EN 1997-1 Eurocode 7

Section 10 – Hydraulic Failure
Section 11 – Overall Stability
Section 12 – Embankments

Trevor L.L. Orr
Trinity College Dublin
Ireland

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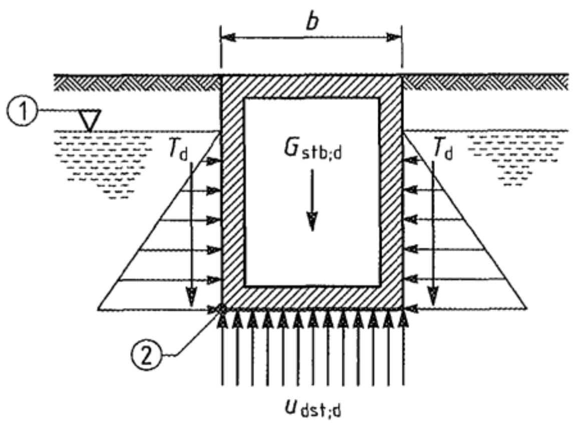
Section 10 – Hydraulic Failure

- Eurocode 7 is mainly concerned with failure modes involving the strength, stiffness or compressibility of the ground, e.g.
  - Bearing resistance failure of spread foundations
  - Failure by rotation of embedded retaining walls, and
  - Excessive settlement of spread foundations
- Section 10 of Eurocode 7 is concerned with hydraulic failure where the strength of the ground is *not significant* in providing resistance and where failure is induced by excessive pore-water pressures or pore-water seepage
- The hydraulic modes of failure include:
  1. Failure by **uplift** (buoyancy)
  2. Failure by **heave**
  3. Failure by **internal erosion**
  4. Failure by **piping**

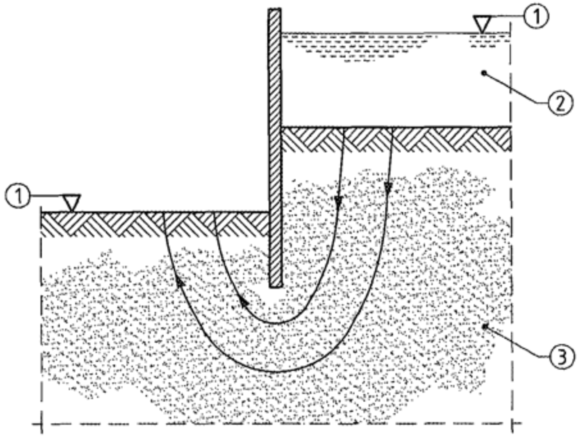
Hydraulic Failure Ultimate Limit States

- In Eurocode 7 the hydraulic failure ULSs are divided into UPL and HYD and recommended partial factor values are provided for each
- A UPL ultimate limit state is "*loss of equilibrium of a structure or the ground due to uplift by water pressure (buoyancy) or other vertical actions*"
  - A typical UPL situation is uplift of a deep basement due to hydrostatic static groundwater pressure
- An HYD ultimate limit state is "*hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients*"
  - A typical HYD situation is heave of the base of a deep excavation due to seepage around a retaining wall
- Since the strength of the ground is not significant in UPL or HYD situations, only one set of recommended partial factors is provided for each of these ULSs, not three Design Approaches as for GEO ULSs

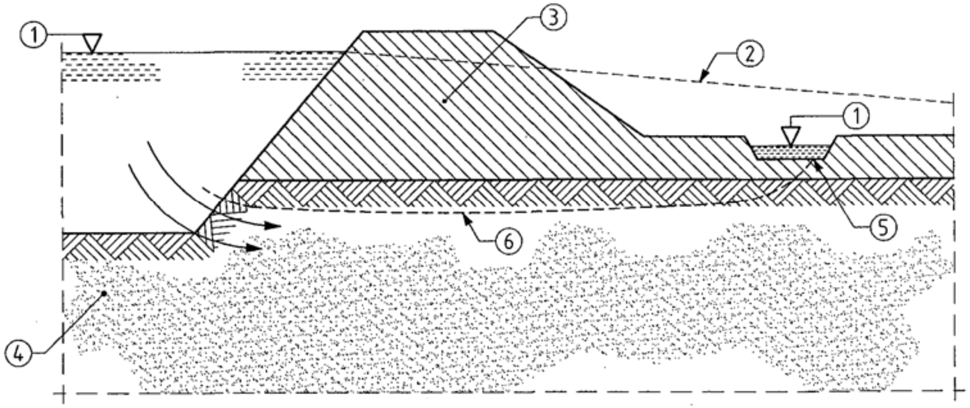
Figures from EN 1997-1 showing Hydraulic Failures



Uplift of a hollow buried cylinder



Conditions that may cause **heave**

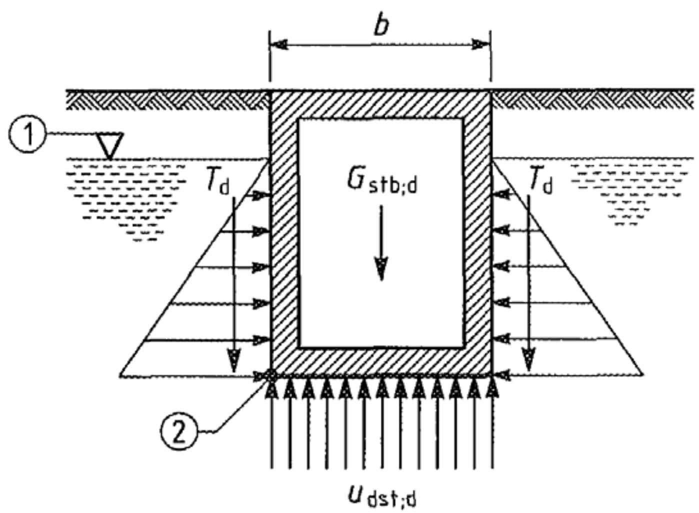


Conditions that may cause **piping**

Stabilising Forces in UPL and HYD

- For both ULS and HYD limit states one needs to check there is not loss of equilibrium with regard to stabilising and destabilising forces
- The stabilising force in UPL is mainly due to the self-weight of structure, but some stabilising force is provided by the ground resistance on the side of structure due to the strength of the ground
- HYD failure occurs when, due to the hydraulic gradient, the pore water pressure at a point in the soil exceeds the effective stress or the upward seepage force on a column of soil exceeds the effective weight of the soil
- Stabilising force in HYD is provided entirely by the weight of the soil
- The strength of the ground is not considered to be involved at all in HYD in resisting the force of the seeping water

UPL Equilibrium Equation



UPL Equilibrium

One equation given:

$$V_{dst;d} \le G_{stb;d} + R_d \qquad 2.8$$

where:

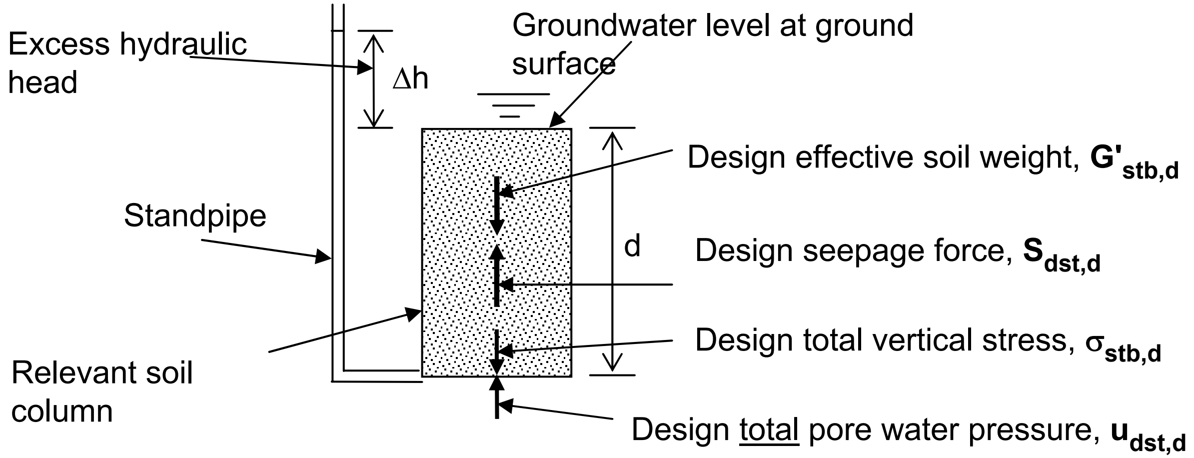
$V_{dst;d}$ = design vertical disturbing load

= $G_{dst;d}$ (design perm. load) + $Q_{dst;d}$ (design var. load)

$G_{dst;d}$ = $b \times u_{dst;d}$ (design uplift water pressure force)

R_d = T_d (design wall friction force)

HYD Equilibrium Equations



Two equations given for HYD equilibrium

First eqn: $u_{dst;d} \leq \sigma_{std;d}$ 2.9a

(total stress eqn. - only equation in Eurocode 7 in terms of stress)

Second eqn: $S_{dst;d} \leq G'_{stb;d}$ 2.9b

(seepage force and submerged weight eqn.)

i.e. $(\gamma_w \, i \, Vol)_d \leq (\gamma' \, Vol)_d$ where $i = \Delta h/d$

$(\gamma_w \, \Delta h/d)_d \leq (\gamma')_d$

$(\gamma_w \, \Delta h)_d \leq (\gamma' d)_d$

$\Delta u_{dst;d} \leq \sigma'_{stb;d}$ (effective stress eqn.)

$\Delta u_{dst;d}$ = design excess pore water pressure

Recommended UPL and HYD Partial Factors

Partial factors	UPL	HYD
Actions, γ_F		
$\gamma_{G;dst}$	1.0	1.35
$\gamma_{G;stb}$	0.9	0.9
$\gamma_{Q;dst33}$	1.5	1.5
Material properties, γ_M plus pile tensile resistance and anchorage resistance		
$\gamma_{\phi'}$	1.25	-
$\gamma_{c'}$	1.25	-
γ_{cu}	1.4	-
$\gamma_{s;t'}$	1.4	-
γ_a	1.4	-

Note:

- In **UPL**, a factor of **1.0** is recommended for **destabilising permanent actions**, e.g. uplift water pressures. The required safety is thus obtained by factoring **stabilising permanent actions by 0.9** and the **soil strength** or **resistance**.
- In **HYD,** no partial material factors are provided as **no soil strength** is involved.

Overall Factor of Safety (OFS) for Uplift

Equation 2.8:

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

For no R_d (i.e. soil resistance on side of buried structure ignored)

$$\gamma_{G;dst} V_{dst;k} = \gamma_{G;stb} G_{stb;k}$$

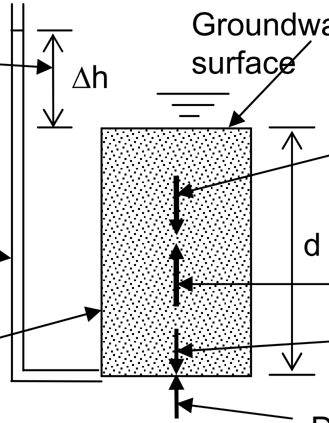
Overall factor of safety (OFS) = $G_{stb;k} / V_{dst;k}$ = $\gamma_{G;dst} / \gamma_{G;stb}$

Applying recommended partial factors

$$\gamma_{G;dst}/\gamma_{G;stb} = 1.0/0.9 = 1.11$$

Hence **OFS = 1.11**

OFS for Heave using EC7 Equations



Equation 2.9b

$$S_{dst;d} \leq G'_{stb;d}$$

$$\gamma_{G;dst} S_{dst;k} \leq \gamma_{G;stb} G'_{stb;k}$$

$$\gamma_{G;dst} \gamma_w \, i \, V \leq \gamma_{G;stb} \gamma' V$$

$$OFS_{(b)} = G_{stb;k} / S_{dst;k} = \gamma_{G;dst} / \gamma_{G;stb}$$

$= \gamma' / (\gamma_w i) = i_c/i$ = critical hydraulic gradient / actual hydraulic gradient

$$\mathbf{OFS_{(b)}} = \gamma_{dst}/\gamma_{stb} = 1.35/0.9 = \mathbf{1.5}$$

Equation 2.9a

$$u_{dst;d} \leq \sigma_{stb;d}$$

$$\gamma_{G;dst}\gamma_w d + \gamma_{G;dst}\gamma_w \Delta h \leq \gamma_{G;stb}\gamma' d + \gamma_{G;stb}\gamma_w d$$

$$\mathbf{OFS_{(a)}} = \gamma_{G;dst} / \gamma_{G;stb} = (\gamma' d + \gamma_w d) / (\gamma_w \Delta h + \gamma_w d)$$

$$= (i_c + 1)/(i + 1) = \mathbf{1.5}$$

$$i_c/i = 1.5 + 0.5/i$$

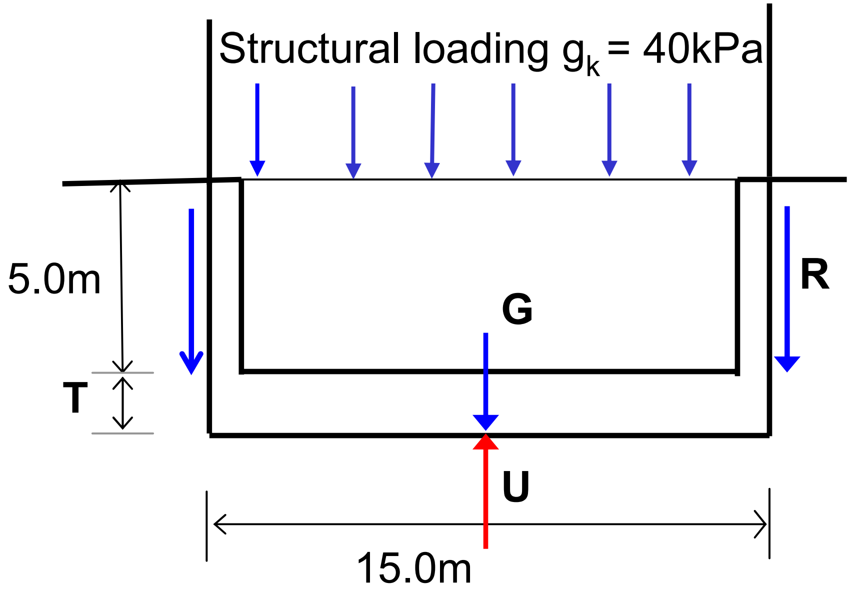
if $i = 0.5$ then **$i_c/i = OFS_{(b)} = 2.5$**

i.e. **more cautious** than using **Eqn. 2.9b** because $\gamma_w d$ occurs on both sides of equation and is **multiplied by different γ values**

Comment on HYD Overall Safety

- **HYD** ultimate limit states include **internal erosion** and **piping** as well as **heave**
- The **OFS** value traditionally used to avoid **piping** is often **very much greater** than the **1.5** provided by the HYD partial factors; e.g. 4.0
- Hence, EN 1997-1 gives **additional provisions** to avoid the occurrence of **internal erosion** or piping
- For internal erosion, it states that:
  - ***Filter criteria shall be used*** *to limit the danger of material transport by internal erosion*
  - *Measures such as* ***filter protection shall be applied*** *at the free surface of the ground*
  - *Alternatively,* ***artificial sheets*** *such as geotextiles may be used*
  - *If the filter criteria are not satisfied, it shall be verified that the design value of the* ***hydraulic gradient*** *is well* ***below the critical hydraulic gradient*** *at which soil particles begin to move.* i_c value depends on the design conditions
- EN 1997-1 states that ***piping*** *shall be prevented by providing sufficient resistance against internal soil erosion through by providing*:
  - sufficient **safety against heave**
  - sufficient **stability of the surface layers**

Uplift Design Example



Design situation:

- Long basement, 15m wide
- Sidewall thickness = 0.3m
- Characteristic structural loading = 40 kPa
- Groundwater can rise to ground surface
- Soil is sand with $\phi'_k = 35^o$, g = 20 kN/m^3
- Concrete weight density = 24 kN/m3

Require base thickness, D

U = Uplift water pressure force = $\gamma_w 15\,(5 + T)$

G = Weight of basement plus structural load

R = Resisting force from soil on side walls

Range of design values obtained: D = 0.42 – 0.85m

Why?

Model for UPL Equilibrium Calculation

Model Assumptions

Include or ignore R ?

$R = A\tau = A\sigma_h'\tan\phi' = AK\sigma_v'\tan\delta'$ where A = sidewall area

What value for K?

K is a function of ϕ' and δ. Should $K = K_0$ or K_a ?

What value for wall friction δ?

Is δ a function of ϕ'? Should $\delta = \phi'$ or $2/3\phi'$?

How should partial factors be applied to obtain R_d?

No UPL resistance factors are provided in EN 1997 to obtain R_d from R_k

i.e. there is no UPL equivalent to DA2

Design according to EN 1997-1 and assuming $\sigma_h' = K_a\sigma_v'$

1) With $R_k = AK_{a;k}\sigma_v'\tan\delta'_k$ **apply partial factor γ_M** to ϕ'_k to obtain $K_{a;d}$ and δ_d as for DA1, C2 and hence get R_d (Clause 2.4.7.4(1))

2) Treat R_κ as a permanent stabilising vertical action and **apply $\gamma_{G;stb}$ to R_k** to obtain R_d (Clause 2.4.7.4(2))

Determination of Design Value of R_d

Assume $K = K_a$ and is obtained from EN 1997-1 for $\delta = 2/3\phi'$

1) **Clause 2.4.7.4(1):** Apply partial factor γ_M to ϕ'_k to obtain $K_{a;d}$ and δ_d and hence R_d

No factors applied: $\phi'_k = 35^o$ and $\delta = 2/3\phi'$ $\rightarrow$ $K_{a;k} = 0.23$

$\delta_k = 2/3\phi'_k = 23.3^o$ $\rightarrow$ $R_k = AK_{a;k}\sigma_v' \tan\delta_k = \mathbf{0.099A\sigma_v'}$

a) γ_M **= 1.25 applied** to obtain ϕ'_d and δ_d by reducing ϕ'_k and hence δ_k

$\phi'_d = 29.3^o$ and $\delta = 2/3\phi'$ $\rightarrow$ $K_{a;d} = 0.29$

$\delta_d = 2/3\phi'_d = 19.5^o$ $\rightarrow$ $R_d = AK_{a;d}\sigma_v' \tan\delta_d = \mathbf{0.103A\sigma_v'}$

Since R is a resistance, need $R_d < R_k$: $R_d = (0.103/0.099)R_k$ $R_d = 1.04\ R_k$ –**unsafe**

b) γ_M **applied to increase ϕ'_k** but to reduce δ

$\phi'_d = 41.2^o$ and $\delta_d = 19.0^o$ $\delta_d/\phi'_d = 19.0/41.2 = 0.46$ $\rightarrow$ $K_{a;d} = 0.18$

$\rightarrow$ $R_d = AK_{a;d}\sigma_v' \tan\delta_d = 0.062A\sigma_v'$

$R_d = (0.062/0.099)R_k$ $\mathbf{R_d = 0.69\ R_k}$ – **safe**

2) **Clause 2.4.7.4(2):** Treat R_κ as a permanent stabilising vertical action and apply $\gamma_{G;stb}$ **to R_k** to obtain R_d

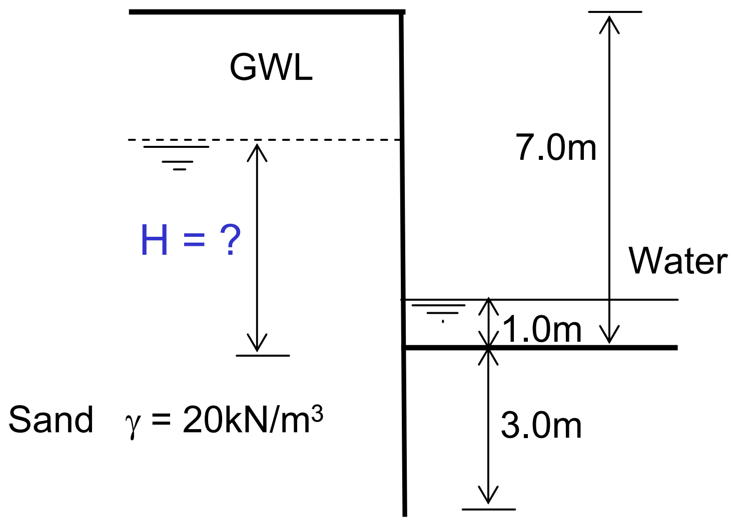
$\mathbf{R_d = \gamma_{G;stb} R_k}$ $\mathbf{R_d = 0.9\ R_k}$ – **safe**

Comments on Uplift Design Example

Reasons for Range of Solutions to Uplift Design Example:

- **Whether R Ignored or included**
- **Model chosen for R = A σ_h' tanδ = A Kσ_v' tanδ**
  - K = K_0 or K_a
  - δ = 0.5ϕ' or (2/3)ϕ'
- **How R_d is obtained**
  - Treated as a resistance or a stabilising action
  - How partial factors are applied
  - What partial factors are applied

Heave Design Example



Design Situation

- 7m deep excavation
- Sheet pile wall
- Pile penetration 3m below excavation level
- 1.0 m water in excavation
- Weight density of sand = 20 kN/m^3

Require H

Height of GWL behind wall above excavation level

Range of design values obtained: H = 1.7 – 6.6m

Why?

Reasons for Range of Solutions to Heave Example

Assumption Regarding PWP distribution around the wall (i.e. pwp at toe of wall)

– Some used equation for pwp at toe from EAU Recommendations
– Some obtained pwp at toe from flownet
– Some assumed a linear dissipation of pwp around wall - this gives least conservative designs

Choice of Equilibrium Equation

– **Some used Equation 2.9a** with partial factors applied to total pwp and total stress. This involves applying different partial factors to hydrostatic pwp on either side of equation and gave an overall factor of safety that is 1.5d/Δh greater than Equation 2.9b
– **Most** design solutions were based on **Equation 2.9b** – i.e. comparing seepage force and effective soil weight

Treatment of Seepage Force

– Some treated seepage force as a variable action
– Most considered it a permanent action

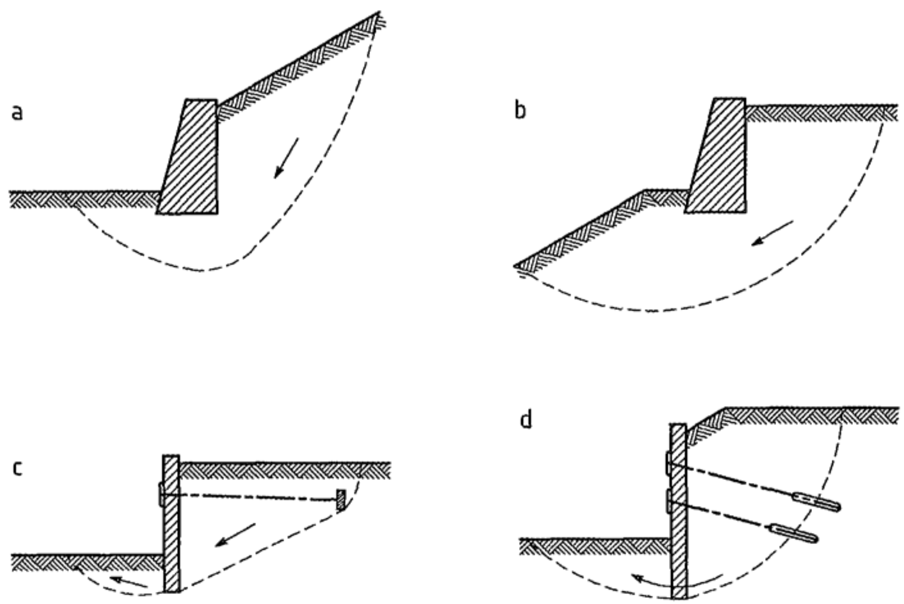
Conclusions on Section 10

- **Uplift** and **heave** ultimate limit states involving failure due to water pressures are **important** in geotechnical design and are **different** to geotechnical designs involving the strength of soil
- Need to clearly **identify** what the **stabilising** actions and the **destabilising actions** are
- This is best achieved by working in terms of **actions** (forces) rather than **stresses**
- Need to apply **partial factors** **appropriately** to get the design stabilising and destabilising actions for both uplift and heave design situations
- **Designs** against uplift and heave failure are **clarified** using Eurocode 7 as the Eurocode equilibrium equations and partial factors provide a better understanding of the design situation

Section 11: Overall Stability

- Overall stability situations are where there is **loss of overall stability** of **the ground** and **associated structures** or where **excessive movements** in the ground cause damage or loss of serviceability in neighbouring structures, roads or services
- **Typical structures** for which an analysis of overall stability should be performed:
  - Retaining structures
  - Excavations, slopes and embankments
  - Foundations on sloping ground. natural slopes or embankments
  - Foundations near an excavation, cut or buried structure, or shore
- It is stated that **a slope analysis** should verify the **overall moment** and **vertical stability** of the sliding mass. If horizontal equilibrium is not checked, inter-slice forces should be assumed to be horizontal.
  - This means that Bishop's method is acceptable, but not Fellenius'

Overall Failure Modes



Examples of **overall failure modes** involving ground failure around retaining structures presented in Section 11 of EN 1997-1

Section 12: Embankments

- Section 12: Embankments of EN 1997 provides the **principles** for the design of **embankments** for **small dams** and for **infrastructure** projects, such as **road embankments**
  - No definition is given for the word “small” but Frank et al. state that it may be appropriate to assume “small dams” include dams (and embankments for infrastructure) up to a height of approximately 10m
- A long list of possible limit states, both **GEO** and **HYD** types, that should be checked is provided including:
  - Loss of overall stability
  - Failure in the embankment slope or crest
  - Failure by internal erosion
  - Failure by surface erosion or scour
  - Excessive deformation
  - Deformations caused by hydraulic actions
- Limit states involving **adjacent structures**, roads and services are included in the list

Particular Features of Section 12

- Since embankments are constructed by placing **fill** and sometimes involve **ground improvement**, the provisions in **Section 5** should be applied
- For embankments on ground with low strength and high compressibility, EN 1997-1 states that ***the construction process shall be specified***, i.e. in Geotechnical Design Report, to ensure that the bearing resistance is not exceeded or excessive movements do not occur during construction
- 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 there is a **separate sub-section** on the **supervision** of the construction of embankments and the **monitoring** of embankments during and after construction in Section 12
- The only other section of Eurocode 7 that has provisions for both supervision and monitoring is the section on **ground anchorages**

Conclusions on Sections 10, 11 and 12

- Sections 10, 11 and 12 set out the **provisions** for designing against **hydraulic failure** and **overall stability** and for the design of **embankments**
- The focus is on the **relevant limit states** to be checked and the **equilibrium** conditions to be satisfied
- **No calculation models** are provided
- The relevance and importance of **other sections** of EN 1997-1 is demonstrated, 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
- **These sections have been accepted by the geotechnical community in Europe**


Thank You