EN 1997-1  Eurocode 7

Section 8 – Anchorages
Section 9 – Retaining structures

Brian Simpson
Arup Geotechnics
EN 1997-1
Geotechnical design – General Rules

1 General
2 Basis of geotechnical design
3 Geotechnical data
4 Supervision of construction, monitoring and maintenance
5 Fill, dewatering, ground improvement and reinforcement
6 Spread foundations
7 Pile foundations
8 Anchorages
9 Retaining structures
10 Hydraulic failure
11 Overall stability
12 Embankments

Appendices A to J
8 Anchorages

8.1 General
8.2 Limit states
8.3 Design situations and actions
8.4 Design and construction considerations
8.5 Ultimate limit state design
8.6 Serviceability limit state design
8.7 Suitability tests
8.8 Acceptance tests
8.9 Supervision and monitoring
8 Anchorages

8.1 General

8.1.1 Scope

(1)P This Section applies to the design of temporary and permanent anchorages used:

— to support a retaining structure;

— to provide the stability of slopes, cuts or tunnels;

— to resist uplift forces on structures.

by transmitting a tensile force to a load bearing formation of soil or rock.

(2)P This Section is applicable to;

— pre-stressed anchorages consisting of an anchor head, a tendon free length and a tendon bond length bonded to the ground by grout;

— non pre-stressed anchorages consisting of an anchor head, a tendon free length and a restraint such as a fixed anchor length bonded to the ground by grout, a deadman anchorage, a screw anchor or a rock bolt.

(3) This Section should not be applied to soil nails.

(4)P Section 7 shall apply to the design of anchorages comprising tension piles.
8.1.2 Definitions

8.1.2.1 permanent anchorage
anchorage with a design life of more than two years

NOTE definition taken from EN 1537:1999

8.1.2.2 temporary anchorage
anchorage with a design life of less than two years

NOTE definition taken from EN 1537:1999

8.1.2.3 acceptance test
load test on site to confirm that each anchorage meets the design requirements

8.1.2.4 suitability test
load test on site to confirm that a particular anchor design will be adequate in particular ground conditions

NOTE definition taken from EN 1537:1999

8.1.2.5 investigation test
load test to establish the ultimate resistance of an anchor at the grout/ground interface and to determine the characteristics of the anchorage in the working load range
8.2 Limit states

(1)P The following limit states shall be considered for anchorages, both individually and in combination:

— structural failure of the tendon or anchor head, caused by the applied stresses;
— distortion or corrosion of the anchor head;
— for grouted anchors, failure at the interface between the body of grout and the ground;
— for grouted anchors, failure of the bond between the steel tendon and the grout;
— for deadman anchorages, failure by insufficient resistance of the deadman;
— loss of anchorage force by excessive displacements of the anchor head or by creep and relaxation;
— failure or excessive deformation of parts of the structure due to the applied anchorage force;
— loss of overall stability of the retained ground and the retaining structure;
— interaction of groups of anchorages with the ground and adjoining structures.
8.4 Design and construction considerations

(13)P Corrosion protection of pre-stressed anchorages shall comply with 6.9 of EN 1537:1999.

8.5.2 Design values of pull-out resistance determined from the results of tests

(1)P The design value of the pull-out resistance shall be derived from the characteristic value using the equation:

\[ R_{a,d} = R_{a,k} / \gamma_a \]  \hspace{1cm} (8.2)

NOTE    The partial factor, \( \gamma_a \), takes into account unfavourable deviations of the anchorage force transmitted into the ground, either in terms of fixed length or of a deadman structure.

(2)P The partial factors \( \gamma_a \) defined in A.3.3.4(1)P shall be used in equation (8.2).

NOTE    The value of the partial factor may be set by the National annex. The recommended values for persistent and transient situations are given in Table A.12.

(3) The characteristic value should be related to the suitability test results by applying a correlation factor \( \tilde{\gamma}_a \).

NOTE    8.5.2(3) refers to those types of anchorage that are not individually checked by acceptance tests. If a correlation factor \( \tilde{\gamma}_a \) is used, it must be based on experience or provided for in the National annex.

8.5.3 Design values of pull-out resistance determined by calculations

(1)P The design value of pull-out resistance shall be assessed according to the principles in 2.4.7 and 2.4.8, where appropriate.
8.5.4 Design value of the structural resistance of the anchorage

(1) P The structural design of the anchorage shall satisfy the following inequality:

\[ R_{a;\text{d}} \leq R_{t;\text{d}} \]  

(8.3)

(2) P The material resistance of the anchorages, \( R_{t;\text{d}} \), shall be calculated according to EN 1992, EN 1993 and EN 1537:1999, as relevant.

(3) P If anchors are submitted to suitability tests, \( R_{t;\text{d}} \) shall take account of the proof load (see 9.5 of EN 1537:1999).

8.5.5 Design value of the anchorage load

(1) P The design value of the anchorage load, \( P_{\text{d}} \), shall be derived from the design of the retained structure as the maximum value of

— the ultimate limit state force applied by the retained structure, and if relevant

— the serviceability limit state force applied by the retained structure.
8.7 Suitability tests

8.8 Acceptance tests

(1) It shall be specified in the design that all grouted anchorages shall be subjected to acceptance tests prior to lock-off and before they become operational.

(2) The procedure for acceptance tests shall follow the rules given in EN 1537:1999 for grouted anchorages.

(3) Where groups of anchorages are crossing with tendon bond lengths at spacings of less than 1.5 m, random control tests should be made after completion of the lock-off action.

8.9 Supervision and monitoring

(1) Supervision and monitoring shall follow the rules given in Section 4 of this standard and 9.10 and 9.11 of EN 1537:1999, where appropriate.
8 Anchorages

- Section depends on EN1537 - Execution of special geotechnical work - Ground anchors
- Not fully compatible with EN1537. Further work on this is underway.
- BS8081 being retained for the time being.
EN1537:1999

Execution of special geotechnical work — Ground anchors

EN1997-1: Anchorages and Retaining structures
EN1537:1999
Execution of special geotechnical work - Ground anchors

1 Scope
2 Normative references
3 Terms, definitions and symbols
4 Specific needs
5 Site investigation
6 Materials and products
7 Design considerations
8 Execution
9 Testing, supervision and monitoring
10 Records
11 Special requirements

Annex A Electrical testing of corrosion protection
Annex B Investigation testing of corrosion protection
Annex C Guidelines for acceptance criteria of viscous corrosion protection compounds and examples of standards for the testing of material properties
Annex D Design of ground anchors
Annex E Examples of anchor testing methods
Annex F Examples of record sheets
Annex E
(informative)

Examples of anchor testing methods

E.1 General

In clause 9 reference was made to the three classes of test commonly adopted in connection with anchors. These are:

a) **Test Method 1**: The anchor is loaded in incremental cycles from a datum load to a maximum test load. Displacement of the anchor head is measured over a time period at the maximum load in each incremental cycle;

b) **Test Method 2**: The anchor is loaded in incremental cycles from a datum load to a maximum test load or to failure. The loss of load at the anchor head is measured over a period of time at the lock-off load and at the maximum load in each incremental cycle;

c) **Test Method 3**: The anchor is loaded in incremental steps from a datum load to a maximum test load. The displacement of the anchor head is measured under maintained load at each loading step.

The essential loading procedures for Test Methods 1, 2 and 3 are shown in Figures E.1, E.2 and E.3.
### Use of partial factors in anchor design and use

<table>
<thead>
<tr>
<th>Source of action or force</th>
<th>Magnitude of force</th>
</tr>
</thead>
<tbody>
<tr>
<td>From wall analysis</td>
<td></td>
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<tr>
<td>SLS design force</td>
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<tr>
<td>DA1.1 ULS design force</td>
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<tr>
<td>DA1.2 ULS design force</td>
<td>♦</td>
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<tr>
<td>ULS design force (greater of Comb 1 and 2)</td>
<td>♦</td>
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<tr>
<td>For anchor design and assessment testing</td>
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<tr>
<td>Required minimum ULS design resistance</td>
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<tr>
<td>Required minimum characteristic resistance</td>
<td>♦</td>
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<tr>
<td>Second check on characteristic resistance</td>
<td>♦</td>
</tr>
<tr>
<td>Required minimum assessment test result</td>
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<tr>
<td>For anchor use</td>
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<tr>
<td>Typical lock-off load</td>
<td>♦♦</td>
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<tr>
<td>Typical preload in acceptance test</td>
<td>♦♦♦♦</td>
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<td>Greater of these two</td>
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<tr>
<td>Design of wall structure</td>
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<tr>
<td>anchors subject to acceptance tests only</td>
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<tr>
<td>Characteristic action for structural design</td>
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<tr>
<td>ULS design action - short term</td>
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<td>Design of wall structure</td>
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<tr>
<td>Design of wall structure</td>
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<td>long term working state</td>
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<tr>
<td>ULS design action - long term</td>
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<tr>
<td>Check for major increase in load with time</td>
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</tr>
</tbody>
</table>

♦ Value of anchor force at indicated stage in the process.

a) SLS force may exceed ULS force in some cases.
b) EN1997-1 and the UK National Annex give γ = 1.1. EN1337-1 gives γ = 1.35
c) This value is not provided by EN1997-1 or the UK National Annex. “If a correlation factor 5. is used, it must be based on experience or provided for in the National annex.”
d) EN1337-1 5.2.3 requires minimum of 1.25.
e) EN1337-1 5.2.1 gives 0.9 to 1.25
f) Including bearing plates, washers and connections. EN1337 contains information relevant to design of tendons.
g) Load factor for variable actions. Reduced factors may be considered for short duration loading.
h) Load factor for permanent actions.
### Partial factors in anchor design

**Use of partial factors in anchor design and use**

<table>
<thead>
<tr>
<th>Source of action or force</th>
<th>➔➔➔➔➔➔ Magnitude of force ➔➔➔➔➔➔</th>
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</tr>
<tr>
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</tr>
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</tbody>
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**EN1997-1: Anchorages and Retaining structures**

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EN 1997-1  Eurocode 7

Section 8 – Anchorages
Section 9 – Retaining structures

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Section 9 – Retaining structures

Fundamentals – Design Approaches

Main points in the code text

Examples:
Comparisons with previous (UK) practice
Comparison between Design Approaches

Lessons from the Dublin Workshop
EN 1997-1  Eurocode 7

Section 9 – Retaining structures

Fundamentals – Design Approaches

Main points in the code text

Examples:
Comparisons with previous (UK) practice
Comparison between Design Approaches

Lessons from the Dublin Workshop
FOS > 1 for characteristic soil strengths

- but not big enough
The slope and retaining wall are all part of the same problem.

Structure and soil must be designed together - consistently.
Approaches to ULS design –
The merits of Design Approach 1 in Eurocode 7
Brian Simpson
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EN 1997-1  Eurocode 7

Section 9 – Retaining structures

Fundamentals – Design Approaches

Main points in the code text

Examples:
- Comparisons with previous (UK) practice
- Comparison between Design Approaches

Lessons from the Dublin Workshop
9 Retaining structures

9.1 General
9.2 Limit states
9.3 Actions, geometrical data and design situations
9.4 Design and construction considerations
9.5 Determination of earth pressures
9.6 Water pressures
9.7 Ultimate limit state design
9.8 Serviceability limit state design
9.2 Limit states

(1) A list shall be compiled of limit states to be considered. As a minimum the following limit states shall be considered for all types of retaining structure:

— loss of overall stability;

— failure of a structural element such as a wall, anchorage, wale or strut or failure of the connection between such elements;

— combined failure in the ground and in the structural element;

— failure by hydraulic heave and piping;

— movement of the retaining structure, which may cause collapse or affect the appearance or efficient use of the structure or nearby structures or services, which rely on it;

— unacceptable leakage through or beneath the wall;

— unacceptable transport of soil particles through or beneath the wall;

— unacceptable change in the ground-water regime.
9.2 Limit states

(2) In addition, the following limit states shall be considered for gravity walls and for composite retaining structures:

— bearing resistance failure of the soil below the base;

— failure by sliding at the base;

— failure by toppling;

and for embedded walls:

— failure by rotation or translation of the wall or parts thereof;

— failure by lack of vertical equilibrium.

(3) For all types of retaining structure, combinations of the above mentioned limit states shall be taken into account, if relevant.
9.3.2 Geometrical data

9.3.2.1 Basic data

(1)P Design values for geometrical data shall be derived in accordance with the principles stated in 2.4.6.3.

9.3.2.2 Ground surfaces

(1)P Design values for the geometry of the retained material shall take account of the variation in the actual field values. The design values shall also take account of anticipated excavation or possible scour in front of the retaining structure.

(2) In ultimate limit state calculations in which the stability of a retaining wall depends on the ground resistance in front of the structure, the level of the resisting soil should be lowered below the nominally expected level by an amount $\Delta a$. The value of $\Delta a$ should be selected taking into account the degree of site control over the level of the surface. With a normal degree of control, the following should be applied:

— for a cantilever wall, $\Delta a$ should equal 10% of the wall height above excavation level, limited to a maximum of 0.5 m;

— for a supported wall, $\Delta a$ should equal 10% of the distance between the lowest support and the excavation level, limited to a maximum of 0.5 m.
9.3.2 Geometrical data

(2) In ultimate limit state calculations in which the stability of a retaining wall depends on the ground resistance in front of the structure, the level of the resisting soil should be lowered below the nominally expected level by an amount $\Delta a$. The value of $\Delta a$ should be selected taking into account the degree of site control over the level of the surface. With a normal degree of control, the following should be applied:

- for a cantilever wall, $\Delta a$ should equal 10% of the wall height above excavation level, limited to a maximum of 0.5 m;

- for a supported wall, $\Delta a$ should equal 10% of the distance between the lowest support and the excavation level, limited to a maximum of 0.5 m.

(3) Smaller values of $\Delta a$, including 0, may be used when the surface level is specified to be controlled reliably throughout the appropriate execution period.

(4) Larger values of $\Delta a$ should be used where the surface level is particularly uncertain.
9.4 Design and construction considerations

9.4.1 General

(1) Both ultimate and serviceability limit states shall be considered using the procedures described in 2.4.7 and 2.4.8.

(2) It shall be demonstrated that vertical equilibrium can be achieved for the assumed pressure distributions and actions on the wall.

(3) The verification of vertical equilibrium may be achieved by reducing the wall friction parameters.

(4) As far as possible, retaining walls should be designed in such a way that there are visible signs of the approach of an ultimate limit state. The design should guard against the occurrence of brittle failure, e.g. sudden collapse without conspicuous preliminary deformations.

(5) For many earth retaining structures, a critical limit state should be considered to occur if the wall has displaced enough to cause damage to nearby structures or services. Although collapse of the wall may not be imminent, the degree of damage may considerably exceed a serviceability limit state in the supported structure.

(6) The design methods and partial factor values recommended by this standard are usually sufficient to prevent the occurrence of ultimate limit states in nearby structures, provided that the soils involved are of at least medium density or firm consistency and that adequate construction methods and sequences are adopted. Special care should be taken, however, with some highly over-consolidated clay deposits in which large at rest horizontal stresses may induce substantial movements in a wide area around excavations.
(8)P The design of retaining structures shall take account of the following items, where appropriate:

- the practicability of constructing the wall to reach a stratum of low permeability, so forming a water cut-off. The resulting equilibrium ground-water flow problem shall be assessed;

- the practicability of forming ground anchorages in adjacent ground;

- the practicability of excavating between any propping of retaining walls;

- the ability of the wall to carry vertical load;

- the ductility of structural components;

- access for maintenance of the wall and any associated drainage measures;

- the appearance and durability of the wall and any anchorages;

- for sheet piling, the need for a section stiff enough to be driven to the design penetration without loss of interlock;

- the stability of borings or slurry trench panels while they are open;

- for fill, the nature of materials available and the means used to compact them adjacent to the wall, in accordance with 5.3.
9.4.2 Drainage systems

(1) If the safety and serviceability of the designed structure depend on the successful performance of a drainage system, the consequences of its failure shall be considered, having regard for both safety and cost of repair. One of the following conditions (or a combination of them) shall apply:

— a maintenance programme for the drainage system shall be specified and the design shall allow access for this purpose;

— it shall be demonstrated both by comparable experience and by assessment of any water discharge, that the drainage system will operate adequately without maintenance.

(2) The quantities, pressures and eventual chemical content of any water discharge should be taken into account.
9.5 Determination of earth pressures

9.5.1 General

(1) Determination of earth pressures shall take account of the acceptable mode and amount of any movement and strain, which may occur at the limit state under consideration.

(2) In the following context the words "earth pressure" should also be used for the total earth pressure from soft and weathered rocks and should include the pressure of ground-water.
9.5 Determination of earth pressures

(4) The amount of mobilised wall friction and adhesion should be considered as a function of:

— the strength parameters of the ground;

— the friction properties of the wall-ground interface;

— the direction and amount of movement of the wall relative to the ground;

— the ability of the wall to support any vertical forces resulting from wall friction and adhesion.

(5) The amount of shear stress, which can be mobilised at the wall-ground interface should be determined by the wall-ground interface parameter $\delta$.

(6) A concrete wall or steel sheet pile wall supporting sand or gravel may be assumed to have a design wall ground interface parameter $\delta_d = k \varphi_{cv_d}$. $k$ should not exceed 2/3 for precast concrete or steel sheet piling.

(7) For concrete cast against soil, a value of $k = 1.0$ may be assumed.

(8) For a steel sheet pile in clay under undrained conditions immediately after driving, no adhesive or frictional resistance should be assumed. Increases in these values may take place over a period of time.
9.5.3 Limiting values of earth pressure

1) Limiting values of earth pressures shall be determined taking account of the relative movement of the soil and the wall at failure and the corresponding shape of the failure surface.

2) Limiting values of earth pressure assuming straight failure surfaces can significantly deviate from the values assuming curved failure surfaces for high angles of internal friction and wall-ground interface parameters $\phi$, and so lead to unsafe results.

NOTE Annex C provides some data of relative movements that cause limiting values of earth pressures.

3) In cases where struts, anchorages or similar elements impose restraints on movement of the retaining structure, it should be considered that the limiting active and passive values of earth pressure, and their distributions, may not be the most adverse ones.

Annex C also provides charts and formulae for the active and passive limit values of earth pressure.
Annex C  Sample procedures to determine limit values of earth pressures on vertical walls

- Based on Caquot and Kerisel (and Absi?).
- No values for adverse wall friction, which can lead to larger $K_a$ and much smaller $K_p$. 
Adverse wall friction may be caused by loads on the wall from structures above, inclined ground anchors, etc.
C.2 Numerical procedure for obtaining passive pressures

- Also provides Ka
- Programmable formulae (though not simple)
- Incorporation in some software (e.g., Oasys FREW, STAWAL)
- Precise source not known (to me), but same values as
- Covers range of adverse wall friction.
- Slightly more conservative than Caquot & Kerisel when $\phi$ and $\delta/\phi$ large – but more correct?
Ka, Kp charts in Simpson & Driscoll

Figure CG.1 Coefficient of active earth pressure, $K_a$, based on EC7 Equation G12

Figure CG.2 Coefficient of passive earth pressure, $K_p$, based on EC7 Equation G12
Comparison with Caquot & Kerisel

Figure CG.3  Ratio between coefficients of active earth pressure from Kerisel and Absi and from Equation G12

Figure CG.4  Ratio between coefficients of passive earth pressure from Kerisel and Absi and from Equation G12
9.7 Ultimate limit state design

9.7.1 General

(3)P Calculations for ultimate limit states shall establish that equilibrium can be achieved using the design actions or effects of actions and the design strengths or resistances, as specified in 2.4. Compatibility of deformations shall be considered in assessing design strengths or resistances.

(5) Calculation methods may be used, which redistribute earth pressure in accordance with the relative displacements and stiffnesses of ground and structural elements.
9.7.2 Overall stability

(1) P The principles in Section 11 shall be used as appropriate to demonstrate that an overall stability failure will not occur and that the corresponding deformations are sufficiently small.

(2) As a minimum, limit modes of the types illustrated in figure 9.1 should be considered, taking progressive failure and liquefaction into account as relevant.

Figure 9.1 — Examples of limit modes for overall stability of retaining structures.
9.7.3 Foundation failure of gravity walls

(1) The principles of Section 6 shall be used as appropriate to demonstrate that a foundation failure is sufficiently remote and that deformations will be acceptable. Both bearing resistance and sliding shall be considered.

(2) As a minimum, limit modes of the types illustrated in Figure 9.2 should be considered.

Figure 9.2 — Examples of limit modes for foundation failures of gravity walls
9.7.4 Rotational failure of embedded walls

(1) It shall be demonstrated by equilibrium calculations that embedded walls have sufficient penetration into the ground to prevent rotational failure.

Figure 9.3 — Examples of limit modes for rotational failures of embedded walls
9.7.5 Vertical failure of embedded walls

(1) It shall be demonstrated that vertical equilibrium can be achieved using the design soil strengths or resistances and design vertical forces on the wall.

(4) The design magnitude and direction of shear stress between the soil and the wall shall be consistent with the check for vertical and rotational equilibrium.

Figure 9.4 — Example of a limit mode for vertical failure of embedded walls
9.7.6 Structural design of retaining structures

(1) Retaining structures, including their supporting structural elements such as anchorages and props, shall be verified against structural failure in accordance with 2.4 and EN 1992, EN 1993, EN 1995 and EN 1996.

(2) As a minimum, limit modes of the types illustrated in Figure 9.5 should be considered.

Figure 9.5 — Examples of limit modes for structural failure of retaining structures
9.7.6 Structural design of retaining structures

(1) Retaining structures, including their supporting structural elements such as anchorages and props, shall be verified against structural failure in accordance with 2.4 and EN 1992, EN 1993, EN 1995 and EN 1996.

(2) As a minimum, limit modes of the types illustrated in Figure 9.5 should be considered.

(3) For each ultimate limit state, it shall be demonstrated that the required strengths can be mobilised, with compatible deformations in the ground and the structure.

(4) In structural elements, reduction in strength with deformation due to effects such as cracking of unreinforced sections, large rotations at plastic hinges or local buckling of steel sections should be considered in accordance with EN 1992 to EN 1996 and EN 1999.
9.7.7 Failure by pull-out of anchorages

(1) It shall be demonstrated that equilibrium can be achieved without pull-out failure of ground anchorages.

Figure 9.6 — Examples of limit modes for failure by pull-out of anchors.

(2) Anchors shall be designed in accordance with Section 8.

(3) As a minimum, limit modes of the types illustrated in Figure 9.6 (a, b) should be considered.

(4) For deadman anchors, the failure mode illustrated in Figure 9.6 (c) should also be considered.
9.8 Serviceability limit state design

9.8.1 General

(1)P The design of retaining structures shall be checked at the serviceability limit state using the appropriate design situations as specified in 9.3.3.

(2)P Design values of earth pressures for the serviceability limit state shall be derived using characteristic values of all soil parameters.

(5) The design values of earth pressures should be derived taking account of the allowable deformation of the structure at its serviceability limit state. These pressures may not necessarily be limiting values.
9.8.2 Displacements

(1) P Limiting values for the allowable displacements of walls and the ground adjacent to them shall be established in accordance with 2.4.8, taking into account the tolerance to

(2) P A cautious estimate of the distortion and displacement of retaining walls, and the effects on supported structures and services, shall always be made on the basis of comparable experience. This estimate shall include the effects of construction of the wall. The design may be justified by checking that the estimated displacements do not exceed the limiting values.

(3) P If the initial cautious estimate of displacement exceeds the limiting values, the design shall be justified by a more detailed investigation including displacement calculations.

(4) P It shall be considered to what extent variable actions, such as vibrations caused by traffic loads behind the retaining wall, contribute to the wall displacement.

(5) P A more detailed investigation, including displacement calculations, shall be undertaken in the following situations:

— where nearby structures and services are unusually sensitive to displacement;

— where comparable experience is not well established.

(8) The behaviour of materials assumed in displacement calculations should be calibrated by comparable experience with the same calculation model. If linear behaviour is assumed, the stiffnesses adopted for the ground and structural materials should be appropriate for the degree of deformation computed. Alternatively, complete stress-strain models of the materials may be adopted.
EN 1997-1 Eurocode 7

Section 9 – Retaining structures

Fundamentals – Design Approaches

Main points in the code text

Examples:
  Comparisons with previous (UK) practice
  Comparison between Design Approaches

Lessons from the Dublin Workshop
8m propped wall

$K_0 = 2.0$

$1.5$

$1.0$

Clay $\phi'_k = 24^\circ$
$c'_k = 0$
$\gamma_k = 20 \text{ kN} / \text{m}^3$
CASE:

Unplanned overdig (m) | DA1-1 | DA1-2 | EC7 SLS
---|---|---|---
0.5 | 0.5 | 0

Dig level: Stage 1

Stage 2

Characteristic $\phi' (\ )$ | 24 | 24 | 24

$\gamma$ (or M) on tan $\phi'$ | 1 | 1.25 | 1

Design $\phi'$ | 24 | 19.6 | 24

$\delta'/\phi'$ active | 1 | 1 | 1

$\delta'/\phi'$ passive | 1 | 1 | 1

$K_a$ | 0.34 | 0.42 | 0.34

Factor on $K_a$ | 1 | 1 | 1

Design $K_a$ | 0.34 | 0.42 | 0.34

$K_p$ | 4.0 | 2.9 | 4.0

Factor on $K_p$ | 1 | 1 | 1

Design $K_p$ Excd. side Retd. side | 4.0 | 2.9 | 4.0 1.0

$\gamma_Q$ | 1 | 1.3 | 1
## 8m propped wall - length and BM

###TABLE 1 - Results

<table>
<thead>
<tr>
<th>CASE:</th>
<th>DA1 -1</th>
<th>DA1 -2</th>
<th>EC7 SLS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unplanned overdig (m)</td>
<td>0.5</td>
<td>0.5</td>
<td>0</td>
</tr>
<tr>
<td>Design $\phi'$</td>
<td>24</td>
<td>19.6</td>
<td>24</td>
</tr>
<tr>
<td>Design $K_a$</td>
<td>0.34</td>
<td>0.42</td>
<td>0.34</td>
</tr>
<tr>
<td>Design $K_p$</td>
<td>Excd. side</td>
<td>Retd. side</td>
<td></td>
</tr>
<tr>
<td>4.0</td>
<td>2.9</td>
<td>4.0</td>
<td>1.0</td>
</tr>
<tr>
<td>$\gamma_Q$</td>
<td>1</td>
<td>1.3</td>
<td>1</td>
</tr>
<tr>
<td>Computer program</td>
<td>STW</td>
<td>STW</td>
<td>F</td>
</tr>
<tr>
<td>Data file</td>
<td>PROP11</td>
<td>PROP1</td>
<td>BCAP3A</td>
</tr>
<tr>
<td>Wall length (m)</td>
<td>15.1</td>
<td><strong>17.9</strong></td>
<td>17.8</td>
</tr>
<tr>
<td>Max bending moment (kNm/m)</td>
<td>1097</td>
<td>1519</td>
<td>-236</td>
</tr>
<tr>
<td>Factor on bending moment</td>
<td>1.35</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>ULS design bending moment (kNm/m)</td>
<td>1481</td>
<td>1519</td>
<td>-236</td>
</tr>
</tbody>
</table>

* Computed  ** Assumed
Redistribution of earth pressure
Compare CIRIA 104

Bending moment (kNm/m)

-1000  0  1000  2000

5  0  -5  -10  -15  -20  -25

Elevation (m)

506 kN/m

M_{ULS} = 1870 kNm/m

287 kN/m

M_{ULS} = 776 \times 1.5 = 1164 kNm/m

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$10\text{kPa (13kPa)}$

$\phi' = 24^\circ$ ($19.6^\circ$)

$0$ m

$-8$ m ($-8.5$ m)
EN1997-1: Anchorages and Retaining structures
### 8m propped wall - length and BM

<table>
<thead>
<tr>
<th>CASE:</th>
<th>CIRIA Fs</th>
<th>CIRIA Fs</th>
<th>BS 8002</th>
<th>DA1 -1</th>
<th>DA1 -2</th>
<th>EC7 SLS</th>
<th>DA1 -1</th>
<th>DA1 -2</th>
<th>DA1 -2</th>
<th>DA1 -2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unplanned overdig (m)</td>
<td>0</td>
<td>0</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Design φ'</td>
<td>16.5</td>
<td>24</td>
<td>20.4</td>
<td>24</td>
<td>19.6</td>
<td>24</td>
<td>24</td>
<td>19.6</td>
<td>19.6</td>
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<tr>
<td>Design Kₐ</td>
<td>0.49</td>
<td>0.36</td>
<td>0.41</td>
<td>0.34</td>
<td>0.42</td>
<td>0.34</td>
<td>0.34</td>
<td>0.42</td>
<td>0.42</td>
<td>0.42</td>
</tr>
<tr>
<td>Design Kₚ Excd. side</td>
<td>2.1</td>
<td>3.4</td>
<td>2.8</td>
<td>4.0</td>
<td>2.9</td>
<td>4.0</td>
<td>1.0</td>
<td>2.9</td>
<td>1.0</td>
<td>1.0</td>
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<tr>
<td>Retd. side</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>γ₀</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1.3</td>
<td>1</td>
<td>1</td>
<td>1.3</td>
<td>1.3</td>
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<td>STW</td>
<td>STW</td>
<td>STW</td>
<td>FREW</td>
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<td>PROP4</td>
<td>PROP5</td>
<td>PR1B-03</td>
<td>PROP11</td>
<td>PROP1</td>
<td>BCAP3A</td>
<td>BCAPBA</td>
<td>BCAP1A</td>
<td>BCAP4A</td>
<td>XBCAP5</td>
</tr>
<tr>
<td>Wall length (m)</td>
<td>20.4</td>
<td>14.1</td>
<td>17.9</td>
<td>15.1</td>
<td>17.9</td>
<td>17.8</td>
<td>17.8</td>
<td>17.8</td>
<td>17.8</td>
<td>17.8</td>
</tr>
<tr>
<td>Max bending moment (kNm/m)</td>
<td>1870</td>
<td>776</td>
<td>1488</td>
<td>1097</td>
<td>1519</td>
<td>-236</td>
<td>-241</td>
<td>1359</td>
<td>-308</td>
<td>-229</td>
</tr>
<tr>
<td>Factor on bending moment</td>
<td>1.5</td>
<td>1.0?</td>
<td>1.35</td>
<td>1</td>
<td>1</td>
<td>1.35</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>ULS design bending moment (kNm/m)</td>
<td>1164</td>
<td>1488?</td>
<td>1481</td>
<td>1519</td>
<td>-236</td>
<td>-325</td>
<td>1359</td>
<td>-308</td>
<td>-229</td>
<td></td>
</tr>
</tbody>
</table>

* Computed    ** Assumed    ## Not used in design
8m excavation - comparison of methods
Redistribution of earth pressure
German practice for sheet pile design - EAB (1996)

Case 1
\[ a \leq 0.1 \cdot H \]
\[ \frac{e_{ah}}{e_{ahu}} \geq 1.0 \]

Case 2
\[ 0.1 \cdot H < a \leq 0.2 \cdot H \]
\[ \frac{e_{ah}}{e_{ahu}} \geq 1.2 \]

Case 3
\[ 0.2 \cdot H < a \leq 0.3 \cdot H \]
\[ \frac{e_{ah}}{e_{ahu}} \geq 1.5 \]


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Grundbau in STAWAL

**Actual Pressures**

- Water Pressure
- Moment
- Shear

**Scale**

- Reduced Level [m]: -14.00, -12.00, -10.00, -8.00, -6.00, -4.00, -2.00, 0.0
- Pressures [kPa]: -240.0, -160.0, -80.0, 0.0, 80.0, 160.0, 240.0
- Shear Force [kN/m]: -240.0, -160.0, -80.0, 0.0, 80.0, 160.0, 240.0

**Bending Moment [kNm/m]**

- 199.3 kN/m

**Reduced Levels**

- 10.59m

**Graph Details**

- Water Pressure
- Moment
- Shear

**Legend**

- Shear
- Moment
- Water Pressure
- Actual Pressures
EN 1997-1  Eurocode 7

Section 9 – Retaining structures

Fundamentals – Design Approaches

Main points in the code text

Examples:
  Comparisons with previous (UK) practice
  Comparison between Design Approaches

Lessons from the Dublin Workshop
Eurocode 7 Workshop
Dublin, 31 March to 1 April 2005

Organised by
European Technical Committee 10
Technical Committee 23 of ISSMGE
GeoTechNet Working Party 2

Retaining Wall Examples 5 to 7
Example 5 – Cantilever Gravity Retaining Wall

- **Design situation**
  - 6m high cantilever gravity retaining wall,
  - Wall and base thicknesses 0.40m.
  - Groundwater level is at depth below the base of the wall.
  - The wall is embedded 0.75m below ground level in front of the wall.
  - The ground behind the wall slopes upwards at 20°

- **Soil conditions**
  - Sand beneath wall: $c'_k = 0, \phi'_k = 34^\circ, \gamma = 19\text{kN/m}^3$
  - Fill behind wall: $c'_k = 0, \phi'_k = 38^\circ, \gamma = 20\text{kN/m}^3$

- **Actions**
  - Characteristic surcharge behind wall 15kPa

- **Require**
  - Width of wall foundation, $B$
  - Design shear force, $S$ and bending moment, $M$ in the wall
Example 5

Sand beneath wall
- $\gamma$ (kN/m$^3$)
- $\phi'_k$ (°) ($c'_k = 0$)
- $\phi'_a$ (°)
- $\delta(k)$ on base (°)
- $\gamma(\delta)
- \delta(d)$ on base (°)

Fill behind wall (slope $\beta = 20^\circ$)
- $\gamma$ (kN/m$^3$)
- $\phi'_k$ (°) ($c'_k = 0$)
- $\phi'_a$ (°)
- $K_a$ ($\delta/\phi = \beta/\phi$)
- $K_a$ ($\delta/\phi = 2\sqrt{3}$)

Wall (concrete)
- $\gamma$ (kN/m$^3$)
- Surcharge (k)
- $\gamma$(surcharge)
- Surcharge (d)

Base width
Max BM in wall (kN/m)
- $\gamma$ (BM)
- Design max BM in wall (kN/m)
Max shear force in wall (kN/m)
- $\gamma$ (SF)
- Design max SF in wall (kN/m)
Lever arm (m)
Example 5

B = ?

Surcharge 15kPa

Sand

Fill

K_a \gamma z

\begin{array}{l}
\text{Fill} \\
\text{Sand} \\
\text{20°} \\
\text{B = ?} \\
\text{6m} \\
\text{0.4m} \\
\text{0.75m}
\end{array}

\text{γ} (\text{kN/m}^3) \\
\phi_k (°) \quad (c' = 0) \\
\gamma(\phi) \\
\phi_d (°) \\
\delta(k) \text{ on base (°)} \\
\gamma(\delta) \\
\delta(d) \text{ on base (°)} \\
\text{Overburden depth (m)} \\
\text{Overburden pressure (kPa)} \\
\gamma(R) \\
\text{Vk/L} \\
\text{Hk/L} \\
\gamma(V) \\
\gamma(H) \\
\text{Vd/L} \\
\text{Hd/L} \\
\text{phi-d} \\
\text{c-d} \\
Nq \\
Nc \\
Ng \\
iq \\
iq \\
ic \\
\text{γ(R)} \\
R \\
\text{γ(R)} \\
Rd \\
B_{req} \\
\delta(k) \text{ on base (°)} \\
\gamma(\delta) \\
\delta(d) \text{ on base (°)} \\
\text{Sliding: spare FOS} \\
\text{EN1997-1: Anchorages and Retaining structures}
Example 5 – Cantilever Gravity Retaining Wall

Example 5 - Gravity wall

1, 2 or 3 – EC7 DA1, DA2 or DA3
b – EC7 DA1 Comb 1 only
N – national method

Contributor

EN1997-1: Anchorages and Retaining structures
Example 5 – Cantilever Gravity Retaining Wall

- **Design situation**
  - 6m high cantilever gravity retaining wall,
  - Wall and base thicknesses 0.40m.
  - Groundwater level is at depth below the base of the wall.
  - The wall is embedded 0.75m below ground level in front of the wall.
  - The ground behind the wall slopes upwards at 20°

- **Soil conditions**
  - Sand beneath wall: \( c' = 0, \phi' = 34^\circ, \gamma = 19\text{kN/m}^3 \)
  - Fill behind wall: \( c' = 0, \phi' = 38^\circ, \gamma = 20\text{kN/m}^3 \)

- **Actions**
  - Characteristic surcharge behind wall 15kPa

- **Require**
  - Width of wall foundation, B
  - Design shear force, S and bending moment, M in the wall

Additional specifications provided after the workshop:
1. The characteristic value of the angle of sliding resistance on the interface between wall and concrete under the base should be taken as 30°.
2. The weight density of concrete should be taken as 25 kN/m3.
3. The bearing capacity should be evaluated using to the EC7 Annex D approach.
4. The surcharge is a variable load.
5. It should be assumed that the surcharge might extend up to the wall (ie for calculating bending moments in the wall), or might stop behind the heel of the wall, not surcharging the heel (ie for calculating stability).
Example 5 – Cantilever Gravity Retaining Wall

Footings 3.75m wide

Characteristic

ULS

\[ E \{ \gamma_F, F_{\text{rep}}; X_k, a_d \} = E_d \leq R_d = R \{ \gamma_F, F_{\text{rep}}; X_k, a_d \} / \gamma_R \]

\[ B' = 2.61 \text{m} \]

\[ B' = 2.17 \text{m} \]
### Example 5 – Cantilever Gravity Retaining Wall

<table>
<thead>
<tr>
<th>Column no.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Base width</strong></td>
<td>3.75</td>
<td>3.75</td>
<td>3.75</td>
<td>3.75</td>
<td>3.75</td>
</tr>
<tr>
<td><strong>Eccentricity (m)</strong></td>
<td>0.57</td>
<td>0.57</td>
<td>0.57</td>
<td>0.79</td>
<td>0.79</td>
</tr>
<tr>
<td><strong>Effective width B’ (m)</strong></td>
<td>2.61</td>
<td>2.61</td>
<td>2.61</td>
<td>2.17</td>
<td>2.17</td>
</tr>
<tr>
<td><strong>Vertical force kN/m</strong></td>
<td>690</td>
<td>941</td>
<td>690</td>
<td>941</td>
<td>690</td>
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<tr>
<td><strong>Horizontal force kN/m</strong></td>
<td>207</td>
<td>285</td>
<td>285</td>
<td>285</td>
<td>285</td>
</tr>
<tr>
<td><strong>Inclination H/V</strong></td>
<td>0.30</td>
<td>0.30</td>
<td>0.41</td>
<td>See note</td>
<td>0.41</td>
</tr>
<tr>
<td><strong>R (kN/m)</strong></td>
<td>1392</td>
<td>1373</td>
<td>879</td>
<td>659</td>
<td>659</td>
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<tr>
<td><strong>γ(R)</strong></td>
<td>1.0</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
</tr>
<tr>
<td><strong>Rd (kN/m)</strong></td>
<td>1392</td>
<td>981</td>
<td>628</td>
<td>471</td>
<td>471</td>
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<tr>
<td><strong>Rd/Vd</strong></td>
<td>2.02</td>
<td>1.04</td>
<td>0.91</td>
<td>0.50</td>
<td>0.68</td>
</tr>
</tbody>
</table>

**Column no. 1**: Characteristic values of all parameters.
**Column no. 2**: Characteristic eccentricity and inclination; forces and resistance factored.
**Column no. 3**: Characteristic eccentricity; unfavourable (horizontal) force and resistance factored. Favourable (vertical) force not factored in deriving inclination or comparison with resistance.
**Column no. 4**: Unfavourable (horizontal) force and resistance factored. Favourable (vertical) force not factored in deriving inclination or eccentricity, but factored for comparison with resistance.
**Column no. 5**: Unfavourable (horizontal) force and resistance factored. Favourable (vertical) force not factored in deriving inclination or eccentricity, or for comparison with resistance.

---

**EN1997-1: Anchorages and Retaining structures**
Example 5 – Cantilever Gravity Retaining Wall

Example 5 - Gravity wall

BENDING MOMENT kNm/m

<table>
<thead>
<tr>
<th>0</th>
<th>1000</th>
<th>800</th>
<th>600</th>
<th>400</th>
<th>200</th>
<th>0</th>
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<tr>
<td>0</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>G</td>
</tr>
<tr>
<td>1</td>
<td>1=N</td>
<td>N</td>
<td>N</td>
<td>N</td>
<td>2=N</td>
<td>b</td>
</tr>
<tr>
<td>2</td>
<td>2=N</td>
<td>b</td>
<td>b</td>
<td>1</td>
<td>2</td>
<td>3</td>
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</tbody>
</table>

© Example 5 – Gravity wall

C:\BX\BX-C\EC7\Dublin\[Dublin-results.xls] 27-Jun-05  21:43

EN1997-1: Anchorages and Retaining structures

EUROCOTES Background and Applications

ARUP
Example 5 – Cantilever Gravity Retaining Wall

Example 5 - Gravity wall

SHEAR FORCE  kN/m

0  50  100  150  200  250  300

1 N

2=N

0  1  2  3  4  5  6  7  8  9  10  11  12  13  14  15  16  17

EN1997-1: Anchorages and Retaining structures
Example 5 – Cantilever Gravity Retaining Wall

• **Serviceability:**
  - No criteria in the instructions
  - Mainly ignored
  - \( \frac{1}{2}(K_a + K_0) \)?
  - Middle third ?

• **Very large range of results**

• **Importance of sequence of calculation and factoring**
  - this is the main difference between the design approaches for this problem

• **Factors of safety must allow for errors and misunderstanding**
Example 6 – Embedded sheet pile retaining wall

- **Design situation**
  - Embedded sheet pile retaining wall for a 3m deep excavation with a 10kPa surcharge on the surface behind the wall

- **Soil conditions**
  - Sand: $c'_k = 0$, $\phi'_k = 37^\circ$, $\gamma = 20\text{kN/m}^3$

- **Actions**
  - Characteristic surcharge behind wall 10kPa
  - Groundwater level at depth of 1.5m below ground surface behind wall and at the ground surface in front of wall

- **Require**
  - Depth of wall embedment, $D$
  - Design bending moment in the wall, $M$
Example 6 – Embedded sheet pile retaining wall

- **Design situation**
  - Embedded sheet pile retaining wall for a 3m deep excavation with a 10kPa surcharge on the surface behind the wall

- **Soil conditions**
  - Sand: $c'_k = 0$, $\phi'_k = 37^\circ$, $\gamma = 20\text{kN/m}^3$

- **Actions**
  - Characteristic surcharge behind wall 10kPa
  - Groundwater level at depth of 1.5m below ground surface behind wall and at the ground surface in front of wall

- **Require**
  - Depth of wall embedment, $D$
  - Design bending moment in the wall, $M$

**Additional specifications provided after the workshop:**
1. The surcharge is a variable load.
2. The wall is a permanent structure.
Example 6 – Embedded sheet pile retaining wall

- Huge range of results
- Values of $K_p$?
  - $C&K$ / EC7 / Coulomb ??
- What about overdig?
- 2.4.7.1(5) Less severe values than those recommended in Annex A may be used for temporary structures or transient design situations, where the likely consequences justify it.

Figure CG.4  Ratio between coefficients of passive earth pressure from Kerisel and Absi and from Equation G12
Example 7 – Anchored sheet pile quay wall

• **Design situation**
  - Anchored sheet pile retaining wall for an 8m high quay using a horizontal tie bar anchor.

• **Soil conditions**
  - Gravelly sand - $\phi' = 35^\circ$, $\gamma = 18\text{kN/m}^3$ (above water table) and 20kN/m$^3$ (below water table)

• **Actions**
  - Characteristic surcharge behind wall 10kPa
  - 3m depth of water in front of the wall and a tidal lag of 0.3m between the water in front of the wall and the water in the ground behind the wall.

• **Require**
  - Depth of wall embedment, D
Example 7 – Anchored sheet pile quay wall

- **Design situation**  
  - Anchored sheet pile retaining wall for an 8m high quay using a horizontal tie bar anchor.

- **Soil conditions**  
  - Gravelly sand - $\phi'_k = 35^\circ$, $\gamma = 18\text{kN/m}^3$ (above water table) and 20kN/m$^3$ (below water table)

- **Actions**  
  - Characteristic surcharge behind wall 10kPa  
  - 3m depth of water in front of the wall and a tidal lag of 0.3m between the water in front of the wall and the water in the ground behind the wall.

- **Require**  
  - Depth of wall embedment, D

Additional specifications provided after the workshop:

1. The surcharge is a variable load.
2. The wall is a permanent structure.
3. The length of the wall is to be the minimum allowable.
Example 7 – Anchored sheet pile quay wall

Example 7 - Bending moments

- not the end of the design

BENDING MOMENT kNm/m
Economies of up to 30% due to plastic design
The significance of yield in structural elements

### Table 5-1: Classification of cross-sections

<table>
<thead>
<tr>
<th>Classification</th>
<th>Z-profile</th>
<th>U-profile</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><img src="image" alt="Diagram" /></td>
<td><img src="image" alt="Diagram" /></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Class 1</th>
<th>- the same boundaries apply as for class 2</th>
<th>- a rotation check has to be carried out</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 2</td>
<td>$\frac{b/t_f}{\varepsilon} \leq 45$</td>
<td>$\frac{b/t_f}{\varepsilon} \leq 37$</td>
</tr>
<tr>
<td>Class 3</td>
<td>$\frac{b/t_f}{\varepsilon} \leq 66$</td>
<td>$\frac{b/t_f}{\varepsilon} \leq 49$</td>
</tr>
</tbody>
</table>

$\varepsilon = \frac{235}{\sqrt{f_y}}$

<table>
<thead>
<tr>
<th>$f_y$ [N/mm²]</th>
<th>240</th>
<th>270</th>
<th>320</th>
<th>355</th>
<th>390</th>
<th>430</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon$</td>
<td>0,99</td>
<td>0,93</td>
<td>0,86</td>
<td>0,81</td>
<td>0,78</td>
<td>0,74</td>
</tr>
</tbody>
</table>

**Key:**
- $b$: width of the flat portion of the flange, measured between the corner radii, provided that the ratio $r/t_f$ is not greater than 5,0; otherwise a more precise approach should be used;
- $t_f$: thickness of the flange for flanges with constant thickness;
- $r$: midline radius of the flanges between the webs and the flanges;
- $f_y$: yield strength.

---

**EN1997-1:** Anchorages and Retaining structures

**EUROCODES** Background and Applications

**ARUP**
Example 7 – Anchored sheet pile quay wall

- Large range of results
- SSI important
- Optimise: length, BM, anchor force?
- Design doesn’t end at the bending moment
- Nobody considered SLS
The wall must be 12m long. What tie force is required?
As a cantilever, length would be about 14m.
DA1 Comb 2 gives a tie force of 75kN

$\phi_k' = 35^\circ$

No water

Bending moment (kNm/m)

Top 0

Toe -11.86m

Tie force = 75 KN/m
But characteristic calculation gives zero tie force, for 12m length.
EN 1997-1  Eurocode 7

Section 9 – Retaining structures

Fundamentals – Design Approaches
Slopes and walls all one problem
Design Approaches matter!

Main points in the code text
Good basic check lists
Values of $K_a$ and $K_p$
Overdig
Not enough attention to SLS (by users, at least)

Examples:
Results broadly similar to existing practice
DAs: big effect on gravity walls; small effect on embedded

Lessons from the Dublin Workshop
Very wide range of results
Effect of DAs for gravity walls and $K_p$ for embedded
Human error important – partly offset by safety factors
Need to work with EC3-5
EN 1997-1  Eurocode 7

Section 8 – Anchorages
Section 9 – Retaining structures

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Arup Geotechnics