground failure.
A_b = The area of the pile section at the toe.
$q_{s,k}$ = The (average) friction strength at interface between soil and shaft.
A_s = Total area of the shaft.

4.2.2 Friction piles, β method

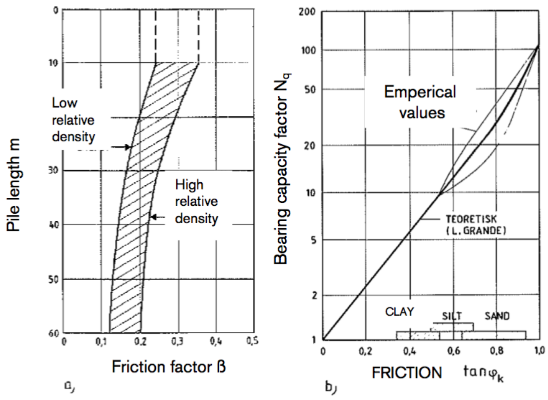


Figure 1: *Values of the friction factor β and the bearing capacity factor N_q (From "Norsk peleveiledning")*

Shaft resistance:

$$q_s = \beta \cdot \bar{\sigma_v'} \tag{4.30}$$

End resistance:

$$q_b = N_q \cdot \sigma_v' \tag{4.31}$$

where; σ_v' = The effective vertical stress in the soil.
$\bar{\sigma_v'}$ = The average effective vertical stress in the soil along pile.
β = Friction factor.
N_q = Bearing capacity factor.

4.2.3 Cohesion piles

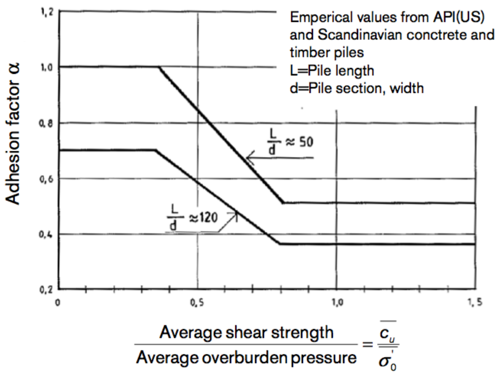


Figure 2: *Values of the adhesion factor α (From "Norsk peleveiledning")*

Shaft resistance:

$$q_s = \alpha \cdot \bar{c_u} \tag{4.32}$$

End resistance:

$$q_b = N \cdot c_u \tag{4.33}$$

where; c_u = The undrained shear strength.
$\bar{c_u}$ = The average undrained shear strength.
α = Adhesion factor.
N = 6-9, end bearing capacity factor.

4.2.4 Design value of bearing capacity (for ultimate limit state)

$$R_{c,d} = \frac{1}{\gamma_{Rd}} \cdot \left(\frac{q_b \cdot A_b}{\gamma_b} + \frac{q_s \cdot A_s}{\gamma_s} \right) \tag{4.34}$$

where; γ_{Rd} = Model factor.
γ_b = Toe resistance factor.
γ_s = Shaft resistance factor.

Tabell 4.2 Partialkoefficienter för slagna pålar, CFA och grävpålar enligt VVFS 2009:19.

Partialkoefficient	*Slagna pålar Tabell A.6(S)*	*CFA och grävpålar Tabell A.7(S) samt A.8(S)*
γ_b	1,2	1,3
γ_s	1,2	1,3
γ_t	1,2	1,3
$\gamma_{s,t}$	1,3	1,4

Tabell 4.3 Modellosäkerheter för friktionspålar (GEO).

Beräkningsmodell / provningsmetod	Modellfaktorer γ_{Rd}
Geostatisk metod (baserad på friktionsvinkel) för pålar i friktionsjord	1,6
Dimensionering av pålar baserat på sonderingsresultat med CPT	1,4
Dimensionering av pålar baserat på övriga sonderingsmetoder, t ex HfA, SPT och Tr, med verifiering av jordart genom provtagning.	1,5
Statisk provbelastning	1,0
Dynamisk provbelastning utvärderad endast med CASE-metoden. CAPWAP-analys bör utföras för mantelburna pålar	1,2
Dynamisk provbelastning med signalmatchning med CAPWAP-analys.	0,85
Dragbelastning utvärderad från CAPWAP. Dessutom bör en reduktionsfaktor för dragbelastning på 0,7 användas.	1,3
Pålslagningsformler med eller utan fjädringsmätning	Tillåts ej
Slagningssimulering (sk WEAP-analys)	Tillåts ej

Tabell 4.4 Modellosäkerheter för kohesionspålar (GEO).

Beräkningsmodell / provningsmetod	Modellfaktorer γ_{Rd}
Odränerad analys (α-metod). Metoden ska alltid användas för lösa leror.	1,1
Dränerad analys (β-metod)	1,2
Statisk provbelastning	1,0
Dynamisk provbelastning, CASE-metod	Tillåts ej
Dynamisk provbelastning med signalmatchning med CAPWAP-analys. Kalibrering mot statisk provbelastning enligt EN 1997-1, avsnitt 7.5.3(1)	1,0
Pålslagningsformler med eller utan fjädringsmätning	Tillåts ej
Slagningssimulering (sk WEAP-analys)	Tillåts ej

4.2.5 Settlements

Action effect in a pile:

$$E = Q_\infty + \int_0^z q_s \, dA \tag{4.35}$$

The resistance at depth z:

$$R = R_{toe} + \int_z^{L_p} q_s \, dA \tag{4.36}$$

where; Q_∞ = Long-time loading of the pile.
R_{toe} = Toe resistance.
q_b = Shaft resistance.
L_p = Length of pile.

Redistribution of the load:

$$\Delta\sigma = \frac{Q}{L_p - z_n} \frac{z - z_n}{B[B + (z - z_n)]} \qquad \text{for} \qquad z_n < z < L_p \tag{4.37}$$

$$\Delta\sigma = \frac{Q}{[B + (z - L_p)][B + (z - z_n)]} \qquad \text{for} \qquad z > L_p \tag{4.38}$$

where; z_n = Depth on the neutral plane.
B = Width of the foundations.

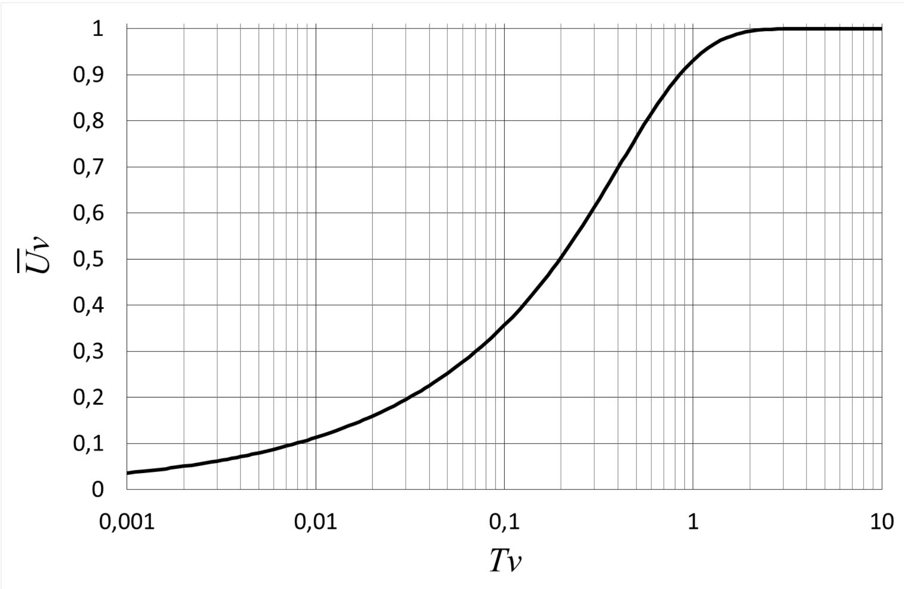
5 Settlements, Time dependence

5.2 Coefficient of consolidation

For the case with constant permeability:

$$c_v = k \cdot \frac{M}{\gamma_w} \tag{5.1}$$

where; c_v = Coefficient of consolidation [m^2/s or $m^2/year$]
k = Permeability [m^2/s] or [$m^2/year$]
M = Compression modulus [kPa]
γ_w = Unit weight of water, [kN/m^3]



Time factor:

$$T_v = c_v \cdot \frac{t}{d^2} \tag{5.2}$$

where; T_v = Time factor
c_v = Coefficient of consolidation [m^2/s or $m^2/year$]
t = time [s or year]
d = Half of soil depth, [m]

The settlement at time t:

$$\delta_t = U_v \cdot \delta_{tot} \tag{5.3}$$

where; U_v = The average degree of consolidation
δ_t = Settlement at time t
δ_{tot} = Total or final settlement

6 Retaining structures

6.1.1 Active earth pressure

For clay:

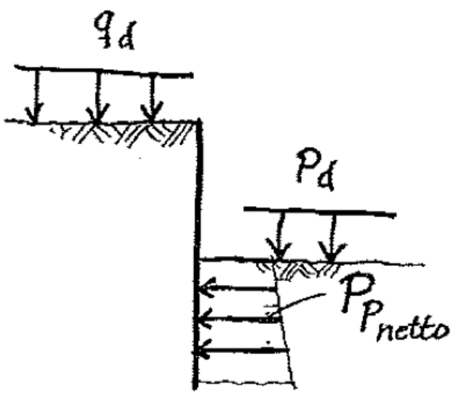
$$P_{ad} = (\sigma_z - 2 \cdot c_{ud}) \tag{6.1}$$

For frictional material:

$$P_{ad} = (\sigma_z - p_w) \cdot \tan^2(45^\circ - \frac{\phi'_d}{2}) + p_w \tag{6.2}$$

where; P_{ad} = Active earth pressure, in [kPa]
σ_z = Vertical stress in the soil, in [kPa]
c_{ud} = Material cohesion, in [kPa]
p_w = Ground water pressure [kPa]
ϕ'_d = Angle of internal friction [deg]

6.1.2 Passive earth pressure



For clay:

$$P_{pd} = (\sigma_z + 2 \cdot c_{ud}) \tag{6.3}$$

For frictional material:

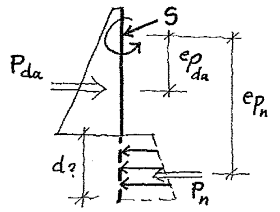
$$P_{pd} = (\sigma_z - p_w) \cdot \tan^2(45^\circ + \frac{\phi'_d}{2}) + p_w \tag{6.4}$$

Earth pressure if clay exists at the bottom of the excavation pit:

$$P_{net,d} = N_{cb} \cdot c_{ud} - (\gamma \cdot H + q_d) \tag{6.5}$$

where; P_{pd} = Passive earth pressure, in [kPa]
σ_z = Vertical stress in the soil, in [kPa]
N_{cb} = Bearing capacity factor
H = Depth to bottom of excavation pit, in [m]
c_{ud} = material cohesion, in [kPa]
p_w = Ground water pressure [kPa]
ϕ'_d = Angle of internal friction [deg]

6.1.3 Static equilibrium



Length of the sheet pile wall:

$$P_{net,d} \cdot e_{pn} - \gamma_{Sda} \cdot P_{da} \cdot e_{pda} \tag{6.6}$$

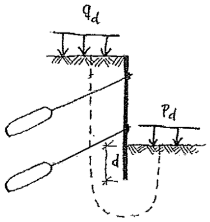
Anchor or strut force:

$$S_{net,d} = P_{net,d} - \gamma_{Sda} \cdot P_{ad} \tag{6.7}$$

where; P_{ad} = Active earth pressure, in [kPa]
γ_{Sda} = Partial coefficient
$P_{net,d}$ = Resulting net pressure, in [kPa]
$S_{net,d}$ = Horizontal force at equilibrium, in [kPa]

6.1.4 Stability of the excavation bottom

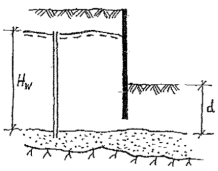
Reliability condition, bottom heave:



$$1.1\, \gamma_d \, (\sigma_{vHd} - p_d) \leq \gamma_{Rdp} \, N_{cb} \, c^*_{ud}$$

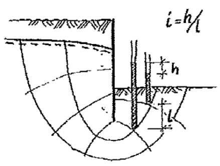
where; γ_{Rdp} = Partial coefficient
c^*_{ud} = mean value of the undrained shear strength below escavation bottom

Condition for hydraulic uplift:



$$\gamma_w \, H_w \leq \frac{0.9\, \gamma_{soil} \, d}{1.1\, \gamma_d}$$

Condition for hydraulic erosion:



$$i < i_{crit} = \frac{\gamma'_{soil}}{\gamma_w \, \gamma_{Rd}}$$

γ_{Rd} = 1.5 for coarse material and 2.5 for silt.

7 Lime/Cement columns

7.1 For the elastic zone- zone B:

$$M_{block} = a \cdot E_{col} + (1 - a) \cdot M_{soil} \tag{7.1}$$

where; M_{block} = Modulus of compression for soil-coloumn composite, [kPa]
E_{col} = Modulus of elasticity for LCC, [kPa]
M_{soil} = Modulus of compression for the soil, [kPa]
a = Proportion of column area per total unit area, i.e $(\pi r^2/s^2)$

Strain in block:

$$\varepsilon_{block} = \frac{\Delta\sigma_{block}}{M_{block}} \tag{7.2}$$

where; $\Delta\sigma_{block}$ = Average stress increase in block, [kPa]
ε_{block} = Strain level in block

Stress increase in soil:

$$\Delta\sigma_{soil} = \Delta\sigma_{block} \cdot \frac{M_{soil}}{M_{block}} \tag{7.3}$$

Stress increase in column:

$$\Delta\sigma_{col} = \Delta\sigma_{block} \cdot \frac{E_{col}}{M_{block}} \tag{7.4}$$

7.2 For the plastic zone - zone A:

The strength of the column:

$$f_{LCC} = 1.5 \cdot c_{u,col} + 3 \cdot \sigma'_{h,col} \tag{7.5}$$

where; $c_{u,col}$ = The undrained shear strength of the columns, [kPa]
$\sigma'_{h,col}$ = The horizontal effective stress in the soil against the column, [kPa]
f_{LCC} = The strength of the column, [kPa]

The horizontal effective stress, $\sigma'_{h,col}$ can be calculated as:

$$\sigma'_{h,col} = K_0 \cdot \sigma'_{v0} + 0.5 \cdot \Delta\sigma'_{soil} \tag{7.6}$$

Stress levels in soil and columns:

$$\Delta\sigma_{col,pl} = f_{LCC} - \sigma'_{v0} \tag{7.7}$$

$$\Delta\sigma_{soil} = \frac{\Delta\sigma_{block} - a \cdot \Delta\sigma_{col,pl}}{1 - a} \tag{7.8}$$

where; σ'_{v0} = The effective stress in the columns before the application of the load, [kPa]
$\sigma'_{col,pl}$ = Stress level in the columns, in [kPa]
f_{LCC} = The strength of the column, [kPa]
a = Proportion of column area per total unit area, i.e $(\pi r^2/s^2)$

Using the above the stress in the soil for $z = 0$ and $z = Zlim$ can be solved as:

$$\Delta\sigma_{soil}(0) = \frac{2 \cdot q - 3 \cdot a \cdot c_{u,col}}{2 + a} \tag{7.9}$$

$$\Delta\sigma_{soil}(Z_{lim}) = \Delta\sigma_{block}(Z_{lim}) \cdot \frac{M_{soil}}{M_{block}} \tag{7.10}$$

Stress levels for soil/column block in zone A

$$\Delta\sigma_{blocklim} = M_{block} \cdot \frac{1.5 \cdot c_{u,col} + \sigma'_{v0}}{E_{col} - 1.5 \cdot M_{soil}} \tag{7.11}$$

where; $c_{u,col}$ = The undrained shear strength of the columns, [kPa]
σ'_{v0} = The effective stress in the columns before the application of the load, [kPa]
M_{block} = Modulus of compression for soil-coloumn composite, [kPa]
E_{col} = Modulus of elasticity for LCC, [kPa]
M_{soil} = Modulus of compression for the soil, [kPa]

7.3 Stress increase at the border between zone A and B

Columns extend to firm ground:

$$\Delta\sigma_{block} = q \tag{7.12}$$

where; q = The applied load, in [kPa]
$\Delta\sigma_{block}$ = Average stress increase in block, [kPa]

Columns which do not reach firm ground:

$$\Delta\sigma_{block} = \eta_{LC} \cdot q + (1 - \eta_{LC}) \cdot q \cdot \frac{B}{(B + z)} \tag{7.13}$$

where; q = The applied load, in [kPa]
$\Delta\sigma_{block}$ = Average stress increase in block, [kPa]
η_{LC} = Key factor
B = Width of the load given in [m]
z = Depth in [m]

The Key factor η_{LC} determining what portion of the external load is transmitted unchanged to the bottom of the stabilized soil:

$$\eta_{LC} = \left(\frac{D}{H}\right)^{1/v} \tag{7.14}$$

where; H = Depth down to firm layer, in [m]
D = Length of columns, in [m]

and v is calculated as:

$$v = \left(\frac{M_{block}}{M_{soil}}\right)^{0.1} - \left(\frac{M_{soil}}{M_{block}}\right)^{0.1} \tag{7.15}$$