Practical Design to Eurocode 2

The webinar will start at 12.30
## Course Outline

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<th>Title</th>
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<td>Jenny Burridge</td>
<td>Introduction, Background and Codes</td>
</tr>
<tr>
<td>2</td>
<td>28 Sep</td>
<td>Charles Goodchild</td>
<td>EC2 Background, Materials, Cover and effective spans</td>
</tr>
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<td>3</td>
<td>5 Oct</td>
<td>Paul Gregory</td>
<td>Bending and Shear in Beams</td>
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<td>12 Oct</td>
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<td>Paul Gregory</td>
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<td>Charles Goodchild</td>
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<td>Paul Gregory</td>
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<td>Jenny Burridge</td>
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<td>10</td>
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<td>Jenny Burridge</td>
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Lecture 9 Exercise

Lap length for column longitudinal bars
Column lap length exercise

Design information

- C40/50 concrete
- 400 mm square column
- 45mm nominal cover to main bars
- Longitudinal bars are in compression
- Maximum ultimate stress in the bars is 390 MPa

Exercise:
Calculate the minimum lap length using EC2 equation 8.10:

\[ l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{0,\text{min}} \]
Column lap length exercise

Procedure

• Determine the ultimate bond stress, $f_{bd}$  
  EC2 Equ. 8.2

• Determine the basic anchorage length, $l_{b,req}$  
  EC2 Equ. 8.3

• Determine the design anchorage length, $l_{bd}$  
  EC2 Equ. 8.4

• Determine the lap length, $l_0 = \text{anchorage length} \times \alpha_6$
Model Answers

Lap length for column longitudinal bars
Column lap length exercise

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Calculate the minimum lap length using EC2 equation 8.10:

\[ l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b, rqd} \geq l_{0, min} \]
Column lap length exercise

Procedure

• Determine the ultimate bond stress, $f_{bd}$  
  
• Determine the basic anchorage length, $l_{b,req}$  
  
• Determine the design anchorage length, $l_{bd}$  
  
• Determine the lap length, $l_0 = \text{anchorage length} \times \alpha_6$

EC2 Equ. 8.2
EC2 Equ. 8.3
EC2 Equ. 8.4
Solution - Column lap length

Determine the ultimate bond stress, $f_{bd}$

$$f_{bd} = 2.25 \eta_1 \eta_2 f_{ctd}$$

$\eta_1 = 1.0$ ‘Good’ bond conditions

$\eta_2 = 1.0$ bar size $\leq 32$

$$f_{ctd} = \alpha_{ct} \frac{f_{ctk,0.05}}{\gamma_c}$$

$\alpha_{ct} = 1.0$ \hspace{1cm} $\gamma_c = 1.5$

$$f_{ctk,0.05} = 0.7 \times 0.3 \frac{f_{ck}^{2/3}}{\gamma_c} = 0.21 \times 40^{2/3} = 2.456 \text{ MPa}$$

$$f_{ctd} = \alpha_{ct} \frac{f_{ctk,0.05}}{\gamma_c} = 2.456/1.5 = 1.637$$

$$f_{bd} = 2.25 \times 1.637 = 3.684 \text{ MPa}$$
Solution - Column lap length

Determine the basic anchorage length, \( l_{b,\text{req}} \)

\[
l_{b,\text{req}} = \left( \frac{Ø}{4} \right) \left( \frac{\sigma_{sd}}{f_{bd}} \right)
\]

EC2 Equ 8.3

Max ultimate stress in the bar, \( \sigma_{sd} = 390 \, \text{MPa} \).

\[
l_{b,\text{req}} = \left( \frac{Ø}{4} \right) \left( \frac{390}{3.684} \right)
\]

\[
= 26.47 \, Ø
\]

For concrete class C40/50
Determine the design anchorage length, $l_{bd}$

\[ l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 \ l_{b,req} \geq l_{b,min} \]  \hspace{1cm} \text{Equ. 8.4}

\[ l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 (26.47\varnothing) \]  \hspace{1cm} \text{For concrete class C40/50}

For bars in compression $\alpha_1 = \alpha_2 = \alpha_3 = \alpha_4 = \alpha_5 = 1.0$

Hence $l_{bd} = 26.47\varnothing$
Solution - Column lap length

Determine the lap length, $l_0 = \text{anchorage length} \times \alpha_6$

All the bars are being lapped at the same section, $\alpha_6 = 1.5$
A lap length is based on the smallest bar in the lap, 25mm
Hence,

\[ l_0 = l_{bd} \times \alpha_6 \]
\[ l_0 = 26.47 \, \text{Ø} \times 1.5 \]
\[ l_0 = 39.71 \, \text{Ø} = 39.71 \times 25 \]
\[ l_0 = 993 \, \text{mm} \]
Foundations
Outline - Week 10, Foundations

We will look at the following topics:

- **Eurocode 7: Geotechnical design** - Partial factors, spread foundations.
- **Pad foundation - Worked example & workshop**
- **Retaining walls**
- **Piles**
Eurocode 7

Eurocode 7 has two parts:

Part 1: General Rules

Part 2: Ground Investigation and testing
How to...... 6. Foundations

The essential features of EC7, Pt 1 relating to foundation design are discussed.

Note:

This publication covers only the design of simple foundations, which are a small part of EC7.

It should not be relied on for general guidance on EC7.
Limit States

The following ultimate limit states apply to foundation design:

EQU: Loss of equilibrium of the structure

STR: Internal failure or excessive deformation of the structure or structural member

GEO: Failure due to excessive deformation of the ground

UPL: Loss of equilibrium due to uplift by water pressure

HYD: Failure caused by hydraulic gradients
### Categories of Structures

<table>
<thead>
<tr>
<th>Category</th>
<th>Description</th>
<th>Risk of geo-technical failure</th>
<th>Examples from EC7</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Small and relatively simple structures</td>
<td>Negligible</td>
<td>None given</td>
</tr>
<tr>
<td>2</td>
<td>Conventional types of structure - no difficult ground</td>
<td>No exceptional risk</td>
<td>Spread foundations</td>
</tr>
<tr>
<td>3</td>
<td>All other structures</td>
<td>Abnormal risks</td>
<td>Large or unusual structures</td>
</tr>
</tbody>
</table>
EC7 - ULS Design

EC7 provides for three Design Approaches

UK National Annex - Use Design Approach 1 - DA1

For DA1 (except piles and anchorage design) there are two sets of combinations to use for the STR and GEO limit states.

- Combination 1 - generally governs structural resistance
- Combination 2 - generally governs sizing of foundations
## STR/GEO ULS - Actions partial factors

<table>
<thead>
<tr>
<th>Combination 1</th>
<th>Permanent Actions</th>
<th>Leading variable action</th>
<th>Accompanying variable actions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unfavourable</td>
<td>Favourable</td>
<td>Main</td>
</tr>
<tr>
<td>Exp 6.10</td>
<td>1.35G_k</td>
<td>1.0G_k</td>
<td>1.5Q_k</td>
</tr>
<tr>
<td>Exp 6.10a</td>
<td>1.35G_k</td>
<td>1.0G_k</td>
<td></td>
</tr>
<tr>
<td>Exp 6.10b</td>
<td>1.25G_k</td>
<td>1.0G_k</td>
<td>1.5Q_k</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Combination 2</th>
<th>Permanent Actions</th>
<th>Leading variable action</th>
<th>Accompanying variable actions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unfavourable</td>
<td>Favourable</td>
<td>Main</td>
</tr>
<tr>
<td>Exp 6.10</td>
<td>1.0G_k</td>
<td>1.0G_k</td>
<td>1.3Q_k</td>
</tr>
</tbody>
</table>

**Notes:**
- If the variation in permanent action is significant, use $G_{k,j,\text{sup}}$ and $G_{k,j,\text{inf}}$.
- If the action is favourable, $γ_{Q,i} = 0$ and the variable actions should be ignored.
## Factors for EQU, UPL and HYD

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Permanent Actions</th>
<th>Variable Actions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unfavourable</td>
<td>Favourable</td>
</tr>
<tr>
<td>EQU</td>
<td>1.1</td>
<td>0.9</td>
</tr>
<tr>
<td>UPL</td>
<td>1.1</td>
<td>0.9</td>
</tr>
<tr>
<td>HYD</td>
<td>1.35</td>
<td>0.9</td>
</tr>
</tbody>
</table>
## Partial factors - material properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Combination 1</th>
<th>Combination 2</th>
<th>EQU</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle of shearing resistance</td>
<td>$\gamma_\varphi$</td>
<td>1.0</td>
<td>1.25</td>
<td>1.1</td>
</tr>
<tr>
<td>Effective cohesion</td>
<td>$\gamma_c'$</td>
<td>1.0</td>
<td>1.25</td>
<td>1.1</td>
</tr>
<tr>
<td>Undrained shear strength</td>
<td>$\gamma_{cu}$</td>
<td>1.0</td>
<td>1.4</td>
<td>1.2</td>
</tr>
<tr>
<td>Unconfined strength</td>
<td>$\gamma_{qu}$</td>
<td>1.0</td>
<td>1.4</td>
<td>1.2</td>
</tr>
<tr>
<td>Bulk density</td>
<td>$\gamma_Y$</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>
The Geotechnical Report should:

- be produced for each project (if even just a single sheet)
- contain details of:
  - the site,
  - interpretation of ground investigation report,
  - geotechnical recommendations,
  - advice

Foundation design recommendations should state:

- bearing resistances,
- characteristic values of soil parameters and
- whether values are SLS or ULS, Combination 1 or Combination 2 values
EC7 Section 6

Three methods for design:

- **Direct method** - check all limit states:
  - [Formula]: $q_{ult} = c'N_c s_c d_i g_i b_c + q'N_q s_q d_q g_q b_q + \gamma'N_i \gamma s_i \gamma d_i \gamma g_i \gamma b_i / 2$

  Where:
  - $c$ = cohesion
  - $q$ = overburden
  - $\gamma$ = body-weight
  - $N_i$ = bearing capacity factors
  - $s_i$ = shape factors
  - $d_i$ = depth factors
  - $i_i$ = inclination factors
  - $g_i$ = ground inclination factors
  - $b_i$ = base inclination factors

  "We just bung it in a spreadsheet"

Settlement often critical

See *Decoding Eurocode 7* by A Bond & A Harris, Taylor & Francis
EC7 Section 6

Three methods for design:

- Direct method - check all limit states
- Indirect method - experience and testing used to determine SLS parameters that also satisfy ULS
- Prescriptive methods - use presumed bearing resistance (BS8004 quoted in NA).
Spread Foundations

Design procedures in:

How to design concrete structures using Eurocode 2
6. Foundations

Eurocode 7: Geotechnical design

Scope
All foundations should be designed so that the soil safely resists the actions applied to the structure. The design of any foundation consists of two components: the geotechnical design and the structural design of the foundation itself. However, for some foundations (e.g. flexible or floating), the effect of the interaction between the soil and structure may be critical and must also be considered. Geotechnical design is covered by Eurocode 7, which supersedes several current British Standards including BS 5930, BS 4085 and BS 8007. The new Eurocode marks a significant change in geotechnical design as the limit state principles are used throughout and this should ensure consistency between the Eurocodes. There are two parts to Eurocode 7, Part 1: General Rules and Part 2: Ground Investigation and Testing. Guidance on the design of retaining walls can be found in Chapter 5.

The essential features of Eurocode 7, Part 1 relating to foundation design are detailed in this chapter. It should be emphasised that this publication covers only the design of simple foundations, which are a small part of the scope of Eurocode 7. Therefore, it should not be relied on for general guidance on this Eurocode.

Limit states
The following ultimate limit states (ULS) should be satisfied for geotechnical design: they each have their own combinations of actions. (For an explanation of Eurocode terminology please refer to Chapter 1, originally published as Introduction to Eurocode2.)

- EDQ: Loss of equilibrium of the structure
- STR: Internal failure or excessive deformation of the structure or structural member
- GEO: Failure due to excessive deformation of the ground
- SPL: Loss of equilibrium due to lift by water pressure
- PFD: Failure caused by hydraulic gradients

In addition, the serviceability limit state (SLS) should be satisfied: it will usually be clear that one of the limit states will govern the design and therefore it will not be necessary to carry out checks for all of them, although it is considered good practice to record that they have all been considered.

Geotechnical Categories
Eurocode 7 recommends three Geotechnical Categories in establishing the geotechnical design requirements for a structure (see Table 1).
Procedure for depth of spread foundations

1. Start
2. Obtain soil parameters from Ground Investigation report
3. Design using direct method?
   - Yes: Size foundation (geotechnical design) using the worst of Combinations 1 or 2 (ULS) for actions and geotechnical material properties. Combination 2 will usually govern.
   - No: Use prescriptive method. Size foundation (geotechnical design) using SLS for actions and presumed bearing resistance.
4. Is there an overturning moment?
   - Yes: Check overturning using EQU limit state for actions and GEO Combination 2 for material properties.
   - No: Design foundation (structural design) using the worst of Combinations 1 and 2 (ULS) for actions and geotechnical material properties.
Pressure distributions

SLS pressure distributions

ULS pressure distribution
Load cases

EQU : $0.9 \, G_k + 1.5 \, Q_k$ (assuming variable action is destabilising e.g. wind, and permanent action is stabilizing)

STR : $1.35 \, G_k + 1.5 \, Q_k$ (Using (6.10). Worse case of Exp (6.10a) or (6.10b) could be used)
Plain Concrete Strip Footings & Pad Foundations:

Cl. 12.9.3, Exp (12.13)

\[
\frac{0.85 \cdot h_f}{a} \geq \sqrt{\frac{3 \sigma_{gd}}{f_{c_{td,pl}}}}
\]

where:
\(\sigma_{gd}\) is the design value of the ground pressure

- as a simplification \(h_f/a \geq 2\) may be used
Plain Concrete Strip Footings & Pad Foundations

<table>
<thead>
<tr>
<th>allowable pressure</th>
<th>C16/20</th>
<th>C20/25</th>
<th>C25/30</th>
<th>C30/37</th>
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<tbody>
<tr>
<td>( \sigma_{gd} )</td>
<td>( h_F/a )</td>
<td>( h_F/a )</td>
<td>( h_F/a )</td>
<td>( h_F/a )</td>
</tr>
<tr>
<td>50</td>
<td>70</td>
<td>0.65</td>
<td>0.60</td>
<td>0.55</td>
</tr>
<tr>
<td>100</td>
<td>140</td>
<td>0.92</td>
<td>0.85</td>
<td>0.78</td>
</tr>
<tr>
<td>150</td>
<td>210</td>
<td>1.12</td>
<td>1.04</td>
<td>0.95</td>
</tr>
<tr>
<td>200</td>
<td>280</td>
<td>1.29</td>
<td>1.21</td>
<td>1.10</td>
</tr>
<tr>
<td>250</td>
<td>350</td>
<td>1.45</td>
<td>1.35</td>
<td>1.23</td>
</tr>
</tbody>
</table>

Example:

- Cavity wall 300 wide carrying 80 kN/m onto 100 kN/m² ground:
  - \( b_f = 800 \) mm
  - \( a = 250 \) mm
  - \( h_f = \) say assuming C20/25 concrete
  - \( 0.85 \times 250 = 213 \) say 225 mm
Reinforced Concrete Bases

• Check critical bending moments at column faces
• Check beam shear and punching shear

For punching shear the ground reaction within the perimeter may be deducted from the column load
Pad foundation

Worked example
Worked Example

Design a square pad footing for a 350 \times 350 \text{mm} column carrying $G_k = 600 \text{kN}$ and $Q_k = 505 \text{kN}$. The presumed allowable bearing pressure of the non-aggressive soil is $200 \text{kN/m}^2$.

Answer:

Category 2. So using prescriptive methods:

Base area: $(600 + 505)/200 = 5.525 \text{m}^2$

=> 2.4 \times 2.4 \text{base} \times 0.5 \text{m (say) deep.}$
Worked Example

Loading = 1.35 \times 600 + 1.5 \times 505
= 1567.5 \text{ kN}

ULS bearing pressure = \frac{1567.5}{2.4^2}
= 272 \text{ kN/m}^2

Critical section at face of column

\[
M_{Ed} = 272 \times 2.4 \times 1.025^2 / 2
= 343 \text{ kNm}
\]

\[
d = 500 - 50 - 16
= 434 \text{ mm}
\]

\[
K = 343 \times 10^6 / (2400 \times 434^2 \times 30)
= 0.025
\]

Use C30/37 concrete
Worked Example

\[ z = 0.95d = 0.95 \times 434 = 412 \text{mm} \]

\[ A_s = \frac{M_{Ed}}{f_{yd}z} = \frac{343 \times 10^6}{(435 \times 412)} = 1914 \text{mm}^2 \]

\[ \Rightarrow \text{Provide 10H16 @ 250 c/c b.w (2010 mm}^2) (804 \text{ mm}^2/\text{m}) \]

Beam shear:

Check critical section d away from column face

\[ V_{Ed} = 272 \times (1.025 - 0.434) = 161 \text{kN/m} \]

\[ n_{Ed} = \frac{161}{434} = 0.37 \text{MPa} \]

\[ \rho = \frac{2010}{(434 \times 2400)} = 0.0019 = 0.19\% \]

\[ n_{Rd,c} \text{ (from table) = 0.42MPa} \]

\[ => \text{beam shear ok.} \]
Worked Example

Punching shear:

Basic control perimeter at 2d from face of column

\[ \nu_{Ed} = \beta V_{Ed} / u_i d < \nu_{Rd,c} \]

\( \beta = 1, \ u_i = (350 \times 4 + 434 \times 2 \times 2 \times \pi) = 6854\text{mm} \)

\[ V_{Ed} = \text{load minus net upward force within the area of the control perimeter} \]

\[ = 1567.5 - 272 \times (0.35^2 + \pi \times 0.868^2 + 0.868 \times 0.35 \times 4) \]

\[ = 560\text{kN} \]

\[ \nu_{Ed} = 0.188 \text{MPa}; \ \nu_{Rd,c} = 0.42 \text{ (as before)} \implies \text{ok} \]
Introduction
This chapter covers the analysis and design of reinforced concrete retaining walls to Eurocode 7 and 8, including retaining walls up to 2 m high and stopped banisters with up to two storeys high. These limits have been chosen so that simplifications can be made in the geometrical design. The self-weight of these walls, including the self-weight of fill, if any, plays a significant role in supporting the retained material. The chapter also covers the analysis and design of retaining walls, which are primarily on piled raft foundations and facing arrangements to support the retained material.

The content of this chapter is largely based on the Eurocode 7 Part 1:2007 and the Eurocode 8:2004.

Geotechnical Categories
Eurocode 7 Part 1 defines three Geotechnical Categories that can be used to establish geotechnical design requirements. These are

1. Geotechnical Category 1: Where the mean soil suction is 50 kPa or less and the soil suction is less than 100 kPa.
2. Geotechnical Category 2: Where the mean soil suction is 50 kPa or less and the soil suction is greater than 50 kPa.
3. Geotechnical Category 3: Where the mean soil suction is greater than 50 kPa.

Limit States
The design of retaining walls requires consideration of two limit states:

1. Serviceability Limit State (SLS): This limit state requires that the wall does not exhibit excessive settlement, heave, or tilt.
2. Ultimate Limit State (ULS): This limit state requires that the wall does not fail in tension, compression, or shear.
Ultimate Limit States for the design of retaining walls

a) Overall stability
b) Sliding
c) Toppling
d) Bearing
e) Structural failure
Calculation Model A

Rankine theory
Model applies if $b_n \geq h_a \tan (45 - \phi_d/2)$
Calculation Model B

Inclined ‘virtual’ plane theory
Model applies to walls of all shapes and sizes
### General expressions

<table>
<thead>
<tr>
<th>General</th>
<th>Model A</th>
<th>Model B</th>
</tr>
</thead>
<tbody>
<tr>
<td>$W_s = b_s H \gamma_{k,c}$</td>
<td>$h = t_b + H + b_h \tan \beta$</td>
<td>$h_b = t_b + H$</td>
</tr>
<tr>
<td>$W_b = t_b B \gamma_{k,c}$</td>
<td>$W_f = b_h \left( H + \frac{b_h \tan \beta}{2} \right) \gamma_{k,f}$</td>
<td>$W_{k,f} \approx \frac{b_h H}{2} \gamma_{k,f}$</td>
</tr>
<tr>
<td>$b_h = B - b_s - b_t$</td>
<td>$L_f \approx b_t + b_s + \frac{b_h}{2}$</td>
<td>$L_f = b_t + b_s + \frac{b_h}{3}$</td>
</tr>
<tr>
<td>$L_s = b_t + \frac{b_s}{2}$</td>
<td>$\Omega = \beta$</td>
<td>$\theta = \tan^{-1} \left( \frac{b_h}{h_b} \right)$</td>
</tr>
<tr>
<td>$L_b = \frac{B}{2}$</td>
<td>$L_{vp} = B$</td>
<td>$\Omega = \theta$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$L_{vp} = B - \frac{b_h}{3}$</td>
</tr>
</tbody>
</table>
Overall design procedure

1. Start
2. Check overall stability of the site
3. Carry out initial sizing of the wall
4. Determine characteristic material properties (see Figure 6)
5. Calculate the characteristic value of actions (see Figure 6)
6. Calculate the design value of actions (see Figure 6)
7. Design against sliding (see Figure 7)
8. Design against toppling (see Figure 9)
9. Design against bearing failure (see Figure 10)
10. Structural design (see Figure 13)
11. Sufficient capacity?
   - Yes: Design complete
   - No, revise geometry

Comprehensive flowchart showing the overall design procedure for a wall, including steps to check stability, determine material properties, calculate characteristic and design values, and account for various types of failure modes.
Initial sizing

\[ b_s \approx t_b \approx h/10 \text{ to } h/15 \]

\[ B \approx 0.5h \text{ to } 0.7h \]

\[ b_t \approx B/4 \text{ to } B/3 \]
Overall design procedure

1. Start
2. Check overall stability of the site
3. Carry out initial sizing of the wall
4. Determine characteristic material properties (see Figure 6)
5. Calculate the characteristic value of actions (see Figure 6)
6. Calculate the design value of actions (see Figure 6)
7. Design against sliding (see Figure 7)
8. Design against toppling (see Figure 9)
9. Design against bearing failure (see Figure 10)
10. Structural design (see Figure 13)

Sufficient capacity?

- Yes: Design complete
- No, revise geometry
Determine characteristic material properties
- Weight density ($\gamma_{k,t}$) and angle of shearing resistance ($\phi'_{k,t}$) of backfill
- Weight density ($\gamma_{k,frq}$), undrained strength ($c_{uk,frq}$), and angle of shearing resistance ($\phi'_{k,frq}$) of foundation soil
- Weight density ($\gamma_{k,c}$) and cylinder strength ($f_{c,d}$) of concrete

Determine initial geometry and actions (see Panel 1)
1. Determine initial geometry (see Figure 5)
   - For calculation model B find inclination of virtual plane ($\theta$)
2. Determine lever arms (see Figures 2c or 3c)
3. Calculate forces
   - Characteristic self-weights ($W_{k,i}$, $W_{k,b}$, $W_{k,f}$)
   - Total weight $\Sigma W_{k,i} = W_{k,i} + W_{k,b} + W_{k,f}$
   - Total moment $\Sigma (W_{k,i} \cdot L_i) = (W_{k,i} \cdot L_i) + (W_{k,b} \cdot L_b) + (W_{k,f} \cdot L_f)$

Carry out separately for both Combinations 1 and 2

Determine design material properties and earth pressures (see Panel 2)
1. Apply partial material factors to obtain design material properties
   - Weight density $\gamma_d = \gamma_k / \gamma_i = \gamma_k$ since $\gamma_i = 1$
   - Angle of shearing resistance $\phi'_d = \tan^{-1}(\tan \phi'_k / \gamma_k)$
   - Undrained strength $c_{ud} = c_{uk} / \gamma_{cu}$
2. Determine earth pressure
   - Earth pressure coefficient $K_{ad} = \text{function of } \phi'_d$ and $\beta$
     (depending on calculation model used)
   - Earth pressure is function of vertical load and $K_{ad}$

9 (Figure 6)

Figure 6 for overall design procedure
## Soil Densities

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Moist bulk density, ( \gamma ) (kg/m(^3))</th>
<th>Saturated bulk density, ( \gamma ) (kg/m(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Loose</td>
<td>Dense</td>
</tr>
<tr>
<td>Granular</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravel</td>
<td>1600</td>
<td>1800</td>
</tr>
<tr>
<td>Well graded sand and gravel</td>
<td>1900</td>
<td>2100</td>
</tr>
<tr>
<td>Coarse or medium sand</td>
<td>1650</td>
<td>1850</td>
</tr>
<tr>
<td>Well graded sand</td>
<td>1800</td>
<td>2100</td>
</tr>
<tr>
<td>Brick hardcore</td>
<td>1300</td>
<td>1750</td>
</tr>
<tr>
<td>Cohesive</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soft clay</td>
<td>1700</td>
<td></td>
</tr>
<tr>
<td>Firm clay</td>
<td>1800</td>
<td></td>
</tr>
<tr>
<td>Stiff clay</td>
<td>1900</td>
<td></td>
</tr>
<tr>
<td>Stiff or hard glacial clay</td>
<td>2100</td>
<td></td>
</tr>
</tbody>
</table>

Ex Concrete Basements
Design value of effective angle of shearing resistance, $\varphi'_d$

$$\tan \varphi'_d = \tan \left( \frac{\varphi'_k}{\gamma_\varphi} \right)$$

where

$$\varphi'_k = \varphi'_{\text{max}}$$ for granular soils and

$$\varphi'$$ for clay soils,

$$\varphi'_{\text{max}}$$ and $$\varphi'$$ are as defined as follows

$$\gamma_\varphi = 1.0 \text{ or } 1.25$$ dependent on the

*Combination* being considered.
Angle of shearing resistance

Granular Soils

Estimated peak effective angle of shearing resistance,
\[ \varphi'_\text{max} = 30 + A + B + C \]

Estimated critical state angle of shearing resistance,
\[ \varphi'_\text{crit} = 30 + A + B, \text{ which is the upper limiting value.} \]

<table>
<thead>
<tr>
<th>Angularity</th>
<th>A (degrees)</th>
<th>Grading</th>
<th>B (degrees)</th>
<th>Number of blows, (N')</th>
<th>C (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rounded</td>
<td>0</td>
<td>Uniform</td>
<td>0</td>
<td>&lt;10</td>
<td>0</td>
</tr>
<tr>
<td>Sub-angular</td>
<td>2</td>
<td>Moderate grading</td>
<td>2</td>
<td>20</td>
<td>2</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>40</td>
<td>6</td>
</tr>
<tr>
<td>Angular</td>
<td>4</td>
<td>Well graded</td>
<td>4</td>
<td>60</td>
<td>9</td>
</tr>
</tbody>
</table>

Ex Concrete Basements
Clay soils

Long term ≡ Granular Soils

<table>
<thead>
<tr>
<th>Plasticity index (%)</th>
<th>15</th>
<th>30</th>
<th>50</th>
<th>80</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varphi'$ (degrees)</td>
<td>30</td>
<td>25</td>
<td>20</td>
<td>15</td>
</tr>
</tbody>
</table>

Ex Concrete Basements
For calculation model A:

\[ K_{ad} = \frac{\cos \beta - \sqrt{\sin^2 \varphi_{df} - \sin^2 \beta}}{\cos \beta + \sqrt{\sin^2 \varphi_{df} - \sin^2 \beta}} \]

\[ P_{ad} = K_{ad} \left( \frac{\gamma_c \gamma_{kf} h^2}{2} + \gamma_Q q_k h \right) \]

For calculation model B:

\[ m_t = \frac{1}{2} \left( \cos^{-1} \left( \frac{\sin \beta}{\sin \varphi'_{df}} \right) + \varphi'_{df} - \beta \right) \]

\[ K_n = \left( \frac{1 + \sin^2 \varphi'_{df}}{1 - \sin \varphi'_{df} \sin (2m_t - \varphi'_{df})} \right) e^{-2 \left( m_t + \beta - \varphi'_{df} - \theta \right) \tan \varphi'_{df}} \]

\[ K_q = K_n \cos^2 \beta \]

\[ K_y = K_n \cos \beta \cos (\beta - \theta) \]

\[ P_{ad} = \gamma_c \gamma_{kf} h^2 + \gamma_Q K_q q_k h \]
Overall design procedure
Design against sliding (Figure 7)

Determine horizontal thrust on virtual plane (see Panel 2)
- \( H_{Ed} = P_{ad} \cos \Omega \)

If appropriate determine undrained sliding resistance on underside of wall base
- \( H_{Rd} = c_{ud,fdn} B \)

Determine drained sliding resistance on underside of wall base
- Calculate vertical action \( V_d = \gamma_a \sum_i W_{k,i} \)
- Calculate vertical traction from \( P_{ad} \) (Panel 2)
- Calculate drained sliding resistance, \( H'_{Rd} \) (Panel 3)

Verify \( H_{Ed} \leq H_{Rd} \) (if appropriate) and \( H_{Ed} \leq H'_{Rd} \)

If necessary, repeat the above with a shear key
Sliding Resistance

Undrained sliding resistance:

\[ H_{Rd} = c_{ud,fdn} B \]

Drained sliding resistance:

\[ \delta_{d,fdn} = \varphi_{cvd,fdn} \]

\[ H'_{Rd} = V_d \tan \delta_{d,fdn} = \left[ \gamma_{G,fav} \sum_i W_{k,i} \right] \tan \varphi_{cvd,fdn} \]

With a shear key:

\[ H'_{Rd} \approx \left[ \gamma_{G,fav} \sum_i W_{k,i} \right] \tan \varphi'_{d,fdn} \sqrt{1 + \left( \frac{\Delta t_0}{B} \right)^2} \]
Overall design procedure

1. Start
2. Check overall stability of the site
3. Carry out initial sizing of the wall
4. Determine characteristic material properties (see Figure 6)
5. Calculate the characteristic value of actions (see Figure 6)
6. Calculate the design value of actions (see Figure 6)
7. Design against sliding (see Figure 7)
8. Design against toppling (see Figure 9)
9. Design against bearing failure (see Figure 10)
10. Structural design (see Figure 13)

- Carry out separately for both Combinations 1 and 2
- Sufficient capacity? (Yes/No, revise geometry)
- Design complete
Design against Toppling

Determine destabilising moment $M_{Ed}$ about point $E$.
(see Panels 1 and 2)  
$$M_{Ed} = P_{ad} \cos \Omega \frac{h}{3}$$

Determine stabilising moments about point $E$
- Wall stem $M_{d,s} = W_{d,s} L_s$
- Wall base $M_{d,b} = W_{d,b} L_b$
- Backfill $M_{d,f} = W_{d,f} L_f$
- From active thrust, $M_{d,vp} = P_{ad} \sin \Omega L_{vp}$
- $M_{Rd} = M_{d,s} + M_{d,b} + M_{d,f} + M_{d,vp}$

Verify $M_{Ed} \leq M_{Rd}$
Overall design procedure
Design against bearing failure

Determine bearing pressure (see Panels 1 and 2)
\[ q_{Ed} = \left( V_d + P_{ad} \sin \Omega \right) / B' \]

Determine undrained bearing capacity, \( q_{Rd} \)
(if appropriate, see Panel 4)

Determine drained bearing capacity \( q'_{Rd} \) (see Panel 4)

Verify \( q_{Ed} \leq q_{Rd} \) (if appropriate) and \( q_{Ed} \leq q'_{Rd} \)
Expressions for bearing resistance

\[
e = \frac{B}{2} - \left\{ \frac{\gamma_c \sum_i (W_{k,i} L_i) + (P_{ad} \sin \Omega \ L_{vp})}{V_d + (P_{ad} \sin \Omega)} - \left[ P_{ad} \cos \frac{h}{3} \right] \right\}
\]

\[
B' = B - 2e
\]

Undrained bearing resistance:

\[
q_{Rd} = c_{ud,fdn} (\pi + 2) i_c + q_d,
\]

\[
i_c = \frac{1}{2} \left\{ 1 + \sqrt{1 - \frac{P_{ad} \cos \Omega}{B' c_{ud,fdn}}} \right\}
\]

Drained bearing resistance:

\[
q'_{Rd} = q'_{q} N_q i_q + (\gamma_{d,fdn} - \gamma_w) \frac{B'}{2} N_q i_q + c' N_c i_c
\]

\[
i_q = \left\{ 1 - \frac{P_{ad} \cos \Omega}{\gamma_c \sum_i W_{k,i} + (P_{ad} \sin \Omega)} \right\}^{2}
\]

\[
i_q = i_q^{1.5}
\]

\[
i_c = i_q - (1 - i_q)/N_{ctan \varphi}
\]

Figure 11
Bearing capacity factors, \( N \), from ground properties

Values for \( N_c, N_y \) and \( N_q \) can be obtained from Figure 11
Overall design procedure

1. Start
2. Check overall stability of the site
3. Carry out initial sizing of the wall
4. Determine characteristic material properties (see Figure 6)
5. Calculate the characteristic value of actions (see Figure 6)
6. Calculate the design value of actions (see Figure 6)
7. Design against sliding (see Figure 7)
8. Design against toppling (see Figure 9)
9. Design against bearing failure (see Figure 10)
10. Structural design (see Figure 13)
11. Sufficient capacity?
   - Yes: Design complete
   - No, revise geometry
Structural design

Check the stem of the retaining wall for compaction pressures or at-rest conditions as appropriate:
- Maximum design moment = $M_s$ (see Panel 5 or 6)
- Maximum design shear force = $V_s$
- Refer to Chapter 3, originally published as *Slabs*\textsuperscript{11} for the design expressions to use to check bending, shear and deflection

Check the toe and heel of the wall in bending and shear using the expressions in Panel 5 to determine applied forces and the design expressions from Chapter 3 to check bending, shear and deflection
Remember: Load and Partial Factor Combinations

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Comb. 1</th>
<th>Comb. 2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Actions</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Permanent action: unfavourable</td>
<td>$Y_{G,unf}$</td>
<td>1.35</td>
<td>1.0</td>
</tr>
<tr>
<td>Permanent action: favourable</td>
<td>$Y_{G,fav}$</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Variable action</td>
<td>$Y_Q$</td>
<td>1.50</td>
<td>1.30</td>
</tr>
<tr>
<td><strong>Soil Properties</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Angle of shearing resistance</td>
<td>$Y_\phi$</td>
<td>1.0</td>
<td>1.25</td>
</tr>
<tr>
<td>Effective cohesion</td>
<td>$Y_{c'}$</td>
<td>1.0</td>
<td>1.25</td>
</tr>
<tr>
<td>Undrained shear strength</td>
<td>$Y_{cu}$</td>
<td>1.0</td>
<td>1.4</td>
</tr>
<tr>
<td>Unconfined strength</td>
<td>$Y_{qu}$</td>
<td>1.0</td>
<td>1.4</td>
</tr>
<tr>
<td>Bulk density</td>
<td>$Y_y$</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>
Flexural and axial resistance of piles

‘Uncertainties related to the cross-section of cast in place piles and concreting procedures shall be allowed for in design’

‘In the absence of other provisions’, the design diameter of cast in place piles without permanent casing is less than the nominal diameter $D_{nom}$:

- $D_d = D_{nom} - 20$ mm for $D_{nom} < 400$ mm
- $D_d = 0.95 \times D_{nom}$ for $400 \leq D_{nom} \leq 1000$ mm
- $D_d = D_{nom} - 50$ mm for $D_{nom} > 1000$ mm

ICE Specification for piling and embedded retaining walls (ICE SPERW)

B1.10.2 states ‘The dimensions of a constructed pile or wall element shall not be less than the specified dimensions’. A tolerance of 5% on auger diameter, casing diameter, and grab length and width is permissible.
Flexural and axial resistance of piles

- The partial factor for concrete, $\gamma_c$, should be multiplied by a factor, $k_f$, for calculation of design resistance of cast in place piles without permanent casing.
- The UK value of $k_f = 1.1$, therefore $\gamma_{c,pile} = 1.65$
- “If the width of the compression zone decreases in the direction of the extreme compression fibre, the value $\eta f_{cd}$ should be reduced by 10%”
Bored piles

Reinforcement should be detailed for free flow of concrete.

Minimum diameter of long. reinforcement  = 16mm
Minimum number of longitudinal bars = 6

[BUT - BS EN 1536 *Execution of special geotechnical work Bored Piles* says 12 mm and 4 bars!]

Minimum areas:

<table>
<thead>
<tr>
<th>Pile cross section: $A_c$</th>
<th>Min area of long. rebar, $A_{s,bpmin}$</th>
<th>Pile diameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_c \leq 0.5 , m^2$</td>
<td>$\geq 0.5% , A_c$</td>
<td>$&lt; 800 , mm$</td>
</tr>
<tr>
<td>$0.5 , m^2 &lt; A_c \leq 1.0 , m^2$</td>
<td>$\geq 2500 , mm^2$</td>
<td></td>
</tr>
<tr>
<td>$A_c &gt; 1.0 , m^2$</td>
<td>$\geq 0.25% , A_c$</td>
<td>$&gt;1130 , mm$</td>
</tr>
</tbody>
</table>
Minimum reinforcement

Minimum area of reinforcement, $A_{s,\text{bpmin}}$ (mm$^2$) vs. Pile diameter, mm.
Workshop

Design a pad foundation for a 300mm square column taking

\[ G_k = 600\text{kN}, \ Q_k = 350\text{kN}. \]

Permissible bearing stress = 225kPa.

Concrete for base C30/37.

Work out

• size of base,
• tension reinforcement and
• any shear reinforcement.
Workshop Problem

Category 2, using prescriptive methods

Base size: \((G_k + Q_k)/\text{bearing stress} = \ldots\)

\[= \ldots \text{m}^2\]

\[\Rightarrow \ldots \times \ldots \text{base} \times \ldots \text{mm deep (choose size of pad)}\]

Use C 30/37 (concrete)

Loading = \(\gamma_g \times G_k + \gamma_q \times Q_k = \ldots\) = \ldots kN

ULS bearing pressure = \(\ldots/\ldots^2\) = \ldots kN/m²

Critical section at face of column

\(M_{Ed} = \ldots \times \ldots \times \ldots^2 / 2 = \ldots\) kNm

\(d = \ldots - \text{cover} - \text{assumed } \varnothing = \ldots = \ldots\) mm

\(K = M/\varnothing^2 f_{ck} = \ldots\)
Workshop Problem

\[ z = \text{_____} \cdot d = \text{_____} \times \text{_____} = \text{_____} \text{mm} \]

\[ A_s = \frac{M_{Ed}}{f_{yd}} \cdot z = \text{_____} \text{mm}^2 \]

- Provide H\____@______ c/c (\________\text{mm}^2\)

Check minimum steel

\[ 100\frac{A_{s,prov}}{bd} = \text{______} \]

For C30/37 concrete \( A_{s,\text{min}} = \text{____} \) ∴ OK/not OK

Beam shear

Check critical section \( d \) away from column face

\[ V_{Ed} = \text{_____} \times \text{_______} = \text{_______} \text{kN/m} \]

\[ \nu_{Ed} = \frac{V_{Ed}}{d} = \text{_______} \text{MPa} \]

\[ \rho = \frac{\text{_____}}{(\text{______} \times \text{_______})} = \text{_____} = \text{_____} \% \]

\[ \nu_{Rd,c} \text{ (from table)} = \text{_____} \text{MPa} \quad \therefore \text{beam shear OK/not OK}. \]
Workshop Problem

Punching shear

Basic control perimeter at 2d from face of column

\[ v_{Ed} = \beta V_{Ed} / u_i d < v_{Rd,c} \]

\[ \beta = 1, \quad u_i = \quad = \quad \text{mm} \]

\[ V_{Ed} = \text{load minus net upward force within the area of the control perimeter} \]

\[ = \quad - \quad \times (\quad) \]

\[ = \quad \text{kN} \]

\[ v_{Ed} = \quad \text{MPa}; \quad v_{Rd,c} = \quad \text{(as before)} \quad => \quad \text{ok/not ok} \]
End of Lecture 10
(and the course!)

Emails:
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cgoodchild@concretecentre.com
jburridge@concretecentre.com
Model answer: Workshop Problem

Category 2, using prescriptive methods
Base size: \( \frac{(600 + 350)}{225} = 4.22 \text{m}^2 \)
\[ \Rightarrow 2.1 \times 2.1 \text{ base} \times 450 \text{mm (say) deep} \]
Use C30/37
Loading = \( 1.35 \times 600 + 1.5 \times 350 = 1335 \text{kN} \)
ULS bearing pressure = \( \frac{1335}{2.1^2} = 303 \text{kN/m}^2 \)
Critical section at face of column
\[ M_{Ed} = 303 \times 2.1 \times (1.05-0.15)^2 / 2 = 258 \text{kNm} \]
\[ d = 450 - 50 - 16 = 384 \text{mm} \]
\[ K = 258 \times 10^6 / (2100 \times 384^2 \times 30) = 0.028 \]
Model answer: 
Workshop Problem

\[ z = 0.95d = 0.95 \times 384 = 365\text{mm} \]
\[ A_s = \frac{M_{Ed}}{f_{yd}z} = \frac{258 \times 10^6}{(435 \times 365)} = 1626\text{mm}^2 \]

\[ \text{Provide H16 @ 250 c/c (1688mm}^2) \text{ (804 mm}^2/\text{m}) \]

Check minimum steel

\[ 100A_{s,prov}/bd = 100 \times 1688 / 2100 / 384 = 0.209 \]

For C30/37 concrete \( A_{s,\text{min}} = 0.151 \) \( \therefore \) OK

Beam shear

Check critical section \( d \) away from column face

\[ V_{Ed} = 303 \times (0.9 - 0.384) = 156\text{kN/m} \]
\[ \nu_{Ed} = 156 / 384 = 0.41\text{MPa} \]
\[ \nu_{Rd,c} (\text{from table}) = 0.44\text{MPa} \implies \text{beam shear ok.} \]
Model answer: Workshop Problem

Punching shear

Basic control perimeter at 2d from face of column

\[ v_{Ed} = \beta V_{Ed} / u_i d < v_{Rd,c} \]

\( \beta = 1, \ u_i = (300 \times 4 + 384 \times 2 \times 2 \times \pi) = 6025\text{mm} \)

\[ V_{Ed} = \text{load minus net upward force within the area of the control perimeter) } \]
\[ = 1335 - 303 \times (0.3^2 + \pi \times .768^2 + .768 \times .3 \times 4) \]
\[ = 467\text{kN} \]

\[ v_{Ed} = 467 \times 10^3 / (6025 \times 384) = 0.202\text{MPa}; \]

\[ v_{Rd,c} = 0.44 \text{ (as before)} \Rightarrow \text{ok} \]