Geotechnical aspects of building design (EN 1997)

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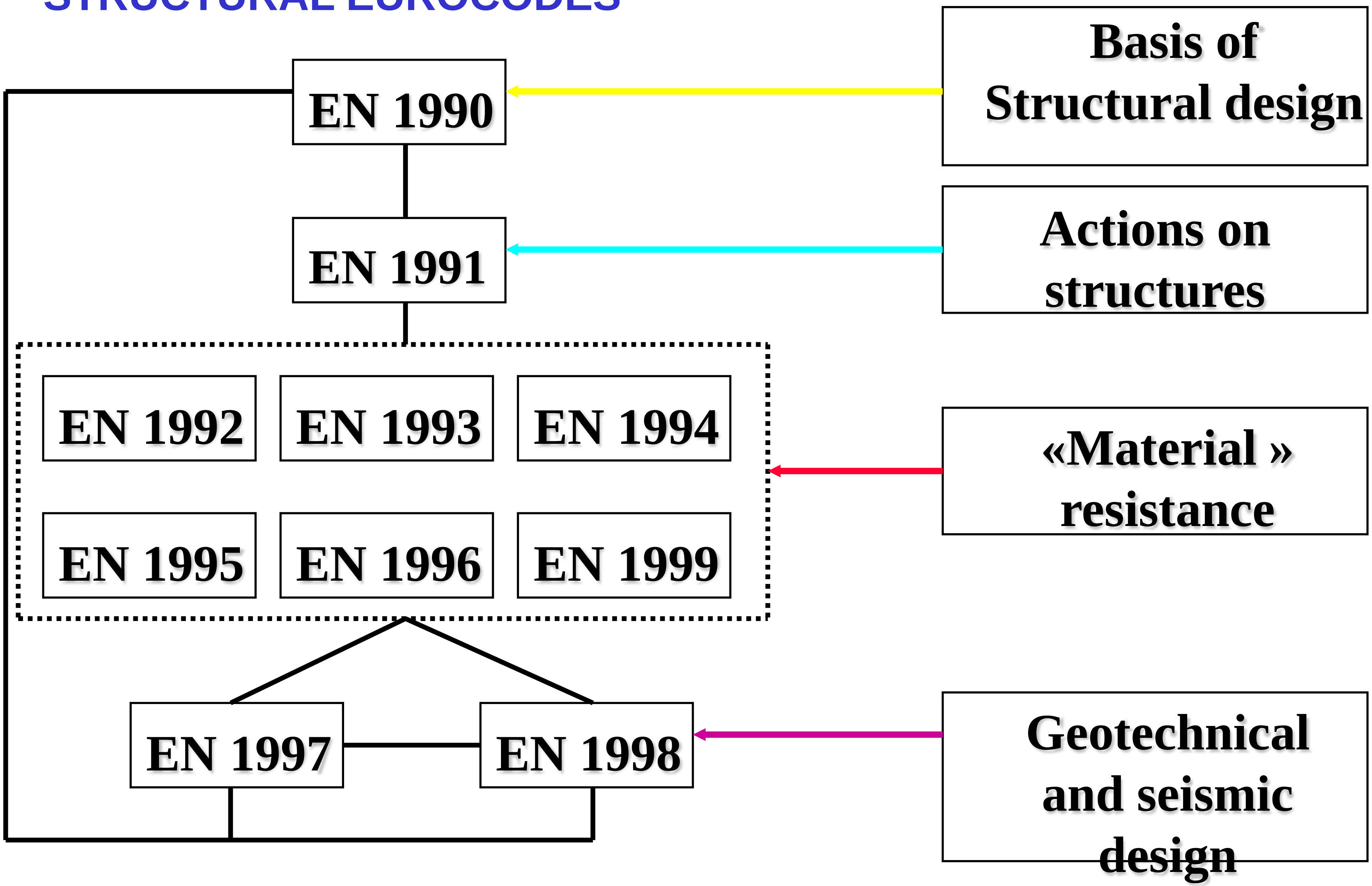
Outline

1. General presentation of Eurocode 7 ‘Geotechnical design’
    - Contents of Part 1 and 2
    - Specific aspects of EN 1997-1
    - 3 ULS-Design Approaches (DAs)
    - SLS and allowable movements of foundations
    - Spread foundations
    - Principles of embedded wall design

2. Application to building design
    - Geotechnical data
    - Column B2
        - ULS-bearing capacity
        - ULS-sliding
        - SLS-settlement

STRUCTURAL EUROCODES



Eurocode 7 – Geotechnical design

EN 1997-1 (2004) : Part 1 - General rules

EN 1997-2 (2007) : Part 2 - Ground investigation and testing

Contents of Part 1 (EN 1997-1)

Section 1 General
Section 2 Basis of geot design
Section 3 Geotechnical data
Section 4 Supervision of construction, monitoring and maintenance
Section 5 Fill, dewatering, ground improv and reinfor
Section 6 Spread foundations *
Section 7 Pile foundations
Section 8 Anchorages
Section 9 Retaining structures *
Section 10 Hydraulic failure
Section 11 Site stability
Section 12 Embankments

EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

EN 1997-1

November 2004

ICS 91.120.20

Supersedes ENV 1997-1:1994

English version

Eurocode 7: Geotechnical design - Part 1: General rules

Eurocode 7: Calcul géotechnique - Partie 1: Règles générales

Eurocode 7: Entwurf, Berechnung und Bemessung in der Geotechnik - Teil 1: Allgemeine Regeln

This European Standard was approved by CEN on 23 April 2004.

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EUROPÄISCHES KOMITEE FÜR NORMUNG

Management Centre: rue de Stassart, 36 B-1050 Brussels

Ref. No. EN 1997-1:2004: E

> + number of Informative annexes with geotechnical models

Contents of Part 2 (EN 1997-2)

Section 1 General

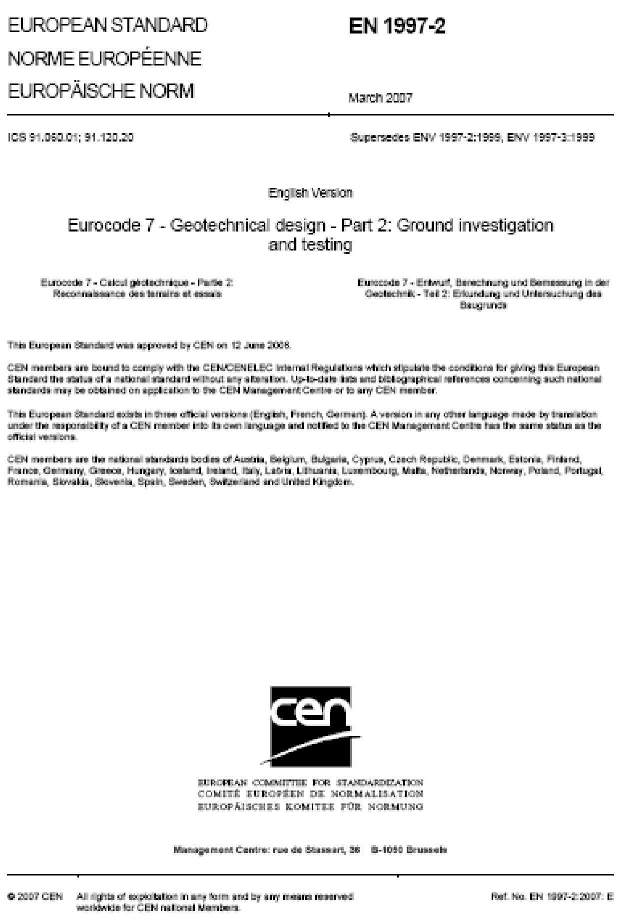
Section 2 Planning and reporting of ground investigations

Section 3 Drilling, sampling and gw measurements

Section 4 Field tests in soils and rocks

Section 5 Laboratory tests on soils and rocks

Section 6 Ground investigation report



EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

EN 1997-2

March 2007

ICS 91.060.01; 91.120.20

Supersedes ENV 1997-2:1999, ENV 1997-3:1999

English Version

Eurocode 7 - Geotechnical design - Part 2: Ground investigation and testing

Eurocode 7 - Calcul géotechnique - Partie 2: Reconnaissance des terrains et essais

Eurocode 7 - Entwurf, Berechnung und Bemessung in der Geotechnik - Teil 2: Erkundung und Untersuchung des Baugrunds

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Ref. No. EN 1997-2:2007: E

> Also a number of Informative annexes

Clauses on :

CPT(U), PMT, FDT, SPT, DP, WST, FVT, DMT, PLT

Objectives, specific requirements, evaluation of test results, use of test results and derived values

Annexes with examples on use of results and derived values for geotechnical design

preparation of soil specimens for testing
preparation of rock specimens for testing
tests for classif., identif. and description of soils
chemical testing of soils and groundwater
strength index testing of soils
strength testing of soils
compressibility and deformation testing of soils
compaction testing of soils
permeability testing of soils
tests for classification of rocks
swelling testing of rock material
strength testing of rock material

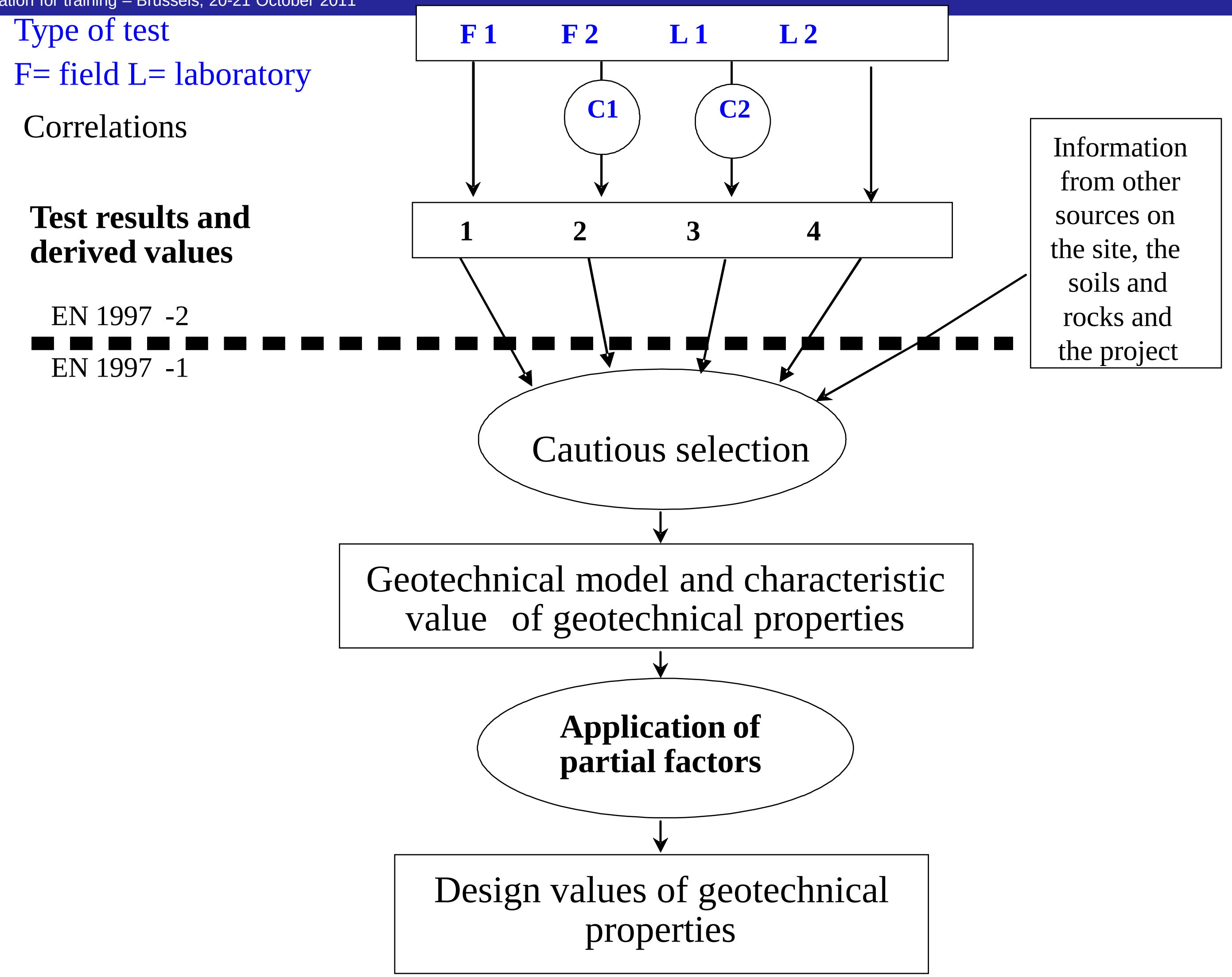
Results of test standards EN 1997-2 Annex A

Field test	Test results
CPT/CPTU	q_c , f_s , R_f (CPT) / q_t , f_s , u (CPTU)
Dynamic probing	N_{10} (DPL, DPM, DPH); N_{10} or N_{20} (DPSH)
SPT	N , E_r (SPT), soil description
Pressuremeters (PMT)	E_M ,,p_f , p_{IM} (MPM); expansion curve (all)
Flexible dilatometer (FDT)	E_{FDT}, deformation curve
Field vane test (FVT)	c_{fv} , c_{rv} , torque-rotation curve
Weight sounding test (WST)	continuous record of penetration depth or N_b
Plate loading test	p_u
Flta dilatometer test	P_0 , p_1 , E_{DMT} , I_{DMT} , K_{DMT} (DMT)

Laboratory tests

Soils: **w ; ρ ; ρ_s ; grain size distribution curve ; w_P , w_L ; e_{max} , e_{min} , I_D ; C_{OM} ; C_{CaCO3} ; $C_{SO4}{}^{2-}$, $C_{SO3}{}^{2-}$; C_{cl} ; pH ; compressibility, consolidation, creep curves, E_{oed}, σ'_p or C_s, C_c, σ'_p, C_α ; c_u (lab vane) ; c_u (fall cone) ; q_u ; c_u (UU) ; σ-ε and u curves, σ–paths, Mohr circles ; c', φ' or c_u, c_u=f(σ'c), E' or E_u ; σ-u curve, τ-σ diagram, c', φ', residual parameters ; I_{CBR} ; k (direct lab, field or oedometer)**
Rocks: **w ; ρ and n ; swelling results ; σ_c, E and ν ; I_{s50} ; σ-u curve, Mohr diagram, c', φ', res par ; σ_T ; σ-ε curve, σ–paths, Mohr circles ; c', φ', E and ν**

Geotechnical properties



Specific aspects of Eurocode 7-1

Characteristic values and design values

ULS Design Approaches

SLS and deformations of structures

P The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state.

If statistical methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%.

Ultimate limit states – Eurocode 7-1

- EQU : loss of equilibrium of the structure
- STR : internal failure or excessive deformation of the structure or structural elements
- GEO : failure or excessive deformation of the ground
- UPL : loss of equilibrium due to uplift by water pressure (buoyancy) or other vertical actions
- HYD : hydraulic heave, internal erosion and piping caused by hydraulic gradients

Design values of geotechnical parameters

Design value of a parameter : $X_d = X_k / \gamma_M$

Design values of actions and resistances

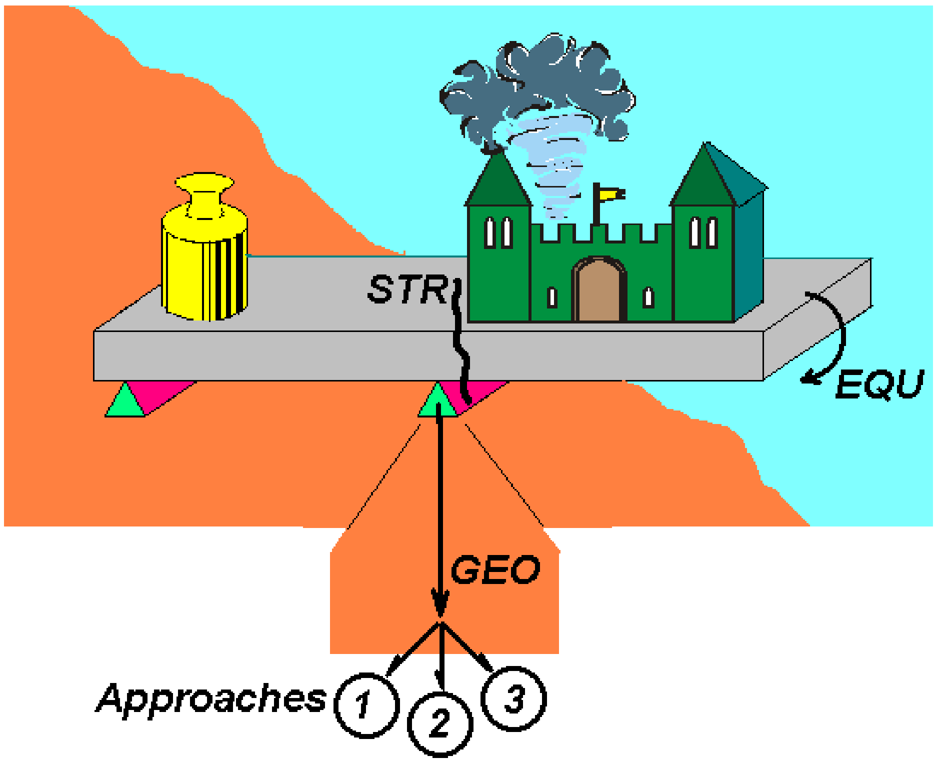
fulfilling for STR/GEO ULS : $E_d \leq R_d$

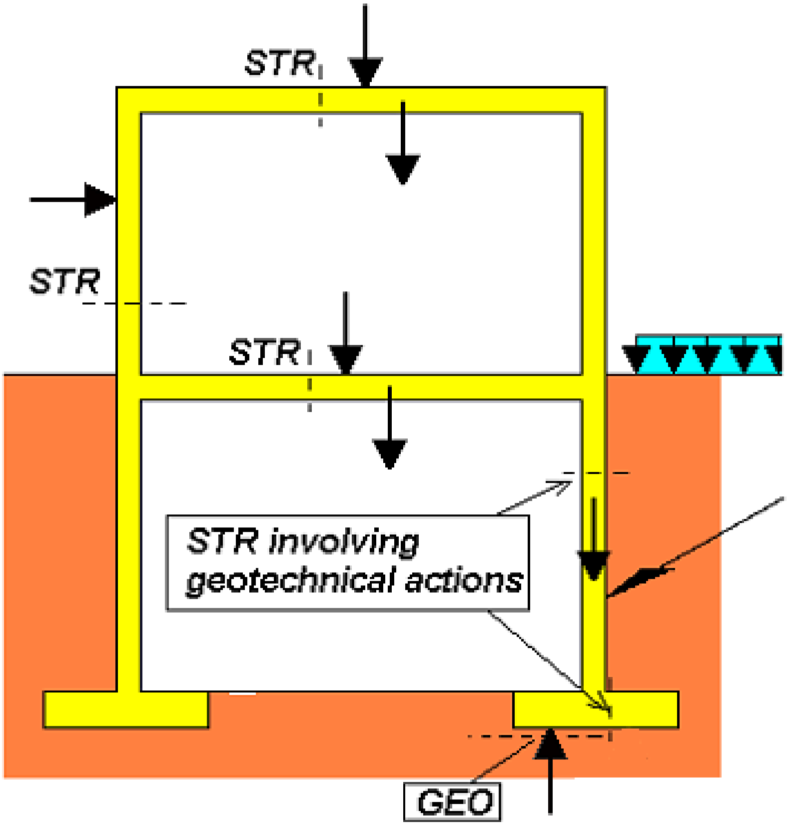
$E_d = E\{\gamma_F.F_k ; X_k / \gamma_M\}$ and $R_d = R\{\gamma_F.F_k ; X_k / \gamma_M\}$

(= “at the source”)

or $E_d = \gamma_E.E\{F_k ; X_k\}$ and $R_d = R\{F_k ; X_k\} / \gamma_R$

EN1990 - Ultimate limit states EQU and STR/GEO





$E_d < R_d$

J.A Calgaro

ULS - STR/GEO : 3 Design Approaches for persistent and transient design situations

Appro-aches	Combinations
1	A1 "+" M1 "+" R1 & A2 "+" M2 "+" R1 Or A2 "+" M1 or M2"+" R4
2	A1 "+" M1 "+" R2
3	A1 or A2 "+" M2 "+" R3

Format : $E_d < R_d$

Action (γ_F)	Symbol	Set A1	Set A2
Permanent			
Unfavourable	γ_G	1,35	1,00
Favourable	γ_G	1,00	1,00
Variable			
Unfavourable	γ_Q	1,50	1,30
Favourable	γ_Q	0	0

Soil parameter (γ_M)	Symbol	Set M1	Set M2
Angle of shearing resistance	$\gamma_{\varphi'}$	1,00	1,25
Effective cohesion	$\gamma_{c'}$	1,00	1,25
Undrained shear strength	γ_{cu}	1,00	1,40
Unconfined strength	γ_{qu}	1,00	1,40
Weight density	γ_{γ}	1,00	1,00

Resistance (γ_R)	Symbol	Set R1	Set R2	Set R3
Bearing capacity	γ_{Rv}	1,00	1,4	1,00
Sliding	γ_{Rh}	1,00	1,1	1,00

← γ_R for Spread foundations

Verifications :

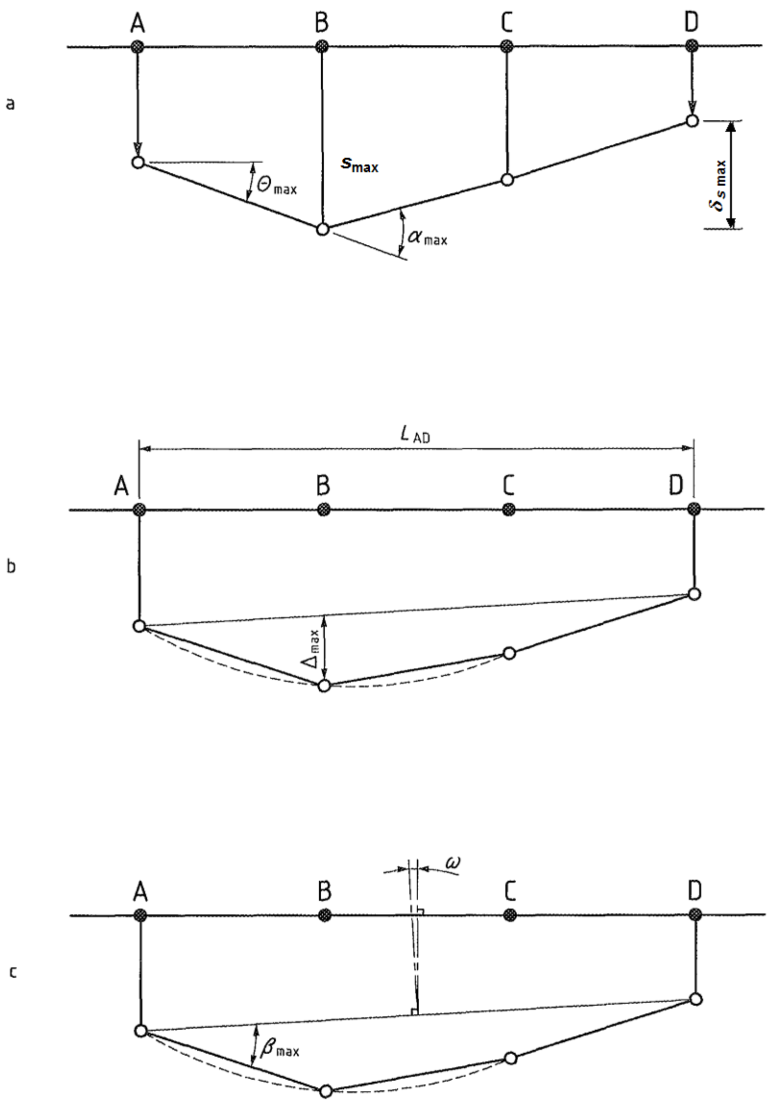
$$E_d \leq C_d$$

C_d = limiting design value of the relevant serviceability criterion

E_d = design value of the effects of actions specified in the serviceability criterion, determined on the basis of the relevant combination

all γ_F and $\gamma_M = 1.0$

EN 1997-1 annex H
Movements and deformations of structures



settlement *s*, differential settlement δs, rotation θ and angular strain α

relative deflection Δ and deflection ratio Δ/L

ω and relative rotation (angular distortion) β

(after Burland and Wroth, 1975)

Foundations of buildings (Eurocode 7, 1994)

* Serviceability limit states (SLS) : $\beta_{max} \approx 1/500$

* Ultimate limit states (ULS) : $\beta_{max} \approx 1/150$

- $s_{max} \approx 50$ mm $\delta_{smax} \approx 20$ mm

Foundations of bridges

Moulton (1986) for 314 bridges in the US and Canada :

* $\beta_{max} \approx 1/250$ (continuous deck bridges)

and $\beta_{max} \approx 1/200$ (simply supported spans)

* $s_{Hmax} \approx 40$ mm

In France, in practice :

ULS : $\beta_{max} \approx 1/250$

SLS : $\beta_{max} \approx 1/1000$ à $1/500$

Bearing resistance:

$$V_d \leq R_d = R_k \;/\; \gamma_{R;v}$$

(R_k : analytical – Annex D, semi-empirical – Annex E or prescriptive - Annex G)

Sliding resistance :

$$H_d \leq R_d + R_{p;d}$$

$$[+\ R_d \leq 0{,}4\ V_d\ ]$$

- drained conditions :

$$R_d = V'_d \tan \delta_d \quad \underline{\text{or}} \quad R_d = (V'_d \tan \delta_k) \;/\; \gamma_{R;h}$$

- undrained conditions

$$R_d = A'c_{u;d} \quad \underline{\text{or}} \quad R_d = (A'c_{u;k}) \;/\; \gamma_{R;h}$$

STR/GEO Ultimate limit states (ULS cntd)

Overall stability

Large eccentricities : special precautions if :

$e/B > 1/3$ (or $0,6\ \phi$)

Structural failure due to foundation movement

Structural design of spread foundation:
see EN 1992

STR/GEO persistent and transient design situations (spread foundations without geotechnical actions)

Design approach	Actions on/from the structure γ_F	Geotechnical resistance γ_R or γ_M (at the source)
1	1,35 and 1,5	$\gamma_{R;v}$ = 1,0 $\gamma_{R;h}$ = 1,0
	1,0 and 1,3	γ_M = 1,25 or 1,4
2	1,35 and 1,5	$\gamma_{R;v}$ = 1,4 $\gamma_{R:h}$ = 1,1
3	1,35 and 1,5	γ_M = 1,25 or 1,4

Serviceability limit states (SLS)

Include both immediate and delayed settlements

Assess differential settlements and relative rotations

Check that limit values for the structure are not reached

Verifications methods

Direct method :

- check each limit states (ULS and SLS)
- check the settlement for the SLSs

Indirect method :

- only a SLS calculation based on experience

Prescriptive method :

- example of the presumed bearing resistance on rocks (Annex G)

Relevant annexes in EN 1997-1

Annex A (normative) Safety factors for ultimate limit states

Informative annexes :

Annex D A sample analytical method for bearing resistance calculation

Annex E A sample semi-empirical method for bearing resistance estimation

Annex F Sample methods for settlement evaluation

Annex G A sample method for deriving presumed bearing resistance for spread foundations on rock

Annex H Limiting foundation movements and structural deformation

“c-φ” model (Annex D)

$$R/A' = c' \times N_c \times b_c \times s_c \times i_c$$

$$+ q' \times N_q \times b_q \times s_q \times i_q$$

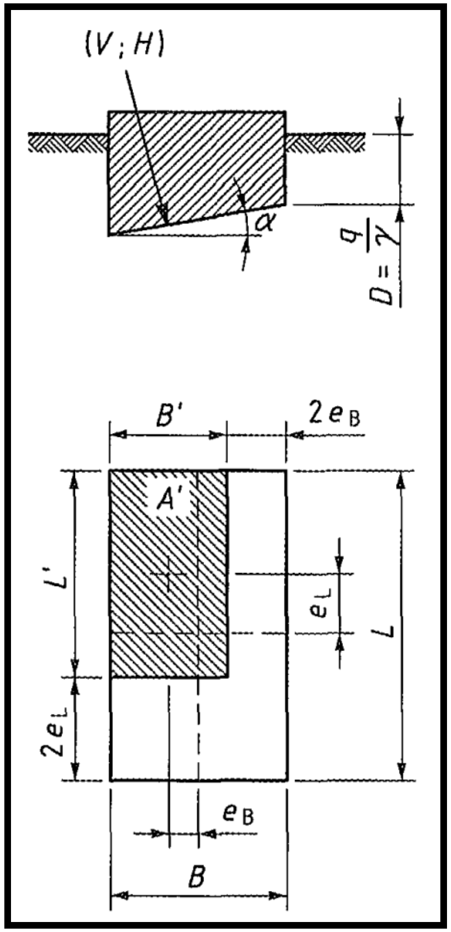
$$+ 0,5 \times \gamma' \times B' \times N_\gamma \times b_\gamma \times s_\gamma \times i_\gamma$$

Pressuremeter model *(annexe E)*

$$R/A' = \sigma_{v0} + k \times p^*_{le}$$

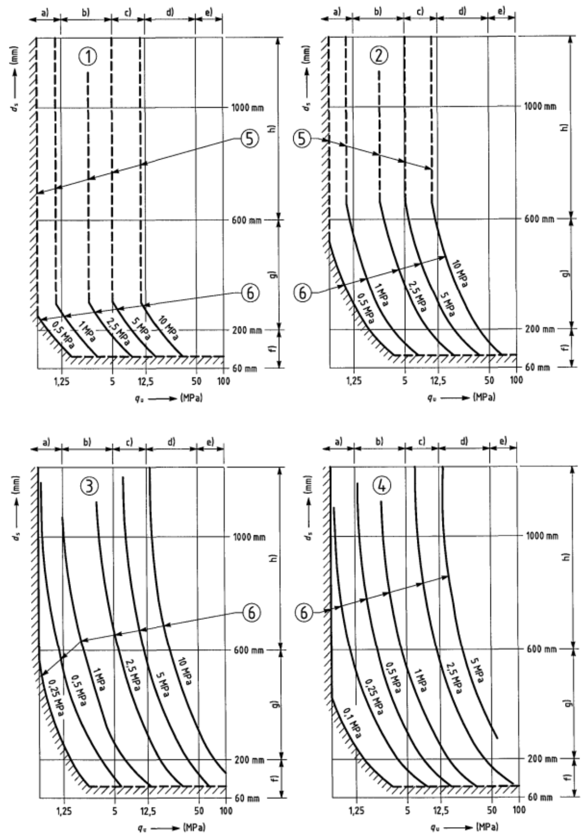
Settlement of foundations (Annex F)

Adjusted elasticity: $s = p \times b \times f / E_m$



EN 1997-1 annex G
Bearing resistance on rocks

Group	Type of rock
1	Pure limestones and dolomites Carbonate sandstones of low porosity
2	Igneous Oolitic and marly limestones Well cemented sandstones Indurated carbonate mudstones Metamorphic rocks, including slates and schist (flat cleavage/foliation)
3	Very marly limestones Poorly cemented sandstones Slates and schists (steep cleavage/foliation)
4	Uncemented mudstones and shales



5 Allowable bearing pressure not to exceed uniaxial compressive strength of rock if joints are tight or 50 % of this value if joints are open,

6 Allowable bearing pressures: a) very weak rock, b) weak rock c) moderately weak rock d) moderately strong rock, e) strong rock

Spacings: f) closely spaced discontinuities g) medium spaced discontinuities h) widely spaced dicontinuities

For types of rock in each of four groups, see Table G.1. Presumed bearing resistance in hatched areas to be assessed after inspection and/or making tests on rock. (from BS 8004)

Relevant annexes in EN 1997-2

Informative annexes :

D.3 Example of a method to determine the settlement for spread foundations from CPT
D.4 Example of a correlation between the oedometer modulus and the cone penetration resistance from CPT
D.5 Examples of establishing the stress-dependent oedometer modulus from CPT results
E.1 Example of a method to calculate the bearing resistance of spread foundations from PMT
E.2 Example of a method to calculate the settlements for spread foundations from PMT
F.3 Example of a method to calculate the settlement of spread foundations from SPT
G.3 Example of establishing the stress-dependent oedometer modulus from DP results
J Flat dilatometer test (DMT)
K.4 Example of a method to calculate the settlement of spread foundations in sand from (PLT)

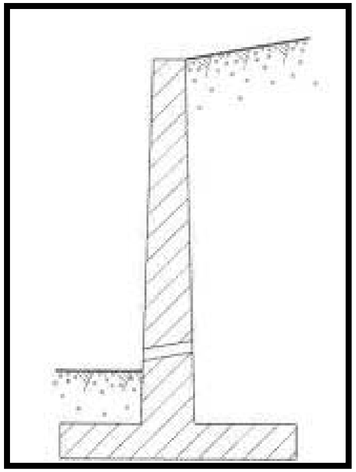
Retaining structures

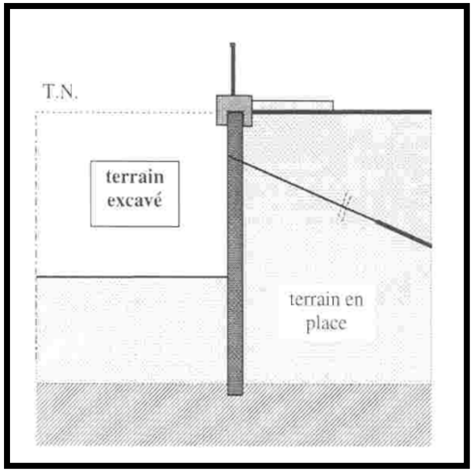
Scope of Eurocode 7 (Section 9)

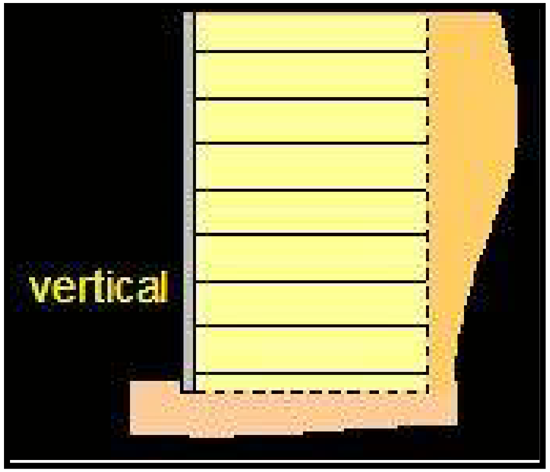
Gravity walls (in stone, concrete, reinforced concrete)

Embedded walls (sheet pile walls, slurry trench walls ; cantilever or supported walls)

Composite retaining structures (walls composed of elements, double wall cofferdams, reinforced earth structures)







Ultimate limit state of embedded walls

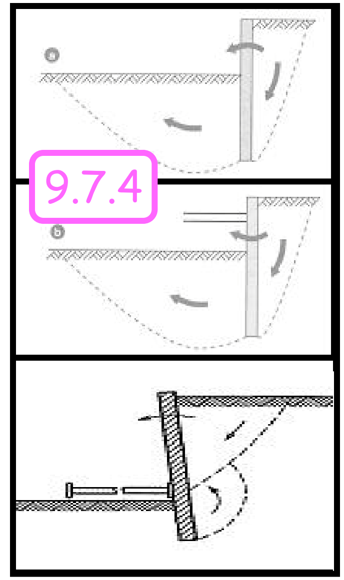
9.7.2 Overall stability
(principles of section 11)

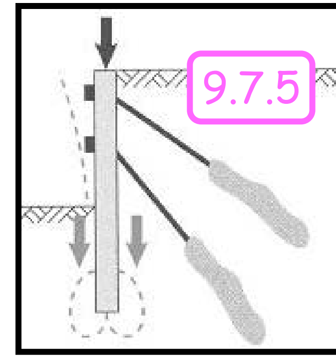
9.7.4 Rotational failure
(lack of passive pressure)

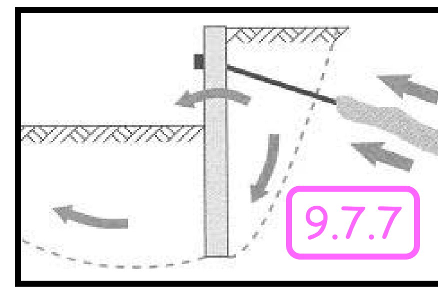
9.7.5 Vertical failure
(principles of sections 7)

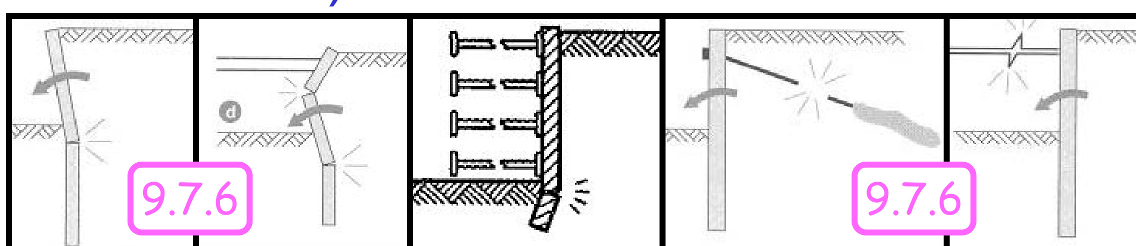
9.7.6 Structural design
(in accordance with EC 2, EC 3, EC5 and EC6)

9.7.7 Failure by pull-out of anchorages
(in accordance with section 8)









9.7.1 (7) : Hydraulic failure (uplift, heave, etc.) (see section 10)

Ultimate limit state of embedded walls

Overall stability (9.7.2) :
Principles of section 11 apply

Rotational failure of embedded walls (9.7.4) :
it shall be demonstrated hat they have sufficient penetration into the ground , the design magnitude and direction of shear stress between the soil and the wall being consistent with the relative vertical displacement

Vertical failure of embedded walls (9.7.5) :
The design magnitude and direction of shear stress between the soil and the wall shall be consistent with the check for vertical and rotational equilibrium

Failure by pull-out of anchorages (9.7.7) : in accordance with section 8 (… under amendment !)

ULS - STR/GEO : 3 Design Approaches for persistent and transient design situations

Approaches	Combinations
1	A1 “+” M1 “+” R1 & A2 “+” M2 “+” R1 Or A2 “+” M1 or M2“+” R4
2	A1 “+” M1 “+” R2
3	A1 or A2 “+” M2 “+” R3

Action (γ_F)	Symbol	Set A1	Set A2
Permanent			
Unfavourable	γ_G	1,35	1,00
Favourable	γ_G	1,00	1,00
Variable			
Unfavourable	γ_Q	1,50	1,30
Favourable	γ_Q	0	0

Soil parameter (γ_M)	Symbol	Set M1	Set M2
Angle of shearing resistance	$\gamma_{\varphi'}$	1,00	1,25
Effective cohesion	$\gamma_{c'}$	1,00	1,25
Undrained shear strength	γ_{cu}	1,00	1,40
Unconfined strength	γ_{qu}	1,00	1,40
Weight density	γ_{γ}	1,00	1,00

Resistance (γ_R)	Symbol	Set R1	Set R2	Set R3
Bearing capacity	$\gamma_{R;v}$	1,0	1,4	1,0
Sliding resistance	$\gamma_{R;h}$	1,0	1,1	1,0
Earth resistance	$\gamma_{R;e}$	1,0	1,4	1,0

← γ_R for Retaining structures

Relevant annexes in EN 1997-1

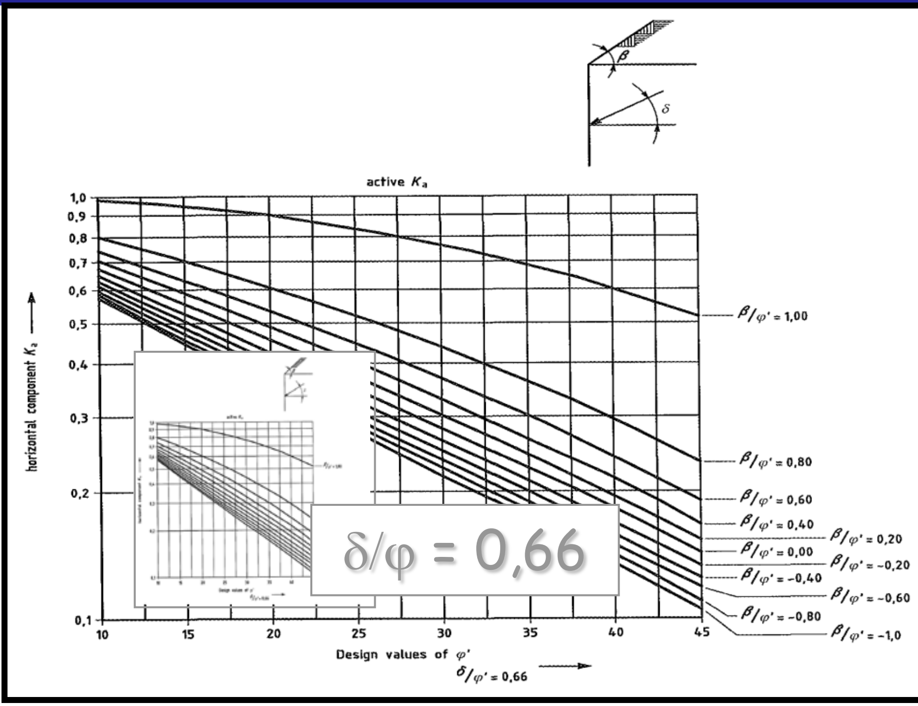
Annex A (normative) Safety factors for ultimate limit states

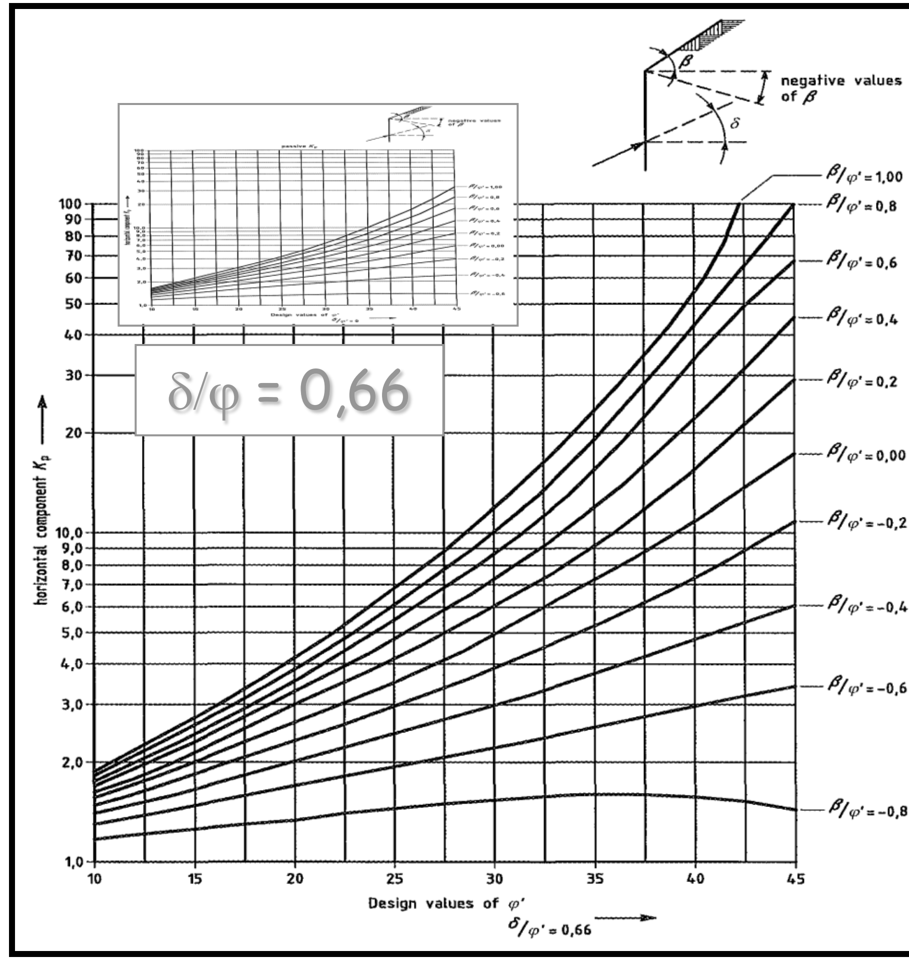
Informative annexes :

Annex C Limit values of earth pressures on vertical walls

Annex H Limiting foundation movements and structural deformation

Active /Passive earth pressures - annex C





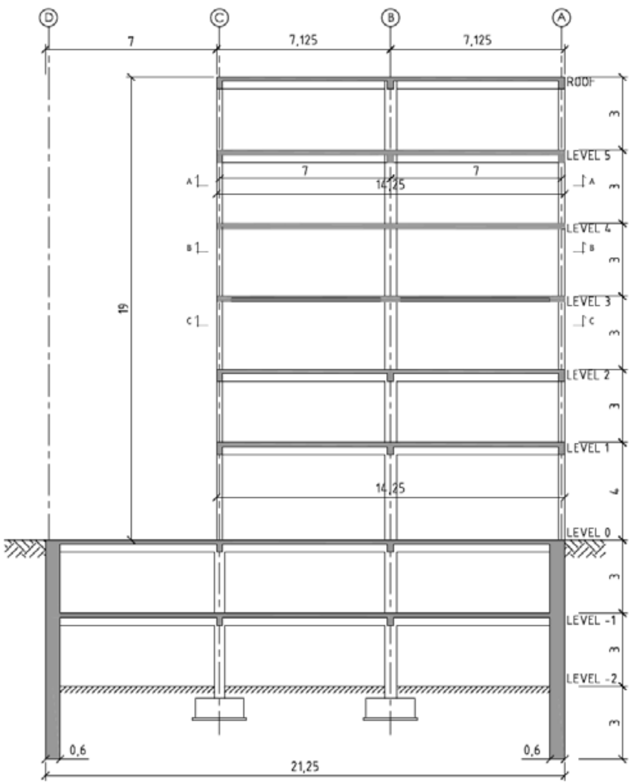
Active/Passive earth pressures

$\beta = -\varphi$ à $+\varphi$

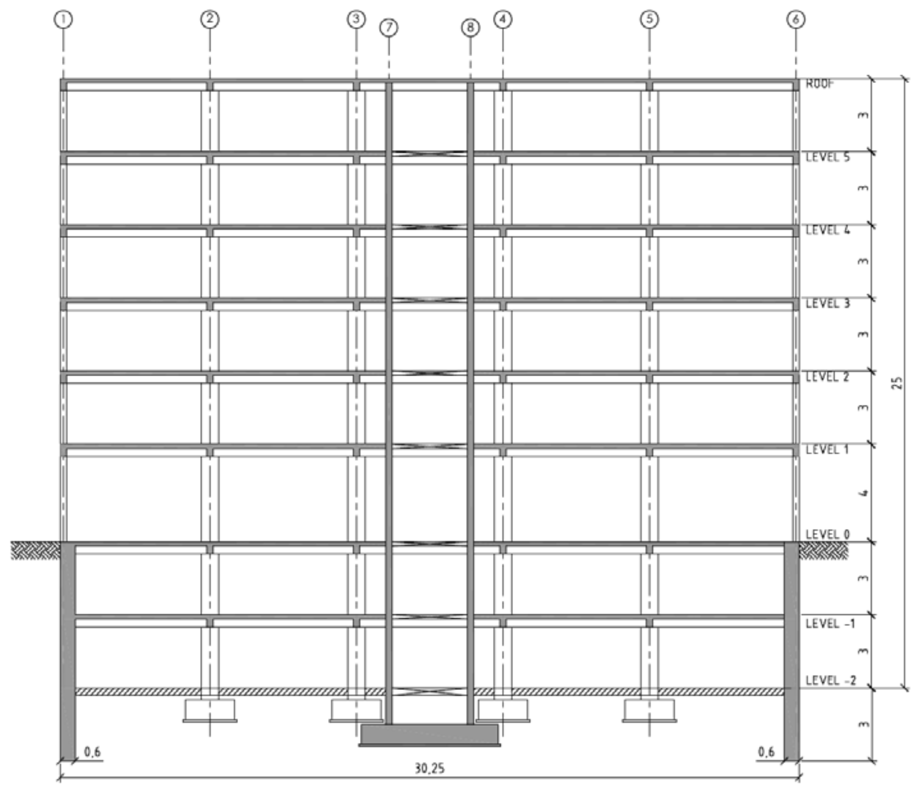
$\delta = 0$; $2/3\varphi$ and φ

Building design

SECTION 1



SECTION 2



Geotechnical data

For the sake of simplicity, in the present study, it is assumed that the whole building is founded on a very stiff clay:

- undrained shear strength (for total stresses analysis, short term) : $c_u = 300$ kPa
- total unit weight : $\gamma_k = 20$ kN/m^3

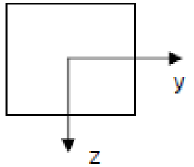
The water-table is assumed to be at natural ground level.

Example of column B2

Table 1. Forces and moments on the foundation of column B2 for ULS – Fundamental combinations (Curbach and Just, 2011)

Column B2

Situation: Foundation
Position of internal forces: at the sole of the Foundation



Ultimate limit state

Superposition	N [KN]	Vy [KN]	Vz [KN]	My [KNm]	Mz [KNm]	Q1	Qi
Combination:	1,35 * G + 1,5 * Q1 + 1,5 * Σ(psi0 * Qi)					Considered load cases	
						Q1	Qi
max My, accordingly N, V und Mz	-4554,80	-3,56	-3,68	3,82	-3,78	101	203 - 206, 10101
max Mz, acc. N, V und My	-4837,96	0,24	2,51	-3,11	0,60	51	203, 205, 206, 10011
max Vy, acc. M, N und Vz	-4990,35	0,48	2,02	-2,71	0,66	10011	51, 203, 205, 206
max Vz, acc. M, N und Vy	-4985,91	-2,81	2,78	-3,26	-2,31	51	202, 1356, 10001, 10121
max N, acc. V und M	-4491,62	-1,83	2,27	-2,73	-1,38	51	202-205
min My, acc. N, V und Mz	-5435,54	-3,83	2,38	-2,98	-3,46	10121	51, 201, 202, 1356, 10031
min Mz, acc. N, V und My	-5359,70	1,26	-1,77	1,44	0,71	10111	101, 201, 202, 1356, 10031
min Vy, acc. M, N und Vz	-5359,70	1,26	-1,77	1,44	0,71	10111	101, 201, 202, 1356, 10031
min Vz, acc. M, N und Vy	-4502,78	-3,18	-4,01	4,08	-3,50	101	204-206
min N, acc. V und M	-5780,18	-4,53	1,54	-2,36	-4,49	10031+1336	201, 10121

Example of column B2

Table 2. Forces and moments on the foundation of column B2 for SLS (Curbach and Just, 2011)

Virtually-permanent combination

Combination:	1,00 * G + 1,00 * Σ(psi2 * Qi)					Considered load cases Qi
max My, accordingly N, V und Mz	-3365,83	-2,75	0,83	-1,24	-2,61	10101
max Mz, acc. N, V und My	-3434,58	-1,95	0,70	-1,18	-1,89	10011
max Vy, acc. M, N und Vz	-3434,58	-1,95	0,70	-1,18	-1,89	10011
max Vz, acc. M, N und Vy	-3506,91	-3,02	0,84	-1,25	-2,86	10121, 1356, 10001
max N, acc. V und M	-3473,34	-0,65	0,69	-1,13	-0,68	1366,10001,10111
min My, acc. N, V und Mz	-3603,82	-3,03	0,87	-1,31	-2,90	10121, 10031, 1356
min Mz, acc. N, V und My	-3585,31	-0,66	0,72	-1,19	-0,72	10111, 10031, 1336
min Vy, acc. M, N und Vz	-3585,31	-0,66	0,72	-1,19	-0,72	10111, 10031, 1336
min Vz, acc. M, N und Vy	-	-	-	-	-	not applicable
min N, acc. V und M	-3618,11	-3,03	0,87	-1,31	-2,90	10121, 10031, 1336

Example of column B2

- ULS – Bearing capacity
- ULS – Sliding resistance
- SLS – Settlement check

Column B2 – ULS Bearing capacity

$$N_d < R_d$$

Resultant actions : most unfavourable case in permanent and transient design situation – see table 1 (to be checked) :

$N_d = - 5.78$ MN

$V_{yd} = -4.53 \times 10^{-3}$ MN

$V_{zd} = -1.54 \times 10^{-3}$ MN

$M_{yd} = -2.36 \times 10^{-3}$ MN.m

$M_{zd} = -4.49 \times 10^{-3}$ MN.m

► $H_d = 4.78 \times 10^{-3}$ MN.m

Note that horizontal loads and moments on this foundation are negligible

Geotechnical resistance (bearing capacity)– see Annex D of EN 1997-1 (CEN, 2004)

$$R = A' (\pi+2)\, c_u\, s_c\, i_c \qquad (4)$$

with $A' = B'\,L' = (B-2e_B).(L-2e_L)$

$s_c = 1+0.2\, B'/L'$

and $$i_c = \frac{1}{2}\left(1+\sqrt{1-\frac{H}{A' c_u}}\right)$$

with H being the resultant horizontal force (resultant of V_y and V_z)

Eccentricity, is calculated by :

- in the transversal (B) direction : $e_B = M_y/N$
- in the longitudinal (L) direction : $e_L = M_z/N$

Note the dependance on the actions, thus on the design approach….

Column B2 – ULS Bearing capacity

For *DA1-1, DA2 and DA3*, $e_B = 4.1 \times 10^{-4\ m}$ (!)
$e_L = 7.8 \times 10^{-4\ m}$ (!), B' ≈ B and L' ≈ L and $s_c = 1.2$

For *DA1-2*, the loads are divided by a factor somewhere between 1.15 and 1.35, depending on the proportion of permanent and variable loads.

Correction factor s_c, and the total resistance R also depend on the Design Approach through γ_M and $\gamma_{R;v}$ (see tables for spread foundations).

Column B2 – ULS Bearing capacity

Design Approach 1

- combination DA1-1: $\gamma_M = 1{,}0$; $\gamma_{R;v} = 1{,}0$

Thus : $c_{ud} = 300$ kPa ; $s_c \approx 1{,}2$, $i_c \approx 1$

and $R_d = 4 \times 5.14 \times 1.2 \times 1 \times 300 \times 10^{-3} / 1.0 = 7.4/1.0$

$= 7.4$ MN and $N_d \leq R_d$ is verified.

- combination DA1-2 : $\gamma_M = 1{,}4$; $\gamma_{R;v} = 1{,}0$

Thus : $c_{ud} = 300/1{,}4 = 214$ kPa ; $s_c \approx 1{,}2$, $i_c \approx 1$

and $R_d = 4 \times 5.14 \times 1.2 \times 1 \times 214 \times 10^{-3} / 1.0 = 5.28/1.0$

$= 5.28$ MN

Let us assume that N_d is equal to N_d for DA1-2 divided by 1.25, thus $N_d = 4.62$ MN and $N_d \leq R_d$ is verified.

Design Approaches 2 and 3

They yield the same safety, because one of the values for the factors γ_M and $\gamma_{R;v}$ is equal to 1,4 and the other one is equal to 1,0.

Thus: R_d = 4x5.14x1.2x1x300x10-3 /1.4 = 5.28 MN.
and $N_d \leq R_d$ is not verified.

The size of the footing should be around :
$A' = 1.4 \times N_d/(\pi+2)\, c_u\, s_c\, i_c \approx 4.37$, that is, say :
B = L = 2,1. The difference is small…

Column B2 – ULS Sliding resistance

$$H_d \leq R_d + R_{p;d}$$

where

H_d horizontal component in the longitudinal direction

R_d is the sliding resistance on the base area of the foundation

$R_{p;d}$ is the passive earth force in front of the spread foundation (will be neglected here).

For undrained conditions : $R_d = \{A'c_u/\gamma_M\}/\gamma_{R;h}$

where

- $A' = B' L' = (B-2e_B).(L-2e_L)$
- $c_u = 300$ kPa is the undrained shear strength of the stiff clay

Column B2 – ULS Sliding resistance

Resultant actions : most unfavourable case in permanent and transient design situation – see table 1 (to be checked) :

$N_d = - 4.50$ MN

$V_{yd} = - 3.18 \times 10^{-3}$ MN

$V_{zd} = - 4.01 \times 10^{-3}$ MN

$M_{yd} = - 4.1 \times 10^{-3}$ MN.m

$M_{zd} = - 3.5 \times 10^{-3}$ MN.m

► $H_d = 5.12 \times 10^{-3}$ MN.m

Note that horizontal loads and moments on this foundation are negligible and e_B and e_L remain negligible and B' ≈ B, L' ≈ L and A' ≈ BL ≈ 4m².

Design Approach 1

- combination DA1-1: $\gamma_M = 1,0$; $\gamma_{R;h} = 1,0$

Thus, $c_{ud} = 300$ kPa and $R_d = 4x0.300 /1.0 = 1.2$ MN and $H_d \leq R_d$ is largely verified.

- combination DA1-2 : $\gamma_M = 1,4$; $\gamma_{R;h} = 1,0$

Thus, $c_{ud} = 300/1.4= 214$ kPa and $R_d = 4x0.214/1,0 = 0.86$ MN, with $H_d < 5.12$ kN. $H_d \leq R_d$ is largely verified.

According to DA1, the foundation is safe with regard to sliding.

Column B2 – ULS Sliding resistance

Design Approach 2 : $\gamma_M = 1{,}0$; $\gamma_{R;h} = 1{,}1$

Thus, $c_{ud} = 300$ kPa

$R_d = 4x0.300 / 1.1 = 1{,}09$ MN

and $H_d \leq R_d$ is largely verified.

Design Approach 3 : $\gamma_M = 1{,}4$; $\gamma_{R;h} = 1{,}0$

Thus, $c_{ud} = 300/1.4 = 214$ kPa

$R_d = 4x0.214/1.0 = 0.86$ MN

and $H_d \leq R_d$ is largely verified.

Column B2 – SLS Settlement check

$$s_d < C_d$$

s_d - Determine the settlement(s)

- Compensated foundation : no settlement (?)
- Empirical Ménard pressuremeter calculation (Informative Annex D2 of EN 1997-2)
- Adjusted elasticity approach (Informative Annex F of EN 1997-1)

c_d - Check the results against … limiting values c_d provided for by the structural engineer !

Column B2 – SLS Settlement check

Settlements are usually derived for SLS-QP combination

For column B2, from Table 2 :

$$N_d = 3.6 \text{ MN}$$

which corresponds to the applied pressure on the ground:

$$q = N_d/(BL) = 3.6/2x2 = 0.9 \text{ MPa}$$

Column B2 – SLS Settlement check

Ménard formula :

$$s = (q - \sigma_{v0}) \times \left[\frac{2B_0}{9E_d} \times \left(\frac{\lambda_d B}{B_0} \right)^a + \frac{\alpha \lambda_c B}{9E_c} \right]$$

- σ_{vo} = 0, as if the soil is loaded from its initial natural level (pessimistic assumption)
- square foundation: B_0 = 0.6m; λ_d = 1.12 and λ_c = 1.1
- overconsolidated clay : α = 1
- $E_M \approx 150\ c_u$ = 45 MPa; thus, $E_d = E_c$ = 45 MPa

Finally, s_{B2} =(0.9-0.00)[1.2(1.12x2/0.6)^1/(9x45)+ 1x1.1x2/(9x45)]
= 0.9 [0.011 + 0.0054] = 0.015 m = 15 mm.

Adjusted elasticity (pseudo-elastic) approach (Annex F)

$$s = q \times B \times f / E_m$$

but how to evaluate E_m ???

Here, it is assumed that $E_m \approx E_u \approx 500$ MPa (from Mair, 2011, Singapore clay matrix, $c_u > 150$ kPa,back-analysis of settlements of buildings on rafts):

$s_{B2} = 0.9x2.0x0.66/500 = 0.0023 =$ 2.3 mm

P1 – SLS Settlement

Allowable relative rotation?

If $\delta s \approx s_{B2}/2$, $L = 6$ m

the relative rotation is for :

$\beta = s_{B2}/2L =$ 1.2×10^{-3} and 0.109×10^{-3} respectively

Annex H of EN 1997-1 (Informative) states that a relative rotation $\beta = 1/500 = 2 \times 10^{-3}$ is quite acceptable

and to conclude, a nice sentence from En 1997-1 :

It should be considered that knowledge of the ground conditions depends on the extent and quality of the geotechnical investigations. Such knowledge and the control of workmanship are usually more significant to fulfilling the fundamental requirements than is precision in the calculation models and partial factors.

Thank you for your kind and patient attention !