



# Shallow foundations – design of spread foundations

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## **Eurocode 7 Part 1, Section 6 : Spread foundations**

§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

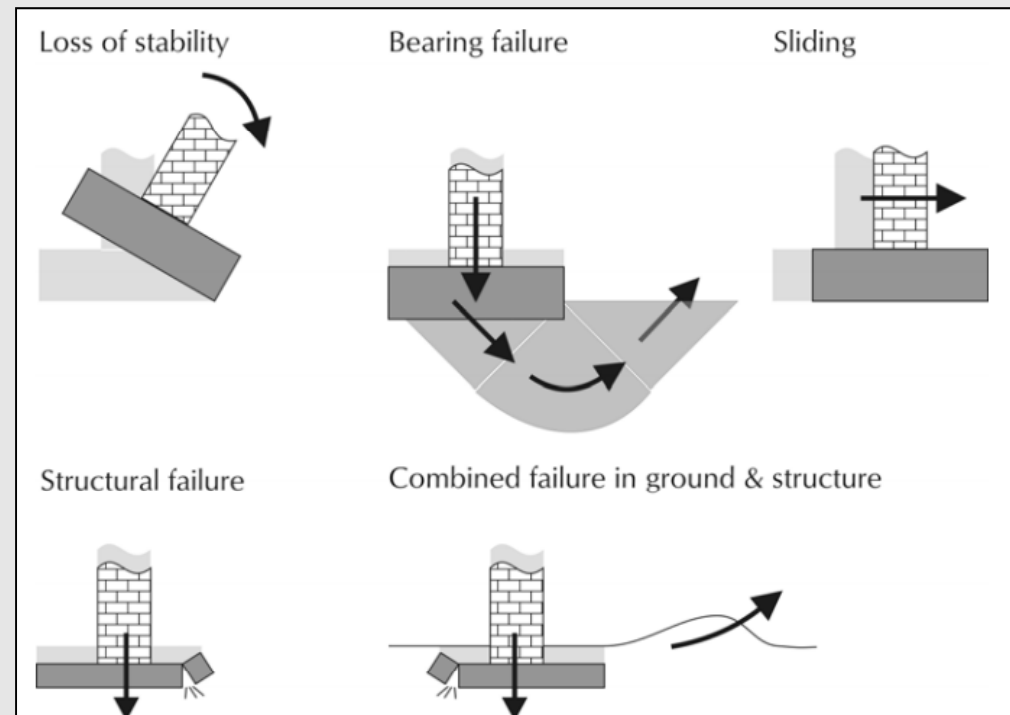
§6.9 Preparation of the subsoil

## §6.1 General

Section 6 of EN 1997-1 applies to pad, strip, and raft foundations and some provisions may be applied to deep foundations, such as caissons.

## §6.2 Limit states

- 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



Some of above are **ultimate limit states** and some are **serviceability limit states** – both need to be considered

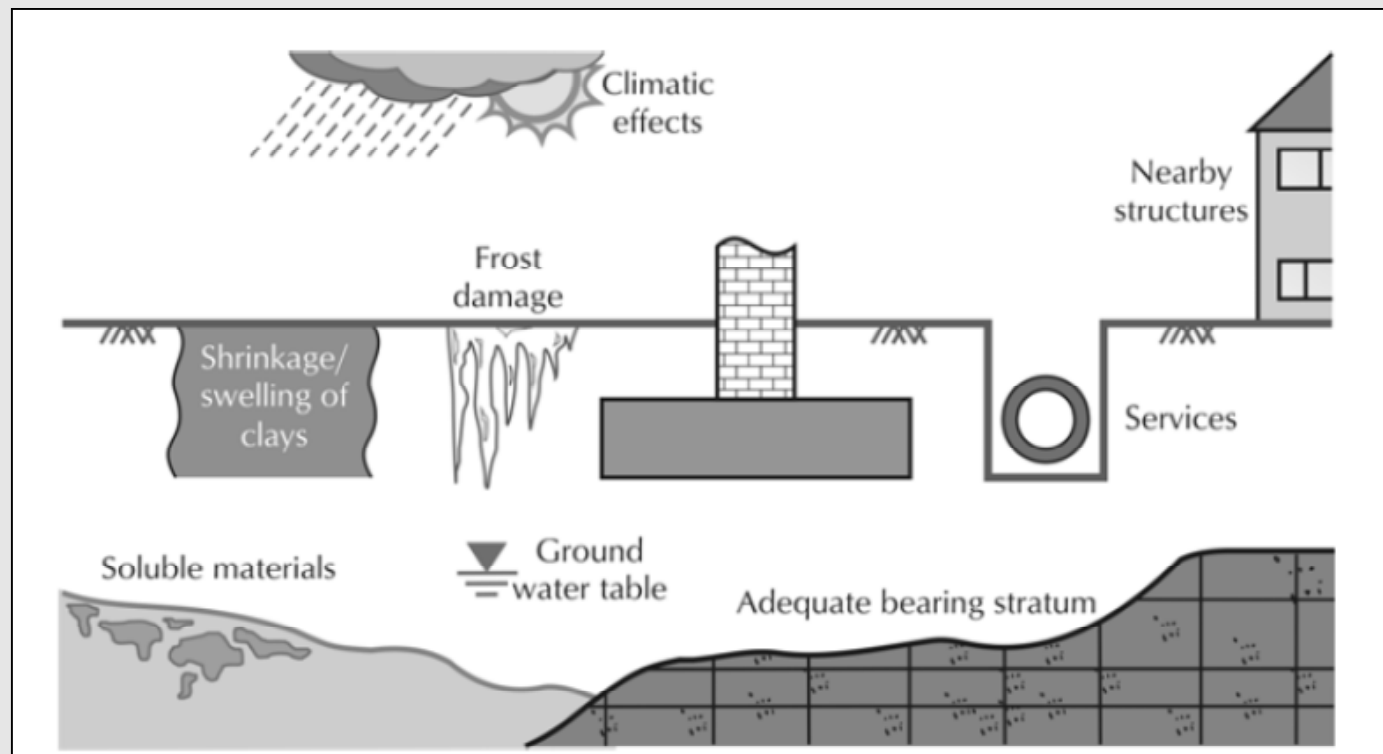
## §6.3 Design situations and Actions

**Design situations** shall be selected in accordance with 2.2. (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; the environment within which the design is set..... 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; traffic loads....)

## §6.4 Design and construction considerations

A number of things that must be considered when choosing the depth of a spread foundation.



One of the following design methods shall be used for shallow foundations:

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

## ULS

Limit state design implies the application of partial factors to actions (or effect of actions) to obtain  $E_d$  and to geotechnical parameters or resistances to obtain  $R_d$ .

$$E_d \leq R_d$$

## SLS

check for

$$E_d \leq C_d$$

$C_d$  is the limiting design value of the effect of an action

## Representation of the design action

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

$G_{kj}$ : characteristic permanent loads

$Q_{ki}$ : characteristic variable loads

$\psi_{0i}$ : factors for combination value of variable loads

$\gamma_{Gj}$ : partial factors for permanent loads

$\gamma_{Qi}$ : partial factors for variable loads



## §6.5 Ultimate limit state design

### Overall stability

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.

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

Table A.14 - Partial resistance factors ( $\gamma_R$ ) for slopes and overall stability

Resistance	Symbol	Set		
		<i>R1</i>	<i>R2</i>	<i>R3</i>
Earth resistance	$\gamma_{R,e}$	1,0	1,1	1,0

## Direct Method

### 1. ULS verifications with the three possible Design Approaches

- DA1 - Combination 1 **A1+ M1+R1**  
- Combination 2 **A2+ M2+R1**
- DA2 **A1+ M1+R2**
- DA3 **(A1 o A2)\*+ M2+R3**

\*(A1 for structural actions and A2 for geotechnical actions)

- Undrained Conditions
- Drained Conditions

### 2. SLS check the performance of the foundation

Partial factors on actions ( $\gamma_F$ ) or the effects of actions ( $\gamma_E$ )				
Action		Symbol	Set	
			A1	A2
Permanent	Unfavourable	$\gamma_G$	1.35	1.0
	Favourable		1.0	1.0
Variable	Unfavourable	$\gamma_Q$	1.5	1.3
	Favourable		0	0

Partial factors for soil parameters ( $\gamma_M$ )			
Soil parameter	Symbol	Value	
		M1	M2
Shearing resistance	$\gamma_\phi^1$	1.0	1.25
Effective cohesion	$\gamma_c$	1.0	1.25
Undrained strength	$\gamma_{cu}$	1.0	1.4
Unconfined strength	$\gamma_{qu}$	1.0	1.4
Weight density	$\gamma_\gamma$	1.0	1.0

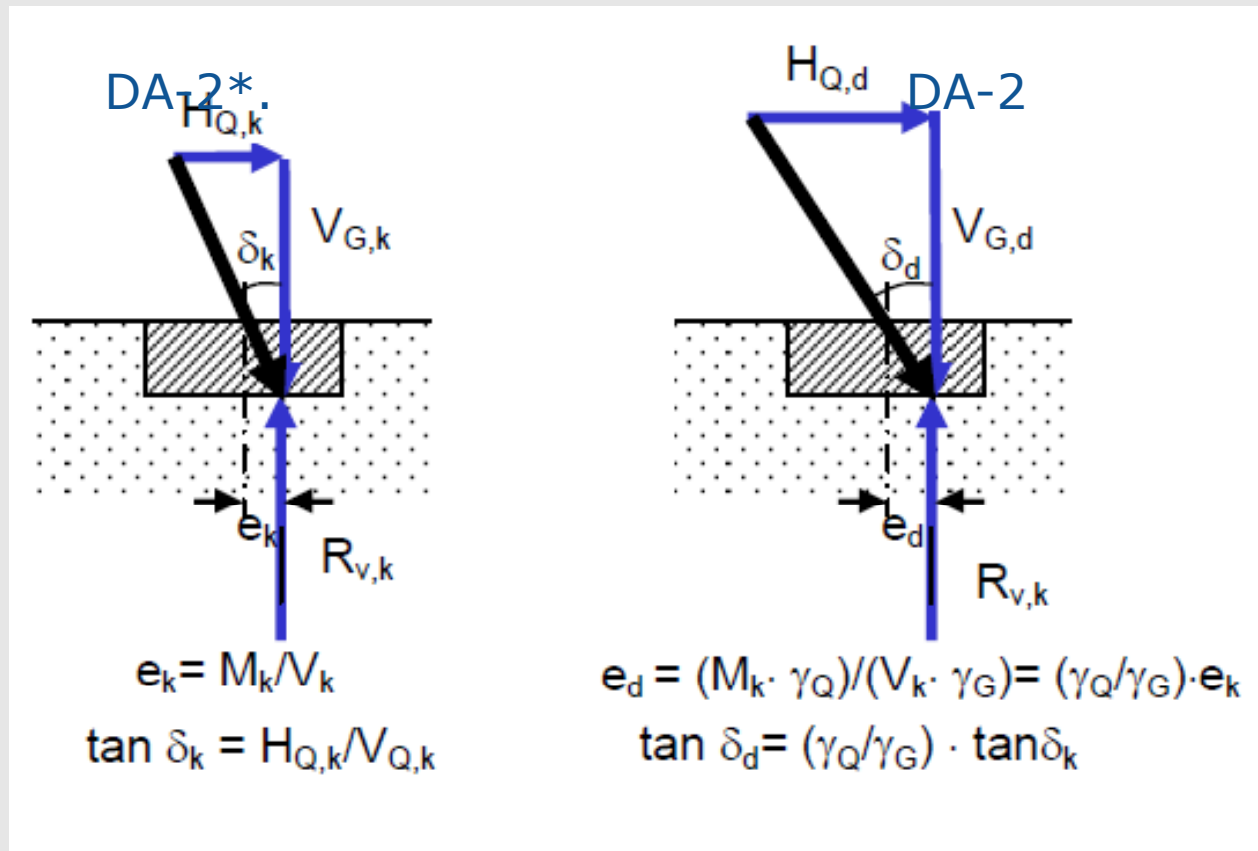
<sup>1</sup> This factor is applied to  $\tan \phi'$

Partial resistance factors for spread foundations ( $\gamma_R$ )				
Resistance	Symbol	Set		
		R1	R2	R3
Bearing	$\gamma_{Rv}$	1.0	1.4	1.0
Sliding	$\gamma_{Rh}$	1.0	1.1	1.0

There are two ways of performing verifications according to Design Approach 2, either by applying them to the actions (at the source) or by applying them to the effect of the actions.

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.



Determination of the ground bearing resistance in design procedures DA-2 and DA-2\*. Design approach DA 2\* gives the most economic (or less conservative) design.

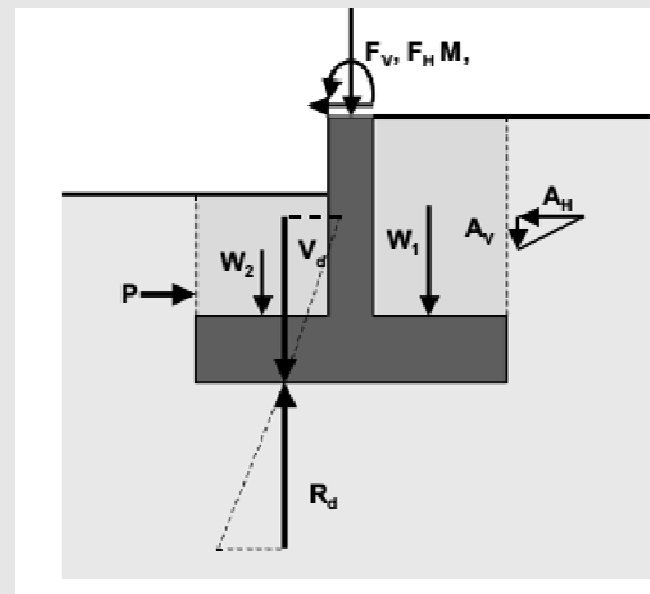
## Bearing resistance

$$V_d \leq R_d$$

$V_d$  should include the self-weight of the foundation and any backfill on it. This equation is a re-statement of the inequality:  $E_d \leq R_d$

### Design action $V_d$

- **Variable vertical load**
- **Permanent vertical load**
  - a) Supported permanent load
  - b) Weight of foundation
  - c) Weight of the backfill
  - d) Loads from water pressures
  - e) Uplift



## Annex D

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

$$R/A' = (2 + \pi) c_u s_c i_c + q \quad \text{UNDRAINED CONDITIONS}$$

with the dimensionless factors for

- the bearing resistance:

$$N_q = e^{\pi \times \tan \phi'} \tan^2(45^\circ + \phi'/2)$$

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

$$N_\gamma = 2 (N_q - 1) \tan \phi'$$

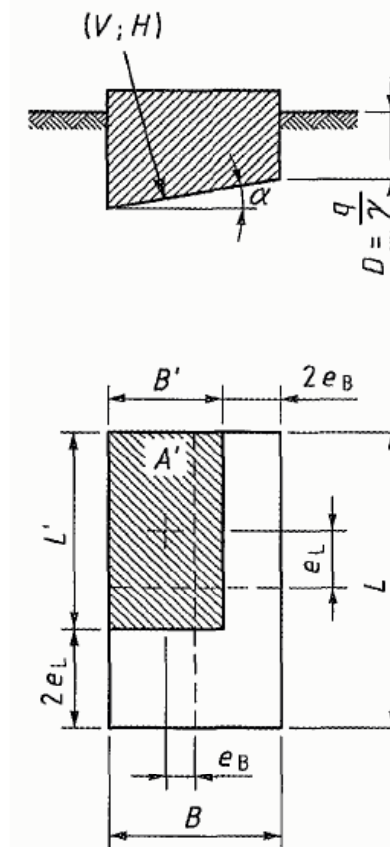
- the inclination of the foundation base:  $b_c, b_q, b_\gamma$
- the shape of foundation:  $s_c, s_q, s_\gamma$
- the inclination of the load:  $i_c, i_q, i_\gamma$

$A'$  = effective foundation area (reduced area with load acting at its centre)

The eccentricity of the action from the centre of the footing should be kept within the following limits (known as the foundation's 'middle-third') to avoid the loss of the contact between footing and ground:

$$e_B \leq B/6 \quad e_L \leq L/6$$

where  $B$  and  $L$  are the footing's breadth and length respectively and  $e_B$  and  $e_L$  are eccentricities in the direction of  $B$  and  $L$ .





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

$$i_q = (1 - 0.70 \times H / (V + A' \times c' \times \cotan\phi'))^m$$

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

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

$$m = m_\theta = m_L \cos^2 \theta + m_B \sin^2 \theta$$

$$i_c = (i_q \times N_q - 1) / (N_q - 1)$$

$$i_\gamma = (1 - H / (V + A' \times c' \times \cotan\phi'))^3$$

$$s_q = 1 + (B' / L') \times \text{sen}\phi' \quad (\text{rectangular shape})$$

$$s_q = 1 + \text{sen}\phi' \quad (\text{square or circular shape})$$

$$s_c = (s_q \times N_q - 1) / (N_q - 1)$$

$$s_\gamma = 1 - 0.30 \times (B' / L') \quad (\text{rectangular shape})$$

$$s_\gamma = 0.70 \quad (\text{square or circular shape})$$

$$b_c = b_q - (1 - b_q) / (N_c \tan \phi')$$

$$b_q = b_\gamma = (1 - \alpha \tan \phi')^2$$

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

*UNDRAINED CONDITIONS*

$$b_c = 1 - 2\alpha / (\pi + 2)$$

$\alpha$  is the inclination of the foundation base to the horizontal

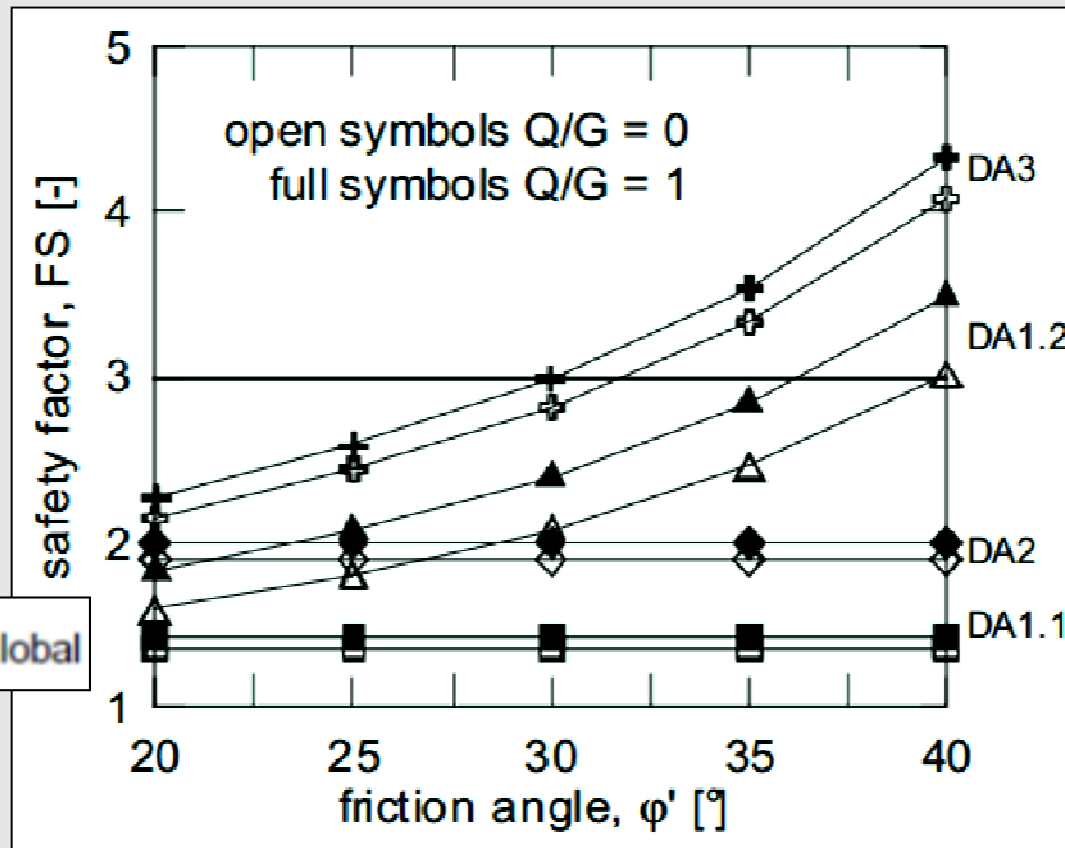
$$s_c = 1 + 0.2 (B'/L') \quad (\text{rectangular shape})$$

$$s_c = 1.2 \quad (\text{square or circular shape})$$

$$i_c = 0.5(1 + \sqrt{(1 - H/(A'c_u))})$$

$$D = z_w = 2\text{m} \quad B/L=1$$

$$\gamma_R \cdot \gamma_{G/Q} \approx \eta_{\text{global}}$$

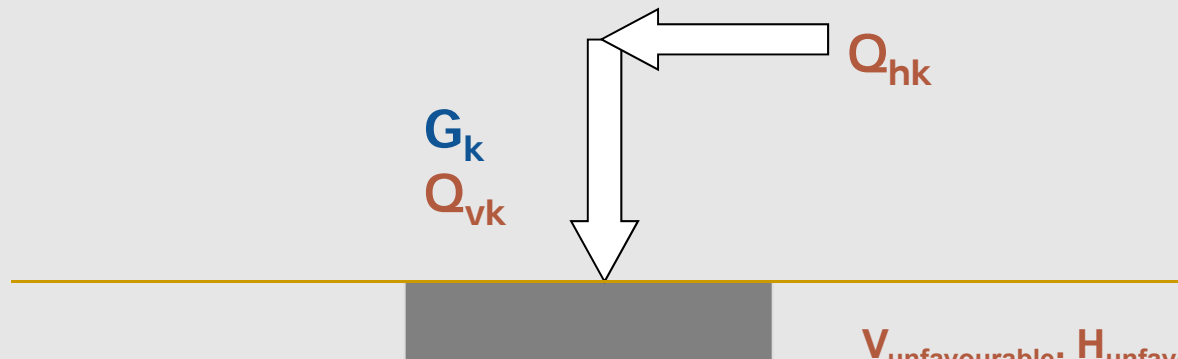


Global safety factor of the foundation designed according to EC7, Mandolini & G. Viggiani, 2004

## Considerations

For drained conditions water pressures must be included as actions. How to apply the partial factors to the weight of a submerged structure? Since the water pressure acts to reduce the value of  $V_d$ , it may be considered as favorable, 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 unfavorable.



As the eccentricity influence the effective base dimension it could be necessary to analyze different load combinations, by considering the permanent vertical load both favourable and unfavourable and by changing the principal variable load.

$V_{\text{unfavourable}} \cdot H_{\text{unfavourable}}$

$$V_d = \gamma_G G_k + \gamma_{Qv} \psi_0 Q_{vk}$$

$$\gamma_G = 1.35. \gamma_{Qv} = 1.5. \gamma_{Qh} = 1.5$$

$$H_d = \gamma_Q Q_{hk}$$

$V_{\text{unfavourable}} \cdot H_{\text{unfavourable}}$

$$V_d = \gamma_G G_k + \gamma_{Qv} Q_{vk}$$

$$\gamma_G = 1.35. \gamma_{Qv} = 1.5. \gamma_{Qh} = 1.5$$

$$H_d = \gamma_{Qh} \psi_0 Q_{hk}$$

$V_{\text{favourable}} \cdot H_{\text{unfavourable}}$

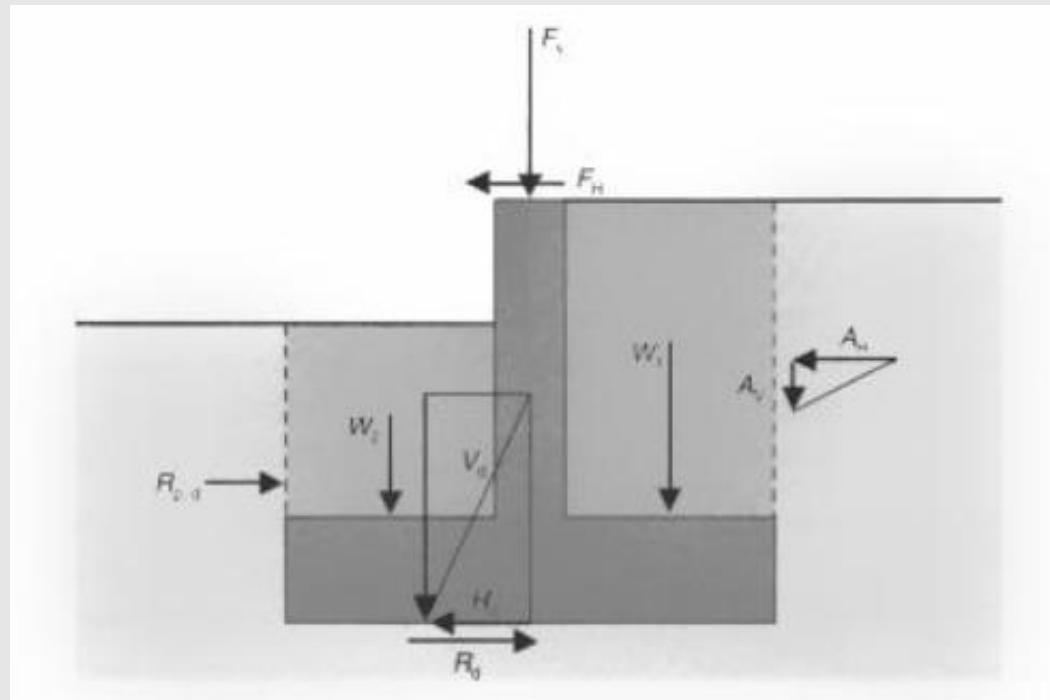
$$V_d = \gamma_G G_k + \gamma_{Qv} Q_{vk}$$

$$\gamma_G = 1.00. \gamma_{Qv} = 0.0. \gamma_{Qh} = 1.5$$

$$H_d = \gamma_{Qh} Q_{hk}$$

## Sliding resistance

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



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 disturbed. So the design friction angle  $\delta_d$  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.

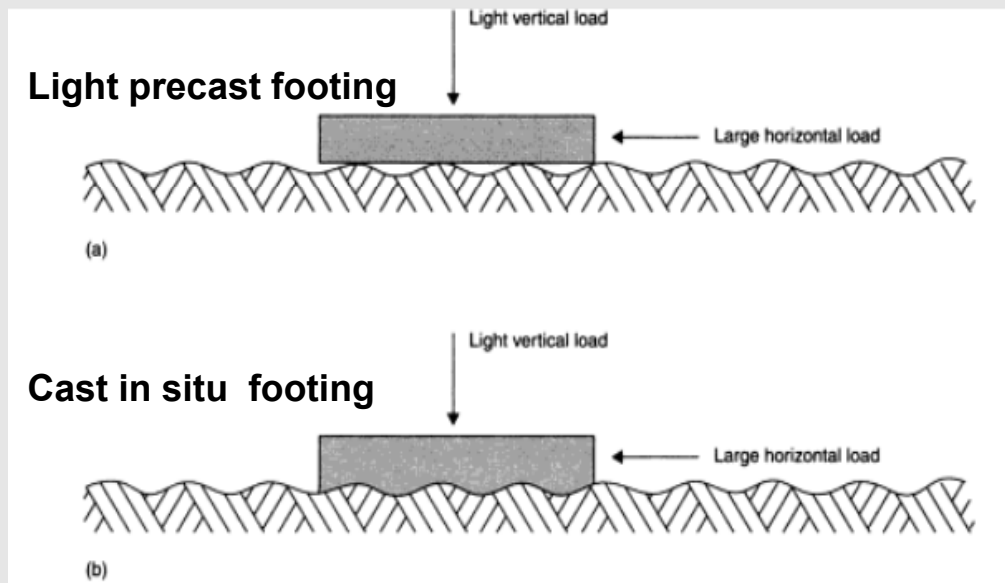
Any effective cohesion  $c'$  should be neglected.

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

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

## Considerations

The maximum available sliding resistance is likely to be mobilized with relatively little movement (and may reduce as large movements take place). Hence it could be difficult to mobilize the maximum value of both  $R_d$  and  $R_{p,d}$ . Considering also the remoulding 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 ( $0.4 V_d$ ).



## §6.6 Serviceability limit state design

**With 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. calculations of vertical displacement should usually be undertaken.

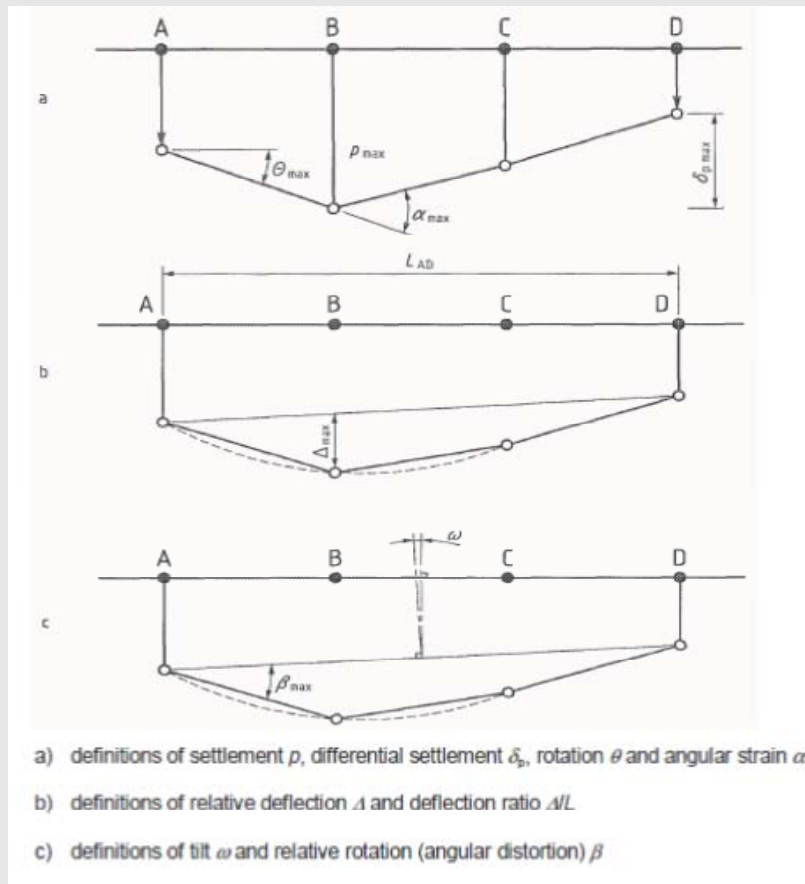
The following three components of settlement have to be considered:

- $s_0$ : 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 taken as 1
- Combination factors are those for characteristic, frequent or quasi permanent combinations ( $\psi_2$ )

## Annex H: definitions



**Ideal approach** (deterministic): solution of the interaction problem and calculation of the stresses induced in the foundation (use of FEM or subgrade reaction models)

**Conventional approach** (semi - empirical):

- $w_{\max}$  calculation
- empirical evaluation of  $\delta$  and  $\beta=f(w_{\max}, \text{foundation, ground})$
- admissibility check for  $\delta$  and  $\beta=f(\text{structure, type of damage})$

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

$$w = p \times b \times f / E_m$$

where:

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

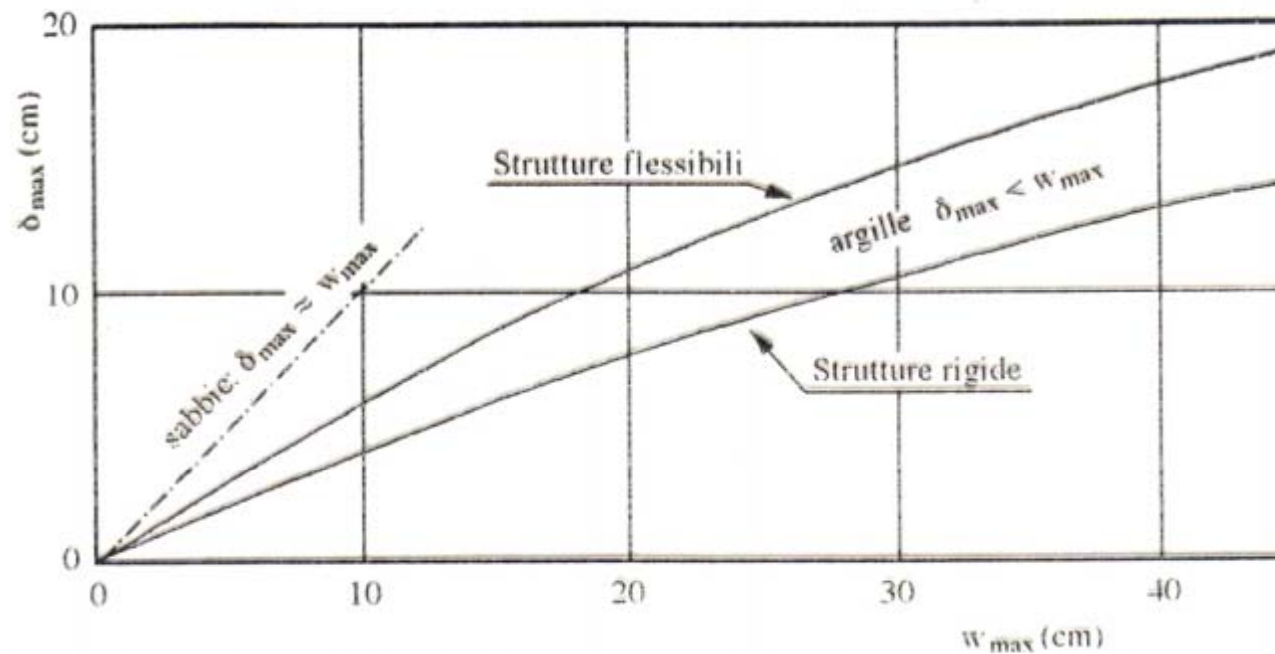
$f$  is an influence settlement coefficient

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

To calculate the settlement caused by consolidation, a confined one-dimensional deformation of the soil may be assumed

## Empirical correlations

Correlazioni empiriche tra  $\delta_{max}$  e  $w_{max}$  (Bjerrum, 1963)



## Limiting values

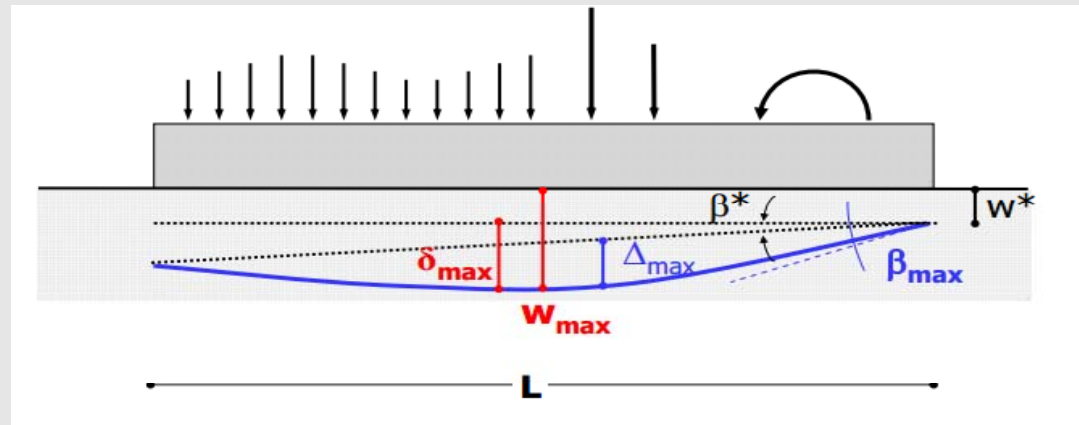
Type of structure	Type of damage/concern	Criterion	Limiting value(s)
Framed buildings and reinforced load bearing walls	Structural damage	Angular distortion	1/150 – 1/250
	Cracking in walls and partitions	Angular distortion	1/500 (1/1000-1/1400) for end bays
	Visual appearance	Tilt	1/300
	Connection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

## Annex H - Limiting values of structural deformation and foundation movement

The maximum acceptable relative rotations for open framed structures, infilled frames and load bearing or continuous brick walls are unlikely to be the same but are likely to 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.

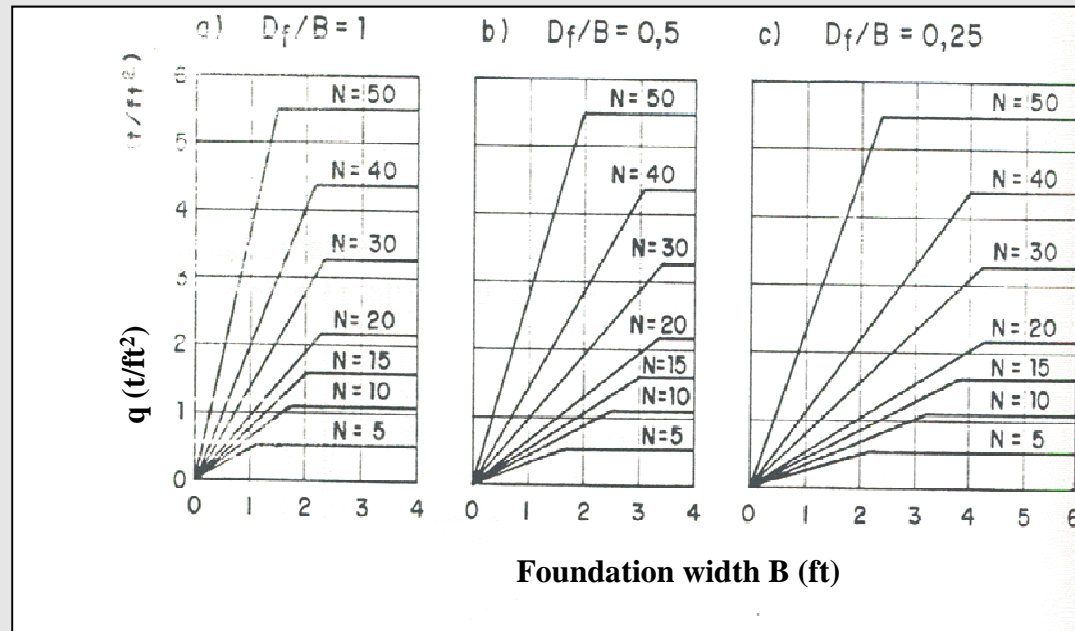
For normal structures with isolated foundations, **total settlements** up to **50 mm** are often acceptable. Larger settlements may be acceptable provided the relative rotations remain within acceptable limits and provided the total settlements do not cause problems with the services entering the structure, or cause tilting etc.



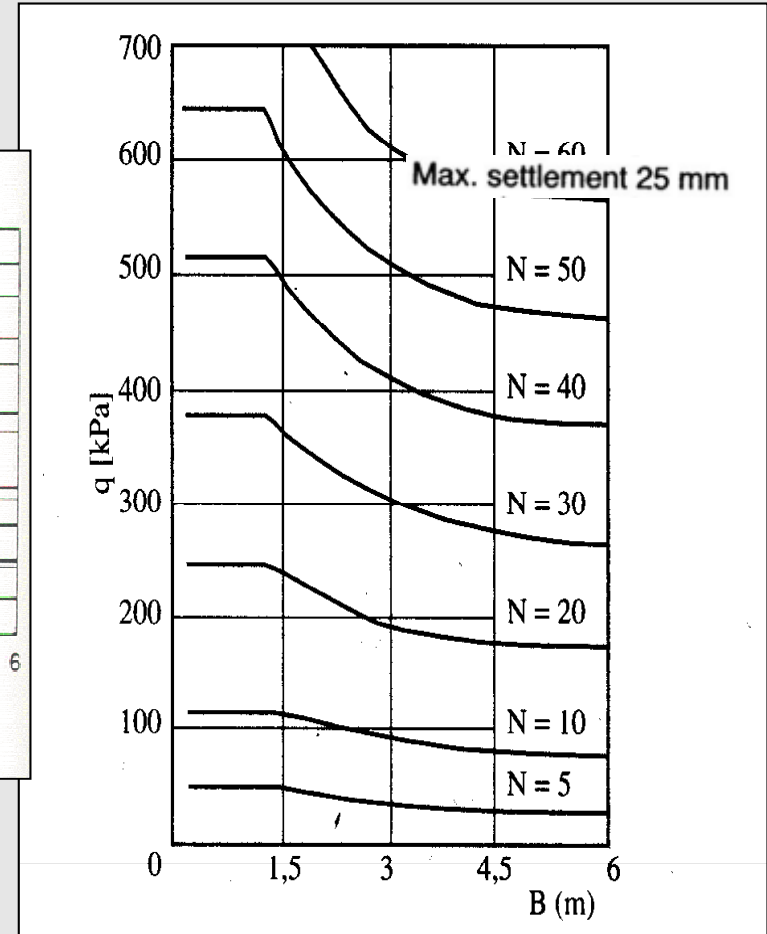


## Foundations of buildings (EC7, Annex H)

- Serviceability limit states (SLS) :  $\beta_{\max} \approx 1/500$
- Ultimate limit states (ULS) :  $\beta_{\max} \approx 1/150$
- $s_{\max} \approx 50 \text{ mm}$



### Terzaghi Peck



As the foundation width increases, the controlling limit state changes from bearing failure (ULS) to excessive settlement (SLS).

- 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 ULS and SLS have a balanced weight for normal design situations.
- This may be not the case with the adopted partial factors in the different countries (as for the Italian Code of Constructions)



# **Geotechnical design with worked examples**

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