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Eurocode 7: Geotechnical Design Worked examples

Worked examples presented at the Workshop "Eurocode 7: Geotechnical Design"
Dublin, 13-14 June, 2013

Support to the implementation, harmonization and further development of the Eurocodes

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CHAPTER 1

Foreword

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The construction sector is of strategic importance to the EU as it delivers the buildings and infrastructure needed by the rest of the economy and society. It represents more than 10% of EU GDP and more than 50% of fixed capital formation. It is the largest single economic activity and it is the biggest industrial employer in Europe. The sector employs directly almost 20 million people. Construction is a key element not only for the implementation of the Single Market, but also for other construction relevant EU Policies, e.g. Sustainability, Environment and Energy, since 40-45% of Europe's energy consumption stems from buildings with a further 5-10% being used in processing and transport of construction products and components.

The *EN Eurocodes* are a set of European standards which provide common rules for the design of construction works, to check their strength and stability against live extreme loads such as fire and earthquakes. In line with the EU's strategy for smart, sustainable and inclusive growth (EU2020), Standardization plays an important part in supporting the industrial policy for the globalization era. The improvement of the competition in EU markets through the adoption of the *Eurocodes* is recognized in the *"Strategy for the sustainable competitiveness of the construction sector and its enterprises"* - COM (2012)433, and they are distinguished as a tool for accelerating the process of convergence of different national and regional regulatory approaches.

With the publication of all the 58 Eurocodes Parts in 2007, the implementation of the *Eurocodes* is extending to all European countries and there are firm steps toward their adoption internationally. The Commission Recommendation of 11 December 2003 stresses the importance of training in the use of the *Eurocodes*, especially in engineering schools and as part of continuous professional development courses for engineers and technicians, which should be promoted both at national and international level. It is recommended to undertake research to facilitate the integration into the *Eurocodes* of the latest developments in scientific and technological knowledge.

In light of the Recommendation, DG JRC is collaborating with DG ENTR and CEN/TC250 "Structural Eurocodes" and is publishing the Report Series 'Support to the implementation, harmonization and further development of the *Eurocodes*' as JRC Scientific and Policy Reports. This Report Series includes, at present, the following types of reports:

1. Policy support documents – Resulting from the work of the JRC in cooperation with partners and stakeholders on 'Support to the implementation, promotion and further development of the *Eurocodes* and other standards for the building sector';
2. Technical documents – Facilitating the implementation and use of the *Eurocodes* and containing information and practical examples (Worked Examples) on the use of the *Eurocodes* and covering the design of structures or its parts (e.g. the technical reports containing the practical examples presented in the workshop on the *Eurocodes* with worked examples organized by the JRC);
3. Pre-normative documents – Resulting from the works of the CEN/TC250 and containing background information and/or first draft of proposed normative parts. These documents can be then converted to CEN technical specifications
4. Background documents – Providing approved background information on current Eurocode part. The publication of the document is at the request of the relevant CEN/TC250 Sub-Committee;
5. Scientific/Technical information documents – Containing additional, non-contradictory information on current Eurocode part, which may facilitate its implementation and use, or preliminary results from pre-normative work and other studies, which may be used in future revisions and further developments of the standards. The authors are various stakeholders involved in *Eurocodes* process and the publication of these documents is authorized by relevant CEN/TC250 Sub-Committee or Working Group.

Editorial work for this Report Series is assured by the JRC together with partners and stakeholders, when appropriate. The publication of the reports type 3, 4 and 5 is made after approval for publication from the CEN/TC250 Co-ordination Group.

The publication of these reports by the JRC serves the purpose of implementation, further harmonization and development of the *Eurocodes*. However, it is noted that neither the Commission nor CEN are obliged to follow or endorse any recommendation or result included in these reports in the European legislation or standardization processes.

This report is part of the so-called Technical documents (Type 2 above) and contains a comprehensive description of the practical examples presented at the workshop *"Geotechnical design with the Eurocodes"* with emphasis on worked examples. The workshop was held on 13-14 June 2013 in Dublin, Ireland and was co-organized with CEN/TC250/Sub-Committee 7, the Ireland's Department of the Environment, Community and Local Government, with the support of CEN and the Member States. The workshop addressed representatives of public authorities, national standardisation bodies, research institutions, academia, industry and technical associations involved in training on the Eurocodes. The main objective was to facilitate training on *Eurocode 7* related to geotechnical design through the transfer of knowledge and training information from the *Eurocode 7* writers (CEN/TC250 Sub-Committee 7) to key trainers at national level and Eurocode users.

The workshop was a unique occasion to compile a state-of-the-art training kit comprising the slide presentations and technical papers with the worked examples for encompassing the most important practical cases of geotechnical design. The present JRC Report compiles all the technical papers and worked examples prepared by the workshop lecturers. The editors and authors have sought to present useful and consistent information in this report. However, it must be noted that the report does not present complete design examples and that the reader may identify some discrepancies between chapters. The chapters presented in the report have been prepared by different authors and are reflecting the different practices in the EU Member States. Users of information contained in this report must decide themselves of its suitability for the purpose for which they intend to use it.

We would like to gratefully acknowledge the workshop lecturers and the members of CEN/TC250 Sub-Committee 7 for their contribution in the organization of the workshop and development of the training material comprising the slide presentations and technical papers with the worked examples. We would also like to thank the Ireland's Department of the Environment, Community and Local Government, and especially John Wickham for the help and support in the local organization of the workshop.

All the material prepared for the workshop (slides presentations and JRC Report) is available to download from the *"Eurocodes: Building the future" website* (http://eurocodes.jrc.ec.europa.eu).

Ispra, October 2013

CHAPTER 2

Basis of design

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2.1. Overview

The Eurocode family of design standards is illustrated in Figure 2.1.1 (*after Bond & Harris, 2008*). It comprises:

- EN 1990, Basis of structural design
- EN 1991, Actions on structures
- EN 1992, Design of concrete structures
- EN 1993, Design of steel structures
- EN 1994, Design of composite concrete and steel structures
- EN 1995, Design of timber structures
- EN 1996, Design of masonry structures
- EN 1997, Geotechnical design
- EN 1998, Design of structures for earthquake resistance
- EN 1999, Design of aluminium structures

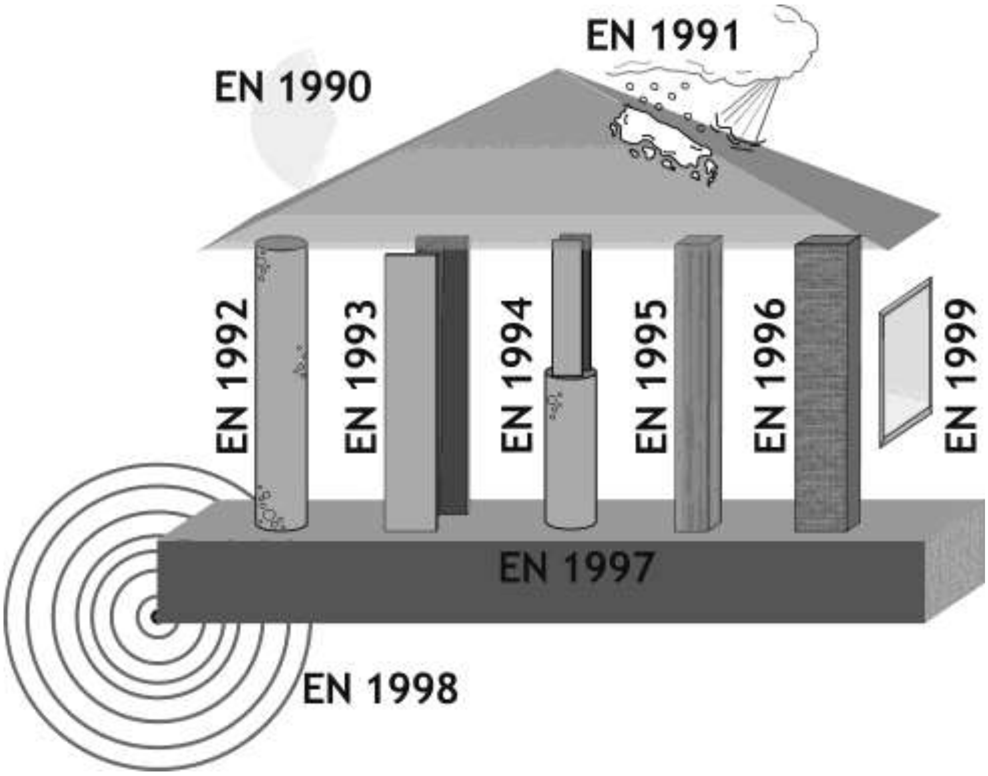

Fig. 2.1.1 Eurocode family of design standards (after Bond and Harris, 2008)

2.2. Design requirements

Eurocode design is based on Principles (general statements, analytical models, and requirements) – where no alternative is permitted – and Application Rules, which are generally recognised rules that comply with the Principles and satisfy their requirements.

A design is deemed to meet the requirements of the Construction Product Directive (and its successors) if the assumptions on which the *Eurocodes* are based are satisfied. The assumptions are that the structures are adequately maintained and used in conjunction with the design assumptions; the construction materials and products are as specified in *ENs 1990-9*; the choice of structural system is made by personnel with appropriate qualifications and experience; and the execution is performed by personnel with appropriate skill and experience and is adequately supervised and quality controlled.

Eurocode 7 includes several requirements regarding management of complexity:

- *"In order to establish minimum requirements for the extent and content of geotechnical investigations, calculations and construction control checks, the complexity of each geotechnical design shall be identified together with associated risks" EN 1997-1 §2.1(8)P*
- *"… a distinction shall be made between light and simple structures and small earthworks for which ... the minimum requirements will be satisfied by experience and qualitative geotechnical investigations, with negligible risk; [and] other geotechnical structures" EN 1997-1 §2.1(8)P* continued
- *"For structures and earthworks of low geotechnical complexity and risk, such as defined above, simplified design procedures may be applied" EN 1997-1 §2.1(9)*

To assist the application of these Principles, *Eurocode 7* introduces Geotechnical Categories:

Table 2.2.1 Geotechnical categories

GC	Includes…	Design requirements	Design procedure
1	Small and relatively simple structures… with negligible risk	Negligible risk of instability or ground movements Ground conditions known to be straightforward No excavation below water table (or such excavation is straightforward)	Routine design & construction methods
2	Conventional types of structure & foundation with no exceptional risk or difficult soil or loading conditions	Quantitative geotechnical data & analysis to ensure fundamental requirements are satisfied	Routine field & lab testing Routine design & execution
3	Structures or parts of structures not covered above	Include alternative provisions and rules to those in *Eurocode 7*	

Examples of structures in Geotechnical Category 2 include: spread, raft, and pile foundations; walls and other structures retaining or supporting soil or water; excavations; bridge piers and abutments; embankments and earthworks; ground anchors and other tie-back systems; and tunnels in hard, non-fractured rock and not subjected to special water tightness or other requirements.

Examples of structures in Geotechnical Category 3 include: very large or unusual structures; structures involving abnormal risks, or unusual or exceptionally difficult ground or loading conditions;

structures in highly seismic areas; and structures in areas of probable site instability or persistent ground movements that require separate investigation or special measures.

2.3. Actions and design situations

Design situations are "sets of physical conditions representing the real conditions occurring during a certain time interval for which the design will demonstrate that relevant limit states are not exceeded" *(EN 1990 §1.5.2.2)*. They are classified as follows:

- Persistent (conditions of normal use)

  Period = same order as design working life (DWL) of structure
- Transient (temporary conditions, e.g. execution or repair

  Period << DWL and high probability of occurrence
- Accidental (exceptional conditions)

  e.g. fire, explosion, impact, local failure
- Seismic (exceptional conditions during earthquake)Overview

Less severe values of the partial factors recommended in *Annex A of EN 1997-1* may be used for temporary structures or transient design situations when the likely consequences justify it *(EN 1997-1 §2.4.7.1 (5))*.

Table 2.3.1 summarizes the classification of actions according to *EN 1990*:

Table 2.3.1 Classification of actions

Action		**Duration**	**Variation with time**	**Examples**
Permanent	G	Likely to act throughout reference period	Negligible or monotonic	Self-weight of structures, fixed equipment and road-surfacing; indirect actions[§] caused by shrinkage and uneven settlements
Variable	Q		Neither negligible nor monotonic	Imposed loads on building floors, beams and roofs; wind*; snow*
Accidental	A	Usually short	Significant magnitude	Explosions, vehicle impact*, seismic* (AE, due to earthquake ground motions)
*may be variable or accidental depending on statistical distribution [§] may be permanent or variable				

2.4. Limit states

Eurocode 7 states that *"For each geotechnical design situation it shall be verified that no relevant limit state … is exceeded" (EN 1997-1 §2.1(1)P)*. Limit states (which include ultimate limit states GEO, STR, EQU, UPL, and HYD and serviceability limit states) should be verified by one or a combination of:

- Use of calculations
- Adoption of prescriptive measures
- Experimental models and load tests
- An observational method

Design by calculation is illustrated in Figure 2.4.1 (after *Bond & Harris, 2008*).

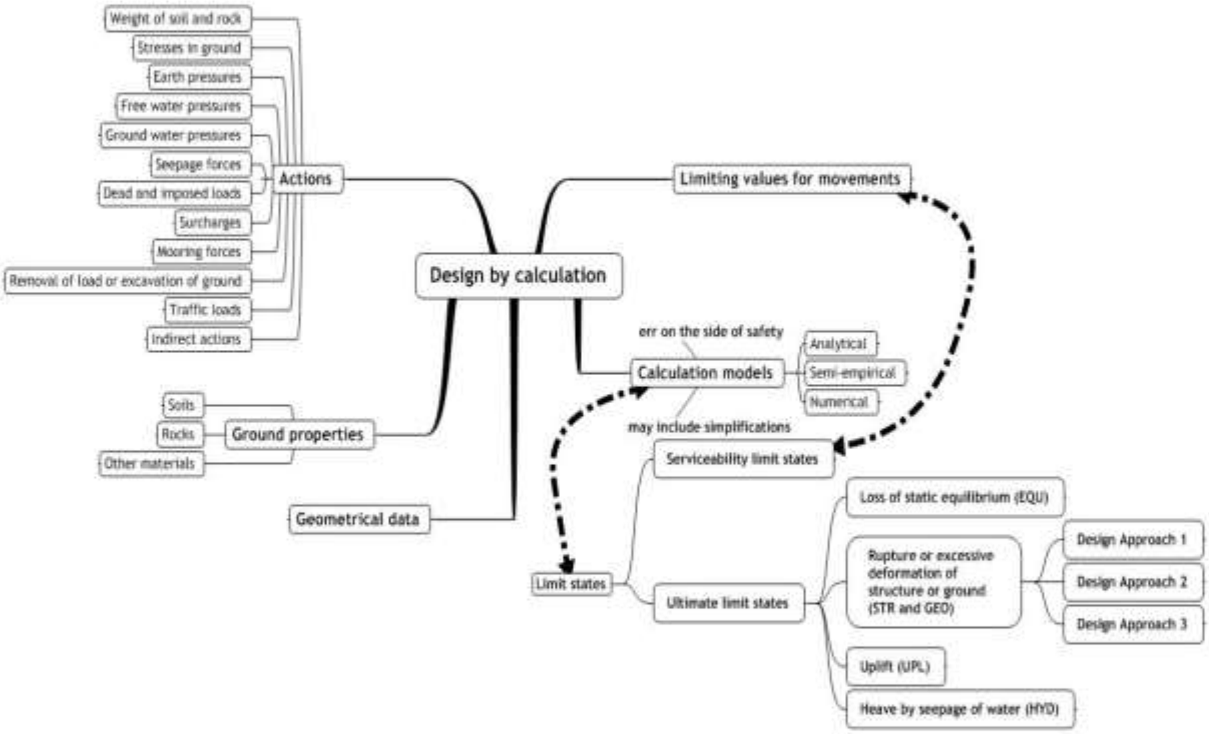

Fig. 2.4.1 Design by calculation (after *Bond and Harris, 2008*)

Verification of the ultimate limit state of strength is expressed in *Eurocode 7* by:

$$E_d \leq R_d$$

where E_d = the design effect of actions and R_d = the corresponding design resistance (*see EN 1997-1 exp. 2.5*).

This expression applies to the following ultimate limit states:

- GEO: 'Failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance'
- STR: 'Internal failure or excessive deformation of the structure or structural elements … in which the strength of structural materials is significant in providing resistance"

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

$$\Lambda_{GEO} = \frac{E_d}{R_d} \leq 100\%$$

where Λ = 'degree of utilization'.

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

$$ODF = \frac{R_d}{E_d} \geq 1{,}0$$

where ODF = 'overdesign factor' (= 1 / Λ).

According to *Eurocode 7*, *"The manner in which equations [for GEO/STR] are applied shall be determined using one of three Design Approaches. Design Approaches apply ONLY to STR and GEO limit states. Each nation can choose which one (or more) to allow" (EN 1997-1 §2.4.7.3.4.1(1)P)*. In Germany, the Design Approaches only apply to GEO, with STR remaining within the domain of structural engineers.

In Design Approach 1, factors are applied to actions alone (in Combination 1) and mainly to material factors (in Combination 2). In Design Approach 2, factors are applied to actions (or effects of actions) and to resistances simultaneously. In Design Approach 3, factors are applied to structural actions (but not to geotechnical actions) and to material properties simultaneously. Further information about the differences between the Design Approaches is given by *Bond and Harris (2008)* and *Frank et al. (2007).*

Figures 2.4.2 and 2.4.3 (after *Bond, 2013*) summarize the choices of Design Approach that have been made for the design of shallow foundations and slopes, by different countries in Europe.

Fig. 2.4.2 National choice of Design Approach for shallow foundations (after Bond, 2013)

Fig. 2.4.3 National choice of Design Approach for slopes (after *Bond, 2013*)

Verification of static equilibrium is expressed in *Eurocode 7* by:

$$E_{dst;d} \leq E_{stb;d} + T_d$$

where $E_{dst;d}$ = the design value of the effect of destabilising actions; $E_{stb;d}$ = the design value of the effect of stabilizing actions; and T_d = the design value of any stabilizing shear resistance of the ground or of structural elements.

This expression applies to the following ultimate limit state:

- EQU: *'Loss of equilibrium of the structure or the ground considered as a rigid body, in which the strengths of structural materials and the ground are insignificant in providing resistance'*

Verification of serviceability is expressed in *Eurocode 7* by:

$$E_d \leq C_d$$

where E_d = design effect of actions (e.g. displacement, distortion) and C_d = design constraint (i.e. limiting value of design effect). According to *EN 1990, "partial factors … should normally be taken equal to 1,0"*.

2.5. Supervision, monitoring, and maintenance

To ensure the safety and quality of a structure, the following shall be undertaken, as appropriate: the construction processes and workmanship shall be supervised; the performance of the structure shall be monitored during and after construction; the structure shall be adequately maintained *(EN 1997-1 §4.1(1)P).*

2.6. Summary of key points

Design requirements:

- Complexity of design
- Geotechnical Categories (GC1-3)

Geotechnical design by...

- Prescriptive measures
- Calculation
- Observation or testing

Limit states

- Overall stability
- Ultimate limit states (GEO, STR, EQU, UPL, and HYD)
- Serviceability limit states

2.7. Worked example – combinations of actions

A worked example illustrating the way in which actions should be combined according to *EN 1990*, in a way that is suitable for geotechnical design of foundations, is included in the Annex to this report.

References

Bond, A. J. (2013). Implementation and evolution of Eurocode 7, Modern Geotechnical Design Codes of Practice, Arnold et al. (eds), Amsterdam: IOS Press, pp3-14.

Bond, A. J., and Harris, A. J. (2008). Decoding Eurocode 7, London: Taylor & Francis, 598pp.

EN 1997-1: 2004. Eurocode 7: Geotechnical design. Part 1: General rules. CEN.

Frank, R., Bauduin, C., Kavvadas, M., Krebs Ovesen, N., Orr, T., and Schuppener, B. (2004). Designers' guide to EN 1997-1: Eurocode 7: Geotechnical design — General rules, London: Thomas Telford.

Frank, R., Schuppener, B., Vogt, N., Weißenbach, A. (2007), Verification of ultimate limit states in geotechnical design in France and Germany, Revue européenne de génie civil, vol. 11, n° 5, Mai 2007, p. 621-641, France.

CHAPTER 3

Shallow foundations

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3.1. Scope

This chapter deals with the geotechnical design of spread foundations according to *Eurocode 7*. *Section 6 of Eurocode 7 Part 1* presents the different aspects to be considered for designing shallow foundations of buildings, bridges, walls, isolated columns etc. It applies to pad, strip, and raft foundations and some provisions may be applied to deep foundations, such as caissons.

Section 6 is organized in the following subsections:

§6.1. General

§6.2. Limit states

§6.3. Actions and design situations

§6.4. Design and construction considerations

§6.5. Ultimate limit state design

§6.6. Serviceability limit state design

§6.7. Foundations on rock; additional design considerations

§6.8. Structural design of foundations

§6.9. Preparation of the subsoil

Moreover, *Eurocode 7* has the following five informative annexes that are specifically referred to in *Section 6* in relation to shallow foundation design that give useful information and guidance about bearing resistance calculation, limiting values of structural deformations for serviceability of constructions and foundation movements:

D. A sample analytical method for bearing resistance calculation;

E. A sample semi-empirical method for bearing resistance estimation;

F. Sample methods for settlement evaluation;

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

H. Limiting values of structural deformation and foundation movement.

3.2. Design situations and limit states

3.2.1. INTRODUCTION

In *§6.2 EC7* most common limit states for spread foundations are listed (here schematically represented in Fig.3.2.1):

- loss of overall stability
- bearing resistance failure
- failure by sliding
- combined failure in the ground and in the structure
- structural failure due to foundation movement
- excessive settlements
- excessive heave due to swelling, frost and other causes

- unacceptable vibrations

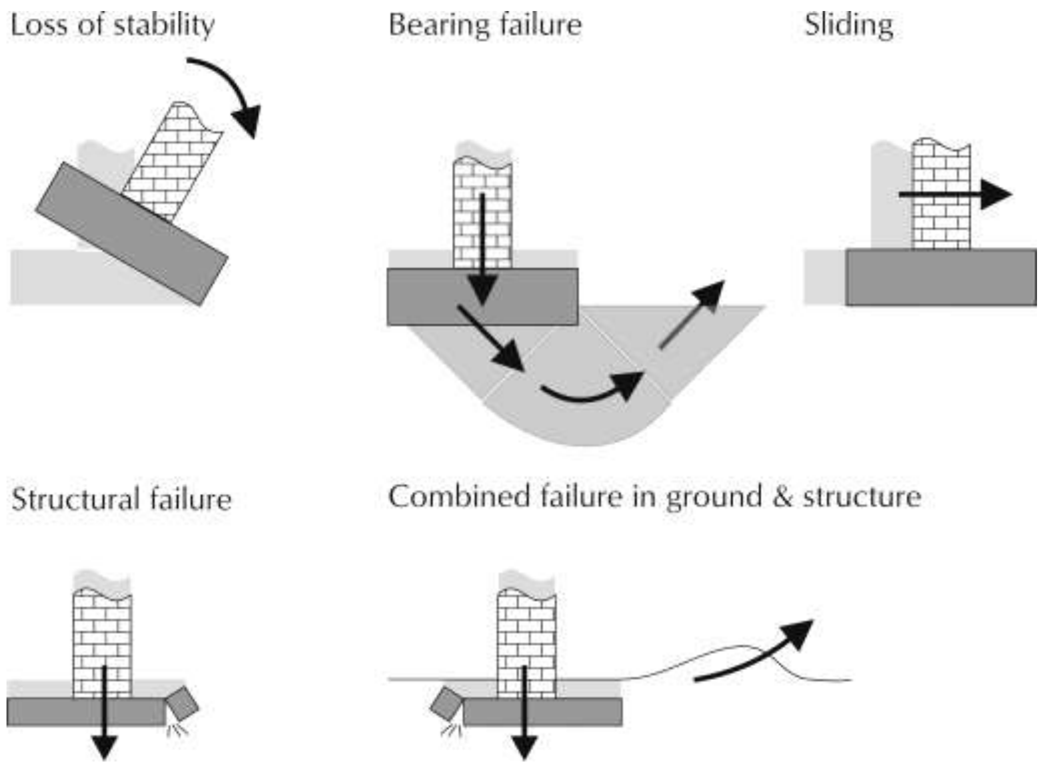

Fig.3.2.1 Sketches of limit states (from Bond & Harris, 2008)

Some of above are ultimate limit states and some are serviceability limit states. The design philosophy of *Eurocode 7* is that both types of limit states need to be considered as possibly critical in selecting the dimensions of the foundations.

Actions have to be selected in accordance to the relevant *Eurocodes*, particularly, *EN1990* and *EN1991*. Special consideration is given to design situations to be considered: the actions, their combinations and load cases; overall stability; the disposition and classification of the various soils and elements of construction; dipping bedding planes; underground structures; interbedded hard and soft strata; faults, joints and fissures; possible instability of rock blocks; solution cavities; and the environment within which the design is set that includes earthquakes, subsidence, interference with existing constructions).

Actions include: weight of soil and water; earth pressures; free water pressure, wave pressure; seepage forces; dead and imposed loads from structures; surcharges; mooring forces; removal of load and excavation of ground; and traffic loads.

Based on common experience, the most important design and construction considerations to choose the foundation depth are given in *§6.4 of EN1997-1.* These are schematically represented in Fig.3.2.2.

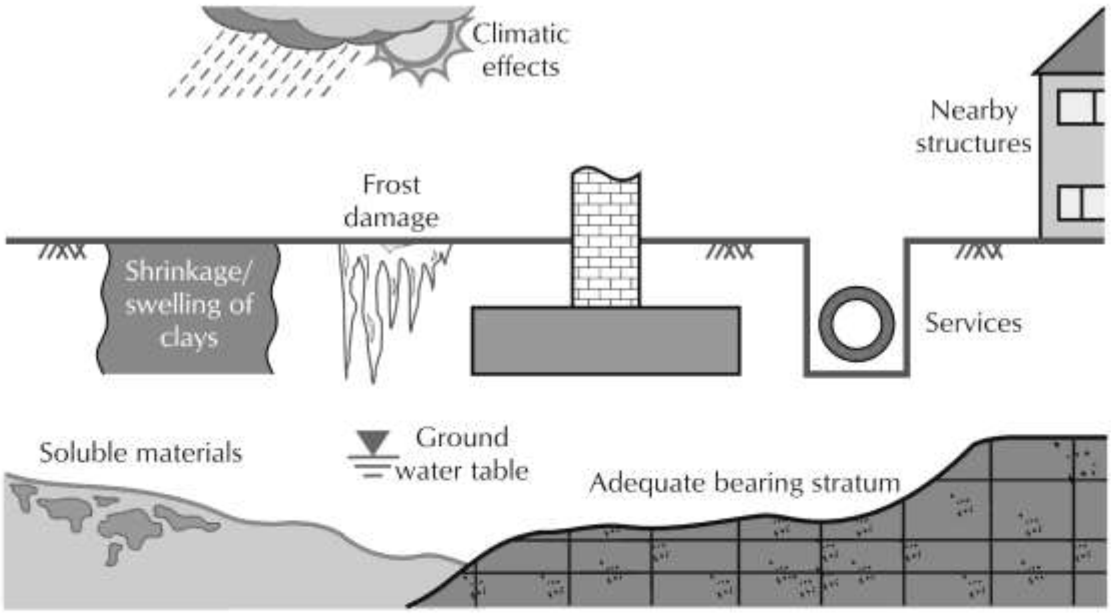

Fig.3.2.2 Selection of the depth of a shallow foundation (from *Bond & Harris, 2008*)

It is clear that most of the foundation problems can be anticipated and avoided if the depth of a foundation is appropriately selected. One of the design methods given in **Error! Reference source not found.** shall be used to analyze the limit states for shallow foundations.

Table 3.2.1 Methods to analyze limit states (after *Bond & Harris, 2008*)

Method	Description	Constraints
Direct	Carry out separate analyses for each limit state, both ultimate (ULS) and serviceability (SLS)	(ULS) Model envisaged failure mechanism
		(SLS) Use a serviceability calculation
Indirect	Use comparable experience with results of field & laboratory measurements & observations	Choose SLS loads to satisfy requirements of all limit states
Prescriptive	Use conventional & conservative design rules and specify control of construction	Use presumed bearing resistance

3.2.2. DESIGN INEQUALITIES

Ultimate limit state designs to *Eurocode 7* require the application of partial factors to actions (or effect of actions) to obtain E_d and to geotechnical parameters or resistances to obtain R_d that are used in the following general inequality:

$$E_d \leq R_d$$

For serviceability limit state designs the inequality to be checked is:

$$E_d \leq C_d$$

where E_d is the design value of the effect of the actions, for example the settlement, calculated using partial factors of unity, and C_d is the limiting value of the effect of an action, for example the limiting value of the structural deformation or foundation movement as values given in *Annex H, EN 1997-1*.

Representation of the design action:

$$E_d = \sum_{j\geq 1} \gamma_{Gj} G_{kj} + \gamma_{Q1} Q_{k1} + \sum_{i>1} \gamma_{Qi} \psi_{0i} Q_{ki}$$

where:

G_{kj} = characteristic permanent loads

Q_{ki} = characteristic variable loads

ψ_{0i} = factors for combination value of variable loads

γ_{Gj} = partial factors for permanent loads

γ_{Qi} = partial factors for variable loads

Representation of resistance:

$$R_d = R\left\{F_{rep}\ ; X_k / \gamma_M ; a_d\right\}\ / \gamma_R$$

where:

Fr_{ep} = representative value of actions

X_k = characteristic value of geotechnical parameters

a_d = design value of geometrical data

γ_M = partial factors for geotechnical parameters

γ_R = partial factors for resistances

3.2.3. OVERALL STABILITY OF SPREAD FOUNDATIONS

Overall stability (ULS) check has to be performed for foundations on sloping ground, natural slopes or embankments and for foundations near excavations, retaining walls or buried structures, canals, etc.

For such situations, it shall be demonstrated using the principles described in the relevant *Section 11, EN1997-1* that a stability failure of the ground mass containing the foundation is sufficiently improbable.

With DA-1 and DA-3 the stability check is carried out using (almost) the same partial factors. DA-2 is slightly more conservative if ϕ'_k is not too large.

When checking overall stability using DA2, the partial factors on resistances in Table 3.2.2 *(Table A.14, EC7, Annex A)* need to be considered.

Table 3.2.2 Partial resistance factors (γ_R) for slopes and overall stability (*Table A.14, EC7, Annex A*)

Resistance	Symbol	Set		
		R1	***R2***	***R3***
Earth resistance	$\gamma_{R,e}$	1,0	1,1	1,0

3.3. Ultimate Limit State verifications by Direct Method

ULS verifications are carried out with the three possible Design Approaches:

- DA1 – Combination 1: A1 + M1 + R1
- DA1 – Combination 2: A2 + M2 + R1
- DA2: A1+ M1+R2
- DA3: (A1 or A2)* + M2 + R3

*A1 is for structural actions and A2 is for geotechnical actions

When applicable, drained and undrained conditions are analyzed. The sets of partial factors summarized in Tables 3.3.1, 3.3.2. and 3.3.3 are given in *EC7* for the different Design-Approaches:

Table 3.3.1 Partial factors on actions (γ_F) or the effects of actions (γ_E)

Action		Symbol	Set	
			A1	**A2**
Permanent	Unfavourable	γ_G	1,35	1,0
	Favourable		1,0	1,0
Variable	Unfavourable	γ_Q	1,5	1,3
	Favourable		0	0

Table 3.3.2 Partial resistance factors for spread foundations (γ_R)

Resistance	Symbol	Set		
		R1	**R2**	**R3**
Bearing	γ_{Rv}	1,0	1,4	1,0
Sliding	γ_{Rh}	1,0	1,1	1,0

Table 3.3.3 Partial factors for soil parameters (γ_M)

Soil parameter	Symbol	Value	
		M1	**M2**
Shearing resistance	γ_{φ} [1]	1,0	1,25
Effective cohesion	γ_c	1,0	1,25
Undrained strength	γ_{cu}	1,0	1,4
Unconfined strength	γ_{qu}	1,0	1,4
Effective cohesion	γ_c	1,0	1,4
Weight density	γ_{γ}	1,0	1,0
[1] This factor is applied to tan φ’			

Whereas there is a general consensus on how Design Approaches 1 and 3 are applied for ULS verifications, there are two ways of performing verifications according to Design Approach 2: partial factors are either applied to the actions at the source, or to the effect of the actions, at the end of the calculation.

In the design approach referred to as DA-2, the partial factors are applied to the characteristic actions right at the start of the calculation and design values are then used. In the design approach referred to as DA-2*, the entire calculation is performed with characteristic values and the partial factors are introduced only at the end when the ultimate limit state condition is checked.

The resulting designs can be very different since for DA-2 the effective foundation breadth *B*' and length *L*' (that is *B*' = *B*-2*e* and *L*' = *L*-2*e*) are governed by what is called the “design value of eccentricity”, e_d whereas for DA-2* the “characteristic value of eccentricity”, e_k is used. The following **Error! Reference source not found.**.1 clarifies why $e_d >> e_k$.

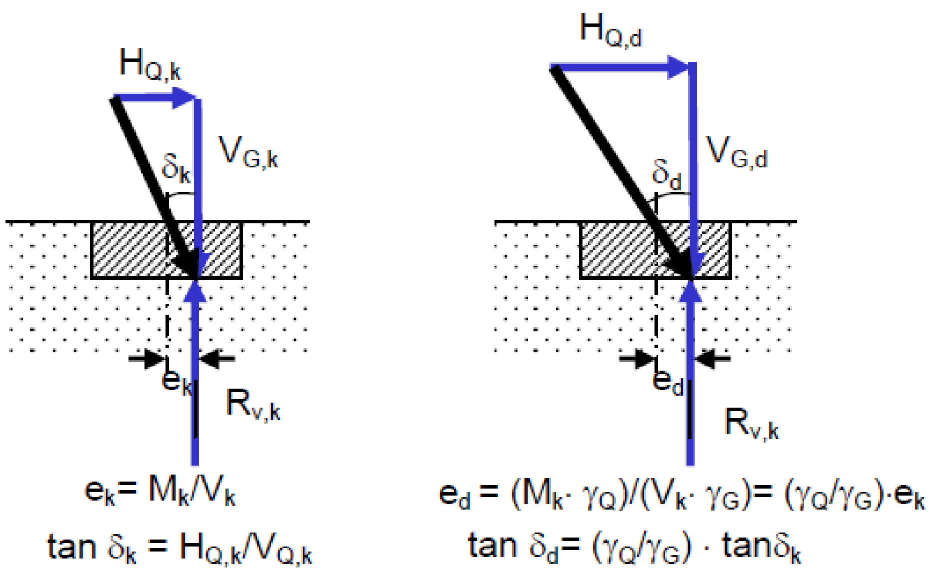

Fig.3.3.1 Characteristic (left) and design (right) values of eccentricity

Design approach DA 2* gives the most economic (or less conservative) design.

3.3.1. BEARING RESISTANCE

The check on the bearing resistance of a spread foundation is a re-statement of the general inequality $E_d \leq R_d$. That is:

$$V_d \leq R_d$$

where V_d is the design action. V_d should include the self-weight of the foundation and any backfill on it.

The design action, V_d includes both variable and permanent vertical loads; this latter includes all the actions shown in **Error! Reference source not found.**3.2 which are:

a) Supported permanent load
b) Weight of foundation
c) Weight of the backfill
d) Loads from water pressures
e) Uplift

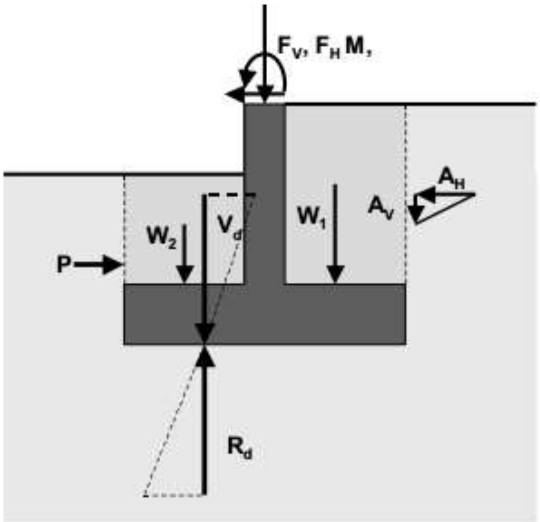

Fig.3.3.2 Actions on a spread foundation

The suggested equations for bearing capacity are given in *Annex D, EN 1997-1*:

For DRAINED CONDITIONS:

$$R/A' = c'N_c b_c s_c i_c + q'N_q b_q s_q i_q + 1/2\gamma' B' N_\gamma b_\gamma s_\gamma i_\gamma$$

For UNDRAINED CONDITIONS:

$$R/A' = (2+\pi) c_u s_c i_c + q$$

Where:

$$N_q = e^{\pi \cdot \tan\varphi'} \tan^2(45° + \varphi'/2)$$

$$N_c = (N_q - 1)\cot\varphi'$$

$$N_g = 2(N_q - 1)\tan\varphi'$$

are dimensionless factors for bearing resistance and

b_c, b_q, b_γ	the inclination of the foundation base
s_c, s_q, s_γ	the shape of foundation
i_c, i_q, i_γ	are the factors for the inclination of the load
$A' = B' \cdot L'$	effective foundation area (reduced area with load acting at its centre)

Eurocode 7 in §6.5.4 requires that special precautions be taken when the eccentricity of the loading exceeds 1/3 of the width of a rectangular; i.e. when e_B exceeds $B/3$ or e_L exceeds $L/3$. It should be noted that this is not the middle third rule, which requires the eccentricity not to exceed $B/6$ or $L/6$ so as to avoid a gap forming between the foundation and the soil if the soil behaves as a purely elastic material. A special precaution to be taken in the case of large load eccentricities is the inclusion of tolerances of up to 0,10 m in the dimensions of the foundation.

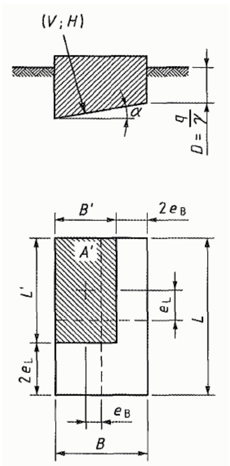

Fig.3.3.3 Definitions of eccentricities e_B and e_L and effective area *A'*

The factors in the bearing capacity equations are calculated as follows:

For DRAINED CONDITIONS:

i_q= (1 – 0,70 H / (V + A' c' cotgϕ'))^m

m=m_B= [2+(B'/L')]/[1+(B'/L')]

m=m_L= [2+(L'/B')]/[1+(L'/B')]

m=m_q =m_L cos$^2 q$ +m_B sin$^2 q$

i_c= (i_q · N_q - 1) / (N_q - 1)

i_g= (1 - H / (V + A' c' cotgϕ'))^3

s_q= 1 + (B' / L') · sinϕ' (rectangular shape)

s_q= 1 + sinϕ' (square or circular shape)

s_c= (s_q N_q - 1) / (N_q - 1)

s_g= 1- 0,30 (B' / L') (rectangular shape)

s_g= 0,70 (square or circular shape)

b_c = b_q - (1-b_q) / (N_c tanϕ')

b_q = $b\gamma$=(1-α tanϕ')2

for UNDRAINED CONDITIONS:

b_c=1–2α /(π+2)

α is the inclination of the foundation base to the horizontal

s_c = 1+ 0,2 (B'/L') (rectangular shape)

$s_c = 1{,}2$ (square or circular shape)

$$i_c = 0{,}5\left(1+\sqrt{(1-H/(A'c_u))}\right)$$

NOTE: For drained conditions, water pressures must be included as actions. The question then arises, which partial factors should be applied to the weight of a submerged structure? Since the water pressure acts to reduce the value of V_d, it may be considered as favourable, while the total weight is unfavourable. Physically however, the soil has to sustain the submerged weight. For the design of structural members, water pressure may be unfavourable.

NOTE: As the eccentricity influences the effective base dimensions it may be necessary to analyze different load combinations, by considering the permanent vertical load as both favourable and unfavourable and by changing the leading variable load.

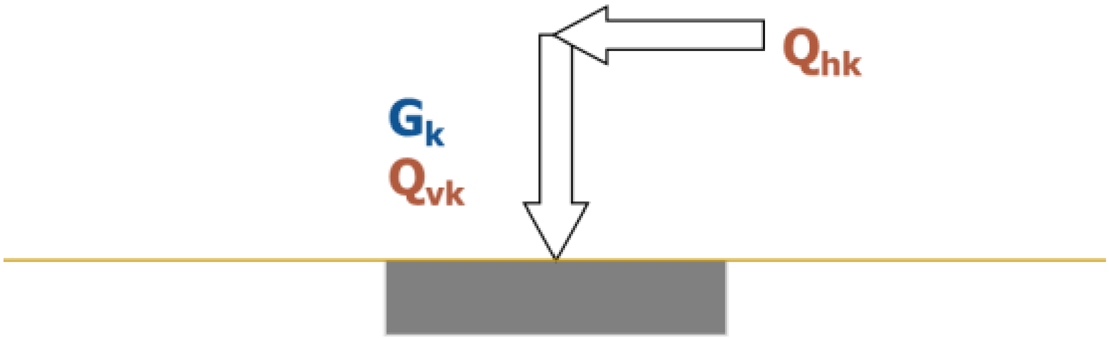

Fig.3.3.4 Vertical and horizontal loads

$V_{unfavorable}$ · $H_{unfavorable}$

$V_d = \gamma_G G_k + \gamma_{Qv} \psi_0 Q_{vk}$; $H_d = \gamma_{Qh} Q_{hk}$

$\gamma_G = 1{,}35$; $\gamma_{Qv} = 1{,}5$; $\gamma_{Qh} = 1{,}5$

$V_{unfavourable}$ · $H_{unfavourable}$

$V_d = \gamma_G G_k + \gamma_{Qv} Q_{vk}$; $H_d = \gamma_{Qh} \psi_0 Q_{hk}$

$\gamma_G = 1{,}35$; $\gamma_{Qv} = 1{,}5$; $\gamma_{Qh} = 1{,}5$

$V_{favourable}$ · $H_{unfavourable}$

$Vd = \gamma_G\ Gk + \gamma Qv\ Qvk$

$Hd = \gamma Qh\ Qhk$

$\gamma_G = 1{,}00$; $\gamma_{Qv} = 0{,}0$; $\gamma_{Qh} = 1{,}5$

3.3.2. SLIDING RESISTANCE

Following the scheme of **Error! Reference source not found.**5, the ULS check for sliding is:

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

where $R_{p,d}$ is the contribution to resistance due to passive thrust that may develop in front of the foundation.

For drained conditions the design shear resistance, R_d, shall be calculated either by factoring the ground properties or the ground resistance as follows:

$$R_d = V'_d \tan \delta_d \text{ or } R_d = (V'_d \tan \delta_k) / \gamma_{Rh}$$

Normally it is assumed that the soil at the interface with concrete is remolded. So the design friction angle δ_d may be assumed equal to the design value of the effective critical state angle of shearing resistance, $\phi'_{cv,d}$, for cast-in-situ concrete foundations and equal to 2/3 $\phi'_{cv,d}$ for smooth precast foundations.

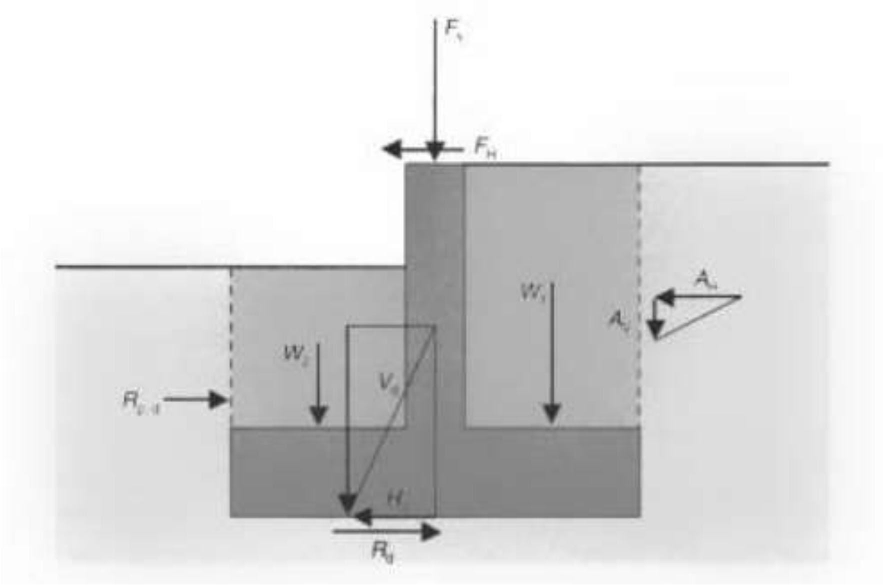

Fig.3.3.5 Actions to be included for checking against sliding

Any effective cohesion *c*' should be neglected.

For undrained conditions, the design shearing resistance, R_d, shall be calculated either by factoring the ground properties or the ground resistance as follows:

$$R_d = A_c\, c_{u,d} \quad \text{or} \quad R_d = (A_c\, c_{u,k})/\gamma_{Rh}$$

NOTE: The maximum available sliding resistance is likely to be mobilized with relatively little movement (and may reduce as large movements take place) whereas the mobilization of the passive earth pressure resistance may require significant movement. Hence it could be difficult to mobilize the maximum value of both R_d and $R_{p,d}$ simultaneously. Considering also the remolding effects of excavation, erosion and shrinkage the passive resistance should be neglected.

In undrained conditions. in some circumstances, the vertical load is insufficient to produce full contact between soil and foundation: the design resistance should be limited to 0,4V_d.

3.4. SLS check the performance of the foundation

When design is carried out by direct methods, settlement calculations are required to check SLS. For soft clays settlement calculations shall always be carried out. For spread foundations on stiff and firm clays in Geotechnical Categories 2 and 3, vertical displacement should usually be calculated.

The following three components of settlement have to be considered:

- s_0: immediate settlement; for fully-saturated soil due to shear deformation at constant volume and for partially-saturated soil due to both shear deformation and volume reduction;
- s_1: settlement caused by consolidation;
- s_2: settlement caused by creep.

In verifications of serviceability limit states, partial factors are normally set to 1.

The combination factors, ψ to be used are those for characteristic, frequent or quasi permanent combinations, which are the ψ_2 values from *EN 1990*.

3.4.1. SOME USEFUL DEFINITIONS

Annex H, EN 1997-1, gives definitions of the relevant quantities for checking the serviceability limit state of spread foundations; settlement, relative settlement and angular distortion are the most important (Fig.3.3.6).

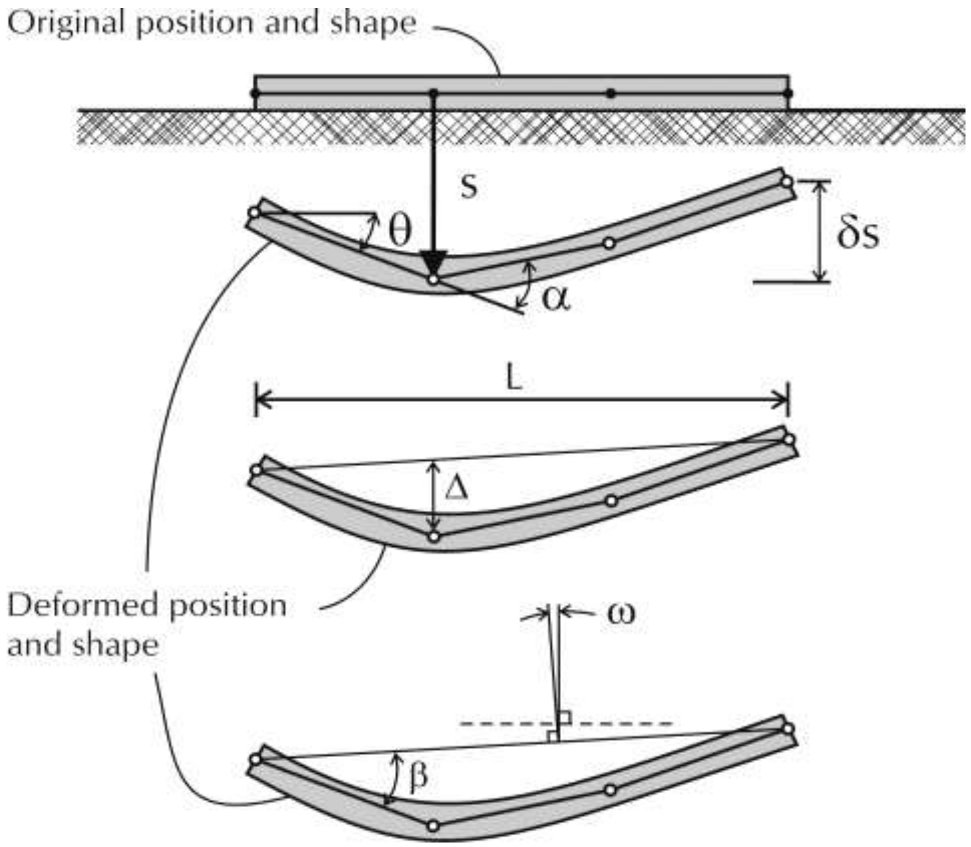

Fig.3.3.6 Definitions for checking SLS from *Annex H of EN 1997-1* (after *Bond and Harris, 2008*)

Spread foundations and superstructures may suffer due to differential settlements and distortions. The maximum acceptable relative rotations for open framed structures, frames and load bearing or continuous brick walls are unlikely to be the same but range from about 1/2000 to about 1/300 to prevent the occurrence of a serviceability limit state in the structure. A maximum relative rotation of 1/500 is acceptable for many structures. The relative rotation likely to cause an ultimate limit state is about 1/150.

For normal structures with isolated foundations, total settlements up to 50 mm are often acceptable. Larger settlements may be acceptable provided the relative rotations remain within acceptable limits and provided the total settlements do not cause problems with the services entering the structure, or cause tilting etc. Acceptable limits for structural deformations are given in the following Table 3..

Table 3.4.1 Limiting deformations and damages of constructions

Type of structure	Type of damage/concern	Criterion	Limiting value(s)
Framed buildings and reinforced load bearing walls	Structural damage	Angular distortion	1/150-1/250
	Cracking in walls and partitions	Angular distortion	1/500 (1/1000-1/1400) for end bays
	Visual appearance	Tilt	1/300
	Connection to services	Total settlement	50-75mm (sands) 75-135mm (clays)
Tall buildings	Operation of lifts & elevators	Tilt after lift installation	1/1200-1/2000
Structures with unreinforced load bearing walls	Cracking by sagging	Deflection ratio	1/2500 (L/H=1) 1/1250 (L/H=5)
	Cracking by hogging	Deflection ratio	1/5000 (L/H=1) 1/2500 (L/H=5)
Bridges - general	Ride quality	Total settlement	100mm
	Structural distress	Total settlement	63mm
	Function	Horizontal movement	38mm
Bridges – multiple span	Structural damage	Angular distortion	1/250
Bridges – single span	Structural damage	Angular distortion	1/200

3.4.2. METHODS FOR SLS CHECK

To perform the SLS check as required by *EC7* different methods are used in practice.

- Deterministic: solve the soil-foundation interaction problem and deduce stresses and deformations of the foundation (by using numerical methods with FEM or subgrade reaction models)
- Semi-empirical: calculate maximum settlement (w_{max})
    a) Evaluation of differential settlement δ and of angular distortion β = f(w_{max}, foundation, ground) using empirical plots (e.g. in Fig.3.3.7, *Bjerrum, 1963*)
    b) Admissibility check for δ and β = f(structure, type of damage)

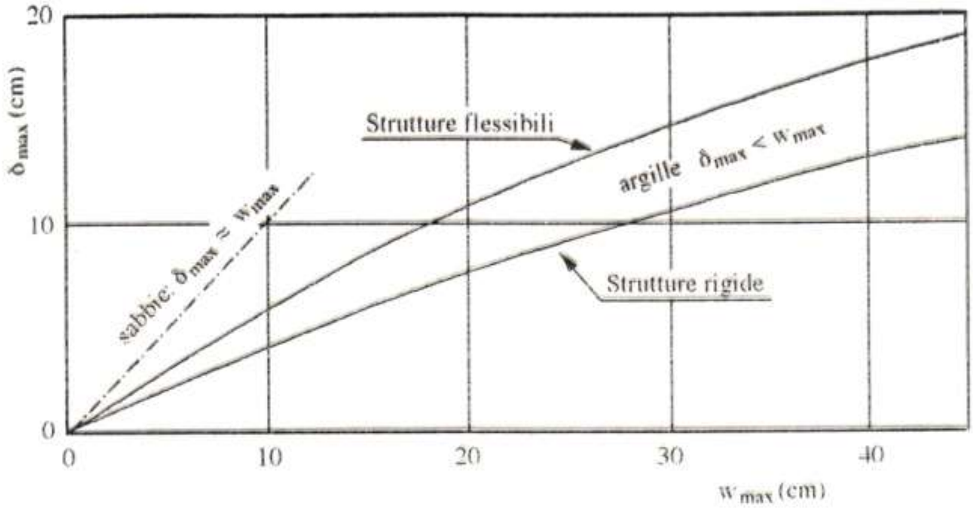

Fig.3.3.7 Empirical correlation between maximum differential settlement δ and maximum settlement (from *Bjerrum, 1963*)[1]

The total settlement of a foundation on cohesive or non-cohesive soil may be evaluated using elasticity theory through an equation of the form:

$w = pbf / E_m$

where:

E_m is the design value of the modulus of elasticity (operative modulus)

f is an influence settlement coefficient

p is the (average) pressure at the base of the foundation

b is the foundation breadth.

To calculate the consolidation settlement, a confined one-dimensional deformation model may be used.

3.4.3. SLS – ULS CHECK BY USING INDIRECT METHODS (SEE TABLE 3.4.1)

Terzaghi & Peck charts (1967) offer an example of using indirect methods for foundation design. Charts give allowable bearing capacity for granular soils and a shallow foundation as a function of the embedment ratio (*D/B*), foundation breadth *B* and corrected blow count *N* from SPT tests. The allowable pressure values imply a settlement less than 25 mm.

The graphs of allowable bearing pressures against foundation width, *B* in Fig.3., which increase linearly for small *B* values and then become constant above a certain *B* value, show that the controlling limit state for pad foundations changes from bearing failure (ULS) for small *B* values to excessive settlement (SLS) for large *B* values.

[1] key: “strutture flessibili” = flexible structures; “strutture rigide” = rigid structures

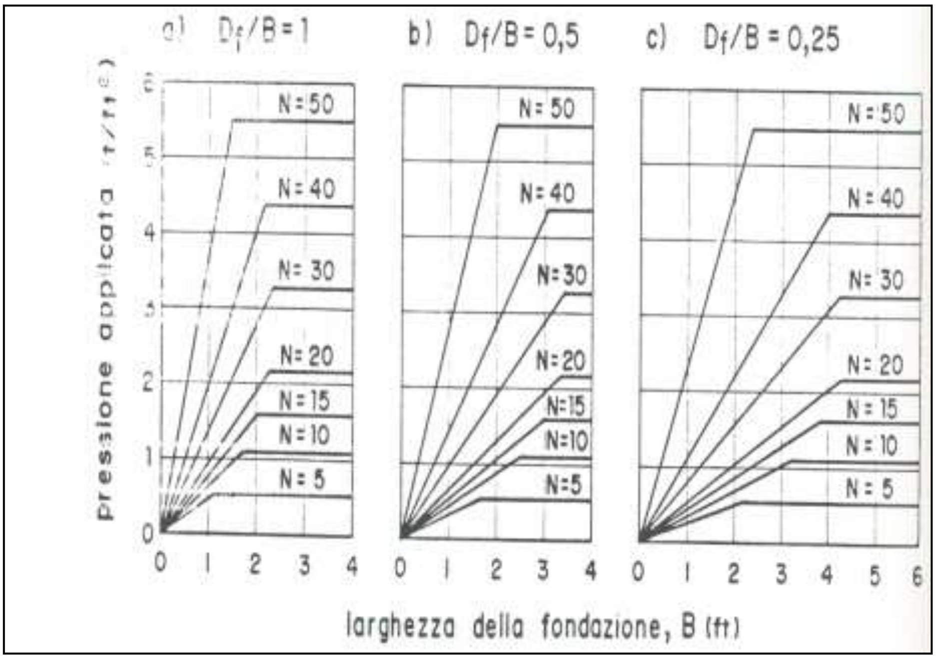

Fig.3.3.8 Allowable bearing pressures from *Terzaghi & Peck (1967)* for large *B*[2]

The reason for this is because, when *B* becomes larger, as shown in Fig.3.3.9 , the allowable bearing pressure must reduce to keep the settlements below the assumed maximum value of 25 mm.

The *Terzaghi & Peck design charts* demonstrate how in practice foundation design can be governed either by ULS or by SLS limit states. A sound foundation design shall always be based on both checks; the calibration of partial factors in *EC7* is such that the ULS and SLS are appropriately balanced for normal design situations.

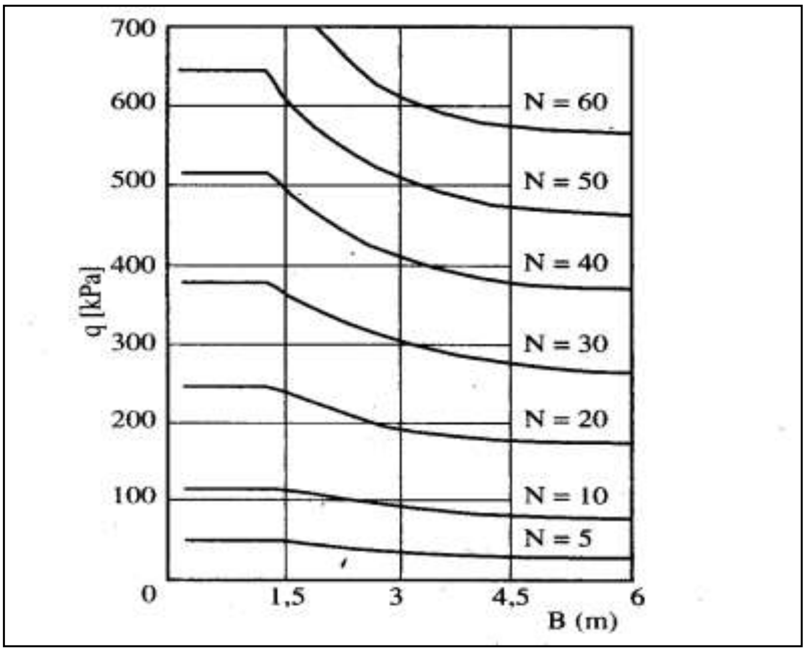

Fig.3.3.9 Allowable bearing pressures from *Terzaghi & Peck (1967)* for large *B*

[2] vertical axis: "applied vertical pressure (t/ft^2)"; horizontal axis: "breadth of the foundation, B (ft)"

3.5. Summary of key points

- *Eurocode 7* does not provide new methods for the designing spread foundations
- The design of spread foundations always implies the execution of a sound geotechnical investigation, the selection of the most appropriate geotechnical model and characteristic values of geotechnical parameters
- The first step in the design of a spread foundation is to fix the required performance of the construction and to select foundation geometry and embedment
- Ultimate and serviceability limit state checks can be carried out using commonly recognized procedures
- *Eurocode 7* provides recommended values of partial factors for ULS and SLS verifications. The particular values to be used in a country are given in that country's National Annex
- *Eurocode 7* suggests acceptable limits for C_d in SLS checks

3.6. Worked example – Design of a strip foundation

A worked example illustrating how the design of a strip foundation of a building can be carried out is included in the Annex to this report.

References

Bjerrum, L. (1963), "Allowable Settlement of Structures," Proc., European Conf. on Soil Mech. and Found. Engr., Weisbaden, Germany, Vol. 3, pp. 135-137.

Bond, A. J., and Harris, A. J. (2008). Decoding Eurocode 7, London: Taylor & Francis, 618pp.

EN 1997-1: 2004. Eurocode 7: Geotechnical design. Part 1: General rules. CEN.

Frank, R., Bauduin, C., Kavvadas, M., Krebs Ovesen, N., Orr, T., and Schuppener, B. (2004). Designers' guide to EN 1997-1: Eurocode 7: Geotechnical design — General rules, London: Thomas Telford.

Terzaghi, K & Peck, R B (1967) Soil mechanics in engineering practice 2nd Ed. Pub. John Wiley& Sons

CHAPTER 4

Retaining structures I

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4.1. Scope

This Chapter concentrates on the design of gravity walls, which are walls of stone or plain or reinforced concrete having a base with or without a heel, ledge, or buttress. A key feature of these walls is that the weight of the wall itself plays a significant role in the support of the retained material. Examples include: concrete gravity walls; spread footing reinforced concrete walls; and buttress walls.

Embedded walls are discussed in Chapter 7.

Composite retaining structures are walls composed of elements of the above two types (gravity and embedded). Examples include: double sheet pile wall cofferdams; earth structures reinforced by tendons, geotextiles, or grouting; structures with multiple rows of ground anchorages or soil nails. Parts of this chapter will be relevant to the design of composite walls.

Section 9 of EN 1997-1 applies to retaining structures supporting ground (i.e. soil, rock or backfill) and/or water and is sub-divided as follows:

- *§9.1. General (6 paragraphs)*
- *§9.2. Limit states (4)*
- *§9.3. Actions, geometrical data and design situations (26)*
- *§9.4. Design and construction considerations (10)*
- *§9.5. Determination of earth pressures (23)*
- *§9.6. Water pressures (5)*
- *§9.7. Ultimate limit state design (26)*
- *§9.8. Serviceability limit state design (14)*

Many provisions from *EN1997-1, Section 6* 'Spread foundations' (discussed in Chapter 3) also apply to gravity walls.

Annex C of Eurocode 7, Part 1 'Sample procedures to determine earth pressures' provides informative text relevant to retaining structures and is sub-divided as follows:

1. Limit values of earth pressure (3 paragraphs)
2. Analytical procedure for obtaining limiting active and passive earth pressures (14)
3. Movements to mobilise earth pressures (4)

4.2. Design situations and limit states

Figures 4.2.1 and 4.2.2 (reproducing *Figures 9.1 and 9.2 of EN 1997-1)* illustrate some common limit modes for overall stability and foundation failure of gravity walls.

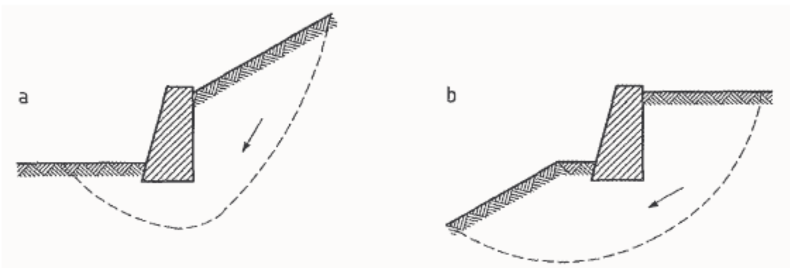

Fig.4.2.3 Limit modes for overall stability of retaining structures

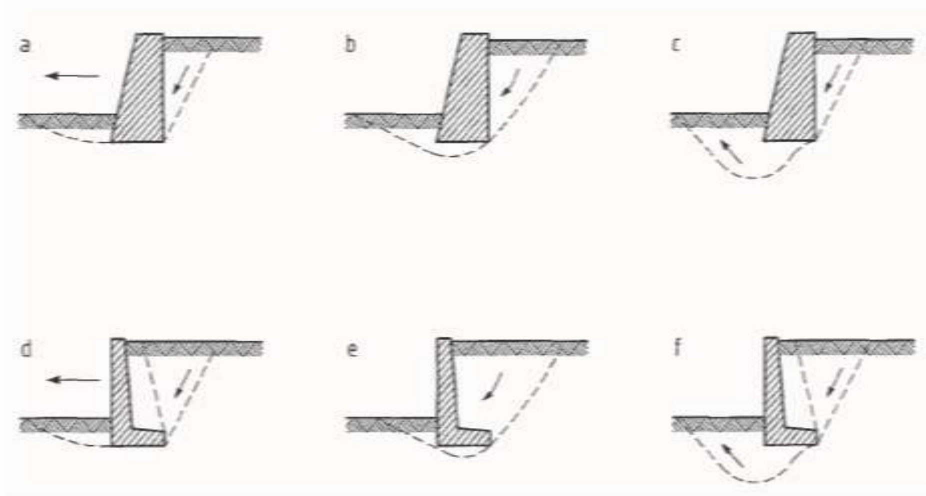

Fig.4.2.2 Limit modes for foundation failures of gravity walls

For ultimate limit state design of gravity walls, the design geometry shall account for anticipated excavation or possible scour in front of the retaining structure. Specifically, with 'normal site control', the retained height of the wall must be increased as follows:

$$H_d = H_{nom} + \Delta H$$

where H_d = design height of wall; H_{nom} = nominal height of wall; and ΔH = allowance for unplanned excavation. Table 4.2.1 below gives the recommended values of ΔH with normal site control in place.

Table 4.2.1 Recommended values of ΔH

Wall type	**ΔH for normal site control**
Cantilever	10% H (up to a maximum of 0,5 m)
Supported	10% of height below lowest support (up to a maximum of 0,5 m)

Eurocode 7's recommendations regarding water levels behind retaining walls distinguish between design situations with and without reliable drainage. When the wall retains medium or low permeability (i.e. mainly fine) soils, the wall should be designed for a water level above formation level.

- Without reliable drainage, the water level should normally be taken at the surface of the retained material.
- With reliable drainage, the water level may be assumed to be below the top of the wall, but there is then a maintenance requirement to ensure the drainage remains reliable.

One source of ambiguity in *Eurocode 7* concerns whether water pressures should be factored or not. For ultimate limit states (ULSs), *EN 1997-1* states "*design values [of groundwater pressures] shall represent the most unfavourable values that could occur during the design lifetime of the structure*." Whereas, for serviceability limit states (SLSs), it states "*design values shall be the most unfavourable values which could occur in normal circumstances [EN 1997-1 §2.4.6.1(6)P]*". Although the first of these statements is easy to interpret, it is not clear what 'normal circumstances' are.

Furthermore, *EN 1997-1* goes on to say "*design values of ground-water pressures may be derived either by applying partial factors to characteristic water pressures or by applying a safety margin to the characteristic water level" [EN 1997-1 §2.4.6.1(8)].*

Bond and Harris (2008) discuss the ways in which this Application Rule has been interpreted by practising engineers – Fig.4.4.3 summarizes some of the more common interpretations.

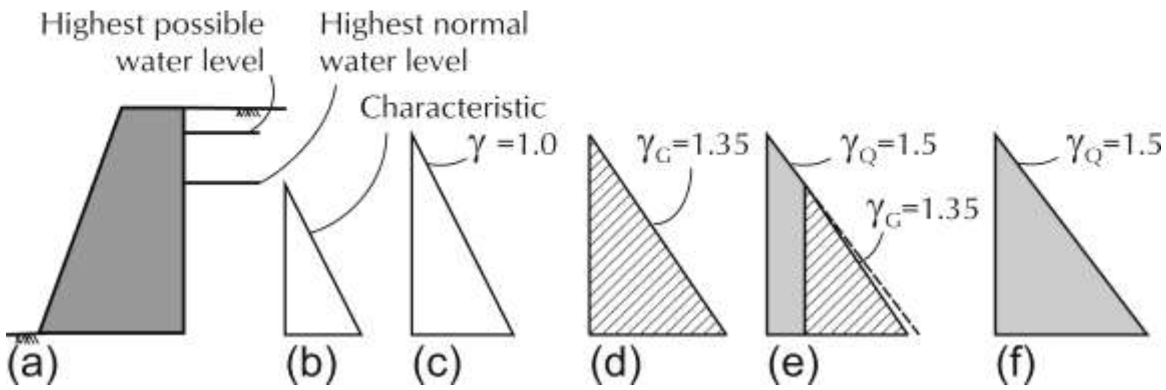

Fig.4.4.3 Possible ways of treating water pressures (after *Bond & Harris, 2008*)

a) Design water levels for ULS and SLS design situations
b) Characteristic water pressures for SLS design situation
c) Design pressures for ULS with no factor applied (γ= 1,0)
d) with the factor on permanent actions (γ_G = 1,35) applied
e) with the factor on permanent actions (γ_G = 1,35) applied to the normal water level and the factor on variable actions (γ_Q = 1,5) applied to any rise in water level
f) with the factor on variable actions (γ_Q = 1,5) applied throughout

Instinctively, many geotechnical engineers consider it wrong to apply partial factors to water pressures, particularly when the density of groundwater is known to a reasonable accuracy. However, the failure to apply partial factors because of their 'unreasonableness' can lead to design situations that are under-designed and may even be unsafe. Because of this, *Bond and Harris (2008)* recommend a balanced approach to the issue, summarized as follows:

- When partial factors γ_G > 1,0 are applied to effective earth pressures, then pore water pressures should also be multiplied by γ_G > 1,0 but calculated from highest normal (i.e. serviceability) water levels – i.e. no safety margin is applied
- When partial factors γ_G = 1,0 are applied to effective earth pressures, then pore water pressures should be multiplied by γ_G = 1,0 but calculated from highest possible (i.e. ultimate) water levels – after an appropriate safety margin has been applied

Fig.4.2.4 illustrates these two design situations. Further clarification of how water pressures should be handled is planned to be included in the next version of *Eurocode 7*, tentatively planned for 2018-2020.

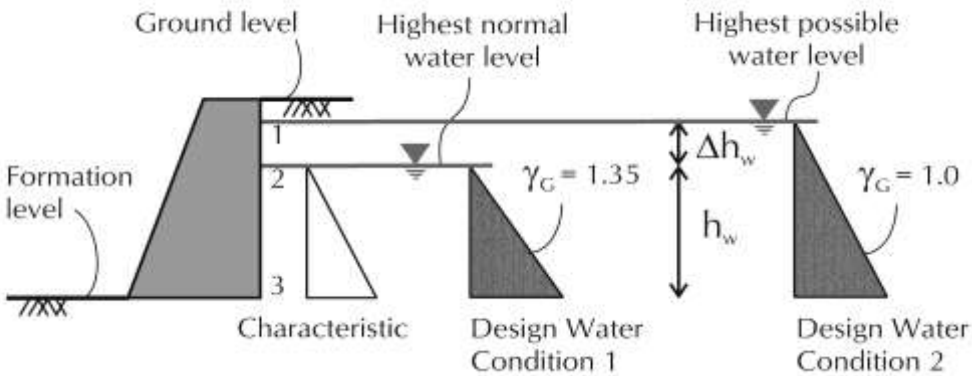

Fig.4.2.4 Recommended treatment of water pressures for design (after *Bond & Harris, 2008)*

4.3. Basis of design for gravity walls

According to EN 1997-1, ultimate limit states GEO and STR must be verified using one of three Design Approaches, as summarized in Table 4.3.1 below. The Design Approaches that have been chosen by different European countries for retaining wall design are shown in Fig.4..

Table 4.3.1 Summary of the Design approaches

Design Approach	**DA1**		**DA2**		**DA3**
Combination	**1**	**2**	**2**	**2***	
Partial factors applied to:	Actions	Material properties	Actions and resistance	Effects of actions and resistance	Structural actions and resistance
Partial factor Sets*	A1+M1+R1**	A2+M2+R1**	A1+M1+R2**	A1+M1+R2**	A1/A2+M2+R3**

* Sets A1-A2 = factors on actions/effects of actions
Sets M1-M2 = factors on material properties
Sets R1-R3 = factors on resistances
** in underlined sets, factors are > 1,0

Fig.4.3.1 National choice of Design Approach for retaining walls (after *Bond, 2013)*

The values of the partial factors that are recommended by *EN 1997-1* for Design Approach 1 Combinations 1 and 2 (DA1-1 and DA1-2) are summarized in Table 4.3.2 below.

Table 4.3.2 Partial factors for Design Approach 1

Parameter		Symbol	DA1-1 A1	DA1-1 M1	DA1-1 R1	DA1-2 A2	DA1-2 M2	DA1-2 R1
Permanent action (*G*)	Unfavourable	γ_G	1,35			1,0		
	Favourable	($\gamma_{G,fav}$)	1,0					
Variable action (*Q*)	Unfavourable	γ_Q	1,5			1,3		
	Favourable	-	(0)			(0)		
Shearing resistance (tan φ)		γ_φ					1,25	
Effective cohesion (*c'*)		γ_c						
Undrained shear strength (c_u)		γ_{cu}		1,0			1,4	
Unconfined compressive strength (q_u)		γ_{qu}						
Weight density (γ)		γ_γ					1,0	
Bearing resistance (R_v)		γ_{Rv}						
Sliding resistance (R_h)		γ_{Rh}			1,0			1,0
Earth resistance (R_e)		γ_{Re}						
Factors given for persistent and transient design situations								

The values of the partial factors that are recommended by EN 1997-1 for Design Approaches 2 and 3 (DA2 and DA3) are summarized in Table 4.3.3 below.

Table 4.3.3 Partial factors for Design Approaches 2 and 3

Parameter		Symbol	DA2/DA2* A1	DA2/DA2* M1	DA2/DA2* R2	DA3 A1#	DA3 A2*	DA3 M2	DA3 R3
Permanent action (G)	Unfavourable	γ_G	1,35			1,35	1.0		
	Favourable	($\gamma_{G,fav}$)	1,0						
Variable action (Q)	Unfavourable	γ_Q	1,5			1,5	1.3		
	Favourable	-	(0)			(0)			
Shearing resistance (tan φ)		γ_φ						1,25	
Effective cohesion (c')		γ_c							
Undrained shear strength (c_u)		γ_{cu}		1,0				1,4	
Unconfined comp. str. (q_u)		γ_{qu}							
Weight density (γ)		γ_γ						1,0	
Bearing resistance (R_v)		γ_{Rv}			1,4				1,0

Parameter	Symbol	DA2/DA2*			DA3			
		A1	M1	R2	A1#	A2*	M2	R3
Sliding resistance (R_h)	γ_{Rh}			1,1				
Earth resistance (R_e) walls	γ_{Re}			1,4				
Earth resistance (R_e) slopes				1,1				
Factors given for persistent and transient design situations # Applied to structural actions; *applied to geotechnical actions								

4.4. Verification of ultimate limit state GEO

4.4.1. REINFORCED CONCRETE WALLS

Fig.4. illustrates the pressures that act on a reinforced concrete wall.

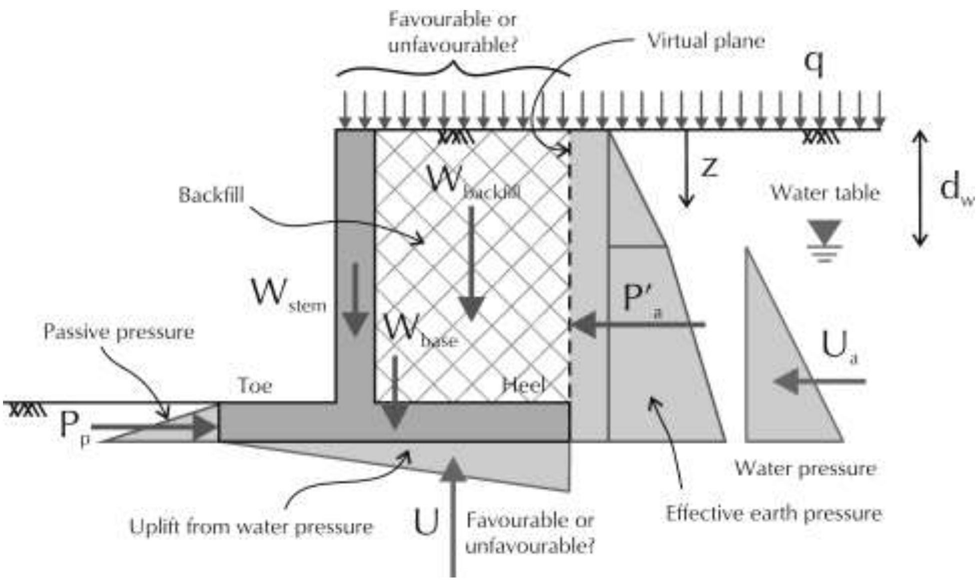

Fig.4.4.1 Pressures on a reinforced concrete wall *(after Bond & Harris, 2008)*

Care must be taken in choosing the values of the partial factors that are applied to the various pressures acting on the wall:

- The surcharge (q) behind the wall – which extends to the wall face – should be considered an unfavourable action for the verification of bearing capacity; but it is a favourable action with regards to resisting overturning and sliding and hence should be curtailed at the virtual place in these design situations
- The thrust from the 'active' water pressure U_a is an unfavourable action
- The uplift from the water pressure beneath the wall base appears, at first glance, to be a favourable action for verification of bearing pressure. However, since the source of this water

pressure is the same as that of the thrust U_a, for compatibility it should be regarded as unfavourable (this is known as the ‘single source principle’)

Annex C of EN 1997-1 (+Corrigendum 1) gives expressions for active and passive earth pressures:

$$\sigma_a = K_a\left(\int_0^z \gamma dz + q - u\right) - 2c\sqrt{K_a(1 + a/c)} + u$$

$$\sigma_p = K_p\left(\int_0^z \gamma dz + q - u\right) + 2c\sqrt{K_p(1 + a/c)} + u$$

where

σ_a, σ_p = active, passive stresses normal to the wall;

K_a, K_p = horizontal active, passive earth pressure coefficients;

γ = weight density of retained ground;

c = ground cohesion;

q = vertical surface load;

z = distance down face of wall;

a = wall adhesion.

The appropriate earth pressure coefficient for active conditions, assuming a Rankine zone forms between the back of the wall and the virtual plane, is given by:

$$K_{a,\beta} = \left(\frac{cos\beta - \sqrt{cos^2\beta - cos^2\varphi}}{cos\beta + \sqrt{cos^2\beta - cos^2\varphi}}\right)cos\beta$$

where

$K_{a,\beta}$ = active earth pressure coefficient for inclined thrust (= $\sigma'_\beta / \sigma'_v$);

φ = angle of shearing of soil;

β = angle of inclination of ground surface;

σ'_β = inclined effective stress;

σ'_v = vertical effective stress.

When β = 0, this reduces to the more familiar:

$$K_a = \frac{1 - \sin\varphi}{1 + \sin\varphi}$$

For the design of the wall stem, it may be necessary to assume at-rest conditions prevail behind the wall, in which case the at-rest earth pressures coefficient K_0 should be used instead of K_a:

$$K_0 = (1 - \sin\varphi) \cdot \sqrt{OCR} \cdot (1 + \sin\beta)$$

where

OCR = overconsolidation ratio (=$\sigma'_{v,max} / \sigma'_v$).

For normally consolidated soil, assume at rest conditions if movement of structure is less than 0,05% of the retained height. This expression is a combination of Meyerhof's equation for K_0 and *Kezdi's* modification for sloping ground.

4.4.2. MASS GRAVITY WALLS

Fig.4.4.2 illustrates the pressures that act on a reinforced concrete wall.

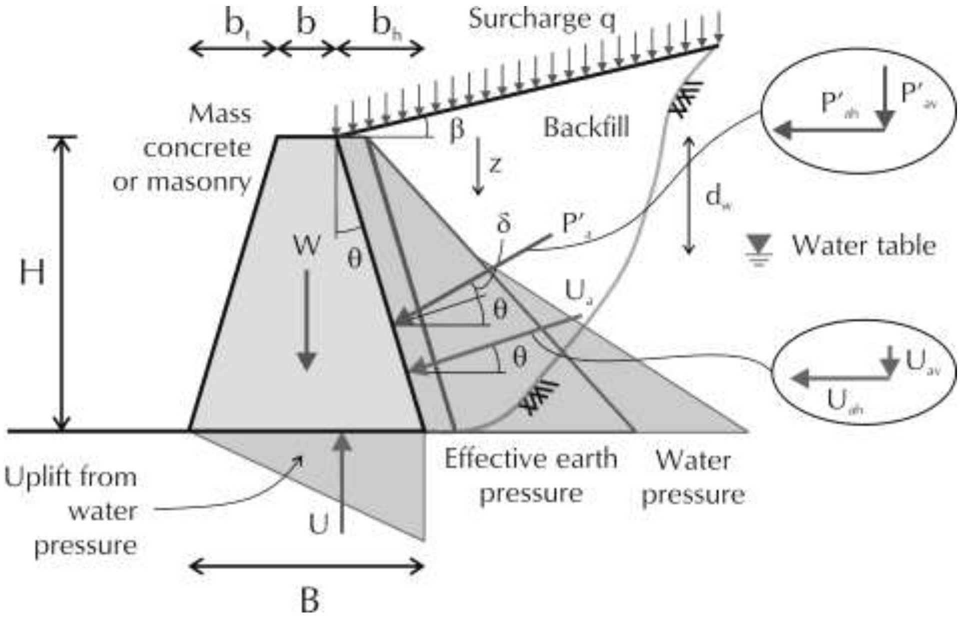

Fig.4.4.2 Pressures on a mass gravity wall (after *Bond & Harris, 2008*)

The earth pressures that act on the back face of the wall may be determined using the charts given in *Annex C of EN 1997-1*, which are were developed by (but are not attributed to) Kerisel and Absi. They are the same charts as appear in *BS 8002*. Kerisel and Absi assumed log-spiral failure surfaces and hence their charts give upper bound values of K_a and K_p. The charts can only be used for walls with vertical back faces. If the back face is inclined, then the following expressions developed by Brinch Hansen (also given in *Annex C of En 1997-1*) may be used:

$$\sigma'_a = K_{a\gamma}\left(\int_0^z \gamma dz - u\right) + K_{aq}q - K_{ac}c; \sigma'_p = K_{p\gamma}\left(\int_0^z \gamma dz - u\right) + K_{pq}q + K_{pc}c$$

$$\left.\begin{matrix} K_{a\gamma} \\ K_{p\gamma} \end{matrix}\right\} = K_n \cdot \cos\beta \cdot \cos(\beta - \theta); \left.\begin{matrix} K_{aq} \\ K_{pq} \end{matrix}\right\} = K_n \cdot \cos^2\beta; \left.\begin{matrix} K_{ac} \\ K_{pc} \end{matrix}\right\} = (K_n - 1) \cdot \cot\varphi$$

$$K_n = \frac{1 \pm \sin\varphi \cdot \sin(2m_w \pm \varphi)}{1 \mp \sin\varphi \cdot \sin(2m_t \pm \varphi)} e^{\pm 2(m_t + \beta - m_w - \theta)\tan\varphi}$$

$$2m_t = \cos^{-1}\left(\frac{-\sin\beta}{\pm\sin\varphi}\right) \mp \varphi - \beta; 2m_w = \cos^{-1}\left(\frac{\sin\delta}{\sin\varphi}\right) \mp \varphi \mp \delta$$

Bond and Harris (2008) have published charts (for vertical walls only) that make these expressions easy to evaluate. Fig.4.4.3 shows one of these charts.

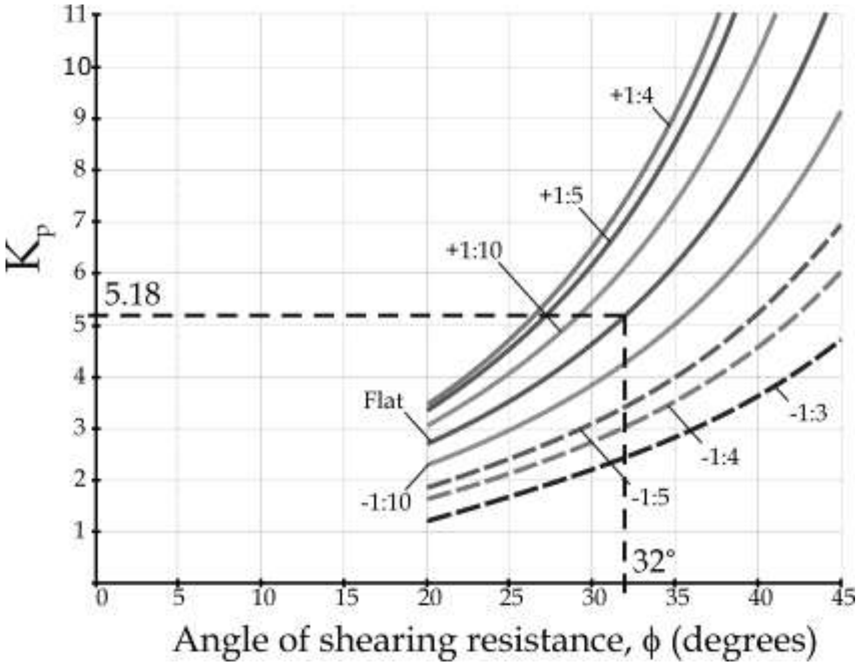

Fig.4.4.3 Chart showing Brinch Hansen's passive earth pressure coefficient *(after Bond & Harris, 2008)*

4.4.3. REINFORCED FILL WALLS

Eurocode 7 does not cover the detailed design of reinforced fill structures. The values of the partial factors given in *EN 1997-1* have not been calibrated for reinforced fill structures. Design of reinforced fill structures is currently carried out to national standards (such as *BS 8006* in the UK). Differences in working practices, geology, and climate, etc. have delayed the development of a single design method accepted throughout Europe.

It is hoped that a future European standard will cover design of these walls.

4.5. Verification of serviceability

Verification of serviceability is expressed in *Eurocode 7* by:

$$E_d \leq C_d$$

where

E_d = design effect of actions (e.g. displacement, distortion);

C_d = design value of the appropriate constraint (i.e. limiting value of the design effect of actions).

For *'conventional structures founded on clays'*, *Eurocode 7* allows settlement calculations to be avoided if an ultimate limit state calculation for bearing resistance satisfies:

$$E_k \leq \frac{R_k}{\gamma_{R,SLS}}$$

where

E_k = characteristic effects of actions;

R_k = characteristic resistance;

$\gamma_{R,SLS}$ = a partial resistance factor ≥ 3,0. (Note: this expression does not appear in *EN 1997-1*, but can be deduced from its text.)

Verification of the serviceability of gravity walls is similar to that of shallow foundations – see Chapter 3 for details.

4.6. Supervision, monitoring, and maintenance

Annex J of EN 1997-1 lists items that need to be considered during supervision of construction of gravity walls, including:

- Verification of ground conditions and of the location and general lay-out of the structure
- Ground-water flow and pore-water pressure regime; effects of dewatering on ground-water table; effectiveness of measures to control seepage inflow; corrosion potential
- Movements, yielding, stability of excavation walls and base; temporary support systems; effects on nearby buildings and utilities; measurement of soil pressures on retaining structures and of pore-water pressure variations resulting from excavation or loading
- Safety of workmen with due consideration of geotechnical limit state

Annex J also lists items that need to be considered with regards to water flow and pore-water pressures, including:

- Adequacy of systems to control pore-water pressures in aquifers where excess pressure could affect stability of base of excavation, including artesian pressures beneath the excavation; disposal of water from dewatering systems; depression of ground-water table throughout entire excavation to prevent boiling or quick conditions, piping and disturbance of formation by construction equipment; diversion and removal of rainfall or other surface water
- Control of dewatering to avoid disturbance of adjoining structures or areas; observations of piezometric levels; effectiveness, operation and maintenance of water recharge systems
- Settlement of adjoining structures or areas
- Effectiveness of sub-horizontal borehole drains

Finally, *Annex J* lists items that need to be considered as part of performance monitoring:

- Settlement at established time intervals of buildings and other structures including those due to effects of vibrations on metastable soils
- Lateral displacement and distortions, especially those related to fills and stockpiles; soil supported structures, such as buildings or large tanks; deep trenches
- Piezometric levels under buildings or in adjoining areas, especially if deep drainage or permanent dewatering systems are installed or if deep basements are constructed
- Deflection or displacement of retaining structures considering: normal backfill loadings; effects of stockpiles; fills or other surface loadings; water pressures
- Flow measurements from drains
- Water tightness

No specific guidance is given in *EN 1997-1* for maintenance.

4.7. Summary of key points

The design of gravity walls to *Eurocode 7* involves checking that the ground beneath the wall has sufficient:

- bearing resistance to withstand inclined, eccentric actions
- sliding resistance to withstand horizontal and inclined actions
- stability to avoid toppling
- stiffness to prevent unacceptable settlement or tilt

Verification of ultimate limit states (ULSs) is demonstrated by satisfying the inequalities:

$$V_d \leq R_d$$

$$H_d \leq R_d + R_{pd}$$

$$M_{Ed,dst} \leq M_{Ed,stb}$$

4.8. Worked example – T-shaped wall

A worked example illustrating the way in which a T-shaped wall may be designed according to *Eurocode 7* is included in the Annex to this report.

References

Bond, A. J. (2013). Implementation and evolution of Eurocode 7, Modern Geotechnical Design Codes of Practice, Arnold et al. (eds), Amsterdam: IOS Press, pp3-14.

Bond, A. J., and Harris, A. J. (2008). Decoding Eurocode 7, London: Taylor & Francis, 618pp.

EN 1997-1: 2004. Eurocode 7: Geotechnical design. Part 1: General rules. CEN.

CHAPTER 5

Ground investigation and testing

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5.1. Overview

Ground investigation and testing for geotechnical design is covered by *EN 1997-2 (2004)*. *EN 1997-2* is intended to be used in conjunction with *EN 1997-1* and provides rules relating to:

- planning and reporting of ground investigations
- general requirements for a number of commonly used laboratory and field tests
- interpretation and evaluation of test results
- establishment of derived values of geotechnical parameters and coefficients

EN 1997-1 covers the establishment of characteristic values. As they are based on ground investigations, the provisions for their determination will therefore also be explained in this chapter.

EN 1997-2 is mainly a standard for the geotechnical engineer and experts for soil and rock testing. Only a small part of the standard is for the designer. This chapter will be restricted to those items which are important for the designer.

Its importance for the designer is stressed in *EN 1997-1 (2004), §2.4.1 (2) "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."*

EN 1997-2 has the following contents:

1. *General*
2. *Planning of ground investigations*
3. *Soil and rock sampling and groundwater measurements*
4. *Field tests in soil and rock*
5. *Laboratory tests on soil and rock*
6. *Ground investigation report*

with 23 Annexes.

EN 1997-2 only gives the general requirements for the field and laboratory tests; their execution is standardized in separate EN ISO standards:

- *EN ISO 22476* with 13 parts for field tests
- *CEN ISO/TS (specifications) 17892* with 12 parts for standard laboratory tests

EN ISO 14688 and *EN ISO 14689* specify the identification of soil and rock, while *EN ISO 22475* standardizes sampling and groundwater measurements.

5.2. Definitions

In *§1.5.3.1 of EN 1997-2 (2004)*, the definition of the term 'derived value' is introduced as a *"value of a geotechnical parameter obtained from test results by theory, correlation or empiricism"*. To distinguish it from a characteristic value of a geotechnical parameter, the derived value is explained by the chart presented in Fig.5.2.1. The process of evaluation test results starts with the numerical results of different field and laboratory tests. The basis for the evaluation of all tests is a theory and – for some of them – corrections and correlations have to be applied. This process is described in *EN 1997-2*.

The next step is the selection of characteristic values of geotechnical properties, taking into account:

- the derived values of the tests performed

- the geotechnical items of the project
- information from other sources on the site, the soils and rock

This second step of the process is covered by *EN 1997-1.*

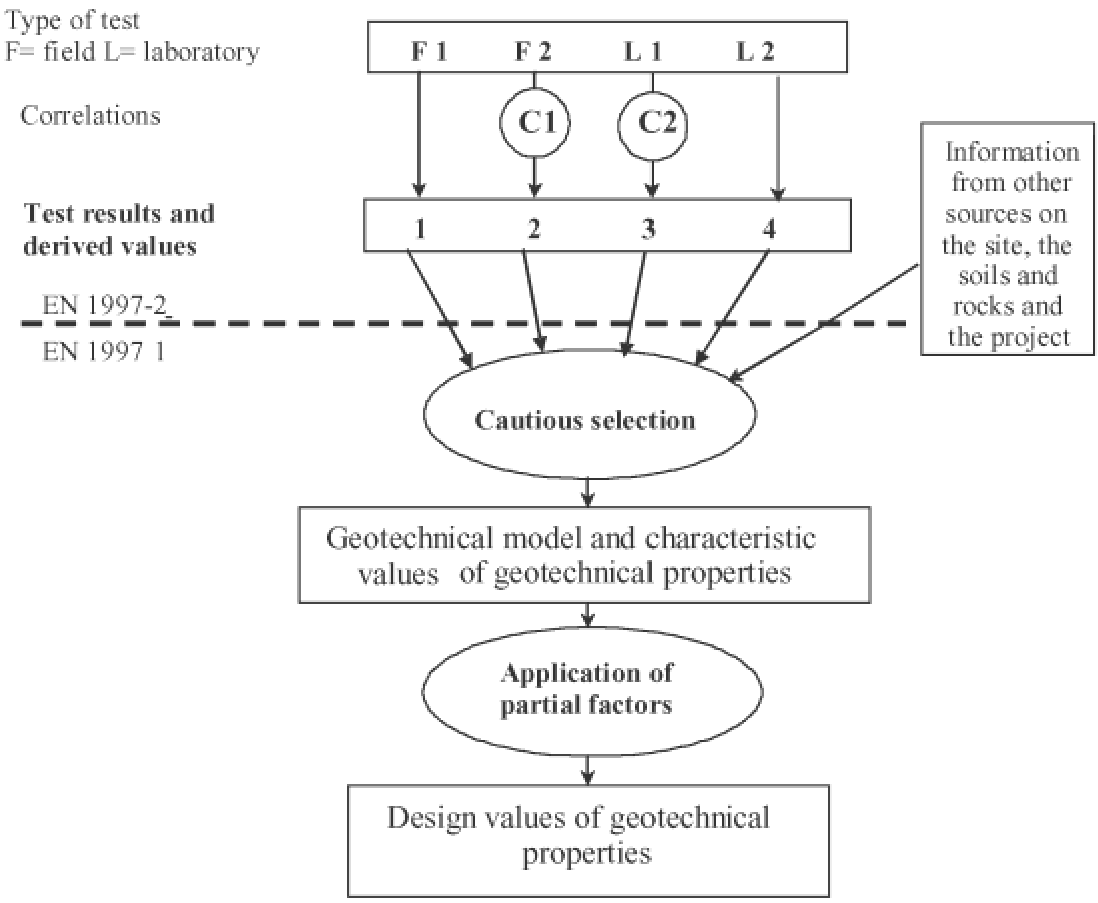

Fig.5.2.1 General framework for the selection of derived and characteristic values of geotechnical properties *(after EN 1997-2)*

5.3. Planning of ground investigations

Ground investigations shall provide a description of ground and groundwater conditions relevant to the proposed works and establish a basis for the assessment of the geotechnical parameters relevant for all construction stages. Such investigations are normally performed in stages, the first one involving desk studies, where (for example) geological maps and descriptions, previous investigations at the site and in the surroundings, aerial photos and previous photos as well as topographical maps are evaluated.

Most important for in situ investigations is the establishment of the locations and depths of the investigation points (for general rules see *§2.4.1.3*). *Annex B.3* (informative) of *EN 1997-2* gives recommendations for the spacing and depth of investigations for different geotechnical structures: The following spacing of investigation points should be used as guidance:

- for high-rise and industrial structures, a grid pattern with points at 15 m to 40 m distance
- for large-area structures, a grid pattern with points at not more than 60 m distance

- for linear structures (roads, railways, channels, pipelines, dikes, tunnels, retaining walls), a spacing of 20 m to 200 m
- for special structures (e.g. bridges, stacks, machinery foundations), two to six investigation points per foundation
- for dams and weirs, 25 m to 75 m distance, along vertical sections.

In this annex also investigation depths are given for the most common geotechnical structures. For example for a pile foundation (see Figure 5.2.2) the following three conditions for the investigation depth z_a should be met:

- $z_a \geq 1{,}0 \cdot b_g$
- $z_a \geq 5{,}0$ m
- $z_a \geq 3 \cdot D_F$

where D_F is the pile base diameter; and b_g is the smaller side of the rectangle circumscribing the group of piles forming the foundation at the level of the pile base.

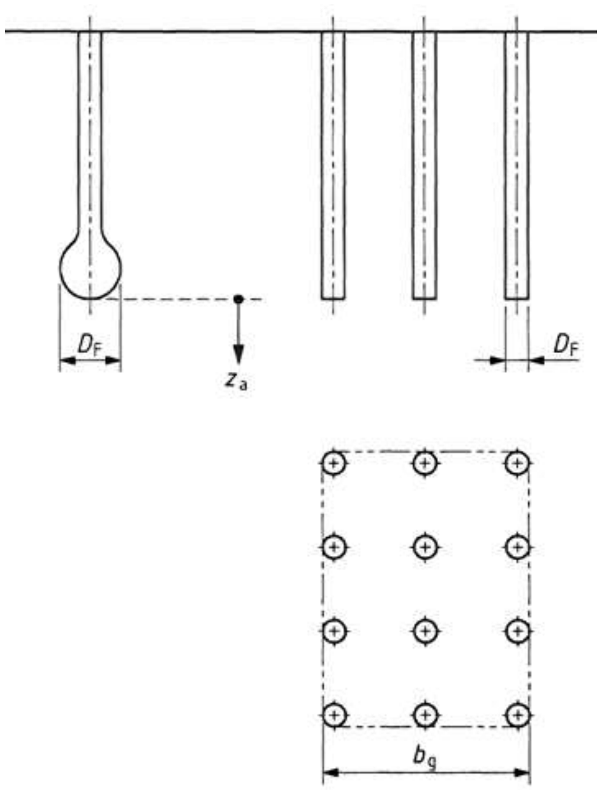

Fig.5.2.2 Investigation depth for pile groups *(after EN 1997-2)*

For sampling, *EN 1997-2* states in *2.4.2.4 (2)P* that *“For identification and classification of the ground, at least one borehole or trial pit with sampling shall be available. Samples shall be obtained from every separate ground layer influencing the behaviour of the structure”. … (7) “Samples should be taken at any change of stratum and at a specified spacing, usually not larger than 3 m. In inhomogeneous soil, or if a detailed definition of the ground conditions is required, continuous sampling by drilling should be carried out or samples recovered at very short intervals“.*

Soil samples for laboratory tests are divided in five quality classes with respect to the soil properties that are assumed to remain unchanged during sampling and handling, transport and storage (see Table 5.3.1) in combination with the sampling category according to *EN ISO 22475-1*. Table 5.3.1 is useful for the specification of sampling for tender purposes.

Table 5.3.1 Quality classes of soil samples (*after EN 1997-2)*

Soil properties / quality class	**1**	**2**	**3**	**4**	**5**
Unchanged soil properties					
particle size	*	*	*	*	
water content	*	*	*		
density, density index, permeability	*	*			
compressibility, shear strength	*				
Properties that can be determined					
sequence of layers	*	*	*	*	*
boundaries of strata - broad	*	*	*	*	
boundaries of strata - fine	*	*	*	*	
Atterberg limits, particle density, organic content	*	*	*		
water content	*	*			
density, density index, porosity, permeability	*	*			
compressibility, shear strength	*				
Sampling category according to EN ISO 22475-1	A				
			B		
					C

5.4. Field Tests

The following field tests are covered in *EN 1997-2:*

- Cone penetration and piezocone penetration tests (CPT, CPTU)
- Pressuremeter tests (PMT)
- Flexible dilatometer test (FDT)
- Standard penetration test SPT
- Dynamic probing tests (DP)
- Weight sounding test (WST)
- Field vane test (FVT)
- Flat dilatometer test (DMT)
- Plate loading test (PLT)

The annexes give valuable information on the evaluation of some field tests, for example see Table 5.3.2 for the cone penetration test.

Table 5.3.2 Effective angle of shearing resistance (φ') and drained Young's modulus of elasticity (E') from cone penetration resistance (q_c)

Density index	Cone resistance (q_c) (from CPT) MPa	Effective angle of shearing resistance, (φ')°	Drained Young's modulus, (E') MPa
Very loose	0,0 – 2,5	29 – 32	< 10
Loose	2,5 – 5,0	32 – 35	10 – 20
Medium dense	5,0 – 10,0	35 – 37	20 – 30
Dense	10,0 – 20,0	37 – 40	30 – 60
Very dense	> 20,0	40 – 42	60 – 90

5.5. Laboratory Tests

The following laboratory tests are covered by *EN 1997-2:*

- Tests for classification, identification and description of soil
- Chemical testing of soil and groundwater
- Strength index testing of soil
- Strength testing of soil
- Compressibility and deformation testing of soil
- Compaction testing of soil
- Permeability testing of soil
- Tests for classification of rocks
- Swelling testing of rock material
- Strength testing of rock material

The annexes give valuable information on the evaluation of some laboratory tests, for example see Table 5.3.3 for the incremental oedometer test.

Table 5.3.3 Incremental oedometer test. Recommended minimum number of tests for one soil stratum (after *EN 1997-2*)

Variability in oedometer modulus E_{oed} (in the relevant stress range)	Comparable experience: None	Medium	Extensive
Range of values of $E_{oed} \geq 50\%$	4	3	2
$20\% <$ Range of values of $E_{oed} < 50\%$	3	2	2
Range of values of $E_{oed} \leq 20\%$	2	2	1[a]
[a] One oedometer test and classification tests to verify compatibility with comparable knowledge (see *Q.1(2), Annex Q, En 1997-2*).			

5.6. Characteristic values of geotechnical data

EN 1997-1 prescribes in *§2.4.5.2(1)P: "The selection of characteristic values for geotechnical parameters shall be based on results and derived values from laboratory and field tests, complemented by well-established experience"*; and the basic principle shall that a *"characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state."* The section contains some general explanations.

§ 2.4.5.2 also gives an important guideline in paragraph (10) and the subsequent note, how to evaluate test data quantitatively:

> *"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%.*
>
> *NOTE In this respect, a cautious estimate of the mean value is a selection of the mean value of the limited set of geotechnical parameter values, with a confidence level of 95%; where local failure is concerned, a cautious estimate of the low value is a 5% fractile."*

Statistics provides formulae to evaluate characteristic values from a set of normally distributed test results. The characteristic value X_k of a normally distributed sample of test results:

$$X_k = X_m\left(1 - k_n V_X\right)$$

where

X_m is the arithmetic mean value of the test results;

V_x is the coefficient of variation;

k_n is a statistical coefficient depending on the number n of test results, the selected probability for the occurrence of X_k and whether the coefficient of variation V_x is known or not.

Values for k_n can be found in tables of textbooks on statistics and also in some textbooks on soil mechanics and ground engineering (see, for example, *Bond and Harris, 2008, Bauduin, 2002*).

Schneider (1999) proposed a much simpler formula for the derivation of the characteristic mean value:

$$c_{u,m,k} = c_{u,m} - 0{,}5 \cdot s_{cu}$$

where s_{cu} is the standard deviation of the sample.

5.7. Summary of key points

Planning of ground investigations:

- location of investigation point
- depth of investigation points
- sampling

Field tests:

- derivation of ground parameter

Laboratory tests:

- minimum number of tests

Characteristic values:

- statistical evaluation

5.8. Worked example – characteristic values of ground parameters

A worked example illustrating the selection of characteristic values for the base and shaft resistance for a pile foundation is included in the Annex to this report.

References

Bauduin, Ch.: (2002): Determination of characteristic values, in Geotechnical Engineering Handbook, Volume 1: Fundamentals, Editor: Ulrich Smoltczyk, Ernst & Sohn

Bond, A. J., and Harris, A. J. (2008). Decoding Eurocode 7, London: Taylor & Francis, 598pp.

EN 1997-1: 2004. Eurocode 7: Geotechnical design. Part 1: General rules. CEN.

EN 1997-2: 2004. Eurocode 7: Geotechnical design - Part 2: Ground investigation and testing. CEN.

Schneider, H. R.: (1999): Determination of characteristic soil properties, in Proceedings of the 12th European Conference on Soil Mechanics and Foundation Engineering, Amsterdam, Balkema, Rotterdam, 1999, Vol. 1, pp 273 - 281.

CHAPTER 6

Slope stability, embankments and hydraulic failure

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6.1. Overview

EN 1997-1 has no separate section on slope stability. The provisions for the design of slopes and embankments are contained in *Section 11: Overall stability* and *Section 12: Embankments*. The provisions in *Section 11* apply to the overall stability of and movements in the ground, whether natural or fill, around foundations, retaining structures, natural slopes, embankments or excavations. The provisions in *Section 12* apply to embankments for small dams and for infrastructure

The provisions for the verification against hydraulic failure are covered in *Section 10 of EN 1997-1*. The terms for the failure modes: uplift (UPL), hydraulic heave, internal erosion and piping (all three abbreviated by HYD) are defined and the verification against these four failures described.

6.2. Slope stability

6.2.1. SCOPE AND CONTENTS

Slope stability is covered by the requirements in *Section 11 of EN 1997-1* against loss of overall stability and where there are excessive movements in the ground causing damage or loss of serviceability in neighbouring structures, roads or services. Typical structures for which an analysis of overall stability should be performed (and mentioned in relevant sections of *Eurocode 7*) are:

- Retaining structures
- Excavations, slopes and embankments
- Foundations on sloping ground, natural slopes, or embankments
- Foundations near an excavation, cut or buried structure, or shore

Some examples of limit modes for overall stability of retaining structures, which are presented in *Section 9 of EN 1997-1*, are shown in Fig.6.2.5.

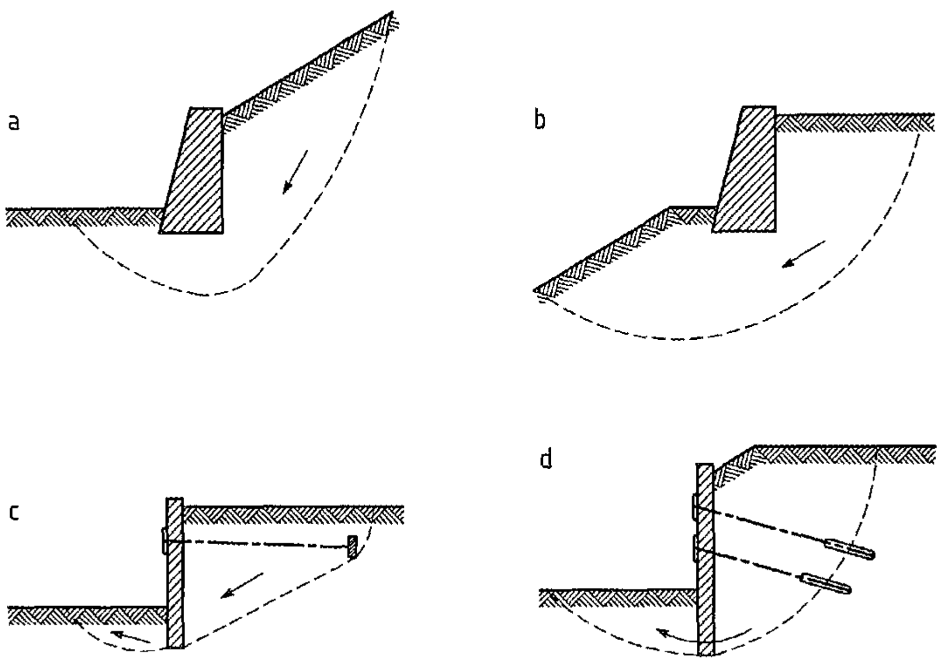

Fig.6.2.5 Examples of limit modes for overall stability of retaining structures

Section 11 of EN 1997-1 has the following sub-sections:

§11.1 General (2 paragraphs)

§11.2 Limit states (2)

§11.3 Actions and design situations (6)

§11.4 Design methods and design considerations (11)

§11.5 Ultimate limit state design (26)

§11.6 Serviceability limit state design(3)

§11.7 Monitoring (2)

6.2.2. ULTIMATE LIMIT STATE DESIGN OF SLOPES

The overall stability of slopes should be checked using the design values of actions, resistance and strengths obtained using the appropriate GEO/STR values for the partial factors. When analysing overall stability, all the relevant modes of failure should be taken into account. *Eurocode 7* does not give any specific inequality to be satisfied for overall stability, nor is any calculation model given. However, with regard to stability analyses of slopes, *EN 1997-1 §11.5.1(4*) states that the mass of soil or rock bounded by the failure surface should normally be treated as a rigid body or several rigid bodies moving simultaneously. The failure surfaces may have a variety of shapes including planar, circular, or more complicated shapes. Alternatively, stability may be checked by limit analysis or by the finite element method.

DA3 is the same as DA1.C2 in the design of slopes since loads on the surface in DA3 are treated as geotechnical actions using the A2 set of partial factors on actions, as in DA1.C2. In DA1, C1 and C2 should both be considered, but DA1.C2 normally controls.

For undrained analyses of slopes, the DA1 partial factors are:

- DA1.C1 $\gamma_G = 1,35$ $\gamma_Q = 1,5$ $\gamma_{cu} = 1,0$
- DA1.C2 $\gamma_G = 1,0$ $\gamma_Q = 1,3$ $\gamma_{cu} = 1,4$

For drained analyses, in the case of DA1.C1 an increase in the vertical load increases the resistance on the shear plane since the shearing resistance is a function of the normal stress due to the soil weight that is causing the instability, so that the margin of safety is unchanged. Thus DA1.C1 does not usually govern. Hence in drained analyses, DA1.C2 governs and the DA1 partial factors are:

- DA1.C2 $\gamma_G = 1,0$ $\gamma_Q = 1,3$ $\gamma_{\phi} = 1,25$

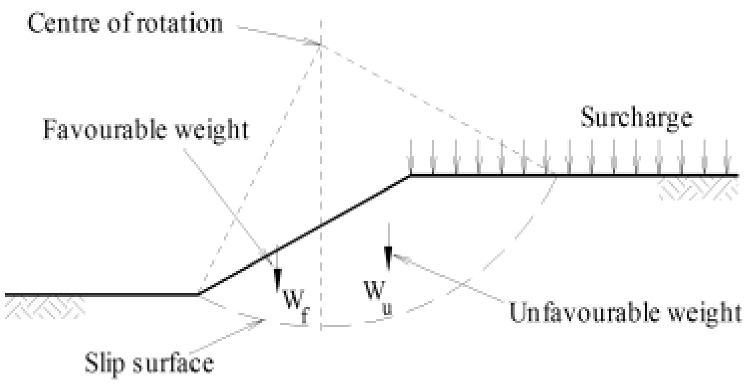

Fig.6.2.6 Rotational slide (after *Orr and Farrell, 1998*)

A rotational slide, which is an example of a loss of overall stability, is shown in Fig.6.2.6 from *Orr and Farrell (1998)*. This figure shows that one part of soil weight is unfavourable and causing the rotational failure, while the other part favourable and acting to stabilise the slope. As all the soil weight should be treated as coming from a "single source" *(see §2.4.2, Note to (9)P)*, the same partial factor should be applied to the unfavourable and favourable components of the soil weight. In practice this means that the partial action factor is applied to the net destabilising load effect. In DA2 different factors are applied to favourable and unfavourable loads. Hence if the single source principle is followed, then the reduced DA2 partial action factor on favourable actions is not applied to the stabilising component of the soil weight with the result that the resulting design is less conservative than if the favourable and unfavourable components of the load were treated separately. For this reason and also because it is not normal practice to separate the soil weight into stabilising and destabilising parts in slope stability analyses and because it is difficult to do so using the method of slices, DA2 is generally not used for slope stability analyses.

6.2.3. SLOPE STABILITY ANALYSES

6.2.3.1. Infinite slope analyses

The example shown in Figure 6.2.3 is an infinite slope at an inclination β with a slip plane parallel to the ground surface and at a depth z. The water table is at a height h above the slip surface and the soil has a weight density γ. The stability of the slope is analysed for the situation when the water rises to the surface, i.e. $h = z$.

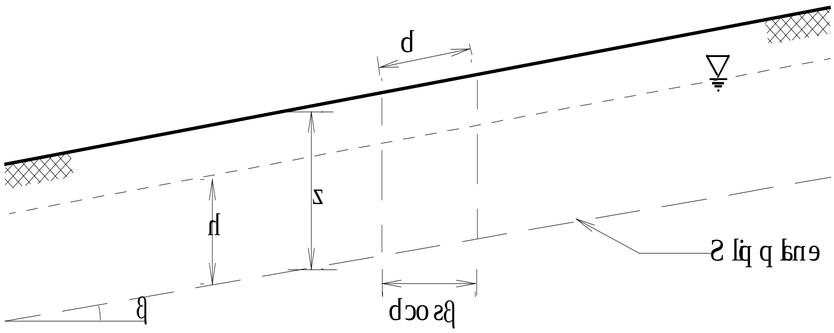

Fig.6.2.7 Infinite slope (after *Orr and Farrell, 1998*)

Analysing the stability of the slope, the equilibrium equation to be satisfied is that the design destabilising action, i.e. the component of the weight causing sliding S_d should be less than or equal to the design resistance force R_d i.e.:

$$S_d \le R_d$$

Considering a column of soil of width b and weight W, the destabilising design sliding force is:

$$S_d = \gamma_G W \sin\beta = \gamma_G \cdot (\gamma \cdot z \cdot b \cdot \cos\beta) \times \sin\beta$$

The design resistance is:

$$\begin{aligned} R_d &= \gamma_G \cdot (W\cos\beta - ub\cos\beta) \cdot \tan\varphi'_d = \\ &= \gamma_G \cdot (\gamma \cdot z \cdot b \cdot \cos\beta - \gamma_w \cdot z \cdot b \cdot \cos\beta) \cdot (\tan\varphi'_k)/\gamma_M = \\ &= \gamma_G \cdot (\gamma' \cdot z \cdot b \cos^2\beta) \cdot (\tan\varphi'_k)/\gamma_M \end{aligned}$$

Substituting for S_d and R_d in the equilibrium inequality and setting γ_M = 1,25 for DA1.C2 gives:

$$\gamma_G\left(\gamma \cdot z \cdot b\right)\cos\beta\sin\beta \le \gamma_G\left(\gamma' \cdot z \cdot b\right)\cos^2\beta \cdot \left(\tan\varphi'_k\right)/1{,}25$$

and hence:

$$\gamma \cdot \tan\beta \le \gamma' \cdot \left(\tan\varphi'_k\right)/1{,}25$$

For comparison, using previous global factor design method, the global factor of safety FOS was given by:

$$\text{FOS} = \frac{\gamma' \tan\varphi'}{\gamma \tan\beta}$$

If FOS = 1,25, this equation becomes:

$$\gamma \cdot \tan\beta \le \gamma' \cdot \left(\tan\varphi'_k\right)/1{,}25$$

so that designs to *Eurocode 7* are the same as previous designs.

6.2.3.2. General stability analyses

Eurocode 7 provides no specific inequality to be satisfied in the case of slope stability analyses. However, *EN 1997-1 §11.5.1(10)* states that a slope stability analysis should verify the overall moment and vertical equilibrium of the sliding mass. It also states that, if horizontal equilibrium is not checked, the interslice forces, i.e. when using the method of slices, should be assumed to be horizontal. This means that some slope stability analysis methods are not acceptable. Information about the equilibrium equations that are satisfied and the interslice force characteristics and relationships in the different methods of slices are given in Table 6.2.3 and Table 6.4 from *Krahn (2004)*. These tables show that:

- Spencer’s method is acceptable because both moment and force equilibrium equations are satisfied
- Bishop’s Simplified method is acceptable because moment equilibrium is satisfied and, while force equilibrium is not satisfied, the interslice forces are horizontal
- Janbu’s method is not acceptable as moment equilibrium is not satisfied
- Fellenius’ method is not acceptable because, while moment equilibrium is satisfied, forces equilibrium is not and the interslice forces are not horizontal

Table 6.2.3 Equations of equilibrium satisfied in methods of slices (*Krahn, 2004*)

Method	Moment Equilibrium	Force Equilibrium
Ordinary or Fellenius	Yes	No
Bishop’s Simplified	Yes	No
Janbu’s Simplified	No	Yes
Spencer	Yes	Yes
Morgenstern-Price	Yes	Yes
Corps of Engineers – 1	No	Yes
Corps of Engineers - 2	No	Yes
Lowe-Karafiath	No	Yes
Janbu Generalized	Yes (by slice)	Yes
Sarma – vertical slices	Yes	Yes

Table 6.4.2 Interslice force characteristics and relationships in methods of slices (Krahn, 2004)

Method	Interslice Normal (E)	Interslice Shear (X)	Inclination of X/E Resultant, and X-E Relationship
Ordinary or Fellenius	No	No	No interslice forces
Bishop's Simplified	Yes	No	Horizontal
Janbu's Simplified	Yes	No	Horizontal
Spencer	Yes	Yes	Constant
Morgenstern-Price	Yes	Yes	Variable; user function
Corps of Engineers – 1	Yes	Yes	Inclination of a line from crest to
Corps of Engineers - 2	Yes	Yes	Inclination of ground surface at top of slice
Lowe-Karafiath	Yes	Yes	Average of ground surface and slice base inclination
Janbu Generalized	Yes	Yes	Applied line of thrust and moment equilibrium of slice
Sarma – vertical slices	Yes	Yes	$X = C + E \cdot \tan\phi$

6.2.3.3. Bishop's simplified method of slices

The global factor of safety F in Bishop's simplified method of slices is equivalent to the partial factor on the soil strength parameters with appropriate partial factors on the actions as shown in the following equations:

$$\tau_{mob} = \frac{c'}{F} + \sigma'_n \frac{\tan\varphi'}{F} = \frac{c'}{\gamma_{M;mob}} + \sigma'_n \frac{\tan\varphi'_k}{\gamma_{M;mob}}$$

$$\gamma_{M;mob} = \frac{1}{\sum \gamma_G W \sin\alpha} \sum \frac{\left\lfloor c'_k b + (\gamma_G W - \gamma_G u b)\tan\varphi'_k \right\rfloor \sec\alpha}{1 + \dfrac{\tan\alpha \tan\varphi'_k}{\gamma_{M;mob}}}$$

In DA1.C1, γ_G = 1,35 is applied to permanent actions, including the soil weight force via the soil weight density, and γ_Q = 1,5 is applied to variable actions when analysing the overall factor of safety, F using the method of slices. Then it is checked that F, which is equal to $\gamma_{M;mob}$, is greater than or equal to 1,0.

In the case of DA1.C2, γ_G = 1,0 is applied to permanent actions, including the soil weight force via the soil weight density, and γ_Q = 1,5 is applied to variable actions when analysing the overall factor of safety, F using the method of slices. Then it is checked that F, which is equal to $\gamma_{M;mob}$, is greater than or equal to 1,25.

6.2.4. SERVICEABILITY LIMIT STATE DESIGN OF SLOPES

With regard to the serviceability limit state design of slopes, *EN 1997-1 §11.6(1)P* states that the design of slopes shall show that the deformations of the ground will not cause a serviceability limit state in structures and infrastructure on or near the particular ground. Since the analytical and numerical methods available at present do not usually provide reliable predictions of the deformations of a natural slope, the occurrence of serviceability limit states should be avoided by one of the following:

- Limiting the mobilised shear strength
- Observing the movements and specifying actions to reduce or stop them, if necessary (i.e. use the Observational Method)

6.3. Embankments

6.3.1. SCOPE AND CONTENTS

EN 1997-1 §12.1(1)P states that the provisions in *Section 12* shall apply to embankments for small dams and for infrastructure. However, no definition is given for the word "small". According to *Frank et al. (2004)*, it may be appropriate to assume "small dams" include dams (and embankments for infrastructure) up to a height of approximately 10 m.

Section 12 Embankments is the shortest section of *EN 1997-1*, being just over four pages long. It has the following sub-sections:

6. *General (2 paragraphs)*
7. *Limit states (2)*
8. *Actions and design situations (8)*
9. *Design methods and design considerations (13)*
10. *Ultimate limit state design (7)*
11. *Serviceability limit state design(4)*
12. *Supervision and monitoring (6)*

Since embankments are constructed by placing fill and sometimes involve ground improvement, the provisions on fill in *Section 5* should be applied. For embankments on ground with low strength and high compressibility, *EN 1997-1 §12.4(4)P* states that the construction process shall be specified, i.e. in the Geotechnical Design Report, to ensure that the bearing resistance is not exceeded or excessive movements do not occur during construction.

6.3.2. LIMIT STATE ANALYSES

Eurocode 7 provides a long list of possible limit states, both ultimate, including GEO and HYD types, and serviceability limit states that should be checked for embankments including the following:

- Loss of overall stability
- Failure in the embankment slope or crest
- Failure by internal erosion
- Failure by surface erosion or scour
- Excessive deformation

Limit states involving adjacent structures, roads and services are also included in the list.

All possible failure modes of an embankment need to be considered. Since embankments are often constructed in different phases, with different load conditions, analyses should be carried out phase by phase and in accordance with the Geotechnical Design Report. The design should show that settlement of an embankment will not cause a serviceability limit state in the embankment or nearby structures or services. The settlement of an embankment should be calculated using the principles of *EN 1997-1, Section 6.6.1, Settlement of foundations.*

6.3.3. MONITORING AND MAINTENANCE

Since the behaviour of embankments on soft ground during construction is usually monitored to ensure failure does not occur, it is often appropriate to use the Observational Method for design. The importance of both supervision and monitoring in the case of embankments is demonstrated by the fact that in *Section 12 of EN 1997-1*, there is a separate *sub-section, 12.7*, with specific provisions for the supervision of the construction of embankments and the monitoring of embankments during and after construction. The only other section of *Eurocode 7* that has specific provisions for both supervision and monitoring is *Section 8* on ground anchorages.

6.4. Hydraulic failure

6.4.1. OVERVIEW AND DEFINITIONS

The provisions of this Section apply to four modes of ground failure induced by pore-water pressure or pore-water seepage, which shall be checked, as relevant:

- failure by uplift (UPL)
- failure by heave (HYD)
- failure by internal erosion (HYD)
- failure by piping (HYD)

As the definitions of hydraulic failures vary considerably in Europe they are defined as follows:

- Failure by uplift (UPL) occurs when pore-water pressure under a structure or a low permeability ground layer becomes larger than the mean overburden pressure (due to the structure and/or the overlying ground layer).
- Failure by heave (HYD) occurs when upwards seepage forces act against the weight of the soil, reducing the vertical effective stress to zero. Soil particles are then lifted away by the vertical water flow and failure occurs (boiling).
- Failure by internal erosion is produced by the transport of soil particles within a soil stratum, at the interface of soil strata, or at the interface between the soil and a structure. This may finally result in regressive erosion, leading to collapse of the soil structure.
- Failure by piping is a particular form of failure, for example of a reservoir, by internal erosion, where erosion begins at the surface, then regresses until a pipe-shaped discharge tunnel is formed in the soil mass or between the soil and a foundation or at the interface between cohesive and non-cohesive soil strata. Failure occurs as soon as the upstream end of the eroded tunnel reaches the bottom of the reservoir (see Fig.6.8.1).

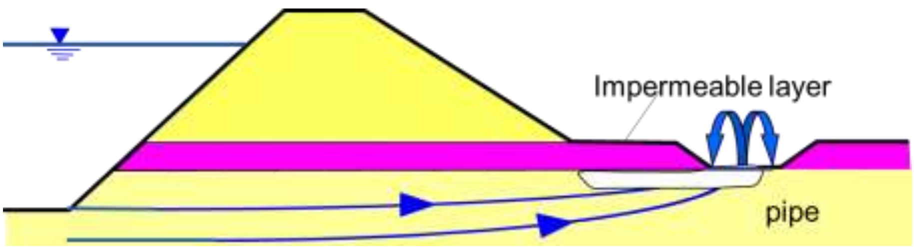

Fig.6.8.1 Example of conditions that may cause piping (*BAW, 2011*)

6.4.2. VERIFICATION OF RESISTANCE TO UPLIFT

According to *EN 1997-1 §2.4.7.4 (1)P* verification for uplift (UPL) shall be carried out by checking that the design value of the combination of destabilising permanent and variable vertical actions $V_{dst;d}$ is less than or equal to the sum of the design value of the stabilising permanent vertical actions $G_{stb;d}$ and of the design value of any additional resistance to uplift R_d (in *EN 1997-1 expression 2.8,* here Eqn 6.1):

$$V_{dst;d} \leq G_{stb;d} + R_d \tag{6.0}$$

where:

$$V_{dst;d} \leq G_{dst;d} + Q_{dst;d}$$

Additional resistance to uplift may also be treated as a stabilising permanent vertical action $G_{stb;d}$.

If a structure is completely below the groundwater level, the water pressure acting on the top of the structure could be regarded as a stabilising action and the water pressure acting on the bottom as a destabilising action. As the stabilising and destabilising actions are multiplied by different partial factor values, the safety against uplift would then depend on the water-depth above the structure. To avoid this misinterpretation the *Note to §2.4.2(9) of EN 1997-1* should be followed: *"Unfavourable (or destabilising) and favourable (or stabilising) permanent actions may in some situations be considered as coming from a single source. If they are considered so, a single partial factor may be applied to the sum of these actions or to the sum of their effects."*

6.4.3. VERIFICATION OF RESISTANCE TO HEAVE

According to *EN 1997-1 §2.4.7.5 (1)P* to avoid the occurrence of a limit state of failure due to heave by seepage of water in the ground, it shall be verified, for every relevant soil column, that the design value of the destabilising total pore water pressure $u_{dst;d}$ at the bottom of the column, or the design value of the seepage force $S_{dst;d}$ in the column is less than or equal to the stabilising total vertical stress $\sigma_{stb;d}$ at the bottom of the column, or the submerged weight $G´_{\text{stb;d}}$ of the same column (*EN 1997-1 expressions 2.9a and 2.9b,* here Eqn.6.2 and Eqn.6.3):

$$u_{dst;d} \leq \sigma_{std;d} \tag{6.0}$$

$$S_{dst;d} \leq G'_{std;d} \tag{6.0}$$

Using the same partial factors, a design with total stresses using Eqn.6.2 provides greater safety than inequality Eqn.6.2. Moreover, using expression Eqn.6.2, the safety becomes dependant of the water depth which is physically not correct. In the next version of *EN 1997-1*, expression Eqn.6.2 needs further explanation so that the hydrostatic water pressure components of the u_{dst} and σ_{stb} are not multiplied by different partial factors as occurs when different partial factors are applied to $u_{dst;k}$ and $\sigma_{stb;k}$.

The determination of the submerged weight $G´_{stb;d}$ and total vertical stress $\sigma_{stb;d}$ at the bottom of the column pose no problems in the verification. The pore water pressure distribution and the seepage force, however, are influenced very strongly by the geometry of the structure and the permeability conditions of the ground. Therefore *EN 1997-1* requires in *§10.3(2)P: "The determination of the characteristic value of the pore-water pressure shall take into account all possible unfavourable conditions, such as:*

- *thin layers of soil of low permeability*
- *spatial effects such as narrow, circular or rectangular excavations below water level"*

This can only be provided by modelling the hydraulic ground conditions by an adequate analytical model based on thorough ground investigations.

6.4.4. INTERNAL EROSION

For internal erosion there is no mathematical formulation of the ultimate limit state. Here *EN 1997-1* states in *§10.4 (1)P "Filter criteria shall be used to assess the danger of material transport by internal erosion. … (5)P If the filter criteria are not satisfied, it shall be verified that the critical hydraulic gradient is well below the design value of the gradient at which soil particles begin to move. (6)P The critical hydraulic gradient for internal erosion shall be established taking into consideration at least the following aspects:*

- *direction of flow*
- *grain size distribution and shape of grains*
- *stratification of the soil"*

The procedure to check the susceptibility of soil to internal erosion is shown in Fig.6..

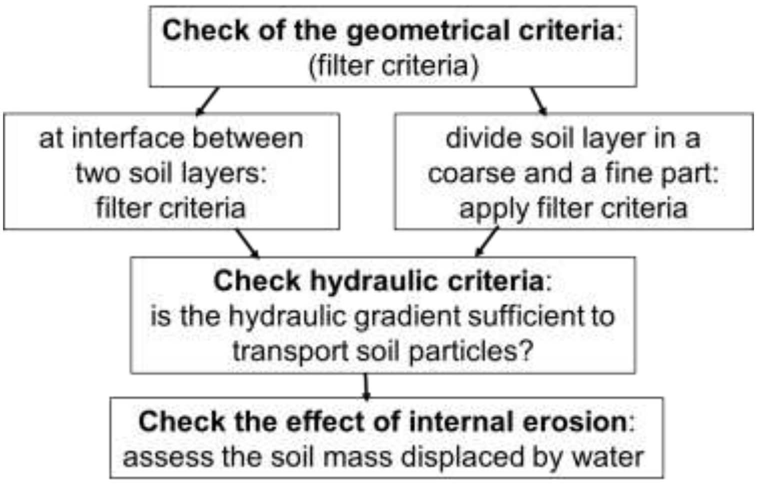

Fig.6.4.2 Steps to check internal erosion (*BAW, 2011*)

6.4.5. FAILURE BY PIPING

Due to the complexity of failure by piping, no mathematical formulation is given in *EN 1997-1* to verify the ultimate limit state. Instead two checks have to be performed. *EN 1997-1 §10.5* prescribes in *(3)P: "Failure by piping shall be prevented by providing sufficient resistance against internal soil erosion in the areas where water outflow may occur"*. If this is not given, a filter layer should be placed on the ground. In the second step it has to be checked that there is *"sufficient safety against failure by heave where the ground surface is horizontal and sufficient stability of the surface layers in sloping ground (local slope stability)" (see EN 1997-1 §10.5(3)P)*. As the hydraulic conditions are strongly influenced by preferred seepage paths due to joints and the interfaces between the structure and the ground, it must be carefully assessed whether a gap and preferred flow path can develop (see Table 6.4.1). These areas or interfaces should be modelled by layers of high permeability. Worked examples can be found in *BAW (2011)*.

Table 6.4.1 Examples for the assessment of the development of gaps or preferred flow paths for structures in dams (*BAW, 2011*)

Interface	No gap	Gaps (cavities) cannot be ruled out
Along driven sheet pile walls	X	
Along vertical, smooth and even walls with a backfilling of cohesionless soil	X	
Beneath cast-in-situ concrete constructed in a dry trench	X	
Between cast-in-situ concrete elements and ground	X	
Between precast elements and ground		X
Beneath slabs with a pile foundation		X

6.5. Summary of key points

6.5.1. SLOPES AND EMBANKMENTS

- *Sections 11 and 12* set out the provisions for the design of slopes and embankments
- The focus in both sections is on the relevant limit states to be checked
- No calculation models are provided
- When using the method of slices for slope stability, some simplified methods are not acceptable
- The relevance and importance of other sections of *EN 1997-1* in the design of embankments is noted, for example:
    - The section on Fill and Ground Improvement
    - The sub-section on the Observational Method
    - The sub-section on the Geotechnical Design Report
    - The section on Supervision and Monitoring

6.5.2. HYDRAULIC FAILURE

- Verification of uplift (UPL)
- Verification of hydraulic heave
- Verification of internal erosion
- Verification of piping

6.6. Worked examples – applying EN1997-1 expressions (2.8) and (2.9) (here Eqn.6.1, Eqn.6.2 and Eqn.6.3)

Worked examples illustrating different approaches to applying expressions Eqn.6.1, Eqn.6.2 and Eqn.6.3 *(expressions (2.8) and (2.9) in 1997-1)* are included in the Annex to this report.

References

BAW, (2011) Code of Practice - Stability of Embankments at German Inland Waterways (downloadable: www.baw.de, > publications, > codes of practice, > MSD (2011))

EN 1997-1: 2004. Eurocode 7: Geotechnical design. Part 1: General rules. CEN.

Frank, R., Bauduin, C., Kavvadas, M., Krebs Ovesen, N., Orr, T., and Schuppener, B. (2004). Designers' guide to EN 1997-1: Eurocode 7: Geotechnical design — General rules, London: Thomas Telford.

Krahn, J. (2004) Stability modelling with SLOPE/W – An Engineering Methodology, GEO-SLOPE/W International Ltd, Canada

Orr, T.L.L. and Farrell E.R. (1998) Geotechnical design to Eurocode 7, Springer-Verlag, London

CHAPTER 7

Retaining structures II

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7.1. Scope

This Chapter concentrates on the design of embedded walls, which are relatively thin walls of steel, reinforced concrete, or timber supported by anchorages, struts, and/or passive earth pressure. The bending capacity of such walls plays a significant role in the support of the retained material. Examples include: cantilever steel sheet pile walls; anchored or strutted steel or concrete sheet pile walls; and diaphragm walls.

Gravity walls are discussed in Chapter 4.

Composite retaining structures are walls composed of elements of the above two types (gravity and embedded). Examples include: double sheet pile wall cofferdams; earth structures reinforced by tendons, geotextiles, or grouting; structures with multiple rows of ground anchorages or soil nails. Parts of this chapter will be relevant to the design of composite walls.

Section 9 of EN 1997-1 applies to retaining structures supporting ground (i.e. soil, rock or backfill) and/or water and is sub-divided as follows:

§7.1. General (6 paragraphs)

§7.2. Limit states (4)

§7.3. Actions, geometrical data and design situations (26)

§7.4. Design and construction considerations (10)

§7.5. Determination of earth pressures (23)

§7.6. Water pressures (5)

§7.7. Ultimate limit state design (26)

§7.8. Serviceability limit state design (14)

Annex C of Eurocode 7 Part 1 'Sample procedures to determine earth pressures' provides informative text relevant to retaining structures and is sub-divided as follows:

1. Limit values of earth pressure (3 paragraphs)
2. Analytical procedure for obtaining limiting active and passive earth pressures (14)
3. Movements to mobilise earth pressures (4)

7.2. Design situations and limit states

Figures 7.2.1, 7.2.2 and 7.2.3 (reproducing here *Figures 9.1, 9.3, 9.4, and 9.6 of EN 1997-1*) illustrate some common limit modes for overall stability and foundation failure of embedded walls.

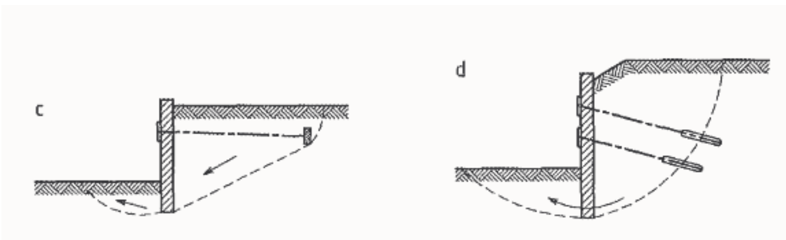

Fig.7.2.1 Limit modes for overall stability of embedded walls

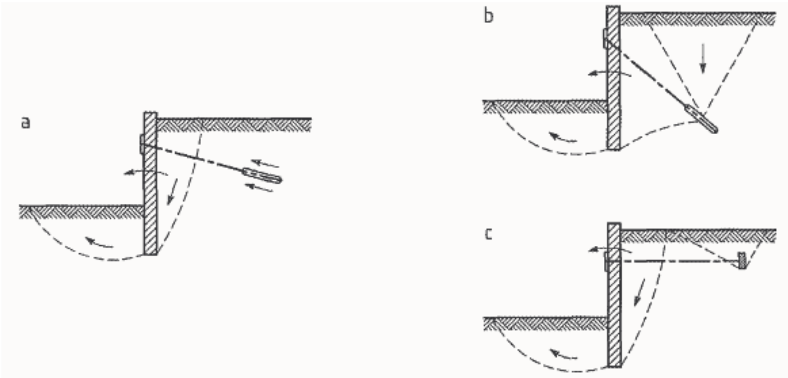

Fig.7.2.2 Limit modes for failure by pull-out of anchors

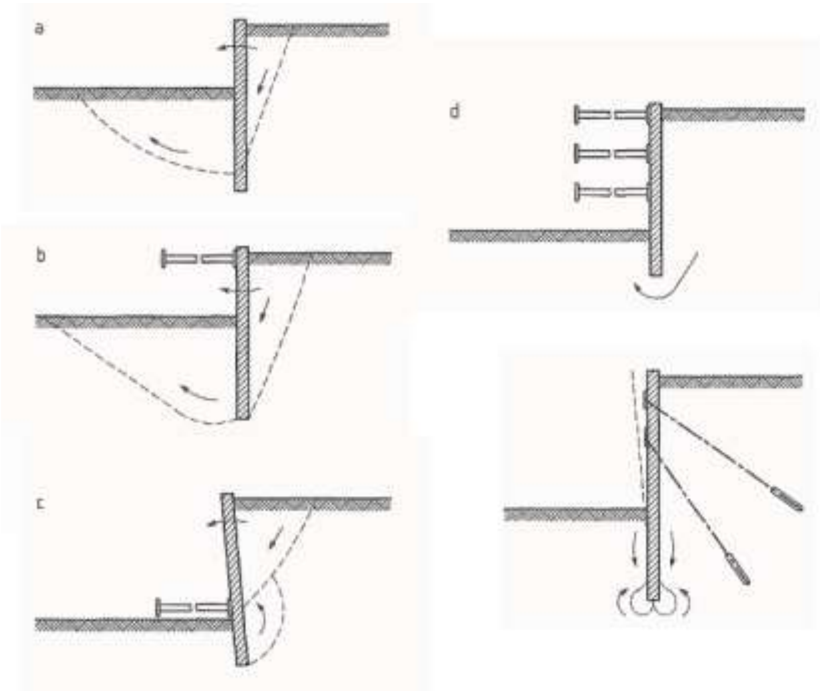

Fig.7.2.3 Limit modes for foundation failures of embedded walls

For ultimate limit state design of embedded walls, the design geometry shall account for anticipated excavation or possible scour in front of the retaining structure. Specifically, with 'normal site control', the retained height of the wall must be increased as follows:

$$H_d = H_{nom} + \Delta H$$

Where H_d = design height of wall; H_{nom} = nominal height of wall and ΔH = allowance for unplanned excavation. Table 7.2.1 below gives the recommended values of ΔH with normal site control in place.

Table 7.2.4 Recommended values of ΔH with normal site control

Wall type	**ΔH for normal site control**
Cantilever	10% H (up to a maximum of 0,5 m)
Supported	10% of height below lowest support (up to a maximum of 0,5 m)

Discussion of *Eurocode 7*'s recommendations regarding water levels and factoring of water pressures is included in Chapter 4 and will not be repeated here.

7.3. Basis of design for embedded walls

According to *EN 1997-1*, ultimate limit states GEO and STR must be verified using one of three Design Approaches, as summarized below. The Design Approaches that have been chosen by different European countries for retaining wall design are shown in Fig.4.3.1.

Table 7.3.5 Summary of Design Approaches

Design Approach	**DA1**		**DA2**		**DA3**
Combination	**1**	**2**	**2**	**2***	
Partial factors applied to:	Actions	Material properties	Actions and resistance	Effects of actions and resistance	Structural actions and resistance
Partial factor Sets*	<u>A1</u>+M1+R1**	A2+<u>M2</u>+R1**	<u>A1</u>+M1+<u>R2</u>**	<u>A1</u>+M1+<u>R2</u>**	<u>A1</u>/A2+<u>M2</u>+R3**
* Sets A1-A2 = factors on actions/effects of actions Sets M1-M2 = factors on material properties Sets R1-R3 = factors on resistances ** in <u>underlined</u> sets, factors are > 1,0					

Fig.7.3.1 National choice of Design Approach for retaining walls (after *Bond, 2013)*

The values of the partial factors that are recommended by *EN 1997-1* for Design Approach 1 Combinations 1 and 2 (DA1-1 and DA1-2) are summarized in Table 7.3.2 below.

Table 7.3.2 Values of the partial factors for Design Approach 1

<table>
<tr><th colspan="2" rowspan="2">Parameter</th><th rowspan="2">Symbol</th><th colspan="3">DA1-1</th><th colspan="3">DA1-2</th></tr>
<tr><th>A1</th><th>M1</th><th>R1</th><th>A2</th><th>M2</th><th>R1</th></tr>
<tr><td rowspan="2">Permanent action (G)</td><td>Unfavourable</td><td>γ_G</td><td>1,35</td><td rowspan="4"></td><td rowspan="4"></td><td rowspan="2">1,0</td><td rowspan="4"></td><td rowspan="4"></td></tr>
<tr><td>Favourable</td><td>($\gamma_{G,fav}$)</td><td>1,0</td></tr>
<tr><td rowspan="2">Variable action (Q)</td><td>Unfavourable</td><td>γ_Q</td><td>1,5</td><td>1,3</td></tr>
<tr><td>Favourable</td><td>-</td><td>(0)</td><td>(0)</td></tr>
<tr><td colspan="2">Shearing resistance (tan φ)</td><td>γ_φ</td><td rowspan="5"></td><td rowspan="5">1,0</td><td rowspan="5"></td><td rowspan="5"></td><td rowspan="2">1,25</td><td rowspan="5"></td></tr>
<tr><td colspan="2">Effective cohesion (c')</td><td>γ_c</td></tr>
<tr><td colspan="2">Undrained shear strength (c_u)</td><td>γ_{cu}</td><td rowspan="2">1,4</td></tr>
<tr><td colspan="2">Unconfined compressive strength (q_u)</td><td>γ_{qu}</td></tr>
<tr><td colspan="2">Weight density (γ)</td><td>γ_γ</td><td>1,0</td></tr>
<tr><td colspan="2">Bearing resistance (R_v)</td><td>γ_{Rv}</td><td rowspan="3"></td><td rowspan="3"></td><td rowspan="3">1,0</td><td rowspan="3"></td><td rowspan="3"></td><td rowspan="3">1,0</td></tr>
<tr><td colspan="2">Sliding resistance (R_h)</td><td>γ_{Rh}</td></tr>
<tr><td colspan="2">Earth resistance (R_e)</td><td>γ_{Re}</td></tr>
<tr><td colspan="9">Factors given for persistent and transient design situations</td></tr>
</table>

The values of the partial factors that are recommended by EN 1997-1 for Design Approaches 2 and 3 (DA2 and DA3) are summarized below.

Table 7.3.3 Values of the partial factors for Design Approaches 2 and 3

<table>
<tr><th colspan="2" rowspan="2">Parameter</th><th rowspan="2">Symbol</th><th colspan="3">DA2/DA2*</th><th colspan="4">DA3</th></tr>
<tr><th>A1</th><th>M1</th><th>R2</th><th>A1#</th><th>A2*</th><th>M2</th><th>R3</th></tr>
<tr><td rowspan="2">Permanent action (G)</td><td>Unfavourable</td><td>γ_G</td><td>1,35</td><td rowspan="4"></td><td rowspan="4"></td><td rowspan="2">1,35</td><td rowspan="2">1.0</td><td rowspan="4"></td><td rowspan="4"></td></tr>
<tr><td>Favourable</td><td>($\gamma_{G,fav}$)</td><td>1,0</td></tr>
<tr><td rowspan="2">Variable action (Q)</td><td>Unfavourable</td><td>γ_Q</td><td>1,5</td><td>1,5</td><td>1.3</td></tr>
<tr><td>Favourable</td><td>-</td><td>(0)</td><td colspan="2">(0)</td></tr>
<tr><td colspan="2">Shearing resistance (tan φ)</td><td>γ_φ</td><td rowspan="2"></td><td rowspan="2">1,0</td><td rowspan="2"></td><td rowspan="2"></td><td rowspan="2"></td><td rowspan="2">1,25</td><td rowspan="2"></td></tr>
<tr><td colspan="2">Effective cohesion (c')</td><td>γ_c</td></tr>
</table>

<table>
<tr><th rowspan="2">Parameter</th><th rowspan="2">Symbol</th><th colspan="3">DA2/DA2*</th><th colspan="4">DA3</th></tr>
<tr><th>A1</th><th>M1</th><th>R2</th><th>A1#</th><th>A2*</th><th>M2</th><th>R3</th></tr>
<tr><td>Undrained shear strength (c_u)</td><td>γ_{cu}</td><td></td><td></td><td></td><td></td><td></td><td rowspan="2">1,4</td><td></td></tr>
<tr><td>Unconfined comp. str. (q_u)</td><td>γ_{qu}</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Weight density (γ)</td><td>γ_γ</td><td></td><td></td><td></td><td></td><td></td><td>1,0</td><td></td></tr>
<tr><td>Bearing resistance (R_v)</td><td>γ_{Rv}</td><td></td><td></td><td>1,4</td><td></td><td></td><td></td><td rowspan="4">1,0</td></tr>
<tr><td>Sliding resistance (R_h)</td><td>γ_{Rh}</td><td></td><td></td><td>1,1</td><td></td><td></td><td></td></tr>
<tr><td>Earth resistance (R_e) walls</td><td rowspan="2">γ_{Re}</td><td></td><td></td><td>1,4</td><td></td><td></td><td></td></tr>
<tr><td>Earth resistance (R_e) slopes</td><td></td><td></td><td>1,1</td><td></td><td></td><td></td></tr>
<tr><td colspan="9">Factors given for persistent and transient design situations
#Applied to structural actions; *applied to geotechnical actions</td></tr>
</table>

7.4. Verification of ultimate limit state GEO

Limiting equilibrium

Annex C of EN 1997-1 (+Corrigendum 1) gives expressions for active and passive earth pressures:

$$\sigma_a = K_a\left(\int_0^z \gamma dz + q - u\right) - 2c\sqrt{K_a(1 + a/c)} + u$$

$$\sigma_p = K_p\left(\int_0^z \gamma dz + q - u\right) + 2c\sqrt{K_p(1 + a/c)} + u$$

where

σ_a, σ_p = active, passive stresses normal to the wall;

K_a, K_p = horizontal active, passive earth pressure coefficients;

γ = weight density of retained ground;

c = ground cohesion;

q = vertical surface load;

z = distance down face of wall;

a = wall adhesion;

u = pore water pressure.

The earth pressures that act on an embedded wall may be determined using the charts given in *Annex C of EN 1997-1*, which were developed by (but are not attributed to) *Kerisel* and *Absi*. They are the same charts as appear in *BS 8002*. *Kerisel* and *Absi* assumed log-spiral failure surfaces and hence their charts give upper bound values of K_a and K_p. The charts can only be used for walls that are vertical.

Alternatively, the following expressions developed by *Brinch Hansen* (also given in *Annex C of EN 1997-1*) may be used:

$$\sigma'_a = K_{a\gamma}\left(\int_0^z \gamma dz - u\right) + K_{aq}q - K_{ac}c; \qquad \sigma'_p = K_{p\gamma}\left(\int_0^z \gamma dz - u\right) + K_{pq}q + K_{pc}c$$

$$\left.\begin{matrix} K_{a\gamma} \\ K_{p\gamma} \end{matrix}\right\} = K_n \cdot \cos\beta \cdot \cos(\beta - \theta); \qquad \left.\begin{matrix} K_{aq} \\ K_{pq} \end{matrix}\right\} = K_n \cdot \cos^2\beta; \qquad \left.\begin{matrix} K_{ac} \\ K_{pc} \end{matrix}\right\} = (K_n - 1) \cdot \cot\varphi$$

$$K_n = \frac{1 \pm \sin\varphi \cdot \sin(2m_w \pm \varphi)}{1 \mp \sin\varphi \cdot \sin(2m_t \pm \varphi)} e^{\pm 2(m_t + \beta - m_w - \theta)\tan\varphi}$$

$$2m_t = \cos^{-1}\left(\frac{-\sin\beta}{\pm\sin\varphi}\right) \mp \varphi - \beta; \qquad 2m_w = \cos^{-1}\left(\frac{\sin\delta}{\sin\varphi}\right) \mp \varphi \mp \delta$$

Bond and Harris (2008) have published charts (for vertical walls only) that make these expressions easy to evaluate. Fig.7..1 shows one of these charts.

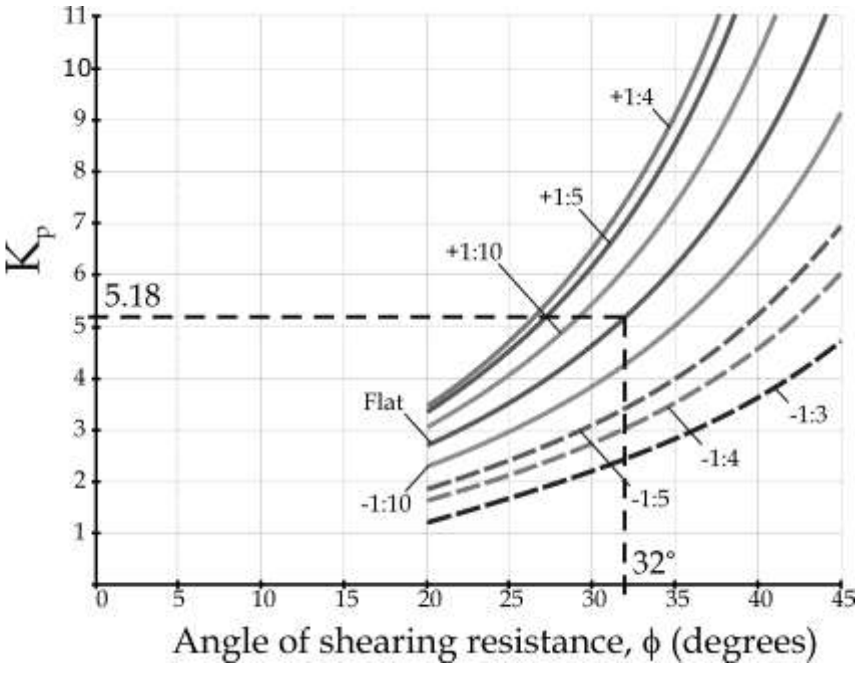

Fig.7.4.1 Chart showing *Brinch Hansen's* passive earth pressure coefficient (after *Bond & Harris, 2008*)

A key parameter when using these charts is the value chosen for the design angle of interface friction δ_d. *Eurocode 7* suggests δ_d should be determined from soil's design constant-volume angle of shearing resistance $\varphi_{cv,d}$ using the expression:

$$\delta_d = k\varphi_{cv,d} = k\tan^{-1}\left(\frac{\tan\varphi_{cv,k}}{\gamma_\varphi}\right)$$

where

k is a constant (= 1 for soil against cast in-situ concrete or ⅔ for soil against precast concrete);

$\varphi_{cv,k}$ is the soil's characteristic constant-volume angle of shearing resistance;

γ_φ is a partial factor.

Advocates of critical state soil mechanics would argue that it is unnecessary to apply a partial factor to $\varphi_{cv,k}$ in this expression, provided its value is chosen carefully, since it is already a pessimistic value. For this reason, the *UK National Annex* states that *'It might be more appropriate to select the design value of φ_{cv} directly'*. A future version *Eurocode 7* might propose a reduced value for the partial factor, for example:

$$\delta_d = k\varphi_{cv,d} = k\tan^{-1}\left(\frac{\tan\varphi_{cv,k}}{\gamma_{\varphi,cv}}\right)$$

with $\gamma_{\varphi,cv}$ = 1,1 (for example).

There is some ambiguity in the interpretation of *Eurocode 7* with regards to the treatment of passive earth pressure. Most engineers' first instinct is to regard passive earth pressure as a resistance – in which case its characteristic value $\sigma_{p,k}$ is divided by a resistance factor γ_{Re} to obtain the design value:

$$\sigma_{p,d} = \frac{\sigma_{p,k}}{\gamma_{\mathrm{Re}}}$$

Typically, in Design Approach 2, γ_{Re} = 1,4.

However, the *NOTE* to *clause 2.4.2(9) of EN 1997-1* states:

> *"Unfavourable (or destabilising) and favourable (stabilising) permanent actions may in some situations be considered as coming from a single source. If … so, a single partial factor may be applied to the sum of these actions or to the sum of their effects"*

This gives the potential to treat passive earth pressure as an action, in which case it may be classified as favourable, that is:

$$\sigma_{p,d} = \gamma_{G,fav}\sigma_{p,k}$$

with $\gamma_{G,fav}$ = 1,0; or it may be classified as unfavourable, in which case:

$$\sigma_{p,d} = \gamma_G\sigma_{p,k}$$

with γ_G = 1,35, typically.

One other treatment is possible – when material properties are factored but actions and resistances are not (as in Design Approach 1, Combination 2) – then:

$$\sigma_{p,d} = \sigma_{p,k}$$

Bond and Harris (2008) studied the effects of these different options and concluded – for Design Approach 1 at least – that treating passive pressure as an unfavourable action, although the least intuitive option, is the most consistent choice. This is equivalent to applying the single source principle and considering the action to be the pressure distribution obtained from the algebraic sum of active and passive earth pressures.

Fig.4..2 illustrates the movement *v* needed for the ground next to an embedded wall to reach a state of active or passive failure, expressed as a percentage of the wall height *h*. When movements are less than that needed for active conditions, consideration should be given to using at-rest earth pressures in the wall's design.

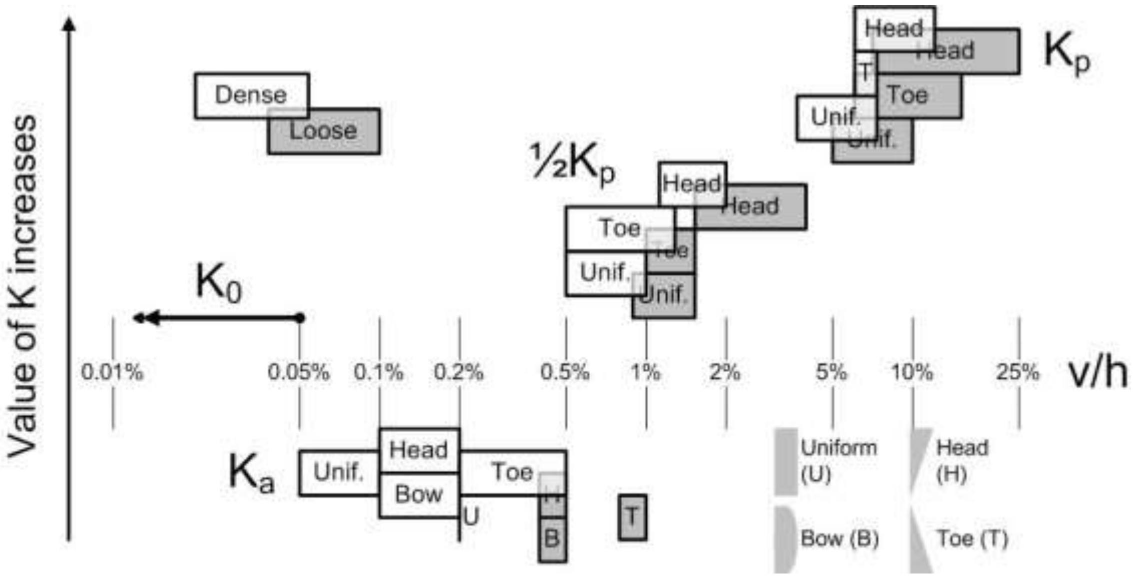

Fig.7.9.2 Normalized movement *v/h* needed to trigger limit states (after *Bond & Harris, 2008*)

7.5. Verification of serviceability

Eurocode 7 requires a cautious estimate of the distortion and displacement of retaining walls, and the effects on supported structures and services, to be made on the basis of comparable experience. This estimate must include the effects of the construction of the wall. The design may be justified by checking that the estimated displacements do not exceed the limiting values (*EN 1997-1 §9.8.2(2)P*).

Displacement calculations must be undertaken:

- where nearby structures and services are unusually sensitive to displacement
- where comparable experience is not well established

Displacement calculations should be considered where the wall:

- retains more than 6 m of cohesive soil of low plasticity
- retains more than 3 m of soils of high plasticity
- is supported by soft clay within its height or beneath its base

This should be interpreted as recommending an explicit calculation for displacements should undertaken whenever the stiffness of the retained or resisting soil is likely to cause unacceptable movements of nearby structures or services.

7.6. Supervision, monitoring, and maintenance

Eurocode 7 does not give specific rules for supervision, monitoring, and maintenance of embedded walls. Instead there are several execution standards that provide the necessary recommendations:

- EN 1536 covers bored piles
- EN 1537 covers ground anchors
- EN 1538 covers diaphragm walls
- EN 12063 covers sheet pile walls

7.7. Summary of key points

The design of embedded walls to *Eurocode 7* involves:

- checking the vertical bearing capacity of the wall
- reducing wall friction owing to vertical loads
- stability calculations based on limiting equilibrium, soil-structure interaction, or fully numerical methods adopting continuous model for the soil and the wall
- careful thought about the way passive earth pressures should be handled – as a resistance, as a favourable action, or as an unfavourable action (invoking the single source principle)

7.8. Worked example – anchored sheet pile wall

A worked example illustrating the way in which an anchored sheet pile wall may be designed according to *Eurocode 7* is included in the Annex to this report.

References

Bond, A. J. (2013). Implementation and evolution of Eurocode 7, Modern Geotechnical Design Codes of Practice, Arnold et al. (eds), Amsterdam: IOS Press, pp3-14.

Bond, A. J., and Harris, A. J. (2008). Decoding Eurocode 7, London: Taylor & Francis, 618pp.

EN 1997-1: 2004. Eurocode 7: Geotechnical design. Part 1: General rules. CEN.

CHAPTER 8

Deep foundations

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8.1. Scope

This Chapter examines the provisions in *Section 7* of *EN 1997-1* for the design of pile foundations. These provisions apply to end-bearing piles, friction piles, tension piles and transversely loaded piles. They apply to piles installed by driving, jacking and screwing or boring, and piles installed with or without grouting. It is noted that the provisions of *Section 7* should not be applied directly to piles that are intended as settlement reducers, as in some piled raft foundations.

Reference is made in *Eurocode 7* to other CEN standards that are relevant to the design of pile foundations. A design standard that is referred to is the part of *Eurocode 3* for the structural design of steel piles:

- *EN 1993-5: Eurocode 3, Part 5: Design of Steel Structures – Piling*

Reference is also made to the following execution standards for the carrying out of piling work:

- *EN 1536:1999 - Bored Piles*
- *EN 12063:1999 - Sheet pile walls*
- *EN 12699:2000 - Displacement piles*
- *EN 14199:2005 - Micropiles*

Another CEN standard that is relevant to the design of piles but not referred to in *Eurocode 7* is the material standard:

- *EN 12794:2005 – Precast concrete products. Foundation piles.*

Section 7 of EN 1997-1 is sub-divided as follows:

§7.1. General (3 paragraphs)

§7.2. Limit states (1)

§7.3. Actions and design situations (18)

§7.4. Design methods and design considerations (8)

§7.5. Pile load tests (20)

§7.6. Axially loaded piles (89)

§7.7. Transversely loaded piles (15)

§7.8. Structural design of piles (5)

§7.9. Supervision of construction (8)

As this list of sub-sections shows, the largest sub-section is sub-section 6 on axially loaded piles, which are the most common type of pile, with 89 out of 167 of the paragraphs in *Section 7*, i.e. 53%. This chapter focuses on the design of axially loaded piles, in the section on ultimate limit state design and in the design examples. Another feature of *Section 7* is that, with 20 pages, it is the longest section of *Eurocode 7*.

8.2. Design situations and limit states

When selecting the design situations for piles, it is necessary to consider the different types of actions to which piles may be subjected. Piles may be loaded axially and/or transversely. They may also be loaded due to displacement of the surrounding soil. This may be due to:

- Consolidation
- Swelling
- Adjacent loads
- Creeping soil
- Landslides, or
- Earthquakes

The loads from the surrounding soil due to these causes need to be considered as they can affect piles by causing downdrag (negative skin friction), heave, stretching, transverse loading and displacement. According to *EN 1997-1 §7.3.3(1)P 1*, if ultimate limit state design calculations are carried out with the downdrag load as an action, its value shall be the maximum, which could be generated by the downward movement of the ground relative to the pile. Usually, in this situation, the design values of the strength and stiffness are upper values, which is a more conservative approach. However, although *EN 1997-1* indicates that downdrag can be included in ultimate limit state calculations, normally downdrag is only relevant in serviceability limit states, causing additional pile settlement.

The limit states that need to be considered in the design of piles are the following:

- Loss of overall stability
- Bearing resistance failure of the pile foundation
- Uplift or insufficient tensile resistance of the pile foundation
- Failure in the ground due to transverse loading of the pile foundation
- Structural failure of the pile in compression, tension, bending, buckling or shear
- Combined failure in the ground and in the pile foundation
- Combined failure in the ground and in the structure
- Excessive settlement
- Excessive heave
- Excessive lateral movement
- Unacceptable vibrations

One of the special features of *Eurocode 7* is that it provides checklists, which are lists of factors to be taken into account or considered in a particular design situation. In the design of piles, a checklist is provided of the factors that affect the selection of the type of pile. *Orr and Farrell (1998)* have presented this checklist in the form of Table 8.3.1 with a column to be ticked when each particular factor has been considered.

8.3. Approaches to pile design and static load tests

EN 1997-1 §7.4(1)P states that the design of piles shall be based on one of the following approaches:

- The results of static load tests, which have been demonstrated, by means of calculations or otherwise, to be consistent with other relevant experience
- Empirical or analytical calculation methods whose validity has been demonstrated by static load tests in comparable situations
- The results of dynamic load tests whose validity has been demonstrated by static load tests in comparable situations

- The observed performance of a comparable pile foundation, provided that this approach is supported by the results of site investigation and ground testing

Table 8.3.1 Checklist of factors affecting selection of pile type (after *Orr and Farrell, 1999*)

Selection of pile type	Checked
The ground and ground-water conditions, including the presence or possibility of obstructions in the ground	
The stresses generated in the pile during installation	
The possibility of preserving and checking the integrity of the pile being installed	
The effect of the method and sequence of pile installation on piles, which have already been installed and on adjacent structures or services	
The tolerances within which the pile can be installed reliably	
The deleterious effects of chemicals in the ground	
The possibility of connecting different ground-water regimes	
The handling and transportation of piles	

The use of static pile load tests for pile design has been underlined in the above text to highlight the emphasis and importance placed on static pile load tests in the design of piles to *Eurocode 7* since the first three approaches all refer to static load tests while the fourth approach refers to observed performance and ground testing. *Eurocode 7* provides the following requirements concerning static load tests:

- Static load tests may be carried out on trial piles, installed for test purposes only, before the design is finalised, or on working piles, which form part of the foundation *(§7.4.1(3))*
- If one pile load test is carried out, it shall normally be located where the most adverse ground conditions are believed to occur *(§7.5.1(4)P)*
- Between the installation of the test pile and the beginning of the load test, adequate time shall be allowed to ensure that the required strength of the pile material is achieved and the pore-water pressures have regained their initial values *(§7.5.1(6)P)*

8.4. Ultimate limit state design of piles

8.4.1. EQUILIBRIUM EQUATION

The equilibrium equation to be satisfied in the ultimate limit state design of axially loaded piles in compression is:

$$F_{c;d} \leq R_{c;d}$$

where $F_{c;d}$ is the design axial compression load and $R_{c;d}$ is the pile compressive design resistance.

8.4.2. DESIGN AXIAL LOAD

The design axial compressive load $F_{c;d}$ is obtained by multiplying the representative permanent and variable loads, G and Q by the corresponding partial action factors γ_G and γ_Q:

$$F_{c;d} = \gamma_G G_{rep} + \gamma_Q Q_{rep}$$

The two sets of recommended partial factors on actions and the effects of actions provided in Table 8.4.1 (representing *Table A3 of Annex A of EN 1997-1*).

Table 8.4.1 Recommended partial factors on actions and effects of actions

Action		**Symbol**	**Set**	
			A1	**A2**
			DA1.C1, DA2 DA3 (structural actions)	**DA1.C2 DA3 (geotech.actions)**
Permanent	Unfavourable	γ_G	1,35	1,0
	Favourable		1,0	1,0
Variable	Unfavourable	γ_Q	1,5	1,3
	Favourable		0	0

The self-weight of the pile should be included when calculating the design axial compressive load, $F_{c;d}$, along with any downdrag, heave or transverse loading. However the common practice of assuming that the weight of the pile is balanced by that of the overburden allowing both to be excluded from $F_{c;d}$ and $R_{c;d}$ is permitted, where appropriate. The pile weight may not cancel the weight of the overburden if:

f) The downdrag is significant

g) The soil is light

h) The pile extends above the ground surface

8.4.3. CHARACTERISTIC PILE RESISTANCE

Eurocode 7 describes three procedures for obtaining the characteristic compressive resistance $R_{c;k}$ of a pile:

a) Directly from static pile load tests

b) By calculation from profiles of ground test results

c) By calculation from ground parameters

In the case of procedures a) and b) *Eurocode 7* provides correlation factors to convert the measured pile resistances or pile resistances calculated from profiles of test results into characteristic resistances. In the case of procedure c), the characteristic pile resistance is calculated from ground parameter values. This is referred to as the “alternative procedure” in the *Note to EN 1997-1 §7.6.2.3(8)*, even though it is the most common method in some countries, for example in Ireland and the UK. Outlines of worked pile design examples using these three procedures are given in *Sections 8, 9 and 10* of this Chapter with links to the complete solutions of these worked examples.

a) Characteristic pile resistance from static load tests

For piles in compression, it is often difficult to identify the ultimate limit state from static load test results because the pile load-settlement plot shows a continuous curvature. In these cases, *Eurocode 7* states that the settlement of the pile top equal to 10% of the pile base diameter should be adopted as the "failure" criterion. The characteristic pile resistance $R_{c;k}$ is then determined directly (i.e. not estimated) from the measured pile resistance $R_{c;m}$ values (ultimate limit state resistances) by applying correlation factors, ξ_1 and ξ_2 , related to number of piles tested, to the mean and minimum measured resistances according to equation:

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

The recommended values for ξ_1 and ξ_2 for n pile load tests, given in *Table A.9 of EN 1997-1 Annex A*, are shown in Table 8.4.1.

Table 8.4.1 Recommended correlation factors ξ_1 and ξ_2 to determine characteristic pile resistance from pile load test results

n	1	2	3	4	≥ 5
ξ_1	1,4	1,3	1,2	1,1	1,0
ξ_2	1,4	1,2	1,05	1,0	1,0

The values for ξ_1 and ξ_2 in Table 8.4.1 show the advantage of carrying out more load tests since the correlation values reduce as the number of load tests increases so that higher $R_{c;k}$ values are determined. For structures which have sufficient stiffness to transfer loads from 'weak' to 'strong' piles, the ξ values may be divided by 1,1 provided they are not less than 1,0.

a) Characteristic pile resistance from profiles of ground test results

Part 2 of EN 1997 includes the following Annexes with methods to calculate the compressive resistance, $R_{c;cal}$ of a single pile from profiles of ground test results:

- D.6 *Example of a correlation between compressive resistance of a single pile and cone penetration resistance* - Tables are provided in this Annex relating the pile's unit base resistances p_b at different normalised pile settlements, *s/D*, and the shaft resistance p_s to average cone penetration resistance q_c values. The values in *Tables D.3* and *D.4* are used to calculate the pile base and shaft resistances in pile (Example 2).
- D.7 *Example of a method to determine the compressive resistance of a single pile* - Equations are provided in this Annex to calculate the maximum base resistance and shaft resistance from the q_c values obtained from an electrical CPT.
- E.3 *Example of a method to calculate the compressive resistance of a single pile* - An equation is provided in this Annex to calculate the ultimate compressive resistance of a pile from the results of an MPM test.

When a number of profiles of tests are carried out, e.g. CPTs, the characteristic total pile compressive resistance $R_{c;k}$ or the base and shaft resistances $R_{b;k}$ and $R_{s;k}$, may be determined directly by applying correlation factors ξ_3 and ξ_4 to the set of pile resistances calculated from the test profiles using, for example, the methods referred to in the Annexes above. This procedure is referred to as the Model Pile procedure by *Frank et al. (2004)*. The values of the correlation factors ξ_3 and ξ_4, depend on the number of test profiles, *n* and they are applied to the mean and minimum $R_{c;cal}$ values according to the following equation:

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

The recommended values for ξ_3 and ξ_4 for *n* profiles of test results, given in *Table A.10 of EN 1997-1*, are shown in Table 8.4.2 and differ from the ξ factor values for design from pile load tests.

Table 8.4.2 Recommended correlation factors ξ_3 and ξ_4 to determine characteristic pile resistance from ground test results

n	1	2	3	4	5	7	10
ξ_3	1,4	1,35	1,33	1,31	1,29	1,27	1,25
ξ_4	1,4	1,27	1,23	1,20	1,15	1,12	1,08

b) Characteristic pile resistance from ground parameters

The characteristic base and shaft resistances may also be determined directly from ground parameters using the following equations given in *EN 1997-1 §7.6.2.3(8)*:

$$R_{b;k} = A_b \times q_{b;k}$$

$$R_{s;k} = \sum A_{s;i} \times q_{si;k}$$

where:

A_b = the nominal plan area of the base of the pile;

A_{si} = the nominal surface area of the pile in soil layer *i*

$q_{b;k}$ = the unit base resistance

$q_{si;k}$ = the unit shaft resistance in soil layer *i*

In this alternative procedure, the characteristic unit pile resistances $q_{b;k}$ and $q_{s;k}$ are calculated using appropriate values for the ground parameters. These would normally be cautious values, i.e. characteristic values. In a note to *EN 1997-1 §7.6.2.3(8)*, it is stated that when the alternative procedure is used to calculate the design pile compressive or tensile resistance, the partial factors γ_b and γ_s may need to be corrected by a model factor larger than 1,0. A number of countries, such as Ireland as explained in Section 8.6.2 below, have introduced such a model factor.

8.4.4. DESIGN COMPRESSIVE RESISTANCE

The design compressive resistance of a pile $R_{c;d}$ may be obtained either by treating the pile resistance as a total resistance or by separating it into base and shaft components. If the pile resistance is treated as a total resistance, the design pile resistance is obtained by dividing the characteristic total resistance $R_{c;k}$ by the relevant partial factor γ_t:

$$R_{c;d} = R_{c;k} / \gamma_t$$

If the pile resistance is separated into base and shaft components, the design resistance is obtained by dividing the characteristic base and shaft resistances, $R_{b;k}$ and $R_{s;k}$, by the relevant partial factors, γ_b and γ_s:

$$R_{c;d} = R_{b;k}/\gamma_b + R_{s;k}/\gamma_s$$

EN 1997-1 provides different sets of recommended partial resistance factor values for driven, bored and CFA piles in *Tables A6, A7* and *A8* of *Annex A.* These are shown in Table 8.4.3. The following features should be noted concerning these partial resistance factor values:

- The R1 partial factor values are greater than 1,0 for bored and CFA piles in compression, but are equal to 1,0 for driven piles
- The R2 partial factors are the same for all three different types of pile
- The R3 partial factors are all equal to 1,0 for all three different types of pile
- The R4 partial factors are all greater than 1,0 and greater than the R2 partial factor values
- For piles in tension, only the shaft resistance factor is relevant and this has the same value for all three types of pile

Table 8.4.3 Recommended partial resistance factors for driven, bored and CFA piles

Resistance	**Symbol**	**Set**			
		R1	**R2**	**R3**	**R4**
Partial resistance factors for driven piles					
Base	γ_b	1,0	1,1	1,0	1,3
Shaft (compression)	γ_s	1,0	1,1	1,0	1,3
Total/combined (compression)	γ_t	1,0	1,1	1,0	1,3
Shaft in tension	$\gamma_{s;t}$	1,25	1,15	1,1	1,6
Partial resistance factors for bored piles					
Base	γ_b	1,25	1,1	1,0	1,6
Shaft (compression)	γ_s	1,0	1,1	1,0	1,3
Total/combined (compression)	γ_t	1,15	1,1	1,0	1,5
Shaft in tension	$\gamma_{s;t}$	1,25	1,15	1,1	1,6
Partial resistance factors for continuous flight auger (CFA) piles					
Base	γ_b	1,1	1,1	1,0	1,45
Shaft (compression)	γ_s	1,0	1,1	1,0	1,3
Total/combined (compression)	γ_t	1,1	1,1	1,0	1,4
Shaft in tension	$\gamma_{s;t}$	1,25	1,15	1,1	1,6

The combinations of sets of partial factor values that should be used for Design Approach 1 are as follows:

DA1.C1: A1 "+" M1 "+" R1

DA1.C2: A2 "+" M1 or M2 "+" R4

It should be noted that, unlike in the case of all other geotechnical design situation, in the design of pile foundations, DA1 is a partial resistance factor rather than a material factor approach since for both C1 and C2 the design resistance is obtained by applying the partial resistance factor Sets R1 or R4, which are mostly ≥ 1,0, to the characteristic base and shaft resistances, and applying the partial material factor Set M1 = 1,0 to the ground parameters; Set M2 > 1,0 is only used for C2 to calculate unfavourable design actions on piles owing e.g. to negative skin friction.

The combinations of sets of partial factor values that should be used for Design Approach 2 are as follows:

DA1.C1: A1 "+" M1 "+" R2

As in the case of Design Approach 1, this is a partial resistance factor approach as Set M1 = 1.0, but the Set R2 values differ from the Set R1 values.

The combinations of sets of partial factor values that should be used for Design Approach 3 are as follows:

DA1.C1: (A1* or A2†) "+" M2 "+" R3

where * and † indicate that the partial action factor Set A1 is applied to structural actions and A2 is applied to geotechnical actions. Since all the R3 values are unity, DA3 should not be used for piles designed from pile load tests or from resistances calculated from profiles of test results as it provides no safety on the resistance.

8.4.5. PILE GROUPS

Piles in a group should be checked for failure of the piles individually and acting as a block. The design resistance shall be taken as the lower value caused by these two mechanisms. Generally a pile block can be analysed as a single large diameter pile. The strength and stiffness of the structure connecting the piles shall be considered. For a stiff structure, advantage may be taken of load redistribution. A limit state will occur only if a significant number of piles fail together; so failure involving only one pile need not be considered. In the case of flexible structures, the weakest pile governs the occurrence of a limit state in the structure. Special attention needs to be given to failure of edge piles by inclined or eccentric loads.

8.5. Serviceability limit state design

The small amount in *Eurocode 7* on the SLS design of compression piles is provided in *§7.6.4* which is called *Vertical displacement of pile foundations (Serviceability of supported structure)*. This includes the Principle that vertical displacements under serviceability limit state conditions shall be assessed and checked against the requirements given. This is a direct design method as it involves checking the serviceability limit state by calculating the pile displacements. However, the application rules to this Principle states that it should not be overlooked that in most cases calculations will provide only an approximate estimate of the displacements of the pile foundation.

§7.6.4.1(2) has the important note that for piles bearing in medium-to-dense soils and for tension piles, the safety requirements (i.e. partial factors) for the ultimate limit state design are normally sufficient to prevent the occurrence of a serviceability limit state in the supported structure. Preventing the occurrence of a serviceability limit state by means of the ultimate limit state partial factors is an indirect design method. Since it is often difficult to predict the settlement of a pile foundation, many countries have adopted this indirect design method for pile foundations and have either increased the ξ values for the ULS design of piles or have included a model factor so as to satisfy the SLS (settlement) as well as the ULS (safety against failure) requirements.

8.6. Pile design in Ireland

8.6.1. PILE DESIGN IN IRELAND FROM GROUND TEST RESULTS

In Ireland it was considered necessary to increase the ξ values in *Table A.10, EN 1997-1* to allow for uncertainties in deriving the characteristic compressive and tensile resistances when designing piles from ground test results. The recommended ξ values given in *Table A.10* and the increased values given in *Table NA.3 of the Irish National Annex (NSAI, 2005)* are shown in Table 8.6.1 It can be seen from these values that all the recommended ξ values have effectively been increased by a model factor of 1,5 in the Irish National Annex.

Table 8.6.1 Recommended correlation factors ξ_3 and ξ_4 and values in the Irish National Annex to determine characteristic pile resistance from ground test results

Number of profiles of tests, *n*		**1**	**2**	**3**	**4**	**5**	**7**	**10**
ξ_3 values on mean calculated resistance	Recommended	1,4	1,35	1,33	1,31	1,29	1,27	1,25
	Irish National Annex	2,1	2,03	2,0	1,97	1,94	1,91	1,88
ξ_4 values on minimum calculated resistance	Recommended	1,4	1,27	1,23	1,20	1,15	1,12	1,08
	Irish National Annex	2,1	1,91	1,85	1,80	1,73	1,68	1,62

8.6.2. PILE DESIGN IN IRELAND USING THE ALTERNATIVE PROCEDURE

In Ireland, in order to allow for uncertainties in deriving the characteristic compressive resistances from ground parameters, it was considered necessary, in accordance with *EN 1997-1 §7.6.2.3(8)* as noted in Section 8.4.3, to correct the recommended partial resistance factors by applying a model factor of 1,75 to the γ_b and γ_s values (*NSAI, 2005*). Hence designing piles in Ireland using the alternative procedure with the model factor of 1,75 included is equivalent to replacing the recommended γ_b and γ_s values in *Tables A.6, A.7* and *A.8* on *EN 1997-1* with the increased values shown in Table 8.6.2.

Table 8.6.2 Recommended partial resistance factors and equivalent Irish values after application of model factor of 1,75

Resistance	Symbol		Set R1	R2	R3	R4
Partial resistance factors for driven piles						
Base	γ_b	Recommended	1,0	1,1	1,0	1,3
		Equivalent Irish values	1,75	1,925	1,75	2,275
Shaft (compression)	γ_s	Recommended	1,0	1,1	1,0	1,3
		Equivalent Irish values	1,75	1,925	1,75	2,275
Partial resistance factors for bored piles						
Base	γ_b	Recommended	1,25	1,1	1,0	1,6
		Equivalent Irish values	2,1875	1,925	1,75	2,8
Shaft (compression)	γ_s	Recommended	1,0	1,1	1,0	1,3
		Equivalent Irish values	1,75	1,925	1,75	2,275
Partial resistance factors for continuous flight auger (CFA) piles						
Base	γ_b	Recommended	1,1	1,1	1,0	1,45
		Equivalent Irish values	1,925	1,925	1,75	2,5375
Shaft (compression)	γ_s	Recommended	1,0	1,1	1,0	1,3
		Equivalent Irish values	1,75	1,925	1,75	2,275

8.7. Summary of key points

The key points concerning the design of pile foundations to *Eurocode 7* are:

- The importance of static pile load tests in the design of piles to *Eurocode 7* is emphasised
- *Eurocode 7* provides an innovative method for determining the characteristic pile resistance directly from the results of pile load tests or from 'profiles of tests' using ξ values
- The ξ values are based on the number of pile load tests or the number of soil test profiles and hence offer more economical designs for more pile load tests or more soil test profiles
- DA1 is a resistance factor approach for the design of piles
- DA3 should not be used for the design of piles from pile load tests or from soil test profiles as its resistance factors are all equal to 1,0
- SLS requirements need to be satisfied and model factors or increased ξ factors in ULS calculations have been introduced in many countries' NAs to provide an indirect design method to ensure that the occurrence of an SLS as well as a ULS is sufficiently unlikely.

8.8. Worked examples – pile design

Worked examples illustrating the design of piles according to *Eurocode 7* are included in the Annex to this report.

References

EN 1997-1: 2004. Eurocode 7: Geotechnical design. Part 1: General rules. CEN.

Frank, R., Bauduin, C., Kavvadas, M., Krebs Ovesen, N., Orr, T., and Schuppener, B. (2004). Designers' guide to EN 1997-1: Eurocode 7: Geotechnical design — General rules, London: Thomas Telford

NSAI (2005) Irish National Annex to Eurocode 7: Geotechnical design Part 1: General rules, National Standards Authority of Ireland, Dublin, 2005

Orr, T.L.L. and Farrell E.R. (1998) Geotechnical design to Eurocode 7, Springer-Verlag, London

ANNEX

Worked examples

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A.1. Introduction

This Annex contains worked examples to accompany the various chapters of this report.

A.2. Worked example to accompany Chapter 2

This worked example illustrates the way in which actions should be combined according to *EN 1990*, in a way that is suitable for geotechnical design of foundations.

A.2.1. DESIGN SITUATION

Consider a n=3 storey building, B_x =48 m by B_y=15 m on plan, which is divided into N_x =8 bays in the building's long direction and N_y =2 bays in its short direction. The height of each storey is h = 3,2 m . The floors of the building are d_{floor} =250 mm thick (Figure A.2.1).

Shear walls, intended to resist overturning, are located at both ends of the building and are t=300 mm thick by b_w=4 m wide on plan. A water tank, d_{tank} =2 m deep by l_{tank} =5 m long by b_{tank} =5m wide sit over the shear wall at one end of the building. The shear walls are supported by strip foundations of length l_{fdn} = 6,5 m, breadth b_{fdn} = 2 m, and thickness d_{fdn} = 1,5 m. The following characteristic imposed/wind actions act on the building:

- roof loading $q_{rf,k}$ = 0,6 kPa
- office floor loading $q_{off,k}$ = 2,5 kPa
- partition loading $q_{par,k}$ = 0,8 kPa
- wind (horizontal) $q_{w,k}$ =1,15 kPa

The characteristic weight density of reinforced concrete is $\gamma_{c,k}$ =25 kN/m^3 and of water $\gamma_{w,k}$ = 10 kN/m^3.

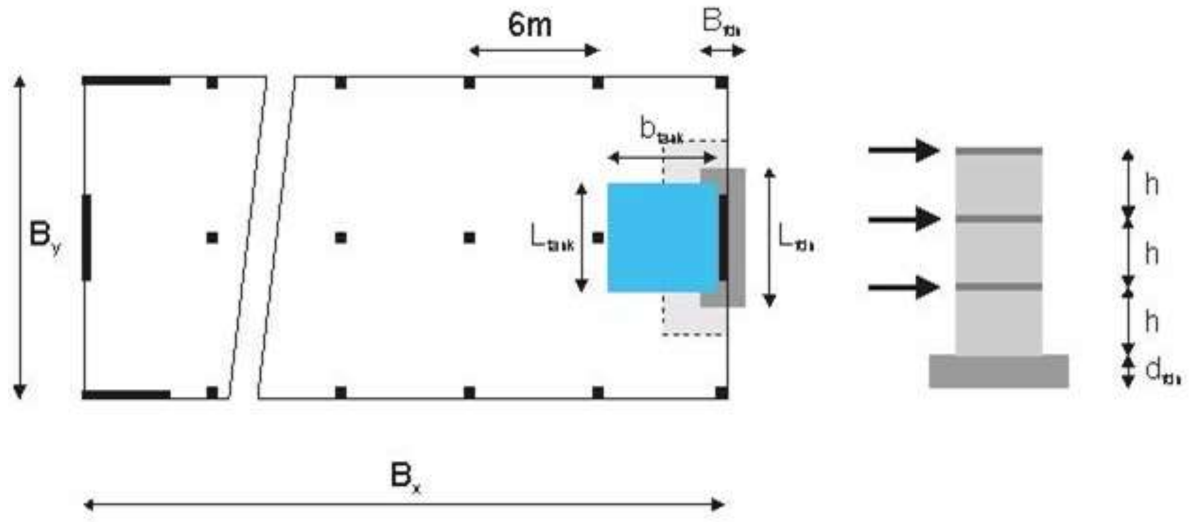


Fig.A.2.1 Plan and section of the 3 storey building

Geometry

Total plan area of building is:

$$A_{tot} = B_x B_y = 720\,\mathrm{m}^2$$

The tributary area above the stability wall has area:

$$A = \left(\frac{B_y + b_w}{2}\right) \cdot \frac{1}{2}\left(\frac{B_x}{N_x}\right) = 28{,}5\,\mathrm{m}^2$$

Characteristic actions -permanent

Self-weight of slabs:

- Floor: $g_{fl,Gk} = \gamma_{c,k}\, d_{floor}$ = 6,25 kPa
- Screed on roof: $g_{scr,Gk}$ =1,5 kPa
- Raised floor: $g_{r\text{-}fl,Gk}$ =0,5 kPa (removable)

Self-weight of water tank on roof - only half total weight is carried by the core wall

$$w_{tank,Gk} = \frac{1}{2}\gamma_{w,k} d_{tank} l_{tank} b_{tank} = 250\,\mathrm{kN} \text{ (removable)}$$

Self-weight of core wall:

$$w_{wall,Gk} = \gamma_{c,k} t_w b_w \left(nh\right) = 288\,\mathrm{kN}$$

Self-weight of pad foundation:

$$W_{fdn,Gk} = \gamma_{c,k} d_{fdn} b_{fdn} l_{fdn} = 488\,\mathrm{kN}$$

Total self-weight of non-removable members (normal to ground):

$$N_{Gk1} = \left(ng_{fl,Gk}A\right)\left(g_{scr,Gk}\mathrm{A}\ \right) + W_{wall,Gk} + W_{fdn,Gk} = 1353\,\mathrm{kN}$$

Total self-weight of removable members (normal to ground):

$$N_{Gk2} = \left\lfloor \left(n-1\right) g_{r-fl,Gk} A \right\rfloor + W_{tank,Gk} = 279\,\mathrm{kN}$$

Characteristic actions - variable

Imposed actions (normal to ground):

- on roof: $N_{rf,Qk} = q_{rf,k} A = 17{,}1\,\mathrm{kN}$
- on floors: $N_{fl,Qk} = \left(n-1\right)\left(q_{off,k} + q_{par,k}\right) A = 188{,}1\,\mathrm{kN}$

Wind actions (horizontal direction):

- on roof: $Q_{w,rf,Qk} = q_{w,k} \dfrac{h}{2}\dfrac{B_x}{2} = 44{,}2\,\mathrm{kN}$
- on each floor: $Q_{w,fl,Qk} = q_{w,k} h \dfrac{B_x}{2} = 88{,}3\ \mathrm{kN}$

Total wind action (normal to ground):

$N_{w,Qk} = 0\text{kN}$

Moment effect of wind action (on ground):

- first floor: $M_{w,Qk1} = Q_{w,fl,Qk} \lfloor (n-2)h + d_{fdn} \rfloor = 415$ kNm
- second floor: $M_{w,Qk2} = Q_{w,fl,Qk} \lfloor (n-1)h + d_{fdn} \rfloor = 698$ kNm
- roof: $M_{w,Qk3} = Q_{w,rf,Qk} (nh + d_{fdn}) = 490$ kNm
- total: $M_{w,Qk} = \sum M_{w,Qk} = 1603$ kNm

A.2.2. COMBINATIONS OF ACTIONS FOR PERSISTENT AND TRANSIENT DESIGN SITUATIONS - ULS VERIFICATION

Combination 1 - wind as leading variable action, vertical actions unfavourable, partial factors from Set B

Partial factors

- on permanent actions: $\gamma_G = \gamma_{G,B} = 1{,}35$
- on variable actions (wind): $\gamma_{Q,w} = \gamma_{Q,B} = 1{,}5$
- on variable actions (imposed loads): $\gamma_{Q,I} = \gamma_{Q,B} = 1{,}5$

Combination factors:

- for wind $\psi_w = 1{,}0$
- for imposed load in office areas (Category B): $\psi_{fl} = \psi_{0,i,B} = 0{,}7$
- for imposed load on roof (Category H): $\psi_{rf} = \psi_{0,i,H} = 0$

Design value of normal action effect:

$$N_{Ed} = \gamma_G (N_{GK1} + N_{GK2}) + \gamma_{Q,w} \psi_w N_{w,Qk} + \gamma_{Q,i} (\psi_{fl} N_{fl,Qk} + \psi_{rf} N_{rf,Qk}) = 2400\text{kN}$$

Design value of moment effect:

$$M_{Ed} = \gamma_{Q,w} \psi_w M_{w,Qk} = 2405\text{kN}$$

Maximum bearing pressure on underside of foundation is:

$$P_{\max,Ed} = \left(\frac{N_{Ed}}{b_{fdn} l_{fdn}} \right) + \left(\frac{6M_{Ed}}{b_{fdn} l_{fdn}^2} \right) = 355\text{kN}$$

Combination 2 - wind as leading variable action, vertical actions favourable, partial factors from Set B

Design value of normal action effect:

$$N_{Ed} = \gamma_{G,fav} (N_{Gk1} + N_{Gk2}) = 1631\,\text{kN}$$

Design value of moment effect:

$$M_{Ed} = \gamma_{Q,w}\psi_w M_{w,Qk} = 2405 \text{ kNm}$$

Maximum bearing pressure on underside of foundation is:

$$P_{\max,Ed} = \left(\frac{N_{Ed}}{b_{fdn} l_{fdn}}\right) - \left(\frac{6M_{Ed}}{b_{fdn} l_{fdn}^2}\right) = -45 \text{kN}$$

Line of action is outside the middle-third and eccentricity is:

$$e = \frac{M_{Ed}}{N_{Ed}} = 1{,}47 \text{kN}$$

Revised maximum bearing pressure on underside of foundation is:

$$P_{\max,Ed} = \frac{8}{3}\frac{N_{Ed}}{\left(l_{fdn} - 2e\right)^2} = 345 \text{kN}$$

Combination 3 - imposed loads as leading variable action, vertical actions unfavourable, partial factors from Set B

Combination factors:

- for wind $\psi_w = \psi_{0,w} = 0{,}6$
- for imposed load in office areas (Category B): $\psi_{fl} = 1$
- for imposed load on roof (Category H): $\psi_{rf} = 1$

Design value of normal action effect:

$$N_{Ed} = \gamma_G \left(N_{Gk1} + N_{Gk2}\right) + \gamma_{Q,w}\psi_w N_{w,Qk} + \gamma_{Q,i}\left(\psi_{fl} N_{fl,Qk} + \psi_{rf} N_{rf,Qk}\right) = 2510 \text{ kN}$$

Design value of moment effect:

$$M_{Ed} = \gamma_{Q,w}\psi_w M_{w,Qk} = 1443 \text{ kN}$$

Maximum bearing pressure on underside of foundation is:

$$P_{\max,Ed} = \left(\frac{N_{Ed}}{b_{fdn} l_{fdn}}\right) + \left(\frac{6M_{Ed}}{b_{fdn} l_{fdn}^2}\right) = 296 \text{kN}$$

Combination 4 - wind as leading variable action, vertical actions unfavourable, partial factors from Set C

Partial factors:

- on permanent actions: $\gamma_G = \gamma_{G,C} = 1$
- on variable actions (wind): $\gamma_{Q,w} = \gamma_{Q,C} = 1{,}3$
- on variable actions (imposed loads): $\gamma_{Q,I} = \gamma_{Q,C} = 1{,}3$

Combination factors:

- for wind: $\psi_w = 1{,}0$

- for imposed load in office areas (Category B): $\psi_{fl} = \psi_{0,i,B} = 0{,}7$
- for imposed load on roof (Category H): $\psi_{rf} = \psi_{0,i,H} = 0$

Design value of normal action effect:

$$N_{Ed} = \gamma_G \left(N_{Gk1} + N_{Gk2}\right) + \gamma_{Q,w}\psi_w N_{w,Qk} + \gamma_{Q,i}\left(\psi_{fl} N_{fl,Qk} + \psi_{rf} N_{rf,Qk}\right) = 1802 \text{ kN}$$

Design value of moment effect:

$$M_{Ed} = \gamma_{Q,w}\psi_w M_{w,Qk} = 2084 \text{ kN}$$

Combination 5 - wind as leading variable action, vertical actions favourable, partial factors from Set C

Design value of normal action effect:

$$N_{Ed} = \gamma_{G,fav}\left(N_{Gk1} + N_{Gk2}\right) = 1631 \text{ kN}$$

Design value of moment effect:

$$M_{Ed} = \gamma_{Q,w}\psi_w M_{w,Qk} = 2084 \text{ kN}$$

Combination 6 - imposed loads as leading variable action, vertical actions unfavourable, partial factors from Set C

Combination factors:

- for wind: $\psi_w = \psi_{0,w} = 0{,}6$
- for imposed load in office areas (Category B): $\psi_{fl} = 1$
- for imposed load on roof (Category H): $\psi_{rf} = 1$

Design value of normal action effect:

$$N_{Ed} = \gamma_G \left(N_{Gk1} + N_{Gk2}\right) + \gamma_{Q,w}\psi_w N_{w,Qk} + \gamma_{Q,i}\left(\psi_{fl} N_{fl,Qk} + \psi_{rf} N_{rf,Qk}\right) = 1898 \text{ kN}$$

Design value of moment effect:

$$M_{Ed} = \gamma_{Q,w}\psi_w M_{w,Qk} = 1250 \text{ kN}$$

A.2.3. COMBINATIONS OF ACTIONS FOR QUASI-PERMANENT DESIGN SITUATIONS - SLS VERIFICATIONS

Combination 7 - wind as leading variable action, vertical actions unfavourable, partial factors for SLS

Partial factors:

- on permanent actions: $\gamma_G = \gamma_{G,SLS} = 1$
- on variable actions (wind): $\gamma_{Q,w} = \gamma_{Q,SLS} = 1$
- on variable actions (imposed loads): $\gamma_{Q,i} = \gamma_{Q,SLS} = 1$

Combination factors:

- for wind: $\psi_w = \psi_{2,w} = 0$
- for imposed load in office areas (Category B): $\psi_{fl} = \psi_{2,i,B} = 0{,}3$
- for imposed load on roof (Category H): $\psi_{rf} = \psi_{2,i,H} = 0$

Design value of normal action effect:

$$N_{Ed} = \gamma_G \left(N_{Gk1} + N_{Gk2}\right) + \gamma_{Q,w}\psi_w N_{w,Qk} + \gamma_{Q,i}\left(\psi_{fl} N_{fl,Qk} + \psi_{rf} N_{rf,Qk}\right) = 1688 \text{ kN}$$

Design value of moment effect:

$$M_{Ed} = \gamma_{Q,w}\psi_w M_{w,Qk} = 0 \text{ kN}$$

Combination 8 - wind as leading variable action, vertical actions favourable, partial factors for SLS

Design value of normal action effect:

$$N_{Ed} = \gamma_{G,fav}\left(N_{Gk1} + N_{Gk2}\right) = 1631 \text{ kN}$$

Design value of moment effect:

$$M_{Ed} = \gamma_{Q,w}\psi_w M_{w,Qk} = 0 \text{ kN}$$

Combination 3 - imposed loads as leading variable action, vertical actions unfavourable, partial factors for SLS

Combination factors:

- for wind: $\psi_w = \psi_{2,w} = 0$
- for imposed load in office areas (Category B): $\psi_{fl} = \psi_{2,i,B} = 0{,}3$
- for imposed load on roof (Category H): $\psi_{rf} = \psi_{2,i,H} = 0$

Design value of normal action effect:

$$N_{Ed} = \gamma_G \left(N_{Gk1} + N_{Gk2}\right) + \gamma_{Q,w}\psi_w N_{w,Qk} + \gamma_{Q,i}\left(\psi_{fl} N_{fl,Qk} + \psi_{rf} N_{rf,Qk}\right) = 1688 \text{ kN}$$

Design value of moment effect:

$$M_{Ed} = \gamma_{Q,w}\psi_w M_{w,Qk} = 0 \text{ kN}$$

A.3. Worked example to accompany Chapter 3

This worked example illustrates how the design of a strip foundation of a building can be carried out to *Eurocode 7*.

Real data from a comprehensive site investigation are given to define the geotechnical model with characteristic geotechnical parameters. Both ULS and SLS checks are shown with some details.

A.3.1. DESIGN SITUATION

Design a strip foundation for the concrete building of Fig.A.3.2 whose destination is civil habitation. It is composed of four floors. The embedment depth of foundation is 1,5 m; the groundwater level is situated at 1,5 m from the ground surface. Allowable settlement is 5 cm.

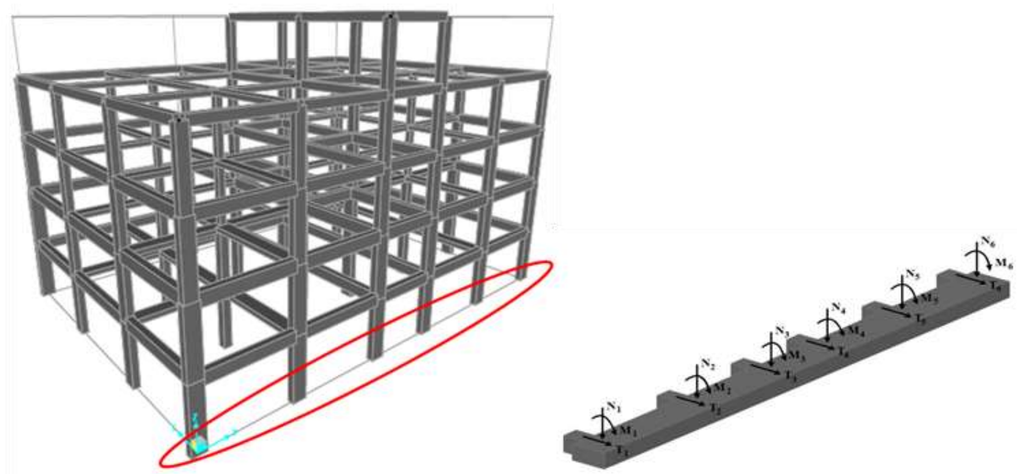


Fig.A.3.2 Arrangement of building

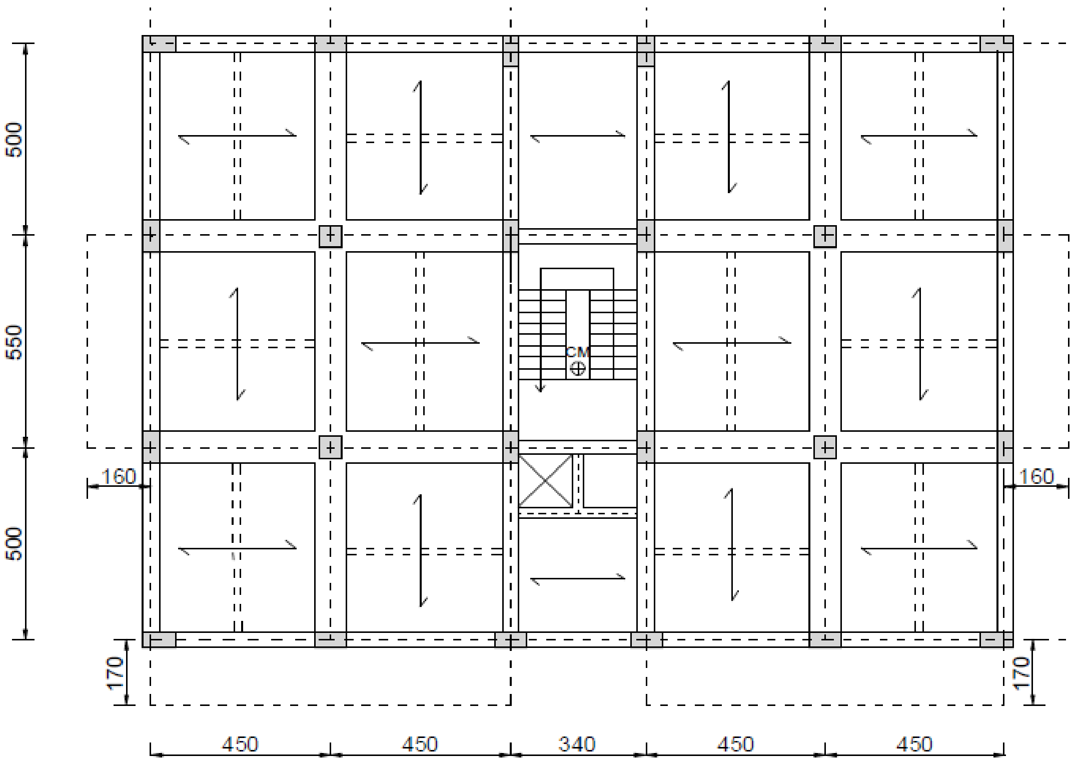


Fig.A.3.3 Main floor framing

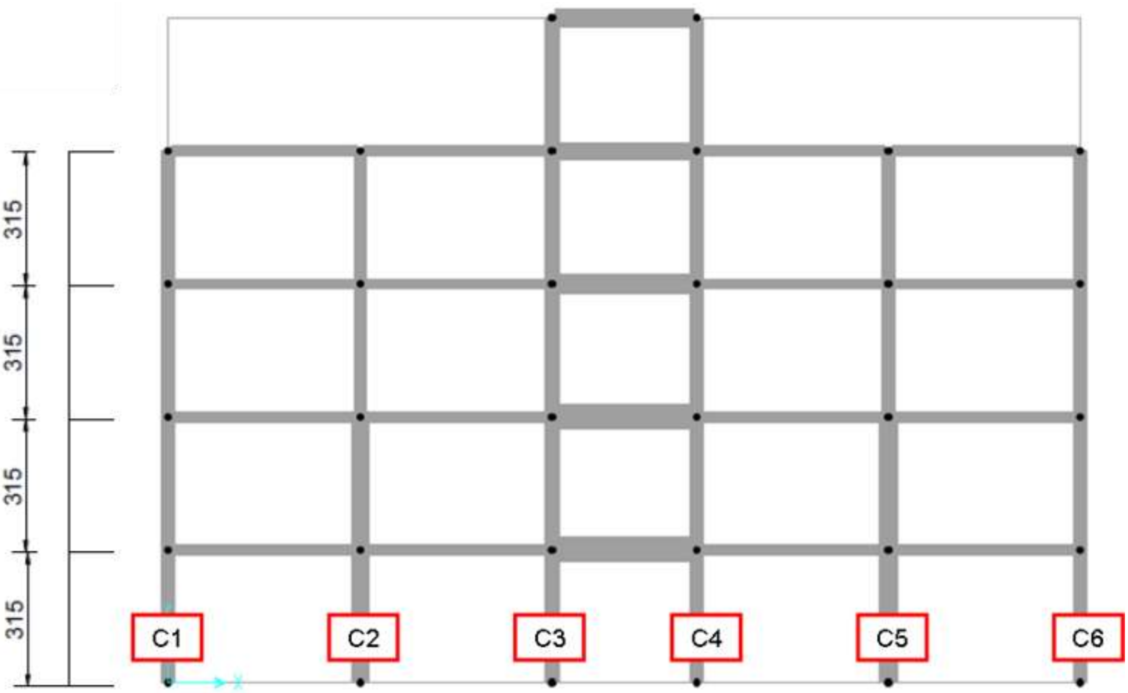


Fig.A.3.4 Vertical cross section

From the structural analysis, the characteristic value of permanent G_k and variable Q_k vertical loads for each of the six columns have been obtained as summarized in Table A.3.1. For no-seismic design situation, shear forces and bending moments can be assumed to be negligible.

Table A.3.1 Vertical action on the columns of the building of Fig.A.3.2

Column	Normal forces (kN)*	
C1	G_k	-460,147
	Q_k	-108,138
C2	G_k	-687,103
	Q_k	-222,355
C3	G_k	-627,8
	Q_k	-154,012
C4	G_k	-623,915
	Q_k	-152,261
C5	G_k	-685,011
	Q_k	-222,231
C6	G_k	-416,982
	Q_k	-108,103

A.3.2. ASSIGNEMENT: THE FOUNDATION WIDTH B TO SATISFY BOTH ULS AND SLS IS REQUIRED

To design the shallow foundation and to meet the assigned requirements, the geotechnical model of the bearing soil has to be defined. This is obtained by assembling data and information from geotechnical investigations as summarized for this example through the following steps:

1. The soil profile is taken from the borehole log;
2. CPTu soil in situ testing gives cone and frictional resistances and the pore pressures with depth;
3. Normalized cone resistance Q_t and Friction Ratio F_r as defined in the Figure A.3.6 and plotted with depth in Figure A.3.7 are used to classify the different soil deposits.

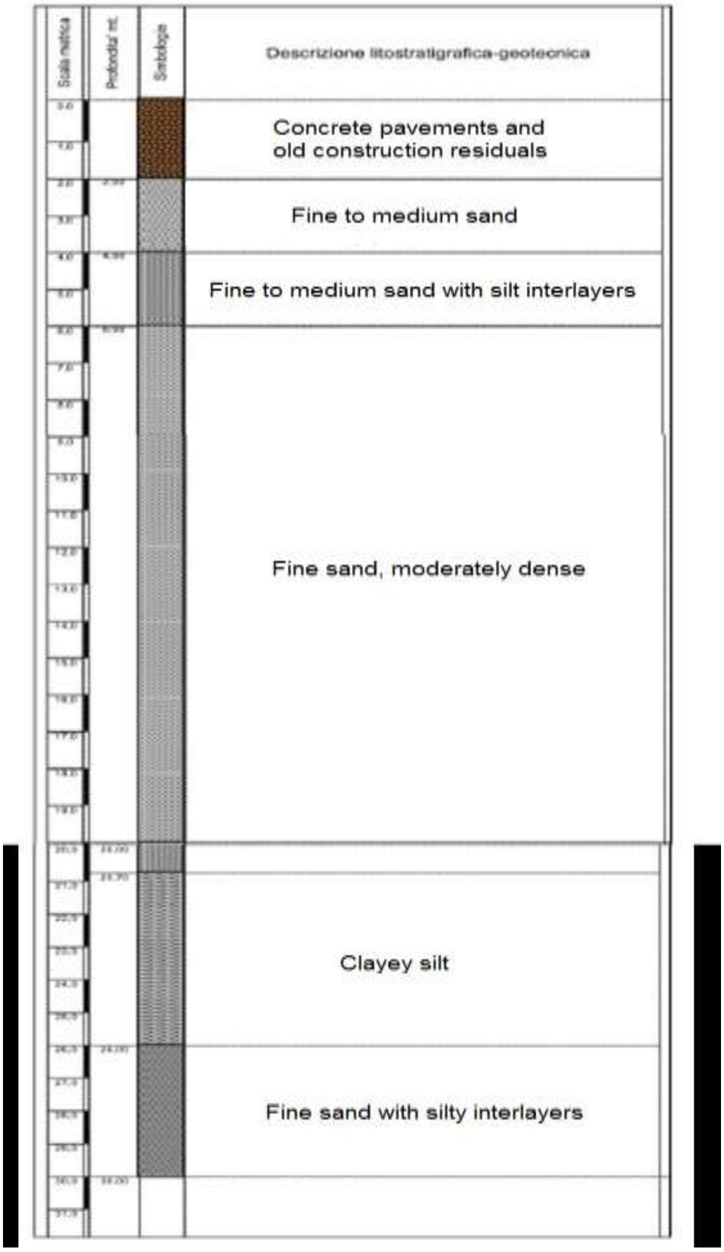


Fig.A.3.5: The soil profile

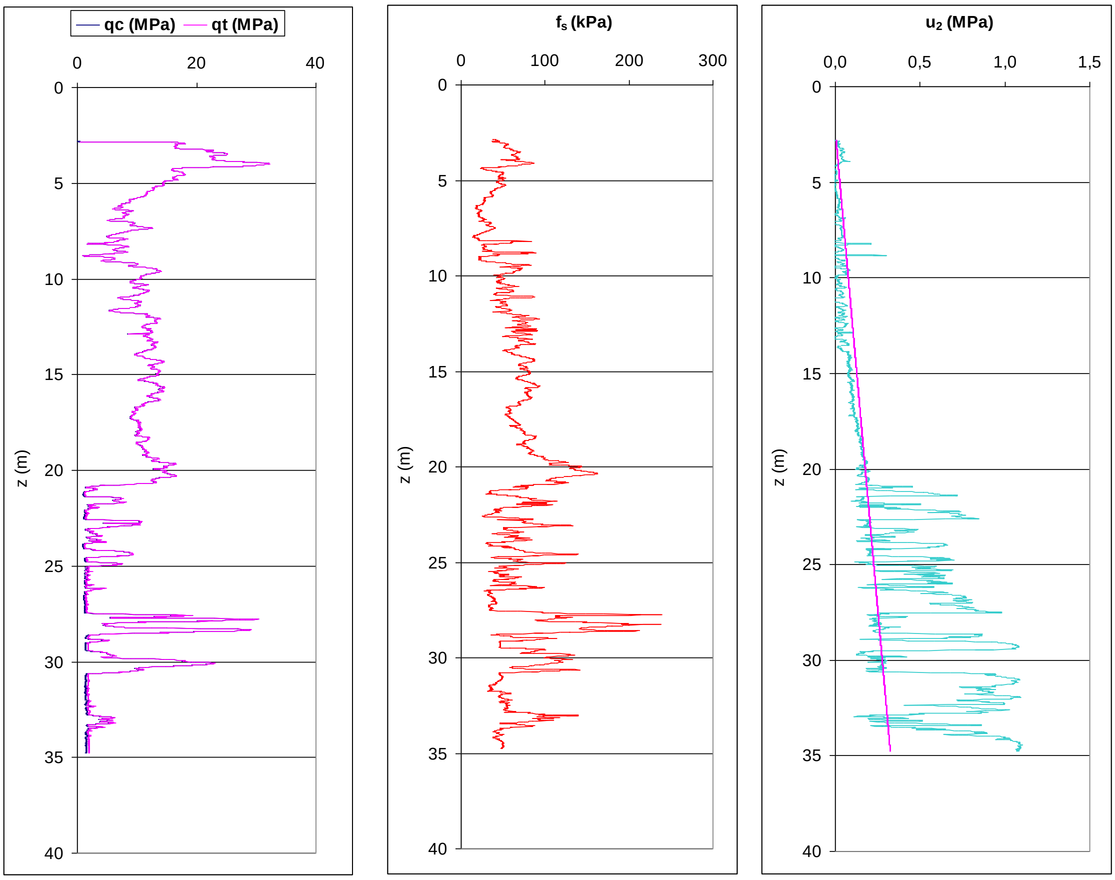


Fig. A.3.6 Results from CPTu testing

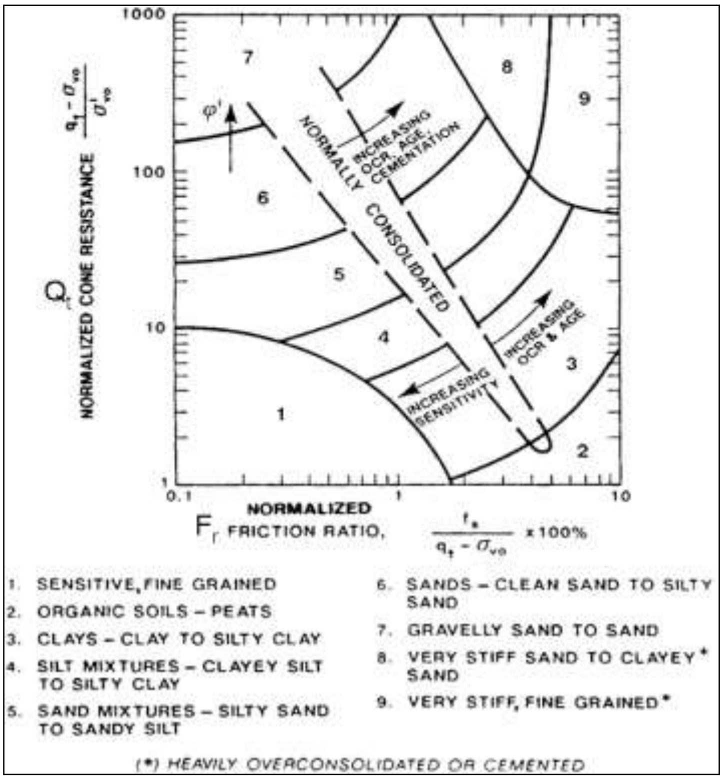


Fig.A.3.7 Normalized chart to analyse CPT test

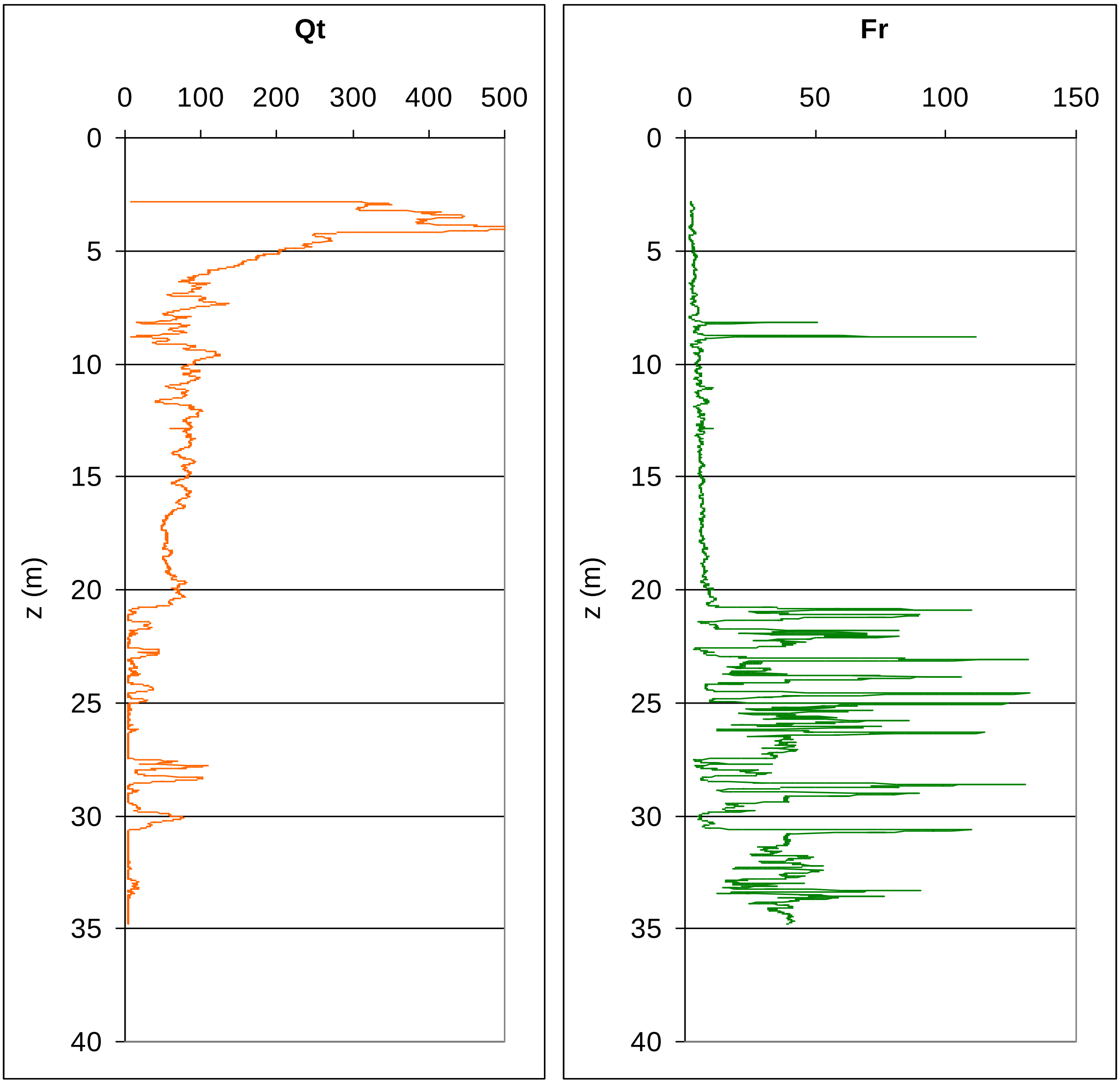


Fig.A.3.8 Normalized point resistance and friction from CPTu tests

More correlations for fine grained soils give:

$c_u=(q_t-\sigma_{v0})/N_k$	undrained shear strength (assumed Nk = 19)
E_u -	undrained modulus (it can be obtained from the plot of Fig.A.3.9)
E_d -	oedometric modulus

E_d (MPa) = $4q_c$ (if q_c > 10 MPa)

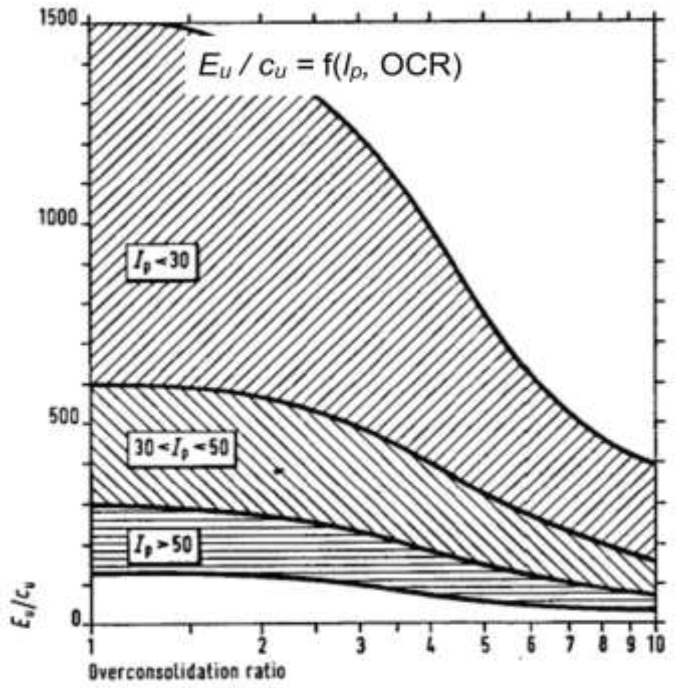
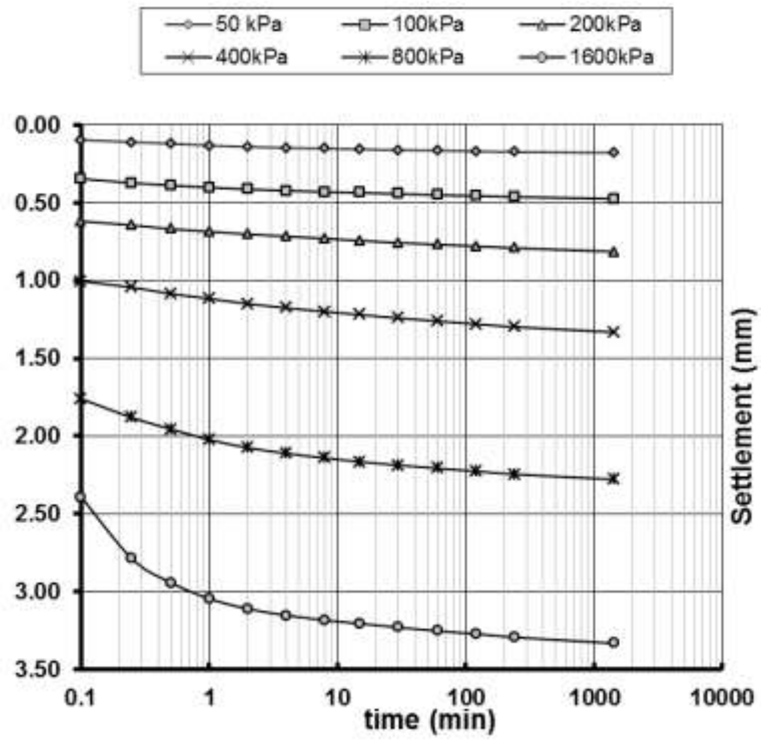


Fig.A.3.9 Empirical correlation for the ratio *Eu/Cu* as a function of the overconsolidation ratio. *Ip* is the soil plasticity index.



Results of one oedometer test (

Fig.A.3.) are used to obtain soil consolidation properties:

$C_v = 2 \cdot 10^{-3}$ cm^2/s - average value of the consolidation coefficient

z = 25 m - sample depth

W =32,74 % - natural water content

γ =19 kN/m^3 - unit weight,

z_w = 2,5 m - water depth

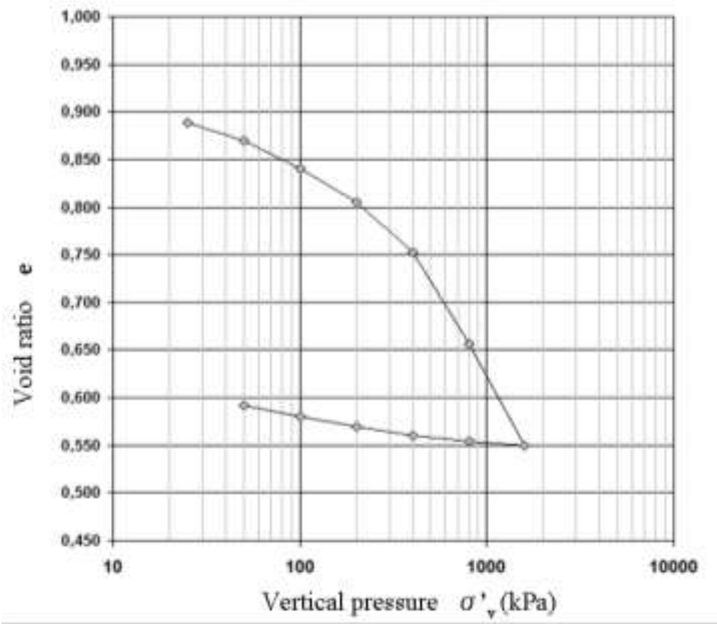


Fig.A.3.10 Results from oedometer testing – vertical pressure

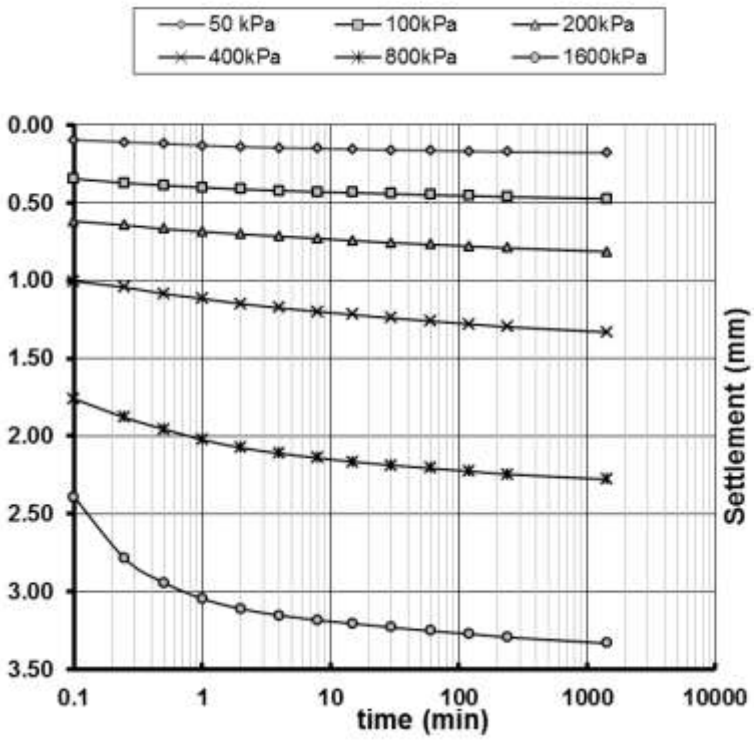


Fig.A.3.10 Results from oedometer testing - settlement

From all the given data, a geotechnical model as shown in Fig.A.3.111 can be finally assumed.

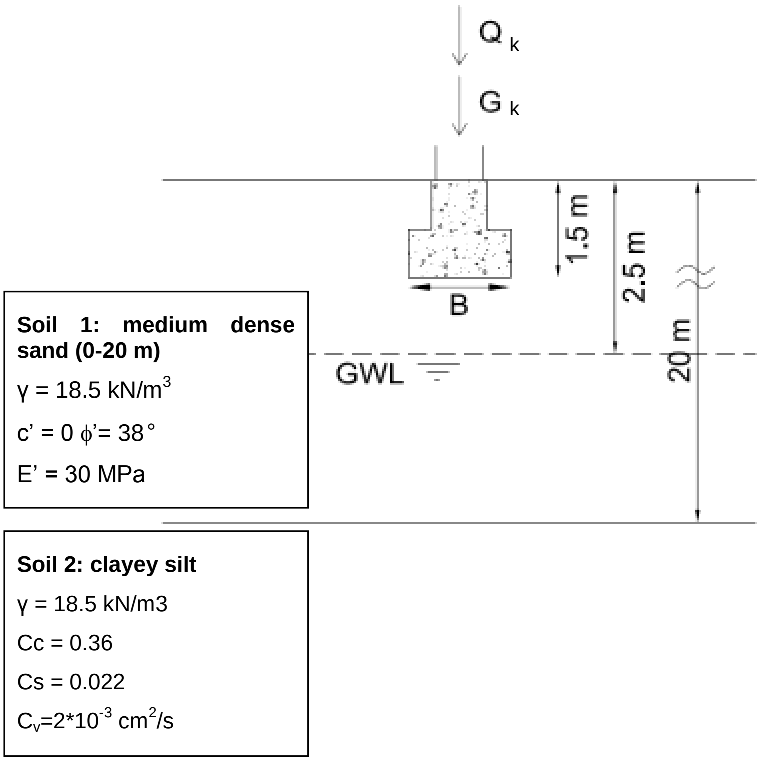


Fig.A.3.111 The soil geotechnical model adopted for the shallow foundation design

A.3.3. DESIGN OF A STRIP FOUNDATION FOR BEARING RESISTANCE

In the following tables are summarized adopted factors and results obtained by using the three different design approaches DA1, DA2 and DA3. For all the cases, ϕ'_k = 38° and the value of B is obtained by imposing $R_d = V_d$ (in DA1 for the 2nd combination only).

Table A.3.1. Adopted factors and results obtained by using design approach DA1

B=1,5 Partial factors	DA1-1 A1+M1+R1	DA1-2 A2+M2+R1
V_d (kN)	7395,74	5661,00
N_q	48,87	23,17
N_γ	74,80	27,72
s_q	1,04	1,04
s_γ	0,95	0,98
R_d (kN)	15320,80	5676,52
R_d / V_d	2,07	1,00

Table A.3.2. Adopted factors and results obtained by using design approach DA2

B=1,21 Partial factors	DA2 A1+M1+R2

V_d (kN)	7160,10
N_q	48,87
N_γ	74,80
s_q	1,03
s_γ	0,98
R_d (kN)	7150,58
R_d / V_d	1,00

Table A.3.3. Adopted factors and results obtained by using design approach DA3

B=1,74	**DA3**
Partial factors	**A1 / A2+M2+R3**
V_d (kN)	7590,75
N_q	23,17
N_γ	27,72
s_q	1,04
s_γ	0,98
R_d (kN)	7612,07
R_d / V_d	1,00

It is clear how design approach 2 gives the smallest value for *B*.

This is also true in general if friction is not too small, as shown by the plot of the ratio R_d/V_d against friction for the three possible design approaches (Fig.A.3.122).

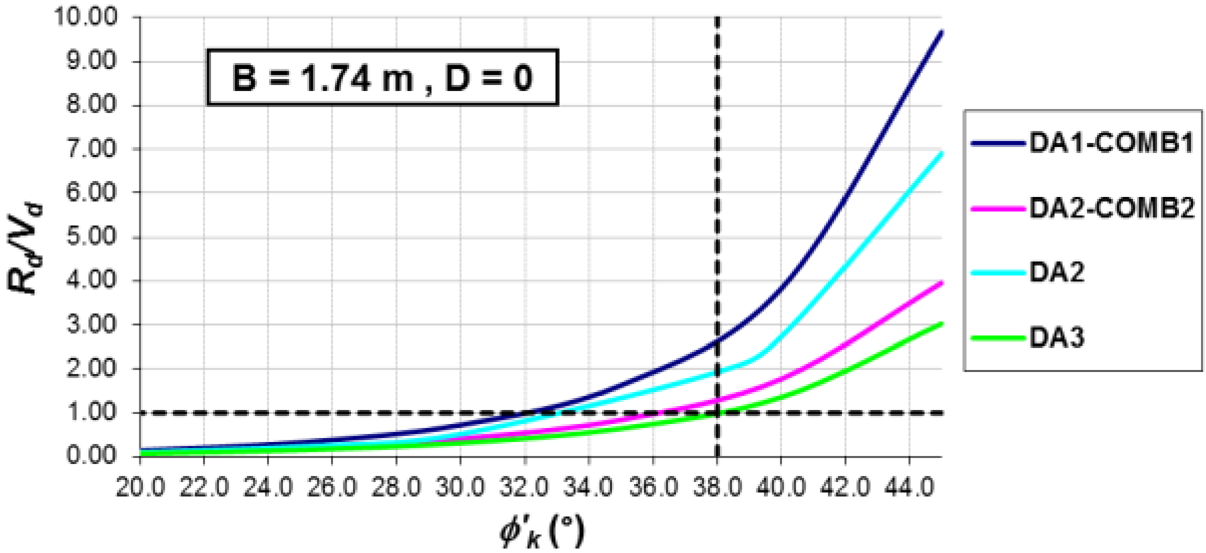


Fig.A.3.122 Graphs of the utilization ratio R_d/V_d with friction ϕ'_k for constant *B* in the three design approaches DA1, DA2 and DA3.

A.3.4. DESIGN OF A STRIP FOUNDATION: SERVICEABILITY CHECK

Calculation of immediate (mean) settlement: use of the linear elastic solution and take the sum of the contributions to total settlement from each *j*-stratum with constant elastic parameters (E_j,v_j) (Figure A.3.13). Use the solution for influence factor $I=\mu_0 \cdot \mu_1$ that can be obtained from the graphs of Fig. A.3.15A.3.14 and A.3.15.

$$s_0 = qB\sum_{i=1}^{n}\frac{I(H_i)-I(H_{i-1})}{E_i}(1-v^2)$$

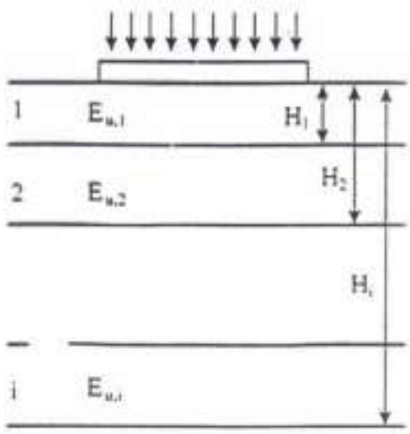


Fig. A.3.133 Calculation of elastic settlement

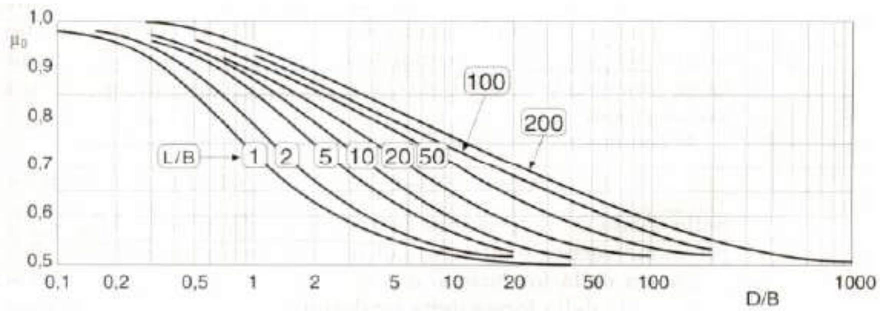


Fig. A.3.144 Graph to obtain μ_0 for calculation of the elastic settlement

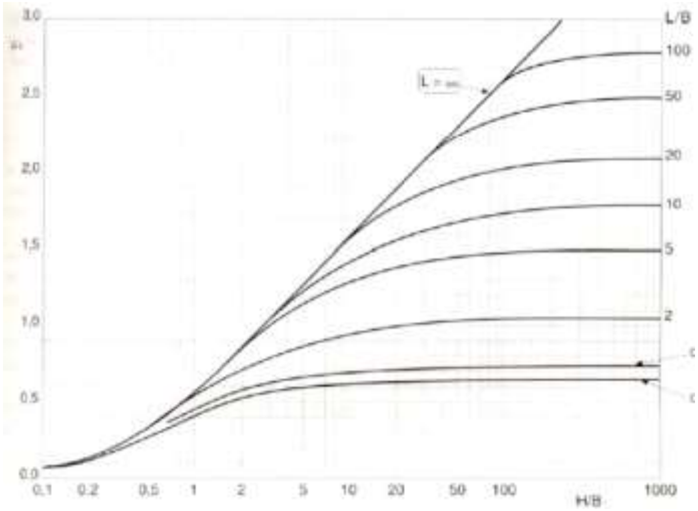


Fig. A.3.15 Graph to obtain μ_1 for calculation of the elastic settlement

Having adopted a footing with B = 1,50 m, stress increments are confined to a very shallow depth and there is no need to account for the change of the stiffness with depth; in this case, just one soil stratum is considered in the settlement calculation. On the other hand, if the footings are assimilated to a single equivalent spread foundation with B equal to the building width, a geotechnical model with two strata is adopted, and each stratum has its appropriate stiffness value. However the thickness of the deforming soil is 50 metres for both schemes. The following Table A. and A.3.5 give details of the two calculations.

Table A.3.4: Settlement calculations for a single footing

q **(kPa)**	167,32
B **(m)**	1,50
L **(m)**	21,40
H **(m)**	50
$\boldsymbol{v}$	0,3
E' **(kPa)**	30000
H / B	666,7
D / B	1
L / B	14,3
$\boldsymbol{\mu_0}$	1
$\boldsymbol{\mu_1}$	1,8
$\boldsymbol{\delta}$ **(m)**	**0,014**

Table A.3.5: Settlement calculations for the spread foundation scheme

q **(kPa)**	64,77
B **(m)**	15,50
L **(m)**	21,40
$\boldsymbol{H_1}$ **(m)**	20
$\boldsymbol{H_2}$ **(m)**	50
$\boldsymbol{v_1}$	0,30
$\boldsymbol{v_2}$	0,50

c_u (kPa)	80,00
E_u (kPa)	32000
E' (kPa)	30000
H_1 / B	1,29
H_2 / B	64,52
D / B	0
L / B	1,38
$\mu_{0\,(1)}$	1,00
$\mu_{1\,(1)}$	0,50
$\mu_{0\,(2)}$	1,00
$\mu_{1\,(2)}$	0,70
δ (m)	**0,021**

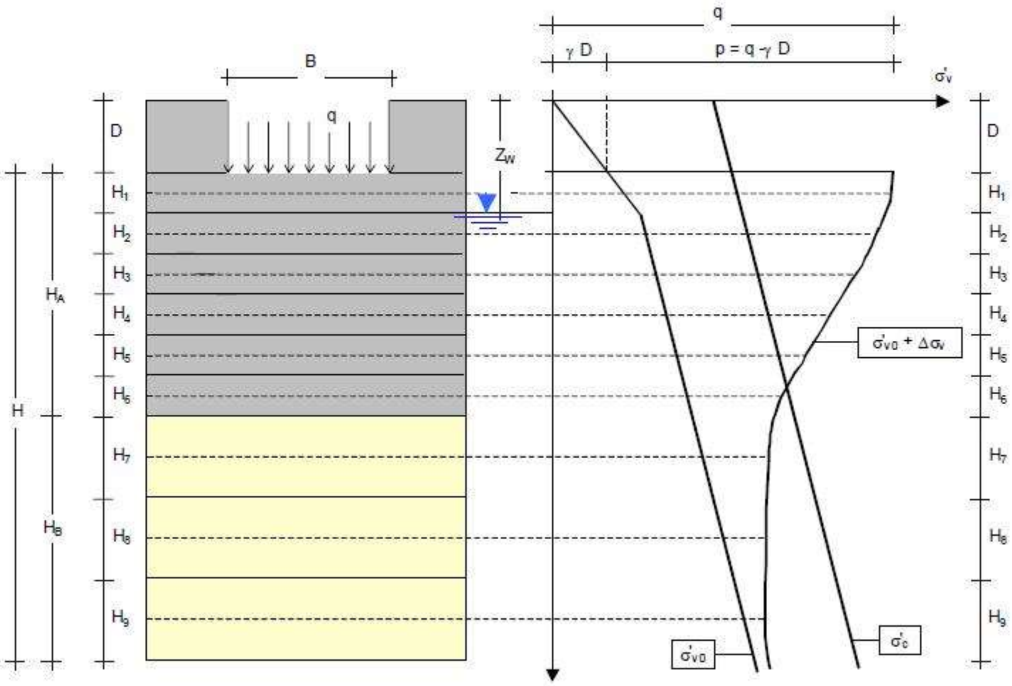
A.3.5. CONSOLIDATION SETTLEMENTS IN OEDOMETRIC CONDITIONS:

$$S_{ed} = \Sigma \Delta H_i$$

$$\Delta H_i = \frac{H_i}{(1+e_0)}\left[C_s \log\left(\frac{\sigma_c^{'}}{\sigma_{v0}^{'}}\right) + C_c \log\left(\frac{\sigma_{v0}^{'} + \Delta\sigma_v}{\sigma_c^{'}}\right)\right]$$

With the calculation scheme shown in the Fig,A.3.6, stress increments (by means of influence factors of Fig.A.3.16) and settlements can be obtained as detailed in Table A.3.6 and in Table A.3.7.

In the first table, settlement calculation was limited at a depth where the stress increment is lower than 10% of the in situ effective stress. This is justified by a small overconsolidation of the clayey soil deposit due to cementation. In the second table this effect is not considered and the settlement calculation was extended to a depth comparable with the building width. Final settlement values are 3,3 cm and 24,3 cm respectively.



Fig,A.3.16: Scheme of the soil stratigraphy for calculation of consolidation settlements

$$\sigma_z = q\frac{1}{4\pi}\frac{2mn\left(m^2+n^2+1\right)^{1/2}}{m^2+n^2+m^2n^2+1}\frac{m^2+n^2+2}{m^2+n^2+1}+\tan^{-1}\frac{2mn\left(m^2+n^2+1\right)^{1/2}}{m^2+n^2-m^2n^2+1}$$

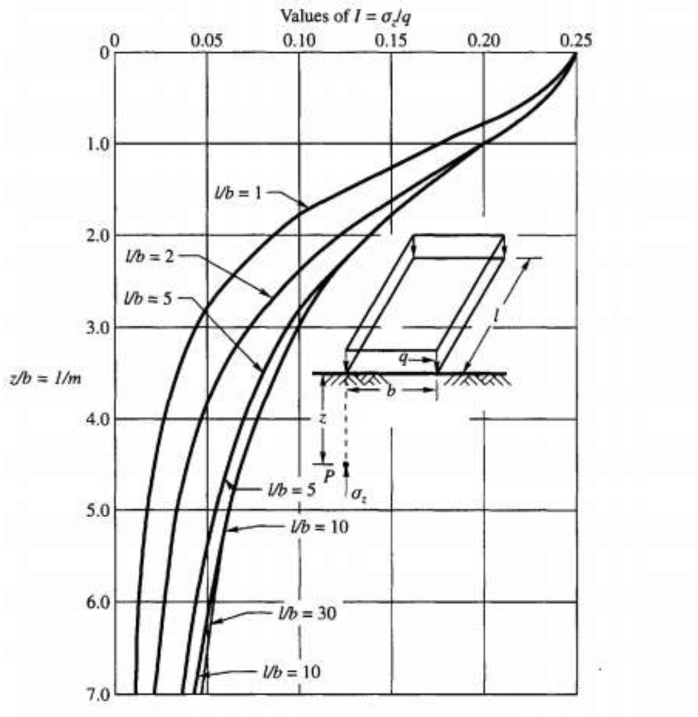


Chart for computing σ_z below the corner of a rectangular foundation (after Steinbrenner, 1934)

Fig.A.3.167 Influence factors for computing vertical stresses

Table A.2.6 Consolidation settlement for the 10% stress increment limitation scheme

layers	Δz	z (from g.s.)	σ_z	u	σ'_z	e_0	$\Delta\sigma_z$	$\Delta\sigma_z/\sigma'_z$	σ'_{z_fin}	e_f	ε_z	S_{ed}
	(m)	(m)	(kPa)	(kPa)	(kPa)		(kPa)		(kPa)			(m)
1,00	0,50	20,25	374,63	174,13	200,50	0,91	21,67	0,11	222,17	0,86	0,023	0,012
2,00	0,50	20,75	383,88	179,03	204,84	0,90	20,87	0,10	225,71	0,86	0,022	0,011
3,00	0,50	21,25	393,13	183,94	20919	0,90	20,10	0,10	229,29	0,86	0,021	0,010
4,00	0,50	21,75	402,38	188,84	213,53	0,90	19,37	0,09	232,90	0,86	0,020	0,010
5,00	0,50	22,25	411,63	193,75	217,88	0,89	18,68	0,09	236,55	0,86	0,019	0,009
6,00	0,50	22,75	420,88	198,65	222,22	0,89	18,01	0,08	240,24	0,86	0,018	0,009
7,00	0,50	23,25	430,13	203,56	226,57	0,89	17,38	0,08	243,95	0,86	0,017	0,009
8,00	0,50	23,75	439,38	208,46	230,91	0,88	16,78	0,07	247,69	0,85	0,016	0,008
9,00	0,50	24,25	448,63	213,37	235,26	0,88	16,20	0,07	251,46	0,85	0,015	0,008
10,00	0,50	24,75	457,88	218,27	239,60	0,88	15,66	0,07	255,26	0,85	0,015	0,007
11,00	0,50	25,25	467,13	223,18	243,95	0,88	15,13	0,06	259,08	0,85	0,014	0,007
12,00	0,50	25,75	476,38	228,08	248,29	0,87	14,63	0,06	262,93	0,85	0,013	0,007
13,00	0,50	26,25	485,63	232,99	252,64	0,87	14,16	0,06	266,79	0,85	0,013	0,006
											s_{tot} =	**0,033**

Table A.3: Consolidation settlement for the full depth scheme

layers	Δz	z (from g.s.)	σ_z	u	σ'_z	$\Delta\sigma_z$	$\Delta\sigma_z/\sigma'_z$	σ'_{z_fin}	e_f	ε_z	S_{ed}
	(m)	(m)	(kPa)	(kPa)	(kPa)	(kPa)		(kPa)			(m)
1,00	2,00	21.00	388.50	181.49	207.02	20.48	0.10	227.50	0.86	0.022	0.043
2,00	2,00	23.00	425.50	201.11	224.40	17.69	0.08	242.90	0.86	0.017	0.035
3,00	2,00	25.00	462.50	220.73	241.78	15.39	0.06	257.17	0.85	0.014	0.029
4,00	2,00	27.00	499.50	240.35	259.16	13.48	0.05	27.64	0.84	0.012	0.024
5,00	2,00	29.00	536.50	259.97	276.54	11.88	0.04	288.42	0.84	0.010	0.020
6,00	2,00	31.00	573.50	279.59	293.92	10.54	0.04	304.46	0.83	0.008	0.017
7,00	2,00	33.00	610.50	299.21	311.30	9.40	0.03	320.70	0.83	0.007	0.014
8,00	2,00	35.00	647.50	318.83	328.68	8.43	0.03	337.11	0.82	0.006	0.012
9,00	2,00	37.00	684.50	338.45	346.06	7.60	0.02	353.65	0.81	0.005	0.010
10,00	2,00	39.00	721.50	358.07	363.44	6.88	0.02	370.31	0.81	0.004	0.009
11,00	2,00	41.00	758.50	377.69	380.82	6.25	0.02	387.07	0.80	0.004	0.008
12,00	2,00	43.00	795.50	397.31	398.20	5.71	0.01	403.90	0.80	0.003	0.007
13,00	2,00	45.00	832.50	416.93	415.58	5.23	0.01	420.81	0.79	0.003	0.006
14,00	2,00	47.00	769.50	436.55	432.96	4.81	0.01	437.76	0.78	0.003	0.005
15,00	2,00	49.00	906.50	456.17	450.34	4.43	0.01	454.77	0.78	0.002	0.005
										s_{tot} =	**0,243**

Patterns of time settlement curves are computed by means of *Terzaghi's* one dimensional consolidation theory. Given the average degree of consolidation U_m, the corresponding time factor T_v is obtained as summarized in Fig.A.3.178:

$$U_m = 2\sqrt{\frac{T_v}{\pi}}; \qquad T_v = \frac{\pi}{4}U_m^2 \qquad \text{per}\, U_m \le 60\%$$

$$T_v = 1{,}781 - 0{,}933\log\left(100 - U_m(\%)\right) \qquad \text{per}\, U_m > 60\%$$

(Terzaghi)

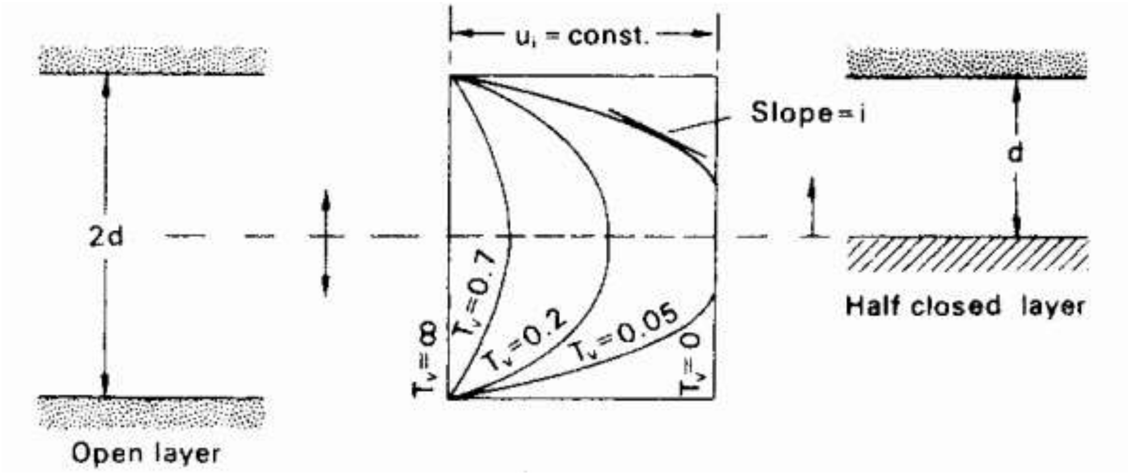


Fig.A.3.17 Main results from Terzaghi's solution of the one dimensional consolidation problem (*d* in the figure is *H* in the text)

$$T_v = \frac{c_v \cdot t}{H^2}$$

The end of consolidation time is strongly influenced by the assumed drainage length H. The selection of the drainage length is based on the interpretation of CPTu tests of Fig. A.3.6 Results from CPTu testingFig. A.3.6; H values are different for the two settlement calculation schemes: if the ten per cent limitation is adopted, only 3 metres of clay is considered, so that $2H$=3 m and the resulting time-settlement plot is given in Fig.A.3.189. If the 10% limitation is removed, the drainage length is the half of the maximum distance between two sandy layers, approximately 7 metres from Fig. A.3.6. For this reason H= 3,5 m in the second scheme and the resulting time-settlement plot is given in Fig.A.3..

Adopting the limitation of 10% for the stress increment, the final settlement is E_d = (3,3 + 2,1) cm. Therefore $E_d \cong C_d$ = 5 cm and the SLS requirement can be considered as satisfied.

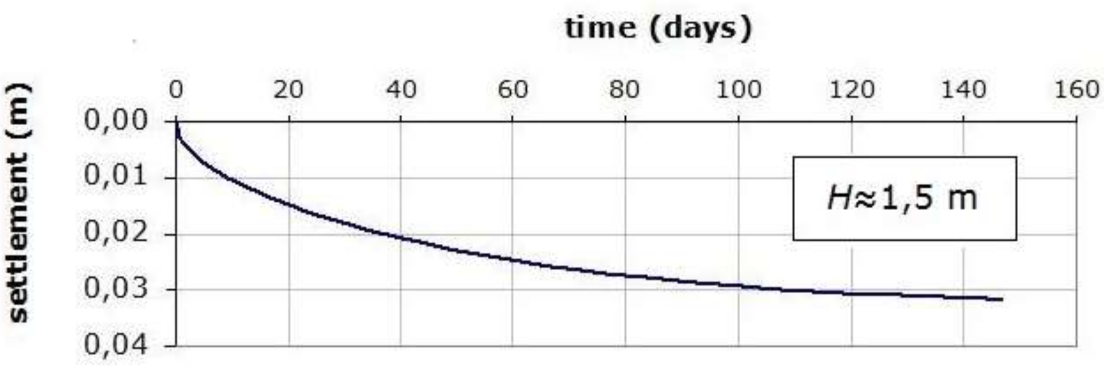


Fig.A.3.189 Time settlement curve for the single footing scheme (*H*=1,5 m)

Fig.A.3.20 Time settlement curve for the whole footing scheme (*H*=3,5 m)

A.4. Worked example to accompany Chapter 4

This worked example illustrates the way in which a T-shaped wall may be designed according to *Eurocode 7*. Two solutions are included – one for Design Approach 1 (DA1) and the other for DA2*.

The design situation involves a T-shaped gravity wall with that is required to support granular fill. This ground and the fill are both dry.

A.4.1. DESIGN SITUATION

Consider a T-shaped gravity wall (Figure A.4.1) with retained height H=6,0 m that is required to support granular fill with characteristic weight density γ_k =19 kN/m^3 and drained strength parameters φ_k = 32,5° and c'_k =0 kPa.

A variable surcharge p_k=5 kPa acts behind the wall on ground that rises at an angle β = 20° to the horizontal.

The dimensions of the wall are as follows: overall breadth (assumed) B=3,9 m; base thickness t_b=0,8 m; toe width b_t=0,95 m; thickness of wall stem t_s=0,7 m. The weight density of reinforced concrete is $\gamma_{c,k}$=25 kN/m^3.

The properties of the ground beneath the wall are the same as the fill. This ground and the fill are both dry.

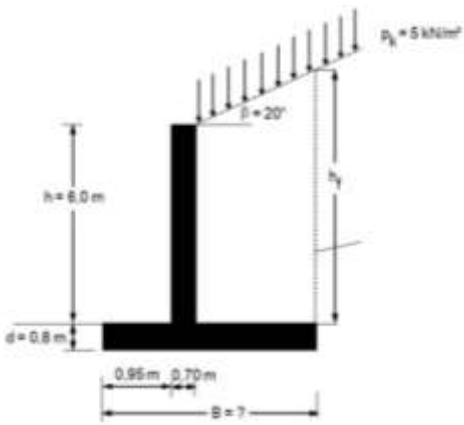


Fig.A.4.1 T-shaped gravity wall

Geometry

Width wall heel is:

$$b_{heel} = B - b_t - t_s = 2{,}25\text{m}$$

Height of fill above wall heel is:

$$h_f = H + b_{heel}\tan(\beta) = 6{,}82\text{m}$$

Height of wall above wall heel including the thickness of base is:

$$H_{heel} = h_f + t_b = 7{,}62\text{m}$$

Depth to base of footing is:

$$d = t_b = 0{,}8\text{m}$$

A.4.2. T-SHAPED WALL: VERIFICATION OF DRAINED STRENGTH (LIMIT STATE GEO) - DESIGN APPROACH 1

Material properties

Partial factors

$$\text{from Set}\begin{pmatrix} M1 \\ M2 \end{pmatrix}: \gamma_\varphi = \begin{pmatrix} 1 \\ 1{,}25 \end{pmatrix} \text{ and } \gamma_c = \begin{pmatrix} 1 \\ 1{,}25 \end{pmatrix}$$

Design angle of shearing resistance:

$$\varphi_d = \text{atan}\left(\frac{\tan(\varphi_k)}{\gamma_\varphi}\right) = \begin{pmatrix} 32{,}5 \\ 27 \end{pmatrix}^\circ$$

Design effective cohesion:

$$c'_d = \frac{c'_k}{\gamma_c} = \begin{pmatrix} 0 \\ 0 \end{pmatrix}\text{kPa}$$

Actions

Characteristic self-weight of wall (permanent action) is:

- wall stem $W_{stem,Gk} = \gamma_{c,k} t_s H = 105\text{kN/m}$
- wall base $W_{base,Gk} = \gamma_{c,k} t_b B = 78\text{kN/m}$

Characteristic total self-weight of wall is:

$$W_{wall,Gk} = W_{stem,Gk} + W_{base,Gk} = 183\text{kN/m}$$

Characteristic total self-weight of backfill is:

$$W_{fill,Gk} = \gamma_k b_{heel} \frac{H + hf}{2} = 274\text{kN/m}$$

Characteristic total self-weight of wall including backfill is then:

$$W_{Gk} = W_{wall,Gk} + W_{fill,Gk} = 457\text{kN/m}$$

Characteristic surcharge (variable) is:

$$q_{Qk} = p_k = 5\text{kPa}$$

$$Q_{Qk} = q_{Qk} b_{heel} = 11{,}3\text{kN/m}$$

Design active earth pressure coefficient (for calculating inclined thrust) is:

$$K_{a\beta,d} = \left(\frac{cos(\beta) - \sqrt{cos(\beta)^2 - \cos(\varphi_d)^2}}{cos(\beta) + \sqrt{cos(\beta)^2 - \cos(\varphi_d)^2}} \right) \cos(\beta) = \begin{pmatrix} 0{,}365 \\ 0{,}486 \end{pmatrix}$$

Equivalent coefficient for calculating horizontal thrust is:

$$K_{ah,d} = K_{a\beta,d} cos(\beta) = \begin{pmatrix} 0{,}343 \\ 0{,}457 \end{pmatrix}$$

Partial factors

$$\text{from Set} \begin{pmatrix} A1 \\ A2 \end{pmatrix} : \gamma_G = \begin{pmatrix} 1{,}35 \\ 1 \end{pmatrix} \text{ and } \gamma_Q = \begin{pmatrix} 1{,}5 \\ 1{,}3 \end{pmatrix}$$

Design thrust (inclined at angle β to the horizontal) from earth pressure on back of virtual plane is:

- from ground $E_{a,Gd} = \overrightarrow{\left[\gamma_G K_{a\beta,d} \left(\frac{1}{2} \gamma_k H_{heel}^2 \right) \right]} = \begin{pmatrix} 271{,}4 \\ 268{,}2 \end{pmatrix} \text{kN/m}$
- from surcharge $E_{a,Qd} = \overrightarrow{\left(\gamma_Q K_{a\beta,d} q_{Qk} H_{heel} \right)} = \begin{pmatrix} 20{,}8 \\ 24{,}1 \end{pmatrix} \text{kN/m}$
- total $E_{a,d} = E_{a,Gd} + E_{a,Qd} = \begin{pmatrix} 292{,}2 \\ 292{,}3 \end{pmatrix} \text{kN/m}$

Horizontal component of design thrust is then:

$$H_{Ed} = E_{a,d} cos(\beta) = \begin{pmatrix} 274{,}6 \\ 274{,}7 \end{pmatrix} \text{kN/m}$$

Vertical/normal component of design weight and thrust:

$$N_{Ed} = \gamma_G W_{Gk} + E_{a,d} sin(\beta) = \begin{pmatrix} 716{,}9 \\ 557 \end{pmatrix} \text{kN/m}$$

Moments about wall toe - bearing design situation

Design overturning moments (destabilizing) about wall toe:

- from ground $M_{Gd} = E_{a,Gd}\left(\frac{1}{3}H_{heel}cos(\beta) - Bsin(\beta)\right) = \begin{pmatrix}285,7\\282,3\end{pmatrix}$kNm/m
- from surcharge $M_{Qd} = E_{a,Qd}\left(\frac{1}{3}H_{heel}cos(\beta) - Bsin(\beta)\right) = \begin{pmatrix}46,8\\54,1\end{pmatrix}$kNm/m
- total design destabilising moment is: $M_{dst,d} = M_{Gd} + M_{Qd} = \begin{pmatrix}332,5\\336,4\end{pmatrix}$kNm/m

Design restoring moments (stabilizing) about wall toe:

- from wall stem $M_{stem,Gd} = \gamma_{G,fav}W_{stem,Gk}\left(b_t + \frac{t_s}{2}\right) = 136,5$kNm/m
- from wall base $M_{base,Gd} = \gamma_{G,fav}W_{base,Gk}\frac{B}{2} = 152,1$kNm/m
- from backfill $M_{fill,Gd} = \gamma_{G,fav}\gamma_k b_{heel}\left[H\left(B - \frac{b_{heel}}{2}\right) + \left(\frac{h_f - H}{2}\right)\left(B - \frac{b_{heel}}{3}\right)\right] = 766,9$kNm/m
- from surcharge $M_{Qd,fav} = \gamma_{G,fav}Q_{Qk}\frac{b_{heel}}{2} = 12,7$kNm/m
- total design stabilising moment is: $M_{stb,d} = M_{stem,Gd} + M_{base,Gd} + M_{fill,Gd} + M_{Qd,fav} = 1068$kNm/m

Line of action of resultant force is a distance from the toe:

$$x = \frac{M_{stb,d} - M_{dst,d}}{N_{Ed}} = \begin{pmatrix}1,03\\1,31\end{pmatrix}\text{m}$$

Eccentricity of actions from centre line of base is:

$$e_d = \frac{B}{2} - x = \begin{pmatrix}0,92\\0,64\end{pmatrix}\text{m}$$

Effective width of base is then:

$$B'_d = B - 2e_d = \begin{pmatrix}2,05\\2,63\end{pmatrix}\text{m}$$

Bearing resistance

Design bearing capacity factors:

$$N_{q,d} = \overrightarrow{\left[e^{\pi\tan(\varphi_d)}\left(\tan\left(45° + \frac{\varphi_d}{2}\right)\right)^2\right]} = \begin{pmatrix}24,6\\13,2\end{pmatrix}$$

$$N_{\gamma,d} = \overrightarrow{\left[2\left(N_{q,d} - 1\right)\tan\left(\varphi_d\right)\right]} = \begin{pmatrix}30,1\\12,4\end{pmatrix}$$

Shape factors (for an infinitely long footing): s_q=1,0 and s_γ = 1,0

Inclination factors: (using m_B = 2 for an infinitely long footing)

$$i_q = \overrightarrow{\left(1 - \frac{H_{Ed}}{N_{Ed} + Ac'_d \cot(\varphi_d)}\right)^{m_B}} = \begin{pmatrix} 0,38 \\ 0,26 \end{pmatrix}$$

$$i_\gamma = i_q^{\frac{m_B+1}{m_B}} = \begin{pmatrix} 0,23 \\ 0,13 \end{pmatrix}$$

Partial factors

from Set R1: $\gamma_{Rv} = 1$

Design bearing resistance (in terms of pressure) is:

- from overburden $q_{Rvq,d} = \overrightarrow{\left(\frac{\gamma_k d N_{q,d} s_q i_q}{\gamma_{Rv}}\right)} = \begin{pmatrix} 142,2 \\ 51,6 \end{pmatrix}$ kPa
- from body-mass $q_{Rv\gamma,d} = \overrightarrow{\left(\frac{1}{2}\frac{B'_d \gamma_k N_{\gamma,d} s_\gamma i_\gamma}{\gamma_{Rv}}\right)} = \begin{pmatrix} 137,6 \\ 40,4 \end{pmatrix}$ kPa
- total $q_{Rv,d} = q_{Rvq,d} + q_{Rv\gamma,d} = \begin{pmatrix} 279,8 \\ 92 \end{pmatrix}$ kPa

Characteristic bearing resistance (in terms of force) is:

$$N_{Rd} = \overrightarrow{(q_{Rv,d} B'_d)} = \begin{pmatrix} 574 \\ 242 \end{pmatrix} \text{kN/m}$$

A.4.3. VERIFICATIONS

Verification of resistance to sliding

Partial factors

from Set R1: $\gamma_{Rh} = 1$

For cast-in-place concrete, interface friction angle is *k*=1 times the constant-volume angle of shearing

Assume $\varphi_{cv,k}$ =30°

$$\delta_d = k\varphi_{cv,k} = 30°$$

Design sliding resistance (drained), ignoring adhesion (as required by *EN 1997-1 exp. 6.3a*)

$$H_{Rd} = \frac{\gamma_{G,fav} N_{Ed} \tan(\delta_d)}{\gamma_{Rh}} = \begin{pmatrix} 413,9 \\ 321,6 \end{pmatrix} \text{kN/m}$$

'Degree of utilization' $\Lambda = \frac{H_{Ed}}{H_{Rd}} = \begin{pmatrix} 66 \\ 85 \end{pmatrix}\%$ or 'Overdesign factor' $\text{ODF} = \frac{H_{Rd}}{H_{Ed}} = \begin{pmatrix} 1,51 \\ 1,17 \end{pmatrix}$

The design is unacceptable if the degree of utilization is > 100% (or overdesign factor is < 1)

Verification of bearing resistance

Design bearing resistance is:

$$N_{Rd} = \begin{pmatrix} 574,4 \\ 241,8 \end{pmatrix} \text{kN/m}$$

Design bearing force is:

$$N_{Ed} = \begin{pmatrix} 716,9 \\ 557 \end{pmatrix} \text{kN/m}$$

'Degree of utilization' $\Lambda = \frac{N_{Ed}}{N_{Rd}} = \begin{pmatrix} 125 \\ 230 \end{pmatrix}\%$ or 'Overdesign factor' $\text{ODF} = \frac{N_{Rd}}{N_{Ed}} = \begin{pmatrix} 0,8 \\ 0,43 \end{pmatrix}$

The design is unacceptable if the degree of utilization is > 100% (or overdesign factor is < 1)

Verification of resistance to toppling

Design de-stabilizing moment is:

$$M_{Ed,dst} = M_{Gd} + M_{Qd} = \begin{pmatrix} 332 \\ 336 \end{pmatrix} \text{kNm/m}$$

Design stabilizing moment is (approximately):

$$M_{Ed,stb} = M_{stb,d} = 1068 \text{kNm/m}$$

'Degree of utilization' $\Lambda = \frac{M_{Ed,dst}}{M_{Ed,stb}} = \begin{pmatrix} 31 \\ 31 \end{pmatrix}\%$ or 'Overdesign factor' ODF=$\frac{M_{Ed,stb}}{M_{Ed,dst}} = \begin{pmatrix} 3,21 \\ 3,17 \end{pmatrix}$

The design is unacceptable if the degree of utilization is > 100% (or overdesign factor is < 1)

A.4.4. T-SHAPED WALL: VERIFICATION OF DRAINED STRENGTH (LIMIT STATE GEO) - DESIGN APPROACH 2*

Material properties

Characteristic material properties are used throughout this calculation.

Actions

Characteristic self-weight of wall (permanent action) is:

- wall stem $W_{stem,Gk} = \gamma_{c,k} t_s H = 105 \text{kN/m}$
- wall base $W_{base,Gk} = \gamma_{c,k} t_b B = 78 \text{kN/m}$

Characteristic total self-weight of wall is:

$$W_{wall,Gk} = W_{stem,Gk} + W_{base,Gk} = 183 \text{kN/m}$$

Characteristic total self-weight of backfill is:

$$W_{fill,Gk} = \gamma_k b_{heel} \frac{H + hf}{2} = 274 \text{kN/m}$$

Characteristic total self-weight of wall including backfill is then:

$$W_{Gk} = W_{wall,Gk} + W_{fill,Gk} = 457\,\text{kN/m}$$

Characteristic surcharge (variable) is:

$$q_{Qk} = p_k = 5\,\text{kPa}$$

$$Q_{Qk} = q_{Qk} b_{heel} = 11{,}3\,\text{kN/m}$$

Design active earth pressure coefficient (for calculating inclined thrust) is:

$$K_{a\beta,k} = \left(\frac{cos(\beta) - \sqrt{cos(\beta)^2 - \cos(\varphi_k)^2}}{cos(\beta) + \sqrt{cos(\beta)^2 - \cos(\varphi_k)^2}} \right) \cos(\beta) = 0{,}365$$

Equivalent coefficient for calculating horizontal thrust is:

$$K_{ah,k} = K_{a\beta,k} cos(\beta) = 0{,}343$$

Characteristic inclined thrust (at angle β to the horizontal) from earth pressure on back of virtual plane is:

- from ground $E_{a,Gk} = K_{a\beta,k} \left(\frac{1}{2} \gamma_k H_{heel}^2 \right) = 201\,\text{kN/m}$
- from surcharge $E_{a,Qk} = K_{a\beta,k} q_{Qk} H_{heel} = 13{,}9\,\text{kN/m}$
- total $E_{a,k} = E_{a,Gk} + E_{a,Qk} = 214{,}9\,\text{kN/m}$

Horizontal component of design thrust is then:

$$H_{Ek} = E_{a,k} cos(\beta) = 202\,\text{kN/m}$$

Vertical/normal component of design weight and thrust:

$$N_{Ek} = \gamma_G W_{Gk} + E_{a,k} sin(\beta) = 530{,}5\,\text{kN/m}$$

Moments about wall toe - bearing design situation

Design overturning moments (destabilizing) about wall toe:

- from ground $M_{Gk} = E_{a,Gk} \left(\frac{1}{3} H_{heel} cos(\beta) - B sin(\beta) \right) = 211{,}6\,\text{kNm/m}$
- from surcharge $M_{Qk} = E_{a,Qk} \left(\frac{1}{3} H_{heel} cos(\beta) - B sin(\beta) \right) = 31{,}2\,\text{kNm/m}$
- total design destabilising moment is: $M_{dst,k} = M_{Gk} + M_{Qk} = 242{,}8\,\text{kNm/m}$

Design restoring moments (stabilizing) about wall toe:

- from wall stem $M_{stem,Gk} = W_{stem,Gk} \left(b_t + \frac{t_s}{2} \right) = 136{,}5\,\text{kNm/m}$

- from wall base $M_{base,Gk} = W_{base,Gk} \dfrac{B}{2} = 152{,}1\text{kNm/m}$

- from backfill $M_{fill,Gk} = \gamma_k b_{heel} \left\lfloor H\left(B - \dfrac{b_{heel}}{2}\right) + \left(\dfrac{h_f - H}{2}\right)\left(B - \dfrac{b_{heel}}{3}\right)\right\rfloor = 766{,}9\text{kNm/m}$

- total design stabilising moment is: $M_{stb,k} = M_{stem,Gk} + M_{base,Gk} + M_{fill,Gk} = 1055{,}5\text{kNm/m}$

Line of action of resultant force is a distance from the toe:

$$x = \frac{M_{stb,k} - M_{dst,k}}{N_{Ek}} = 1{,}53\text{m}$$

Eccentricity of actions from centre line of base is:

$$e_k = \frac{B}{2} - x = 0{,}42\text{m}$$

Effective width of base is then:

$$B'_d = B - 2e_k = 3{,}06\text{m}$$

Bearing resistance

Design bearing capacity factors:

$$N_{q,k} = e^{\pi \tan(\varphi_k)}\left(\tan\left(45° + \frac{\varphi_k}{2}\right)\right)^2 = 24{,}6$$

$$N_{\gamma,k} = 2\left(N_{q,d} - 1\right)\tan\left(\varphi_k\right) = 30{,}1$$

Shape factors (for an infinitely long footing): s_q=1,0 and s_γ = 1,0

Inclination factors: (using m_B = 2 for an infinitely long footing)

$$i_q = \left(1 - \frac{H_{Ek}}{N_{Ek} + Ac'_k \cot(\varphi_k)}\right)^{m_B} = 0{,}38$$

$$i_\gamma = i_q^{\frac{m_B + 1}{m_B}} = 0{,}23$$

Characteristic bearing resistance (in terms of pressure) is:

- from overburden $q_{Rvq,k} = \gamma_k d N_{q,k} s_q i_q = 143{,}3\text{kPa}$

- from body-mass $q_{Rv\gamma,k} = \dfrac{1}{2} B' \gamma_k N_{\gamma,d} s_\gamma i_\gamma = 207{,}8\text{ kPa}$

- total $q_{Rv,k} = q_{Rvq,k} + q_{Rv\gamma,k} = 351{,}1\text{kPa}$

Characteristic bearing resistance (in terms of force) is:

$$N_{Rk} = q_{Rv,k} B' = 1076\text{kN/m}$$

A.4.5. VERIFICATIONS

Verification of resistance to sliding

Partial factors from Set A1: γ_G = 1,35; $\gamma_{G,fav}$ = 1 and γ_Q = 1,5.

Design thrust (at angle β to the horizontal) from earth pressure on back of virtual plane is:

- from ground $E_{a,Gd} = \gamma_G E_{a,Gk} = 271{,}4\,\text{kN/m}$
- from surcharge $E_{a,Qd} = \gamma_Q E_{a,Qk} = 20{,}8\,\text{kN/m}$
- total $E_{a,d} = E_{a,Gd} + E_{a,Qd} = 292{,}2\,\text{kN/m}$

Horizontal component of design thrust is then:

$$H_{Ed} = E_{a,d} cos(\beta) = 274{,}6\,\text{kN/m}$$

Vertical/normal component of design weight and thrust:

$$N_{Ed} = \gamma_G W_{Gk} + E_{a,d} sin(\beta) = 716{,}9\,\text{kN/m}$$

Partial factors

from Set R2: γ_{Rh} = 1,1.

For cast-in-place concrete, interface friction angle is k=1 times the constant-volume angle of shearing

Assume $\varphi_{cv,k}$ =30°

$$\delta_k = k\varphi_{cv,k} = 30°$$

Design sliding resistance (drained), ignoring adhesion (as required by *EN 1997-1 exp. 6.3a*)

$$H_{Rd} = \frac{\gamma_{G,fav} N_{Ek} \tan(\delta_k)}{\gamma_{Rh}} = 278{,}4\,\text{kN/m}$$

'Degree of utilization' $\Lambda = \dfrac{H_{Ed}}{H_{Rd}}$ =99% or 'Overdesign factor' ODF $= \dfrac{H_{Rd}}{H_{Ed}} = 1{,}01$

The design is unacceptable if the degree of utilization is > 100% (or overdesign factor is < 1)

Verification of bearing resistance

Partial factors

from Set R2: γ_{Rv} = 1,4.

Design bearing resistance is:

$$N_{Rd} = \frac{N_{Rk}}{\gamma_{Rv}} = 768{,}4\,\text{kN/m}$$

Design bearing force is:

$$N_{Ed} = 716{,}9\,\text{kN/m}$$

'Degree of utilization' $\Lambda = \dfrac{N_{Ed}}{N_{Rd}} = 93\%$ or 'Overdesign factor' $\mathrm{ODF} = \dfrac{N_{Rd}}{N_{Ed}} = 1{,}07$

The design is unacceptable if the degree of utilization is > 100% (or overdesign factor is < 1)

Verification of resistance to toppling

Design de-stabilizing moment is:

$$M_{Ed,dst} = \gamma_G M_{Gk} + \gamma_Q M_{Qk} = 332\,\mathrm{kNm/m}$$

Design stabilizing moment is (approximately):

$$M_{Ed,stb} = \gamma_{G,fav} M_{stb,k} = 1056\,\mathrm{kNm/m}$$

'Degree of utilization' $\Lambda = \dfrac{M_{Ed,dst}}{M_{Ed,stb}} = 31\%$ or 'Overdesign factor' $\mathrm{ODF} = \dfrac{M_{Ed,stb}}{M_{Ed,dst}} = 3{,}17$

The design is unacceptable if the degree of utilization is > 100% (or overdesign factor is < 1)

A.5. Worked example to accompany Chapter 5

A.5.1. APPLICATION OF STATISTICAL METHODS TO DETERMINE CHARACTERISTIC VALUES OF GROUND PARAMETER

The following example (see Fig.A.5.10) explains the application of *EN 1997-1, paragraph (10) and the NOTE*. The ground has been investigated by 3 borings, where 11 samples have been taken from stiff clay. These samples were tested in the laboratory to determine the strength of the ground. There is some scatter in the undrained shear strength c_u of the soil, but no trend to the strength with increasing depth nor an apparent difference between the borings. So it can be assumed that the undrained shear strength is normally distributed.

The first part of the *NOTE* to *§2.4.5.2(10)* refers to a situation where a cut is planned e.g. for a railway line. Here the geotechnical designer has to verify the stability of the slope and as a first step he has to select the characteristic value for the undrained shear strength c_u. The potential slip surface lies in the volume of ground of which several test results have been performed. So the characteristic value can be determined as a cautious estimate of the mean value with a confidence level of 95%.

The second part of the *NOTE* refers to the situation where we have a footing, for which the designer has to verify the ground bearing capacity. The footing only has a small zone of ground governing the behaviour at the limit state. So this zone might be in an area, where the local strength might be lower. In this case the characteristic value is a local low value (index: $_{l,k}$), defined as a 5% fractile.

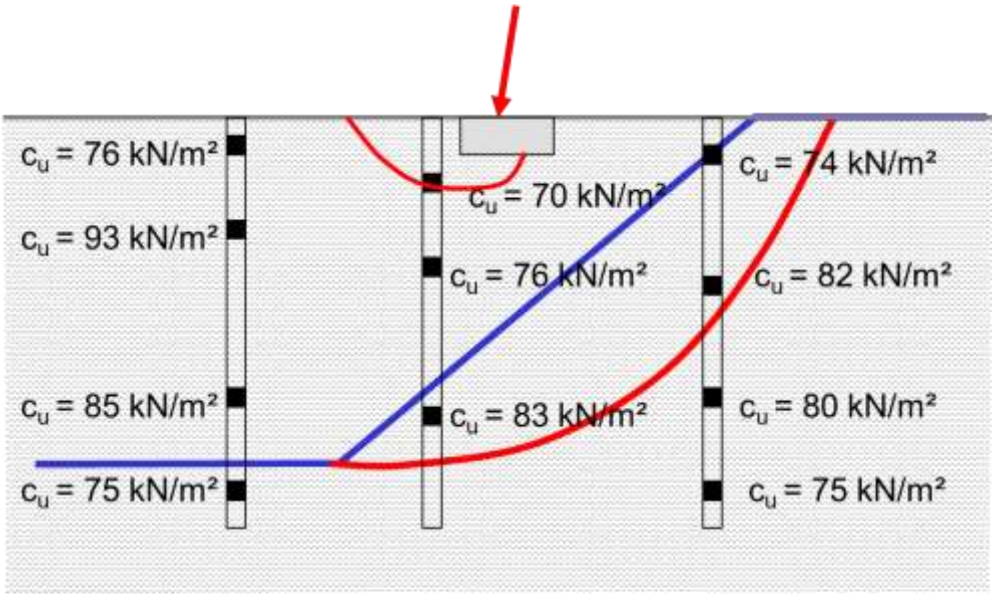


Fig.A.5.10 Examples of the selection of characteristic ground parameters – mean value and local values

The evaluation of the 11 tests gives a mean value of the undrained shear strength of $c_{u,m}$ = 79 kPa with a coefficient of variation V_{cu} = 0,08. The characteristic values for the two situations are shown in Table A.5.5. It can be seen that, in this case, the simple formula of *Schneider (1999)* gives the same result as the refined statistical formula.

Table A.5.5 Statistical evaluation of test results

	Equation	k_n	$c_{u,k}$ (kPa)
Global characteristic value	$c_{u,m,k} = c_{u,m} \cdot (1 - k_{n,m} \cdot V_{cu})$	0,55	75,5
	$c_{u,m,k} = c_{u,m} - 0{,}5 \cdot s_{cu}$	-	75,8
Local characteristic value	$c_{u,l,k} = c_{u,m} \cdot (1 - k_{n,l} \cdot V_{cu})$	1,89	67,2

A.5.2. CHARACTERISTIC VALUES FOR BASE AND SHAFT RESISTANCE OF PILE IN SAND

A building is to be supported on 450 mm diameter bored piles founded entirely in a medium dense to dense sand spaced at 2 m centres. This is a small project for which there will be no load testing. It is believed that settlement in service will not govern design. The sand is a Pleistocene fine and medium sand covered by Holocene layers of loose sand, soft clay and peat; one CPT was carried out (see Fig.A.5.11).

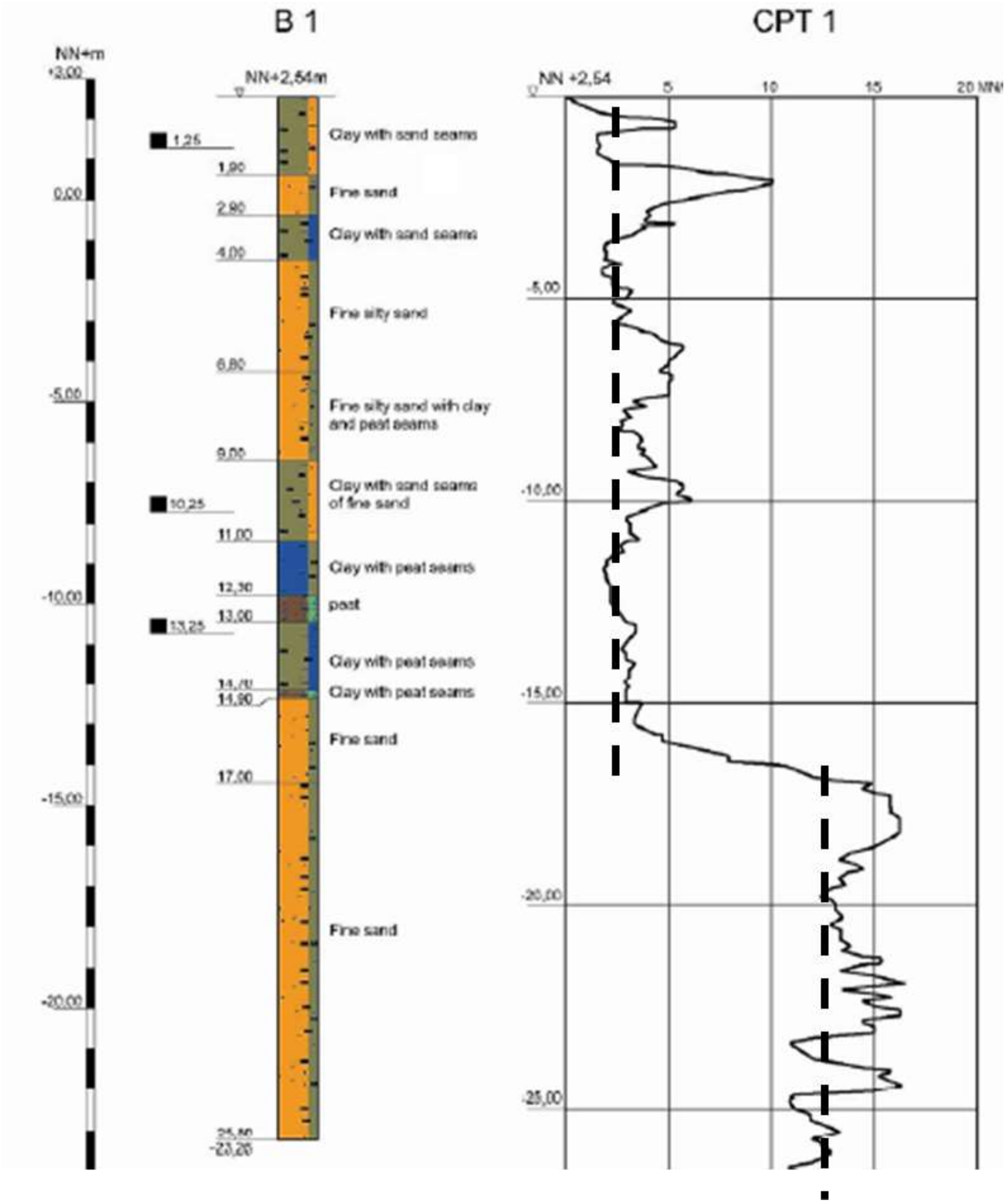


Fig.A.5.11 Ground conditions for the pile in sand

For the design of the piles use will be made of the provisions of *EN 1997-1* given in *§ 7.6.2.3 (8)*. The characteristic values of the base resistance $R_{b;k}$ and the shaft resistance $R_{s,k}$ may be obtained by calculating:

$$R_{b;k} = A_b \cdot q_{b;k} \text{ and } R_{s,k} = \Sigma A_{s,i} \cdot q_{s,i,k} \qquad \text{(A.0)(7.9)}$$

where A_b and $A_{s,i}$ are the base area and the shaft areas in the various strata, $q_{b;k}$ and $q_{s;i;k}$ are characteristic values of base resistance and shaft friction in the various strata, obtained from values of ground parameters."

To determine $q_{b;k}$ and $q_{s;i;k}$ *Tables D.3* and *D.4* of *Annex D* in *EN 1997-2* are used (see Table A.5.6 and

Normalised settlement s/D_s; s/D_b	**Unit base resistance p_b, in MPa, at average cone penetration resistance q_c (CPT) in MPa**			
	q_c=10	**q_c=15**	**q_c=20**	**q_c=25**
0,02	0,70	1,05	1,40	1,75
0,03	0,90	1,35	1,80	2,25
0,10(=s_g)	2,00	3,00	3,50	4,00

NOTE Intermediate values may be interpolated linearly.
In the case of cast in-situ piles with pile base enlargement, the values shall be multiplied by 0,75.
s is the normalised pile head settlement
D_s is the diameter of the pile shaft
D_b is the diameter of the pile base
s_g is the ultimate settlement of pile head

Table A.5.7).

Table A.5.6. Unit base resistance p_b of cast in-situ piles in coarse soil with little or no fines

Normalised settlement s/D_s; s/D_b	**Unit base resistance p_b, in MPa, at average cone penetration resistance q_c (CPT) in MPa**			
	q_c=10	**q_c=15**	**q_c=20**	**q_c=25**
0,02	0,70	1,05	1,40	1,75
0,03	0,90	1,35	1,80	2,25
0,10(=s_g)	2,00	3,00	3,50	4,00

NOTE Intermediate values may be interpolated linearly.
In the case of cast in-situ piles with pile base enlargement, the values shall be multiplied by 0,75.
s is the normalised pile head settlement
D_s is the diameter of the pile shaft
D_b is the diameter of the pile base
s_g is the ultimate settlement of pile head

Table A.5.7. Unit shaft resistance q_s of cast in-situ piles in coarse soil with little or no fines

Average cone penetration resistance q_c (CPT) MPa	**Unit shaft resistance p_s MPa**
0	0
5	0,040
10	0,080
≥ 15	0,120

NOTE Intermediate values may be interpolated linearly

The dashed lines in Fig.A.5.11 show how the CPT was evaluated with respect to the cone resistance q_c which is the basis for establishing the base and shaft resistance of the piles. For the cone resistance a mean value of q_c = 2,5 MN/m² was assumed up to NN –16,5 m and

Normalised settlement s/D_s; s/D_b	**Unit base resistance p_b, in MPa, at average cone penetration resistance q_c (CPT) in MPa**

	q_c=10	q_c=15	q_c=20	q_c=25
0,02	0,70	1,05	1,40	1,75
0,03	0,90	1,35	1,80	2,25
0,10(=s_g)	2,00	3,00	3,50	4,00

NOTE Intermediate values may be interpolated linearly.
In the case of cast in-situ piles with pile base enlargement, the values shall be multiplied by 0,75.
s is the normalised pile head settlement
D_s is the diameter of the pile shaft
D_b is the diameter of the pile base
s_g is the ultimate settlement of pile head

Table A.5.7 then gives a value of $q_{s,k}$ = 0,02 MPa for the shaft resistance. For the base resistance a low mean value of q_c = 12,5 MN/m² below NN was selected from NN –16,5 m. As settlement in service will not govern design the values for a normalized pile settlement for s_g = 0,10·D from

Normalised settlement s/D_s; s/D_b	**Unit base resistance p_b, in MPa, at average cone penetration resistance q_c (CPT) in MPa**			
	q_c=10	**q_c=15**	**q_c=20**	**q_c=25**
0,02	0,70	1,05	1,40	1,75
0,03	0,90	1,35	1,80	2,25
0,10(=s_g)	2,00	3,00	3,50	4,00

NOTE Intermediate values may be interpolated linearly.
In the case of cast in-situ piles with pile base enlargement, the values shall be multiplied by 0,75.
s is the normalised pile head settlement
D_s is the diameter of the pile shaft
D_b is the diameter of the pile base
s_g is the ultimate settlement of pile head

Table A.5.7 was taken. Here local low value for the pile base will be relevant for design: $q_{b,k}$ (s_g)= 2,5 MPa.

A.5.3. CHARACTERISTIC VALUES FOR BASE AND SHAFT RESISTANCE OF PILE IN STIFF CLAY

In this example the same type of pile will be used as in the example with the piles in sand. The settlement in service is limited to 20mm. The clay is an overconsolidated marine clay. There were several different types of tests performed in the clay. In this example I will concentrate on the results of UU-triaxial tests shown in Fig.A.5.12. The UU-tests give as results the undrained shear strength c_u, which are the basis for the characteristic values of the shaft und the base resistance of the piles.

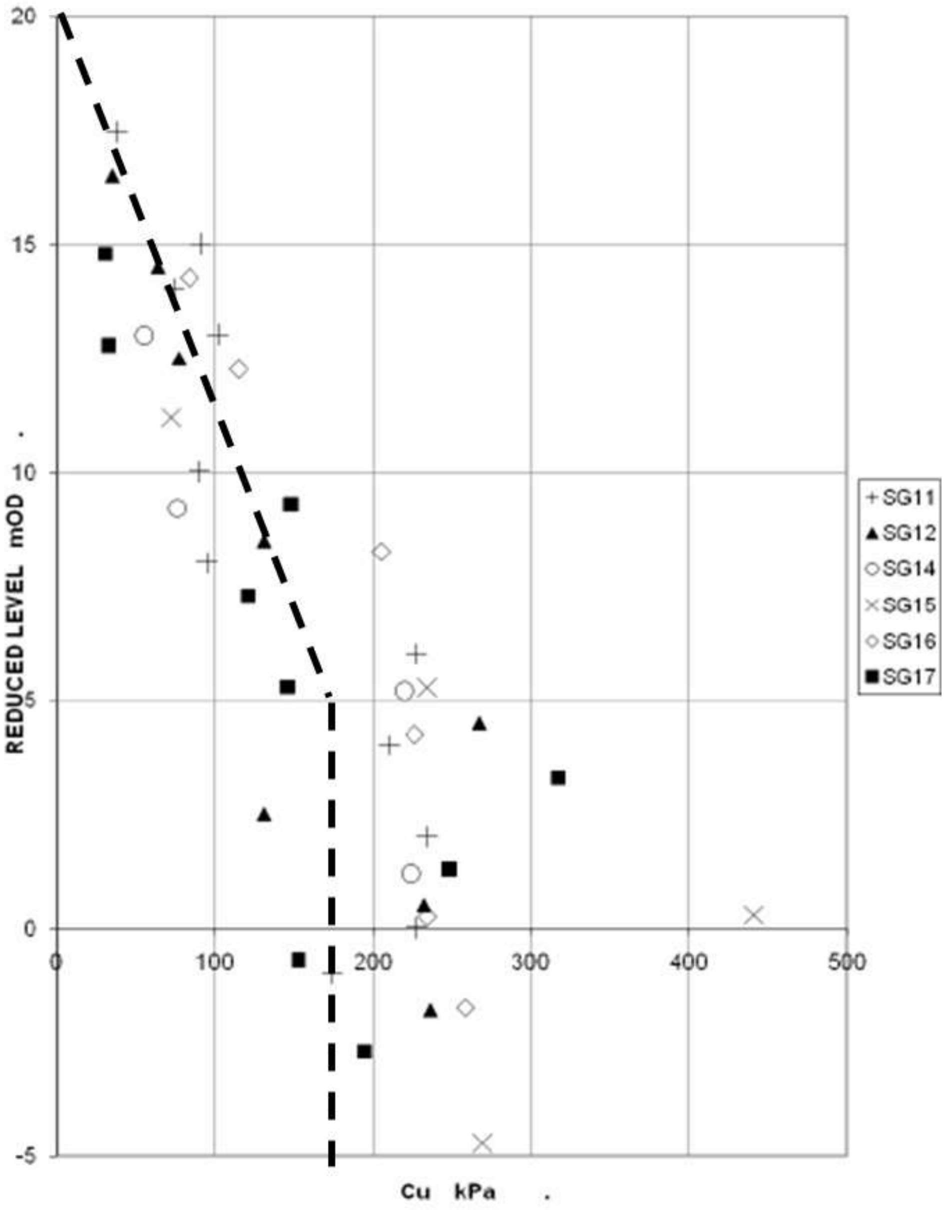


Fig.A.5.12. Undrained shear strength c_u as results of UU triaxial tests with specimen from samples of 6 borings

To evaluate the test results with respect to the base and shaft resistance of piles *Tables B.4* and *B.2* of *Annex B* in *DIN 1054:2002* were used (see Table A.5.8 and Table A.5.9).

Table A.5.8 Unit shaft resistance $q_{s,k}$ of cast in situ piles in clay

Undrained shear strength $c_{u,k}$ MN/m^2	Unit shift resistance $q_{s,k}$ MN/m^2
0,025	0,025
0,10	0,040
≥ 0,20	0,060
Intermediate values may be interpolated linearly	

Table A.5.9 Unit base resistance $q_{b,k}$ of cast in situ piles in clay

Normalized pile settlement s/D_s bzw. s/D_b	Unit base resistance $q_{b,k}$ MN/m² Undrained shear strength $c_{b,k}$ MN/m²	
	0,10	**0,20**
0,02	0,35	0,90
0,03	0,45	1,10
0,10 (s_g)	0,80	1,50
Intermediate values may be interpolated linearly		

From Fig.A.5.12 it was assumed that, from +20,0 mOD to +5,0 mOD, the characteristic value of the undrained shear strength cu increases linearly from 0 to $c_{u,k}$ = 180 kPa. Below +5,0 mOD a value of $c_{u,k}$ = 180 kPa is assumed. Table A.5.8 then gives a linear increase of the shaft resistance from 0 to $q_{s,k}$ = 0,056 MN/m² and with Table A.5.9 a base resistance of $q_{b,k}$ = 0,97 MN/m² assuming a settlement of s=20mm ≈ 0,03D_{pile}=13,5 mm.

A.6. Worked example to accompany Chapter 6

The following worked examples illustrate different approaches to applying *expressions (2.8) and (2.9) of EN 1997-1* (Eqn.6.1, Eqn.6.2 and Eqn.6.3 in Chapter 6).

A.6.1. VERIFICATION AGAINST OF FAILURE OF UPLIFT

The following worked example illustrates different approaches to applying Eqn.6.1 *(expression (2.8), EN 1997-1)* in the verification of uplift. Fig.A.6.1 shows the ground conditions and the cross-section of an excavation, where the groundwater can rise to the ground-level. First the sheet pile walls are placed, then the ground is excavated below the ground water, the struts are installed, the under-water concrete slab is placed and the groundwater is pumped out of the excavation. For this situation the excavation must have a sufficient safety against failure due to uplift. This situation is a transient design situation during construction in which reduced safety factors may be used in Germany.

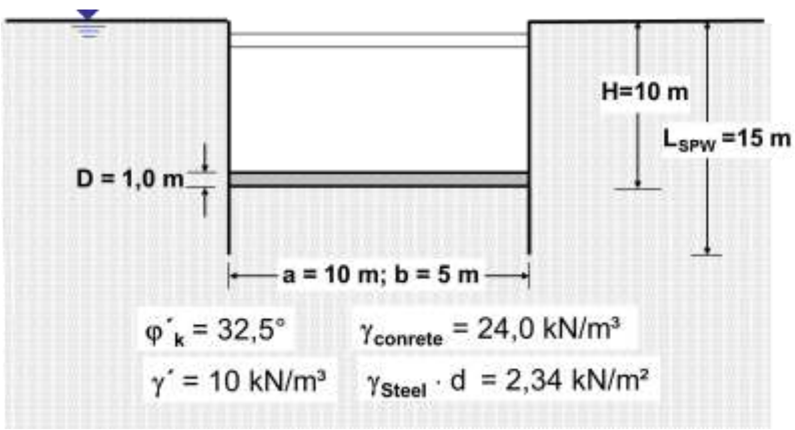


Fig.A.6.1 Cross-section and ground conditions for an excavation

In *EN 1997-1 §2.4.7.4*, we find the following provisions for uplift:

"(1)P Verification for uplift (UPL) shall be carried out by checking that the design value of the combination of destabilising permanent and variable vertical actions ($V_{dst;d}$) is less than or equal to the sum of the design value of the stabilising permanent vertical actions ($G_{stb;d}$) and of the design value of any additional resistance to uplift (R_d):

$V_{dst,d} \leq G_{stb;d} + R_d$ (2.8)

where

$V_{dst,d} = G_{dst;d} + Q_{dst;d}$

(2) Additional resistance to uplift may also be treated as a stabilising permanent vertical action ($G_{stb;d}$)."

The partial factors of safety used in Germany and Ireland are presented in Table A.10.1.

Table A.10.1 UPL partial factors and model factor used in Germany and Ireland

Factor type	**Factor**	**EN 1997-1**	**German NA and DIN 1054**	**Irish NA**
Permanent unfavourable action	$\gamma_{G;dst}$	1,0	1,0	1,0
Permanent favourable action	$\gamma_{G;stb}$	0,9	0,9	0,9
Variable unfavourable action	$\gamma_{Q;dst}$	1,5	1,5	1,5
Angle of shearing resistance	γ_{ϕ}	1,25	1,25	1,25
Pile tensile resistance	$\gamma_{s;t}$	1,4	1,4	2,0
Model factor on wall resistance	η	-	0,8	-

In a first step a UPL-verification with the weight of the structure only is performed.

Self-weight base-slab:

Base-slab: $d = 1{,}0$ m, $\gamma_{concrete} = 24{,}0$ kN/m³

Base-area: $A = a \cdot b = 10{,}0 \cdot 5{,}0 = 50{,}00$ m²

$$G_{concrete} = A \cdot d \cdot \gamma_{concrete} = 50 \cdot 1{,}0 \cdot 24{,}0 = 1200\ kN.$$

Self weight sheet-pile-wall:

$$d_{SPW} = 0{,}03\ m; \qquad \gamma_{steel} = 78{,}0\ kN/m^3$$

Weight per unit area: $g = 78{,}0 \cdot 0{,}03 = 2{,}34$ kN/m²,

$$_0G_{SPW} = L_{SPW} \cdot 2 \cdot (a+b) \cdot g = 15 \cdot 2 \cdot (10+5) \cdot 2{,}34 = 1053\ kN$$

Total characteristic weight of the structure:

$$G_k = G_{concrete} + G_{SPW} = 1200 + 1053 = 2253\ kN$$

Water-pressure: $U_k = H \cdot \gamma_w \cdot b \cdot a$ = 5000 kN

Inserting the values in Eqn.6.1 *(eq. (2.8), EN 1997-1)* with R_d = 0:

$$V_{dst,d} = U_k \cdot \gamma_{G,dst} \leq G_{stb;d} = G_k \cdot \gamma_{G,sbt}$$

$$5000 \cdot 1{,}00 = 5000 \text{ kN} > 2253 \cdot 0{,}90 = 2028 \text{ kN}$$

Hence the requirement is not fulfilled.

In the next step an UPL-verification including wall frictional resistance R is performed. The characteristic value of the wall frictional resistance R_k will be treated as a stabilising action determined as the vertical component of the characteristic active earth pressure reduced by a model factor of η = 0,80:

$$R_k = E_{ah,k} \cdot \tan\delta_a \cdot \eta$$

The wall friction angle is assumed to be δ_a = 2/3 φ'_k. For horizontal surface area and vertical wall the earth pressure coefficient is $K_{ah,k}$ = 0,25 (from *Figure C 1.1 of EN 1997-1* for φ'_k = 32,5°) and tan δ_a = 0,397. The earth pressure is assumed to act only on the outer surface of the sheet pile wall:

$$E_{ah,k} = 2 \cdot (a + b) \cdot 0{,}5 \cdot L^2_{SPW} \cdot \gamma' \cdot K_{ah,k} = 2 \cdot (10 + 5) \cdot 0{,}5 \cdot 15^2 \cdot 10 \cdot 0{,}25 = 8437 \text{ kN}$$

$$R_k = E_{ah,k} \cdot \tan \delta_a \cdot \eta = 8437 \cdot 0{,}397 \cdot 0{,}8 = 2680 \text{ kN}$$

Eqn.6.1 (Eq. (2.8) of EN 1997-1) requires:

$$V_{dst,d} = U_d = 5000 > (G_k + R_k) \cdot \gamma_{G,stb} = (2253 + 2680) \cdot 0{,}90 = 4400 \text{ kN}$$

Hence UPL requirement is not satisfied. So additional tension piles are required.

EN 1997-1 states in §7.6.3 "(3)P For tension piles, two failure mechanisms shall be considered:

- *pull-out of the piles from the ground mass;*
- *uplift of the structure and the block of ground containing the piles."*

The design value of tensile action $F_{t,d}$ from the structure on the tension-pile is determined assuming the ultimate limit state GEO is fulfilled using Design Approach 2 (DA2):

$$F_{t,d} = U_d - (R_d + G_d)$$

$$F_{t,d} = U_k \cdot \gamma_G - (R_k + G_k) \cdot \gamma_{G,inf}$$

Where

γ_G = 1,20 for unfavourable actions (according to *DIN 1054* for transient situations: construction period)

$\gamma_{G,inf}$ = 1,00 for favourable effects of actions.

Inserting the values gives:

$$F_{t,d} = 5000 \cdot 1{,}20 - (2253 + 2680) \cdot 1{,}00 = 6000 - 4933 = 1067 \text{ kN}$$

For tension piles driven piles are used with a diameter of D = 0,5 m and a shaft friction of $q_{s,k}$ = 35 kN/m². The partial factor for these piles is γ_p = 1,40. This gives a design value $R_{t,d}$ for the pile resistance of the total length L_{tot} of the piles:

$$R_{t,d} = L_{tot} \cdot q_{s,k} \cdot \pi \cdot D / \gamma_P = 39{,}25 \cdot L_{tot}$$

With eq. (7.12) of EN 1997-1:

$$F_{t,d} \leq R_{t,d}$$

$$1067 \leq 39{,}25 \cdot L_{tot}$$

$$L_{tot} \geq 27{,}2 \text{ m}$$

Three piles with a length of 10 m are selected, their arrangement below the concrete slab is shown in Fig.A.6.2.

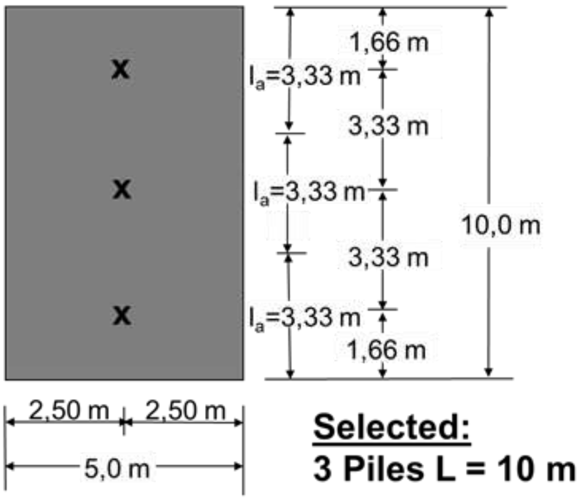


Fig.A.6.2 Arrangement of the tension piles below the concrete slab

For the verification against uplift of structure plus the ground block containing the piles the situation of Fig.A.13.3 is assumed, where Eqn.6.1 *(eq. (2.8), EN1997-1)* now includes the design value of the weight $G_{stb,d}$ of the ground block.

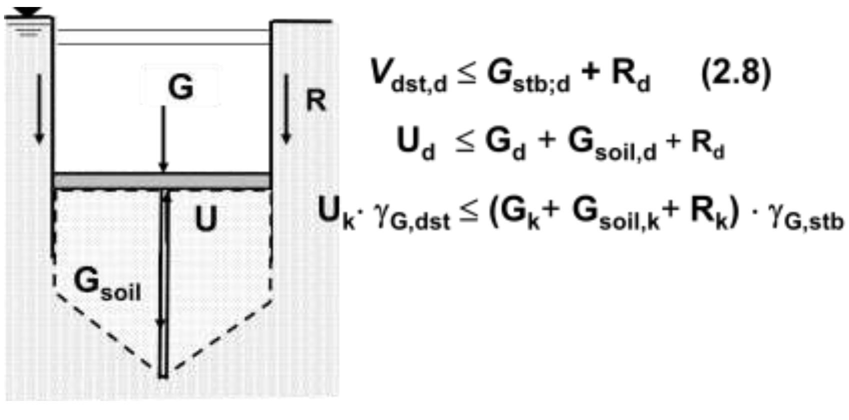


Fig.A.13.3 Assumption for the ground block containing the piles

For the ground block *DIN 1054* gives the following formula for the volume of a prism shown in Fig.A.:

$$G_{soil,k} = n \cdot \left\lfloor l_a \cdot l_b \cdot \left(L - 1/3 \cdot \sqrt{l_a^2 + l_b^2} \cdot \cot\varphi \right) \right\rfloor \cdot \eta \cdot \gamma'$$

Where:

n: number of piles,

L: length of piles

l_a = 5,0 m greater grid distance piles,

l_b = 3,33 m smaller grid distance piles,

γ' = 10,0 kN/m³ submerged weight soil

η = 0,80 model factor for the weight

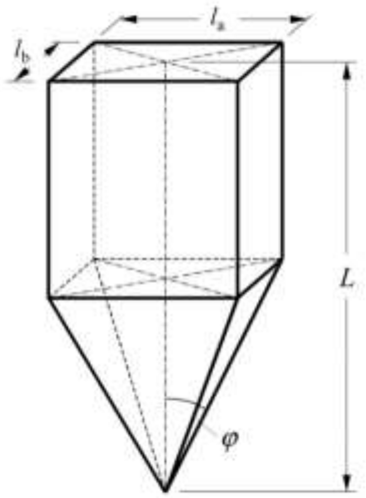


Fig.A.6.4 Assumption for the ground block containing the piles

Inserting the values for the ground block:

$$G_{soil,k} = 3 \cdot (5{,}0 \cdot 3{,}3 \cdot (10 - 1/3\sqrt{5{,}0^2 - 3{,}33^2} \cdot \cot 32{,}5°)) \cdot 8{,}0 \cdot 10 = 2740\,\text{kN}$$

Inserting the values in the extended Eqn.6.1 (*eq. (2.8), EN1997-1*):

$$U_k \cdot \gamma_{G,dst} \leq \left(G_k + R_k + G_{soil,k}\right) \cdot \gamma_{G,stb}$$

5000 ≤ 6906 kN

Hence the requirement for uplift is fulfilled with the tension piles.

Irish verification of uplift failure is carried out either by treating the side-wall force, R as stabilising resistance or, to be conservative and since it may be small due to soil disturbance, by ignoring it.

It should be noted that care is needed when treating R as a resistance and applying γ_ϕ to obtain R_d. This is because if $\varphi'_{k,inf}$ = 32,5^o is used, i.e. the inferior characteristic φ' value, and the UPL factor γ_ϕ = 1,25 from Table A.10.1 is applied to tan($\varphi'_{k,inf}$) to obtain φ'_d = 27,0^o so that δ_d = 2/3φ'_d = 18,0^o giving $K_{a;d}$ = 0,31, the design wall frictional resistance R_d=$E_{ah,d}$ tanδ_d= 2·(10+5)·15^2·0,5·10·0,31·0,325=3400 kN, which is greater than the characteristic value calculated using the values given above, i.e. R_k = $E_{ah,k}$ tanδ_k = 8437·0,397 = 3349 kN, so that this provides no margin of safety, which is not acceptable.

Hence a superior characteristic value $\varphi'_{k,sup}$ needs to be used. This is obtained, after *Bond and Harris (2008)*, by assuming a normal distribution for φ' and a standard deviation of 3^o so that $\varphi'_{k,sup}$ = $32{,}5^o$ + $2\cdot 1{,}624\cdot 3{,}0=42{,}2^o$. Then applying the partial factor of 1,25 as a multiplier to $\tan\varphi'_{k,sup}$ gives $\varphi'_{d,sup}=48{,}6^o$ and hence an R_d value of 2465 kN with a margin of safety of 1,13 on the characteristic value.

Using the Eqn.6.1 *(eq. (2.8), EN 1997-1)*:

$$U_d \leq G_d + R_d$$

$$U_k \gamma_{G,dst} \leq G_k \gamma_{G,stb} + R_d$$

Hence using the characteristic actions given above and the factors in 2 gives:

$$5000 \cdot 1{,}00 = 5000 > 2253 \cdot 0{,}90 \ + \ 2465 \ = 4493\,\text{kN}$$

Hence, treating the wall force as a resistance, the ULP requirement is not satisfied and tensile piles are required.

The Irish verification for the situation with tension piles for GEO using Design Approach 1 involves treating the side wall force as a resistance, as in the UPL analysis above. The superior ϕ'_k value, $42{,}2^o$, is used. For DA1.C1, γ_ϕ = 1,0 and hence R_d = 2781 kN while for DA1.C2, γ_ϕ = 1,25 and hence R_d = 2465 kN.

Using the *eq. (2.5) from EN1997-1* and the symbols defined above:

$$E_d \leq R_d$$

$$E_d = U_k \gamma_G - G_k \gamma_{G,inf} = \ 5000\gamma_G - \ 2253\gamma_{G;inf}$$

$$R_{t,d} = R_{t,k} / \gamma_{s;t} + R_d = L_{tot} q_{s,k} \cdot \pi \cdot D / \gamma_{s;t} + R_d = L_{tot} \cdot 35 \cdot \pi \cdot 0{,}5 / \gamma_{s;t} + \ R_d \ = L_{tot} \cdot 54{,}98 / \gamma_{s;t} + R_d$$

Hence substituting for E_d and R_d in *eq. (2.5) from EN 1997-1* the length of the piles for stability is given by the equation:

$$L_{tot} \geq \gamma_{s;t} (5000\gamma_G - \ 2253\gamma_{G;inf} - \ R_d) \ / \ 54{,}98$$

Substituting the appropriate partial factor values and R_d values in this equation gives the required piles lengths:

DA1.C1: $L_{tot} \geq 1{,}25 \cdot \left(5000 \cdot 1{,}35 - 2253 \cdot 1{,}0 - 2781\right) / 54{,}98 = 39{,}0\,\text{m}$

DA1.C2: $L_{tot} \geq 1{,}6 \cdot \left(5000 \cdot 1{,}0 - 2253 \cdot 1{,}0 - 2465\right) / 54{,}98 = 8{,}2\,\text{m}$

Hence design length is the larger of DA1.C1 and DA1.C2, which 39,0 m and therefore 4 piles, each 10 m long are required. This is more than for the DA2 design given above, which uses a reduced γ_G value of 1,2 for transient situations during the construction period.

It should be noted that due to the large balancing forces, DA1.C1 controls the design in this example, not DA1.C2 as is usually the case.

A.6.2. VERIFICATION AGAINST FAILURE OF HYDRAULIC HEAVE

For the verification against hydraulic heave *EN 1997-1* the following requirement is given in *EN 1997-1 § 2.4.7.5: "(1)P When considering a limit state of failure due to heave by seepage of water in the ground (HYD, see 10.3), it shall be verified, for every relevant soil column, that the design value of the destabilising total pore water pressure ($u_{dst;d}$) at the bottom of the column, or the design value of the seepage force ($S_{dst;d}$) in the column is less than or equal to the stabilising total vertical stress ($\sigma_{stb;d}$) at the bottom of the column, or the submerged weight ($G'_{stb;d}$) of the same column"*:

$$u_{dst;d} \leq \sigma_{stb;d} \qquad (2.9a)$$

$$S_{dst;d} \leq G'_{stb;d} \qquad (2.9b)$$

For the situation in thin column is assumed for which EAU give a formula for the hydraulic head h_r above the ground level of the excavation at the toe of the wall. The embedment length of the wall has previously been determined using ULS-design.

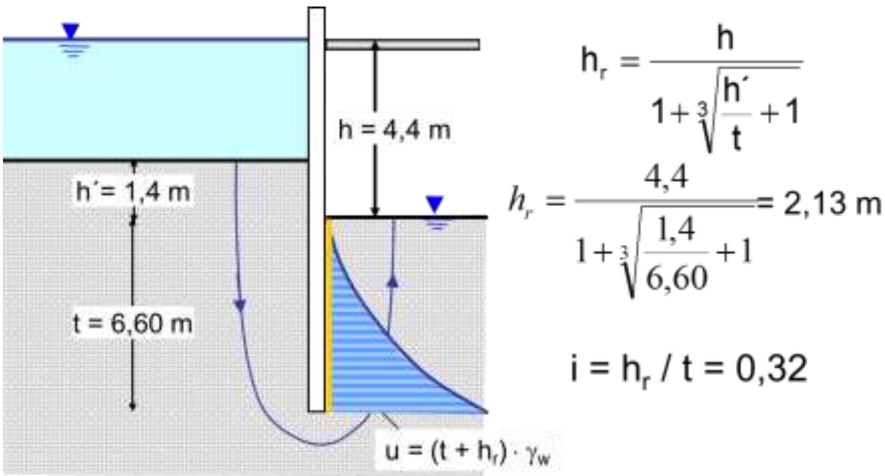


Fig.A.6.5 Geometry of the sheet pile wall and the hydraulic conditions

With the embedment length t and the hydraulic head h_r the hydraulic gradient i in the infinitesimal thin column can be computed. This gives the seepage force:

$$S_k = i\gamma_w V = 0{,}32 \cdot 10V$$

where V is the volume of the column. The characteristic value of the effective weight G'_k of the soil volume is:

$$G'_k = \gamma' V = 10V$$

With partial factors of $\gamma_{G,dst}$ = 1,35 and $\gamma_{G,stb}$ = 0,9 Eqn.6.3 *(eq. (2.9b), EN 1997-1)* becomes:

$$S_k \gamma_{G,dst} \leq G'_k \cdot \gamma_{G,stb}$$

$$0{,}32 \cdot 10 \cdot 1{,}35 = 4{,}32 \;\leq\; 10 \cdot 0{,}90 = 9{,}0$$

Hence there is sufficient safety using Eqn.6.3 *(eq. (2.9b), EN 1997-1)*

Using Eqn.6.2 *(eq. (2.9a), EN 1997-1)* the following values are needed:

$$u_k = (t + h_r)\gamma_w = (6{,}60 + 2{,}13)\cdot 10 = 87{,}3 \text{ kN/m}^2$$

$$s_{stb;k} = t\ (\gamma' + \gamma_w) = 6{,}60 \cdot 20 = 132 \text{kN/m}^2$$

$$u_k \gamma_{G,dst} \leq s_{stb;k} \gamma_{G,stb}$$
$$87{,}3 \cdot 1{,}35 \leq 132 \cdot 0{,}90$$
$$117{,}9 \leq 118{,}8$$

Also Eqn.6.2 *(eq. (2.9a), EN1997-1)* gives sufficient safety against hydraulic heave, however, the margin of safety is clearly smaller.

A.7. Worked example to accompany Chapter 7

This worked example illustrates the way in which an anchored sheet pile wall may be designed according to *Eurocode 7*. Two solutions are included – one for Design Approach 1 (DA1) and the other for DA2*.

The design situation involves a sheet pile wall that retains dense sand. The ground behind the wall is horizontal. The sheet pile is a Z section. An anchor will be installed at an angle to the horizontal to stabilize the wall.

A.7.1. DESIGN SITUATION

Consider a sheet pile wall that retains H_{nom} = 8,0 m of dense sand with characteristic weight density γ_k=20 kN/m^3 and drained angle of shearing resistance φ_k=38°. The ground behind the wall is horizontal and subject to a blanket surcharge (representing traffic loading) - but, for simplicity, we will assume q_k=0kPa. The ground is dry.

The sheet pile is a Z-section with flange thickness t_f =8,5 mm, web thickness t_w = 8,5 mm, web height h=302 mm, clutch-to-clutch breadth b=670 mm, elastic section modulus W_{el}=1400 cm^3/m, and characteristic yield strength f_{yk} =355 MPa.

An anchor with ultimate design resistance of $R_{a,d}$=130 kN/m will be installed at an angle θ=30° to the horizontal to stabilize the wall (Figure A.7.1).

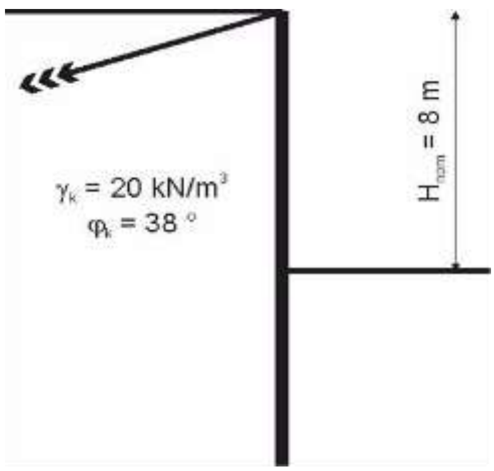


Fig.A.7.1 Anchored sheet pile wall

Geometry

Allowing for an unplanned excavation in ULS verifications, the design retained height of the wall is:

$$H_d = H_{nom} + \min(0{,}1H_{nom};0{,}5\text{m}) = 8{,}5\,\text{m}$$

A.7.2. ANCHORED SHEET PILE WALL: VERIFICATION OF DRAINED STRENGTH (LIMIT STATE GEO) - DESIGN APPROACH 1

Material properties

Partial factors

$$\text{from Set}\begin{pmatrix} M1 \\ M2 \end{pmatrix} : \gamma_\varphi = \begin{pmatrix} 1 \\ 1{,}25 \end{pmatrix}$$

Design angle of shearing resistance:

$$\varphi_d = \text{atan}\left(\frac{\tan(\varphi_k)}{\gamma_\varphi}\right) = \begin{pmatrix} 38 \\ 32 \end{pmatrix}^\circ$$

Characteristic value of soil's constant-volume angle of shearing resistance is assumed to be:

$$\varphi_{cv,k} = 30^\circ$$

Design value of soil's constant-volume angle of shearing resistance is:

$$\varphi_{cv,d} = \min(\varphi_d, \varphi_{cv,k}) = 30^\circ$$

Angle of wall friction is k = 0.67 times the soil's constant-volume angle of shearing resistance:

$$\delta_d = k\varphi_{cv,d} = 20^\circ$$

Earth pressure coefficients from *Annex C of EN 1997-1*:

$$K_{a,h} = \overrightarrow{K_{a\gamma}(\varphi_d;\delta_d;0;0)} = \begin{pmatrix} 0{,}21 \\ 0{,}26 \end{pmatrix}$$

$$K_{p,h} = \overrightarrow{K_{p\gamma}(\varphi_d;\delta_d;0;0)} = \begin{pmatrix} 7{,}39 \\ 5{,}18 \end{pmatrix}$$

Actions

Partial factors

$$\text{from Set}\begin{pmatrix} A1 \\ A2 \end{pmatrix} : \gamma_G = \begin{pmatrix} 1{,}35 \\ 1 \end{pmatrix} \; ; \; \gamma_{G,fav} = 1 \;\text{ and }\; \gamma_Q = \begin{pmatrix} 1{,}5 \\ 1{,}3 \end{pmatrix}$$

'Single source principle' allows

$$\gamma_{G,fav} = \gamma_G = \begin{pmatrix} 1{,}35 \\ 1 \end{pmatrix}$$

Ratio of variable and permanent partial factors is:

$$\gamma_{Q/G} = \frac{\gamma_Q}{\gamma_G} = \begin{pmatrix} 1{,}11 \\ 1{,}3 \end{pmatrix}$$

Assume a depth of embedment

$$d = \begin{pmatrix} 1{,}11 \\ 2{,}01 \end{pmatrix} \text{m}$$

Overturning moment about anchor is:

$$M_{Ed,dst} = \overrightarrow{\left[\gamma_G K_{a,h} \left(\frac{1}{3} \gamma_k (Hd + d)^3 + \frac{1}{2} \gamma_{Q/G} q_k (Hd + d)^2 \right) \right]} = \begin{pmatrix} 1790 \\ 2040 \end{pmatrix} \text{kNm/m}$$

Restoring moment about anchor is:

$$M_{Ed,stb} = \overrightarrow{\left[\gamma_{G,fav} K_{p,h} \left[\frac{1}{2} \gamma_k d^2 \left(H_d + \frac{2}{3} d \right) \right] \right]} = \begin{pmatrix} 1789 \\ 2061 \end{pmatrix} \text{kNm/m}$$

Out of balance moment is:

$$\frac{M_{Ed,dst} - M_{Ed,stb}}{M_{Ed,stb}} = \begin{pmatrix} 0{,}1 \\ -1 \end{pmatrix} \%$$

Active thrust on retained side of wall is:

$$P_{a,Ed} = \overrightarrow{\left[\gamma_G K_{a,h} \left(\frac{1}{2} \gamma_k (H_d + d)^2 + \gamma_{Q/G} q_k (Hd + d) \right) \right]} = \begin{pmatrix} 272 \\ 291 \end{pmatrix} \text{kN/m}$$

Passive thrust on restraining side of wall is:

$$P_{p,Ed} = \overrightarrow{\left[\gamma_{G,fav} K_{p,h} \left(\frac{1}{2} \gamma_k d^2 \right) \right]} = \begin{pmatrix} 190 \\ 209 \end{pmatrix} \text{kN/m}$$

Hence net thrust is:

$$P_{Ed} = P_{a,Ed} - P_{p,Ed} = \begin{pmatrix} 81{,}9 \\ 81{,}7 \end{pmatrix} \text{kN/m}$$

Hence axial force transferred to the anchor is:

$$F_{a,Ed} = \frac{\max\left(P_{Ed1};P_{Ed2}\right)}{\cos\left(\theta\right)} = 94{,}6\,\text{kN/m}$$

The depth of zero shear force in the retaining wall can be found (approximately) from:

$$Z = \sqrt{\overrightarrow{\frac{P_{Ed}}{\gamma_G K_{a,h}\,\frac{1}{2}\gamma_k}}} = \begin{pmatrix}5{,}42\\5{,}57\end{pmatrix}\text{m}$$

... and checked for accuracy using:

$$V_{z,Ed} = P_{Ed} - \overrightarrow{\left[\gamma_G K_{a,h}\left(\frac{1}{2}\gamma_k z^2 + \gamma_{Q/G} q_k z\right)\right]} = \begin{pmatrix}0\\0\end{pmatrix}\text{kN/m}$$

Hence the maximum bending moment in the wall is:

$$M_{Ed} = \overrightarrow{\left[P_{Ed} z - \gamma_G K_{a,h}\left(\frac{1}{6}\gamma_k z^3 + \frac{1}{2}\gamma_{Q/G} q_k z^2\right)\right]} = \begin{pmatrix}296\\303\end{pmatrix}\text{kNm/m}$$

Maximum bending moment from either combination is:

$$M_{Ed} = \max\left(M_{Ed1};M_{Ed2}\right) = 303\,\text{kNm/m}$$

Maximum shear force in the wall is:

$$\mathrm{V_{Ed}} = \max\left(\mathrm{P_{Ed1}};\mathrm{P_{Ed2}}\right) = 81{,}9\,\text{kN/m}$$

A.7.3. VERIFICATIONS

Verification of resistance to overturning

'Degree of utilization' $\Lambda = \frac{M_{Ed,dst}}{M_{Ed,stb}} = \begin{pmatrix}100\\99\end{pmatrix}\%$ or 'Overdesign factor' $\text{ODF} = \frac{M_{Ed,stb}}{M_{Ed,dst}} = \begin{pmatrix}1\\1{,}01\end{pmatrix}$

The design is unacceptable if the degree of utilization is > 100% (or overdesign factor is < 1)

Verification of bending resistance

Partial factor on yield strength of steel is γ_{M0} = 1,0 (from *EN 1993-1-1*)

Factor for reduced shear force in interlocks β_B = 1,0

Design bending resistance of sheet pile section is:

$$M_{c,Rd} = \frac{\beta_B W_{el} f_{yk}}{\gamma_{M0}} = 497\,\text{kNm/m}$$

'Degree of utilization' $\Lambda = \frac{M_{Ed}}{M_{c,Rd}} = 61\%$ or 'Overdesign factor' $\text{ODF} = \frac{M_{c,Rd}}{M_{Ed}} = 1{,}64$

The design is unacceptable if the degree of utilization is > 100% (or overdesign factor is < 1)

Verification of shear resistance

Projected shear area is:

$$Av = \frac{t_w(h-t_f)}{b} = 3724\,\text{mm}^2/\text{m}$$

Design shear resistance of sheet pile section is:

$$V_{pl,Rd} = \frac{A_V f_{yk}}{\sqrt{3}\gamma_{M0}} = 763{,}2\,\text{kNm}$$

'Degree of utilization' $\Lambda = \frac{V_{Ed}}{V_{pl,Rd}} = 11\%$ or 'Overdesign factor' $\text{ODF}=\frac{V_{pl,Rd}}{V_{Ed}} = 9{,}3$

The design is unacceptable if the degree of utilization is > 100% (or overdesign factor is < 1)

Verification of resistance to anchor pull-out

Design pull-out resistance of anchor is:

$$F_{a,Rd} = R_{a,d} = 130\,\text{kN/m}$$

'Degree of utilization' $\Lambda = \frac{F_{a,Ed}}{F_{a,Rd}} = 73\%$ or 'Overdesign factor' $\text{ODF}=\frac{F_{a,Rd}}{F_{a,Ed}} = 1{,}37$.

The design is unacceptable if the degree of utilization is > 100% (or overdesign factor is < 1)

A.7.4. ANCHORED SHEET PILE WALL: VERIFICATION OF DRAINED STRENGTH (LIMIT STATE GEO) - DESIGN APPROACH 2*

Material properties

Partial factors

from Set M1: $\gamma_\varphi = 1$

Characteristic angle of shearing resistance:

$\varphi_k = 38{,}0°$

Characteristic value of soil's constant-volume angle of shearing resistance is assumed to be:

$\varphi_{cv,k} = 30°$

Angle of wall friction is k=0,67 times the soil's constant-volume angle of shearing resistance:

$\delta_k = k\varphi_{cv,k} = 20°$

Earth pressure coefficients from *Annex C of EN 1997-1*:

$$K_{a,h} = \overrightarrow{K_{a\gamma}(\varphi_d;\delta_d;0;0)} = 0{,}21$$

$$K_{p,h} = \overrightarrow{K_{p\gamma}(\varphi_d;\delta_d;0;0)} = 7{,}39$$

Actions

Partial factors

from Set A1: γ_G =1,35; $\gamma_{G,fav}$ =1 and γ_Q=1,5

Partial factor

from Set R2: γ_{Re} =1,4

Ratio of variable and permanent partial factors is:

$$\gamma_{Q/G} = \frac{\gamma_Q}{\gamma_G} = 1{,}11$$

Assume a depth of embedment *d*=2,05 m

Overturning moment about anchor is:

$$M_{Ed,dst} = \overrightarrow{\left[\gamma_G K_{a,h}\left(\frac{1}{3}\gamma_k(Hd+d)^3 + \frac{1}{2}\gamma_{Q/G}q_k(Hd+d)^2\right)\right]} = 2180\,\text{kNm/m}$$

Restoring moment about anchor is:

$$M_{Ed,stb} = \overrightarrow{\left[\frac{\gamma_{G,fav}K_{p,h}\left[\frac{1}{2}\gamma_k d^2\left(H_d + \frac{2}{3}d\right)\right]}{\gamma_{\mathrm{Re}}}\right]} = 2188\,\text{kNm/m}$$

Out of balance moment is:

$$\frac{M_{Ed,dst} - M_{Ed,stb}}{M_{Ed,stb}} = -0{,}4\%$$

Active thrust on retained side of wall is:

$$P_{a,Ed} = \overrightarrow{\left[\gamma_G K_{a,h}\left(\frac{1}{2}\gamma_k(H_d + d)^2 + \gamma_{Q/G}q_k(Hd+d)\right)\right]} = 310\,\text{kN/m}$$

Passive thrust on restraining side of wall is:

$$P_{p,Ed} = \overrightarrow{\left[\frac{\gamma_{G,fav}K_{p,h}\left(\frac{1}{2}\gamma_k d^2\right)}{\gamma_{\mathrm{Re}}}\right]} = 222\,\text{kN/m}$$

Hence net thrust is:

$$P_{Ed} = P_{a,Ed} - P_{p,Ed} = 88{,}2\,\text{kN/m}$$

Hence axial force transferred to the anchor is:

$$F_{a,Ed} = \frac{P_{Ed}}{\cos(\theta)} = 101{,}9\,\text{kN/m}$$

The depth of zero shear force in the retaining wall can be found (approximately) from:

$$Z = \sqrt{\frac{P_{Ed}}{\gamma_G K_{a,h} \frac{1}{2}\gamma_k}} = 5{,}63\,\text{m}$$

... and checked for accuracy using:

$$V_{z,Ed} = P_{Ed} - \left[\gamma_G K_{a,h}\left(\frac{1}{2}\gamma_k z^2 + \gamma_{Q/G} q_k z\right)\right] = 0\,\text{kN/m}$$

Hence the maximum bending moment in the wall is:

$$M_{Ed} = \left[P_{Ed} z - \gamma_G K_{a,h}\left(\frac{1}{6}\gamma_k z^3 + \frac{1}{2}\gamma_{Q/G} q_k z^2\right)\right] = 331\,\text{kNm/m}$$

Maximum shear force in the wall is:

$$\text{V}_{\text{Ed}} = \text{P}_{\text{Ed}} = 88{,}2\,\text{kN/m}$$

A.7.5. VERIFICATIONS

Verification of resistance to overturning

'Degree of utilization' $\Lambda = \frac{M_{Ed,dst}}{M_{Ed,stb}} = 100\%$ or 'Overdesign factor' ODF=$\frac{M_{Ed,stb}}{M_{Ed,dst}} = 1$

The design is unacceptable if the degree of utilization is > 100% (or overdesign factor is < 1)

Verification of bending resistance

Partial factor on yield strength of steel is γ_{M0} =1,0 (from *EN 1993-1-1*)

Factor for reduced shear force in interlocks β_B = 1,0

Design bending resistance of sheet pile section is:

$$M_{c,Rd} = \frac{\beta_B W_{el} f_{yk}}{\gamma_{M0}} = 497\,\text{kNm/m}$$

'Degree of utilization' $\Lambda = \frac{M_{Ed}}{M_{c,Rd}} = 67\%$ or 'Overdesign factor' ODF=$\frac{M_{c,Rd}}{M_{Ed}} = 1{,}5$

The design is unacceptable if the degree of utilization is > 100% (or overdesign factor is < 1)

Verification of shear resistance

Projected shear area is:

$$Av = \frac{t_w(h - t_f)}{b} = 3724\,\text{mm}^2/\text{m}$$

Design shear resistance of sheet pile section is:

$$V_{pl,Rd} = \frac{A_v f_{yk}}{\sqrt{3}\gamma_{M0}} = 763{,}2\,\text{kNm}$$

'Degree of utilization' $\Lambda = \frac{V_{Ed}}{V_{pl,Rd}} = 12\%$ or 'Overdesign factor' ODF=$\frac{V_{pl,Rd}}{V_{Ed}} = 8{,}7$

The design is unacceptable if the degree of utilization is > 100% (or overdesign factor is < 1)

Verification of resistance to anchor pull-out

Design pull-out resistance of anchor is:

$$F_{a,Rd} = R_{a,d} = 130\,\text{kN/m}$$

'Degree of utilization' $\Lambda = \frac{F_{a,Ed}}{F_{a,Rd}} = 78\%$ or 'Overdesign factor' ODF=$\frac{F_{a,Rd}}{F_{a,Ed}} = 1{,}28$.

The design is unacceptable if the degree of utilization is > 100% (or overdesign factor is < 1)

A.8. Worked examples to accompany Chapter 8

A.8.1. PILE DESIGNED FROM STATIC LOAD TEST RESULTS

This worked example illustrates the design of a bored pile from static load test results using Design Approaches 1 and 2 and the recommended partial factor values.

Design situation

Piles are required to support the following loads from a building:

- Characteristic permanent vertical load G_k = 6,0 MN
- Characteristic variable vertical load Q_k = 3,2 MN

The design involves determining the number of piles to support the building. The number of piles is to be determined on the basis of static pile load tests.

Geometry

It has been decided to use bored piles, 1,2 m in diameter and 15 m long.

Measured pile resistance

Static pile load tests have been performed on site on four piles of the same diameter and length as the chosen piles.

The results of the load-settlement curves are plotted in the figure opposite.

In accordance with *§7.6.1.1(3)*, settlement of the pile top equal to 10% of the pile base diameter s_g = (10/100)·1,2·10^3 = 120 mm has been adopted as the "failure" criterion for the piles.

From the load-settlement graphs for each pile (Figure A.8.1) this gives:

- Pile 1 R_m = 2,14 MN
- Pile 2 R_m = 1,96 MN
- Pile 3 R_m = 1,73 MN
- Pile 4 R_m = 2,33 MN

Hence the mean and minimum measured pile resistances are:

$(R_m)_{mean}$ = 2,04 MN

$(R_m)_{min}$ = 1,73 MN

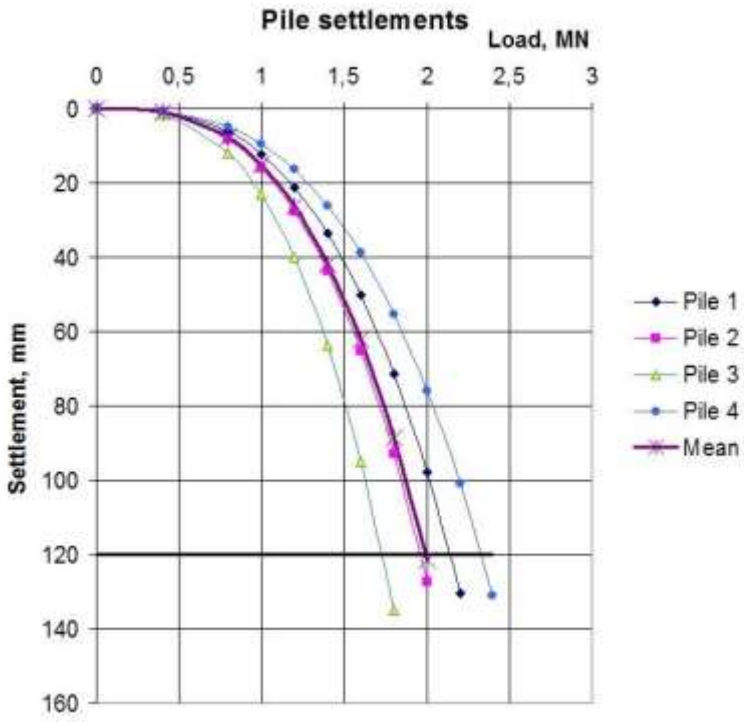


Fig.A.8.1 Load-settlement graphs

Characteristic resistance

The characteristic pile resistance is obtained by dividing the mean and minimum measured pile resistances by the correlation factors ξ_1 and ξ_2 and choosing the minimum value.

For four load tests, recommended ξ values are ξ_1 =1,1 and ξ_2 = 1,0

Hence the characteristic pile resistance

$$R_{c;k} = \min\left\{\frac{2,04}{1,1};\frac{1,73}{1,0}\right\} = \min\{1,85;1,73\} = 1,73\,\text{MN}$$

A.8.1.1. Design Approach 1

Combinations of sets of partial factors

DA1.C1: A1 "+" M1 "+" R1

DA1.C2: A2 "+" M1 "+" R4

Design actions

DA1.C1 $F_{c;d} = \gamma_G G_k + \gamma_Q Q_k = 1{,}35 \cdot 6{,}0 + 1{,}5 \cdot 3{,}2 = 12{,}9$ MN

DA1.C2 $F_{c;d} = \gamma_G G_k + \gamma_Q Q_k = 1{,}0 \cdot 6{,}0 + 1{,}3 \cdot 3{,}2 = 10{,}2$ MN

Characteristic resistances

DA1.C1 $R_{c;d} = R_{c;k} / \gamma_t = 1{,}73 / 1{,}15 = 1{,}50$ MN

DA1.C2 $R_{c;d} = R_{c;k} / \gamma_t = 1{,}73 / 1{,}5 = 1{,}15$ MN

Design equation

$F_{c;d} \leq R_{c;d}$

Hence equating design actions and design resistances for n piles:

DA1.C1 $12{,}9 = 1{,}50n \quad \Rightarrow n = 12{,}9 / 1{,}5 \quad n = 8{,}6$ piles

DA1.C2 $10{,}2 = 1{,}15n \quad \Rightarrow n = 10{,}2 / 1{,}15 \quad n = 8{,}9$ piles

Design number of piles

Hence DA1.C2 controls the DA1 design and the number of piles required is 9.

A.8.1.2. Design Approach 2

Combinations of set of partial factors

DA2 A1 "+" M1 "+" R2

Design actions

DA2 $F_{c;d} = \gamma_G G_k + \gamma_Q Q_k = 1{,}35 \cdot 6{,}0 + 1{,}5 \cdot 3{,}2 = 12{,}9$ MN

Design resistances

DA2 $R_{c;d} = R_{c;k} / \gamma_t = 1{,}73 / 1{,}1 = 1{,}57$ MN

Design equation

$F_{c;d} \leq R_{c;d}$

Hence equating design actions and design resistances:

DA2 $12{,}9 = 1{,}57\ n \qquad \Rightarrow n = 12{,}9 / 1{,}57 \qquad n = 8{,}2$ piles

Design number of piles

Hence using, the DA2, the number of piles required is 9.

A.8.1.3. Design Approach 3

Combinations of sets of partial factors

DA3: A1 (on structural actions) or A2(on geotechnical actions) "+" M2 "+" R3

DA3 not to be used

Since the R3 recommended partial resistance factors used in DA3 are all equal to 1,0, no safety margin is provided if DA3 is used to calculated the design pile resistance from pile load tests and therefore piles should not be designed using DA3 and pile load tests unless the resistance factors are increased.

A.8.1.4. Conclusions from Example 1

The same pile design length, 21m is required for both DA1 and DA2

Since the partial resistance factors are 1,0 for DA3, this Design Approach should not be used for the design of piles from pile load tests unless the resistance factors are increased.

A.1.1. PILE FOUNDATION DESIGNED FROM SOIL TEST PROFILE

This worked example illustrates the design of a bored pile from a soil test profile obtained from a CPT test using Design Approaches 1 and 2.

Design situation

The piles for a building are each required to support the following loads:

- Characteristic permanent vertical load G_k = 300 kN
- Characteristic variable vertical load Q_k = 150 kN

The ground consists of dense sand beneath loose sand with soft clay and peat to 16,5m. One CPT test profile is available. The pile foundation design involves determining the design length, L of the piles.

Geometry

It has been decided to use bored piles with a diameter D = 0,45m.

Material properties

1 CPT was carried out and the results are shown in Figure A.8.2.

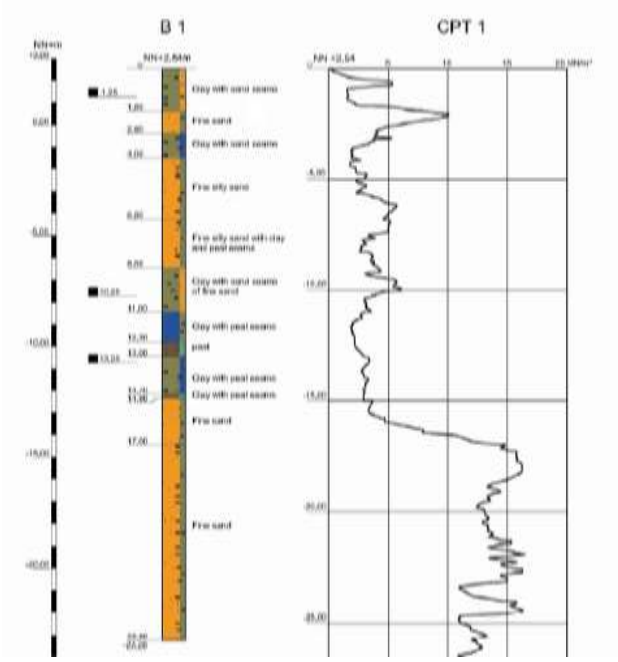


Fig.A.8.2 CPT results

Soil has an upper 11 m layer of loose sand, soft clay and some peat over 5,5 m of clay with peat seams.

- In the upper layer: cautious average q_c = 2,5 MPa

A stronger lower layer of medium to dense sand starts at depth of 16,5m

- In the lower layer: cautious average q_c = 12,5 MPa

Assume the soil above 16,5 m provides no shaft resistance

The pile base and shaft resistances are calculated using Table A.8.1 and Table A.8.2 (*Tables D.3 and D.4 of EN 1997-2)* and, for simplicity, relating the single cautious average q_c value in lower layer of stronger soil to the unit base and shaft resistances, p_b and p_s.

Table A.8.1 Unit base resistance p_b of cast in-situ piles in coarse soil with little or no fines

Normalised settlement s/D_s; s/D_b	**Unit base resistance p_b, in MPa, at average cone penetration resistance q_c (CPT) in MPa**			
	q_c = 10	q_c = 15	q_c = 20	q_c = 25
0,02	0,70	1,05	1,40	1,75
0,03	0,90	1,35	1,80	2,25
0,10 (= s_g)	2,00	3,00	3,50	4,00

NOTE Intermediate values may be interpolated linearly. In the case of cast in-situ piles with pile base enlargement, the values shall be multiplied by 0,75.

s is the normalised pile head settlement
D_s is the diameter of the pile shaft
D_b is the diameter of the pile base
s_g is the ultimate settlement of pile head

Table A.8.2 Unit shaft resistance p_s of cast in-situ piles in coarse soil with little or no fines

Average cone penetration resistance q_c (CPT) MPa	Unit shaft resistance p_s MPa
0	0
5	0,040
10	0,080
$\geq$ 15	0,120
NOTE Intermediate values may be interpolated linearly	

Assume the ULS settlement of the pile head, s_g so that the normalised settlement is 0,1.

Interpret linearly between relevant q_c values to obtain p_b and p_s from these tables:

$$p_b = 2{,}5 \text{ MPa}$$

$$p_s = 0{,}1 \text{ MPa}$$

Characteristic pile resistance

- Pile base cross sectional area: $A_b = \pi \cdot 0{,}45^2/4 = 0{,}159 \text{ m}^2$
- Pile shaft area per metre length: $A_s = \pi \cdot 0{,}45 = 1{,}414 \text{ m}^2/\text{m}$

Length of pile in lower stronger layer providing shaft resistance is L_s.

Calculated compressive pile resistance for the one profile of test results:

$$R_{c;cal} = R_{b;cal} + R_{s;cal} = A_b p_b + A_s L_s p_s = (0{,}159 \cdot 2{,}5 + 1{,}414 \cdot L_s \cdot 0{,}1) \cdot 10^3 \text{kN}$$

$$R_{c;cal} = 398 + 141 \cdot L_s \text{kN}$$

Hence, applying the recommended correlation factors ξ_3 and ξ_4, which are both the same and equal to 1,4 for one profile of test results because the mean and minimum calculated resistances are the same so that ξ_3 and $\xi_4 = \xi = 1{,}4$, and the characteristic base and shaft compressive pile resistances are:

$$R_{b;k} = R_{b;cal} / \xi = 398 / 1{,}4 = 284 \text{kN}$$

$$R_{s;k} = R_{s;cal} / \xi = 141 \cdot L_s / 1{,}4 = 101 \cdot L_s$$

A.8.1.5. Design Approach 1

Design actions

DA1.C1 $F_{c;d} = \gamma_G G_k + \gamma_Q Q_k = 1{,}35 \cdot 300 + 1{,}5 \cdot 150 = 630 \text{ kN}$

DA1.C2 $F_{c;d} = \gamma_G G_k + \gamma_Q Q_k = 1{,}0 \cdot 300 + 1{,}3 \cdot 150 = 495 \text{ kN}$

Design resistances

DA1.C1 $R_{c;d} = R_{b;k} / \gamma_b + R_{s;k} / \gamma_s = 284 / 1{,}25 + 101 L_s / 1{,}0$

DA1.C2 $R_{c;d} = R_{b;k} / \gamma_b + R_{s;k} / \gamma_s = 284 / 1{,}6 + 101\, L_s / 1{,}3$

Design equation

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

Hence equating design actions and design resistances:

DA1.C1 $630 = 284 / 1{,}25 + 101\, L_s / 1{,}0 \quad \Rightarrow L_s = 3{,}99$ m

DA1.C2 $495 = 284 / 1{,}6 + 101\, L_s / 1{,}3 \quad \Rightarrow L_s = 4{,}08$ m

Design pile length

Hence DA1.C2 controls the DA1 design and the DA1 design pile length L=16,5+L_s=21 m.

A.8.1.6. Design Approach 2

Design actions

DA2 $F_{c;d} = \gamma_G G_k + \gamma_Q Q_k = 1{,}35 \cdot 300 + 1{,}5 \cdot 150 = 630$ kN

Design resistances

DA2 $R_{c;d} = R_{b;k} / \gamma_b + R_{s;k} / \gamma_s = 284 / 1{,}1 + 101 \cdot Ls\ /\ 1{,}1$

Design equation

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

Hence equating design actions and design resistances:

DA2 $630 = 284 / 1{,}1 + 101 \cdot Ls / 1{,}1 \quad \Rightarrow Ls = 4{,}05$ m

Design pile length

Hence the DA2 design pile length L=16,5+L_s=21 m.

A.8.1.7. Design Approach 3

As the R3 recommended partial resistance factors used in DA3 are equal to 1,0, no safety margin is provided if these are used in DA3 to calculate the design pile resistance from a CPT test profile. Hence, piles should not be designed from CPT test profiles using DA3 unless a model factor is applied to increase the partial resistance factors

A.8.1.8. Conclusions from Example 2

The same design pile length, 21 m is required for both DA1 and DA2

Since the recommended partial resistance factors are 1,0 for DA3, this Design Approach should not be used for the design of piles from profiles of ground test results unless the partial resistance factors are increased.

A.8.2. PILE FOUNDATION DESIGNED FROM SOIL PARAMETERS

This worked example illustrates the design of a driven pile in Dublin Boulder Clay from soil parameter values using Design Approaches 1, 2 and 3 and using the Irish and German National Annexes.

Design situation

The piles for a proposed building in Dublin are each required to support the following loads:

- Characteristic permanent vertical load G_k = 600 kN
- Characteristic variable vertical load Q_k = 300 kN

The ground consists of about 3m Brown Dublin Boulder Clay over Black Dublin Boulder Clay to great depth. A large number of SPT results are available.

The pile foundation design involves determining the design length, *L* of the piles.

Geometry

It has been decided to use driven piles with a diameter *D* = 0,45 m.

Material properties

Figure A.8.3 shows tests results of SPT N values plotted against depth. Shaft resistance in Brown Dublin Boulder Clay is ignored.

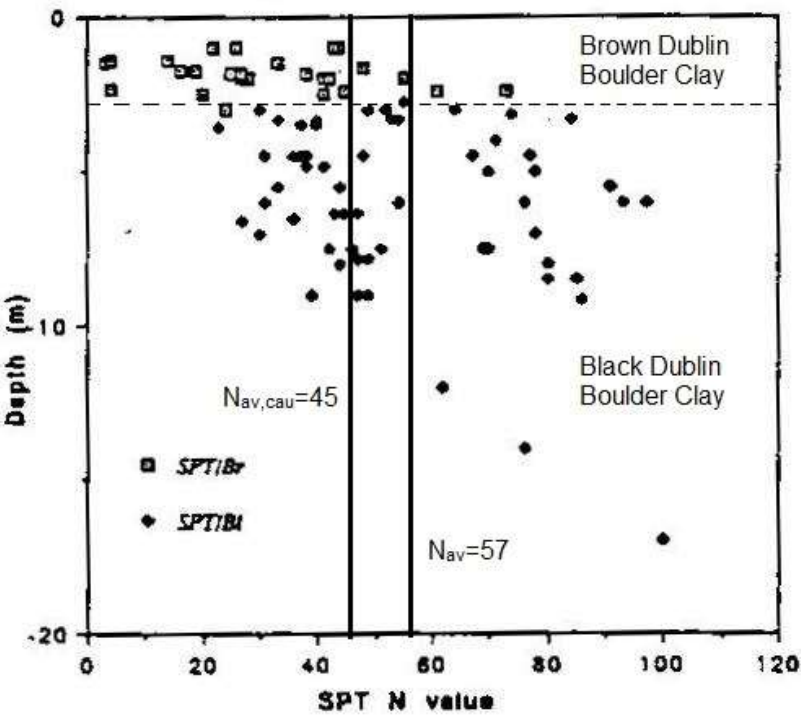


Fig.A.8.3 SPT N values versus depth

The average *N* value in Black Dublin Boulder Clay:

N_{av} = 57

A cautious average *N* value:

$N_{av,cau} = 45$

Plasticity Index of the Dublin Boulder Clay:

$PI = I_P = 14\%$

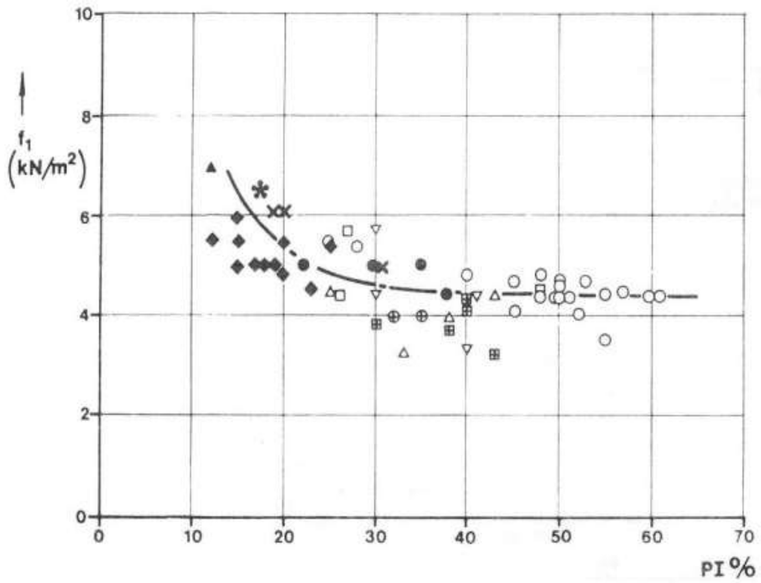


Fig.A.8.4 f_1 vs. PI from *Stroud and Butler*

From Figure A.8.4.:

Adopt $f_1 = 6$

Hence the cautious undrained shear strength:

$c_u = f_1 \cdot N = 270$ kPa

Pile resistances

Pile base cross-sectional area:

$$A_b = \pi D^2 / 4 = \pi \cdot 0{,}452 / 4 = 0{,}159 \text{ m}^2$$

If length of pile in Black Dublin Boulder Clay providing shaft resistance is *Ls*, then pile shaft area is:

$$A_s = \pi D Ls = \pi \cdot 0{,}45 \cdot L_s = 1{,}414 L_s \text{ m}^2$$

Characteristic unit pile base resistance:

$$q_{b;k} = N_q c_u = 9 \cdot 270 = 2430$$

Characteristic unit shaft resistance:

$$q_{s;k} = \alpha c_u = 0{,}4 \cdot 270 = 108$$

Hence characteristic base resistance:

$$R_{b;k} = A_b q_{b;k} = 0{,}159 \cdot 2430 = 386 \text{ kN}$$

Characteristic shaft resistance:

$$R_{s;k} = A_s q_{s;k} = 1{,}414 \cdot L_s \cdot 0{,}4 \cdot 270 = 153Ls$$

A.8.2.1. Design Approach 1

Since the building is being constructed in Dublin, the Irish NA must be used.

The Irish NA requires that the pile partial resistance factors are increased by a model factor of 1,75.

Design actions

DA1.C1 $$F_{c;d} = \gamma_G G_k + \gamma_Q Q_k = 1{,}35 \cdot 600 + 1{,}5 \cdot 300 = 1260 \text{ kN}$$

DA1.C12 $$F_{c;d} = \gamma_G G_k + \gamma_Q Q_k = 1{,}0 \cdot 600 + 1{,}3 \cdot 300 = 990 \text{ kN}$$

Design resistances

DA1.C1 $$R_{c;d} = \frac{R_{b;k}}{\gamma_{bx}\gamma_{R;d}} + \frac{R_{s;k}}{\gamma_{sx}\gamma_{R;d}} == \frac{386}{1{,}0 \cdot 1{,}75} + \frac{153 \cdot L_s}{1{,}0 \cdot 1{,}75} = 221 + 87{,}4L_s \text{ kN}$$

DA1.C2 $$R_{c;d} = \frac{R_{b;k}}{\gamma_{bx}\gamma_{R;d}} + \frac{R_{s;k}}{\gamma_{sx}\gamma_{R;d}} == \frac{386}{1{,}3 \cdot 1{,}75} + \frac{153 \cdot L_s}{1{,}3 \cdot 1{,}75} = 170 + 67{,}3L_s \text{ kN}$$

Design equation

$$F_{c,d} \le R_{c,d}$$

Hence equating design actions and design resistances:

DA1.C1 $1260 = 221 + 87{,}4L_s \quad \Rightarrow L_s = 11{,}9 \text{ m}$

DA1.C2 $990 = 170 + 67{,}3L_s \quad \Rightarrow L_s = 12{,}2 \text{ m}$

Design pile length

Hence DA1.C2 controls the DA1 design and the DA1 design pile length $L = 3 + L_s = 15.5$ m

A.8.2.2. Design Approach 2

As in the case of Design Approach 1, the Irish NA requires the pile partial resistance factors to be increased by a model factor of 1,75.

Design actions

DA2 $$F_{c;d} = \gamma_G G_k + \gamma_Q Q_k = 1{,}35 \cdot 600 + 1{,}5 \cdot 300 = 1260 \text{ kN}$$

Design resistances

DA2 $$R_{c;d} = \frac{R_{b;k}}{\gamma_{bx}\gamma_{R;d}} + \frac{R_{s;k}}{\gamma_{sx}\gamma_{R;d}} == \frac{386}{1,1 \cdot 1,75} + \frac{153 \cdot L_s}{1,1 \cdot 1,75} = 201 + 79,5L_s \text{ kN}$$

Design equation

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

Hence equating design actions and design resistances:

DA2 $1260 = 201 + 79,5L_s \qquad \Rightarrow L_s = 13,3 \text{ m}$

Design pile length

Hence the DA2 design pile length $L = 3,0 + L_s = 16,5 \text{ m}$.

A.8.2.3. Design Approach 3

In DA3, the partial resistance factors are all unity, so the Irish NA requirement to increase the pile partial resistance factors by a model factor of 1,75 is not relevant. The pile design resistances are obtained by applying partial material factors to suitably cautious characteristic soil parameter values. It is assumed that the characteristic undrained strength is the value obtained from the cautious average *N* value = 45.

Design actions

DA3 $F_{c,d} = \gamma_G G_k + \gamma_Q Q_k = 1,35 \cdot 600 + 1,5 \cdot 300 = 1260 \text{ kN}$

Design resistances

DA3 $$R_{c;d} = R_{b;d} + R_{s;d} = A_b q_{b;d} + A_s q_{s;d} = \frac{A_b N_q c_{u;k} / \gamma_{cu}}{\gamma_{R;d}\gamma_b} + \frac{\pi D L_s \cdot 0,4 c_{u;k} / \gamma_{cu}}{\gamma_{R;d}\gamma_s} =$$

$$= \frac{0,159 \cdot 9 \cdot 270 / 1,4}{1,75 \cdot 1,0} + \frac{1,414 \cdot L_s \cdot 0,4 \cdot 270 / 1,4}{1,75 \cdot 1,0} = 158 + 62,3L_s$$

Design equation

$$F_{c;d} \leq R_{c;d}$$

Hence equating design actions and design resistances:

DA3 $1260 = 158 + 62,3L_s \qquad \Rightarrow L_s = 17,7 \text{ m}$

Design pile length

Hence DA3 design pile length $L = 3 + L_s = 21 \text{ m}$

A.8.2.4. Conclusions from Example 3

- The design pile lengths obtained from ground strength parameters using the alternative procedure and the model factor in the Irish National Annex are:

  DA1 L = 15,5 m

  DA2 L = 16,5 m

  DA3 L = 21,0 m

- Application of the model factor of 1,75 as well as the material factor of 1,4 to obtain the design resistance when using DA3, results in DA3 providing a longer design pile length and hence the least economical Design Approach in Ireland
- If the building were to be constructed in Germany, the partial recommended in the German NA have been increased by a model factor of 1,27, compared to the model factor of 1,75 in the Irish NA, which gives a design pile length of 12,0 m in Germany using DA2 compared to 16,5 m using DA2 in Ireland.

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Abstract

This document is a report with worked examples summarizing the general rules, basic design principles and design methods for geotechnical design following Eurocodes. It comprises an overview of Eurocode 7 with focus on the design requirements, actions and design situations, and limit states. Different aspects to be considered for designing shallow foundations, gravity walls, embedded walls and deep foundations are covered in the report. The provisions of Eurocode 7 for ground investigations and testing for geotechnical design, overall stability of and movements in the ground, slopes, hydraulic failure modes and verifications against them are also presented. The Annex contains worked examples to accompany the various chapters of this report.

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