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ABSTRACT
Three examples of retaining walls were prepared for comparison of designs: a gravity wall, a cantilever and an anchored embedded wall. An analysis of the designs submitted is presented here. Despite efforts to avoid unintentional differences, there is a considerable disparity in the results, even among contributors who have apparently used the same Design Approaches. These may due to differing interpretations of the problems, differing calculation methods, or perhaps to errors. Where detail has been provided to the author, it appears that relatively small variations in earth pressure coefficients sometimes lead to significant differences in results, probably outweighing the effects of choice of Design Approach in EC7.

1 INTRODUCTION

On 31 March to 1 April 2005, a workshop was arranged in Dublin under the joint auspices of European Technical Committee 10 and Technical Committee 23 of ISSMGE, and GeoTechNet Working Party 2. A total of ten designs were considered, of which three were retaining walls: a gravity wall (Example 5), an embedded cantilever (Example 6) and an anchored embedded wall (Example 7). Calculations had been submitted in advance for these designs by 15 contributors and an initial comparison was presented at the workshop. Following this, the examples were re-defined, slightly more tightly, and contributors were asked to re-submit their calculations, showing their assumptions, methods and results in a proforma style which facilitated comparisons. In total, 21 contributors have submitted calculations, of which 12 provided full details.

The graphs in this paper show the results which have been obtained. A number in the range 0 to 17 or a letter A to G has been allocated to each document received, as an “anonymous” identifier. Some documents contain several calculations for the same problem, and in some cases several contributions have been received from the same country.

The points on the graphs are annotated as follows to represent the various design approaches:

1 – EC7 Design Approach 1, taking the worst case of Combinations 1 and 2.
2 – EC7 Design Approach 2
3 – EC7 Design Approach 3
N – an existing National method

In the case of Design Approach 1, results for the less severe combination are denoted by “b” if it is Combination 1 (formerly Case B) and by “c” for Combination 2 (formerly Case C).

Results are shown in bold for which details of methods and assumptions have been studied, from which most of the conclusions in this paper have been drawn.
2 EXAMPLE 5 – GRAVITY RETAINING WALL

2.1 Basic method
The design problem set as Example 5 is shown in Figure 1, and the results submitted are summarised in Figures 2 to 4.

The basic steps of a calculation using EC7 Design Approach 1 (DA1) are shown in Figure 5; the notations C1 and C2 are used to indicate combinations 1 and 2 (formerly Cases B and C in ENV 1997). The force on the “virtual back” of the wall (the vertical plane through the rear of the base) is generally taken to be parallel to the ground surface, i.e., at an inclination of 20°; in this particular case this happens to be very close to the value of \( \frac{2}{3} \phi' \), which is used at interfaces between soil and concrete not cast against the ground.

By all design methods, the equilibrium of the wall has to be established, incorporating factors of safety, and this is used to fix the width of the base, B. This requires an iterative calculation in which a value of B is proposed and then equilibrium is checked. It is therefore possible that some of the results submitted do not have B completely minimised.

2.2 Eccentricity and inclination of the base force
The final part of this calculation is the check on bearing capacity for the resultant force, marked as F in Figure 5. This acts centrally on an “effective area” of the base and is inclined to the vertical. Its eccentricity and inclination are therefore critical to the calculation of bearing capacity. Derivation of the force F includes factors of safety, and two approaches have been adopted by contributors in considering eccentricity and inclination when deriving bearing capacity: (a) calculating eccentricity and inclination using the force F derived from factored components, and (b) calculating eccentricity and inclination from the characteristic force F, unfactored. In either case, partial factors of safety are incorporated in the calculation of bearing capacity, and this is then compared with the design value of force F derived from factored components.

The final check is made by comparing the design value of the vertical component, \( V_d \), of force F with the design vertical resistance \( R_d \), so the requirement is \( R_d/V_d \geq 1 \). For typical values related to the footing of the gravity wall, Figure 6 shows how \( R_d/V_d \) varies for width B=3.75m. For the characteristic case, \( R_d/V_k \) is 2.02, and this falls to \( R_d/V_d = 1.04 \) if characteristic eccentricity and inclination are used in obtaining \( R_d \), indicating a near-optimum design. However, significantly lower, unacceptable values of \( R_d/V_d \) are obtained if the horizontal and vertical force components are factored before calculating inclination and eccentricity, particularly since the vertical component is favourable to inclination and eccentricity, and the horizontal component unfavourable. Hence, if approach (a) above is followed the footing has to be made bigger, though it is acceptable according to approach (b).

This issue was discussed during the workshop in relation to spread foundations generally; it is particularly relevant to gravity retaining walls. There appeared to be a majority view that the calculation of eccentricity and inclination should take account of the factored forces, and the author supports this view. In this case, the vertical component depends mainly on the density of the backfill, which is usually fairly well controlled, whereas the horizontal component depends on both the density of the backfill and its strength, which is more uncertain. These features are accommodated in Design Approach 1 Combination 2, in which density and strength are treated separately, but it is less clear how to treat this question in the other design approaches. For retaining walls, the author recommends that this problem is avoided for DA1 Combination 1 by applying the factors to the derived bending moments and shear forces only; this is considered acceptable provided Combination 2 is also checked (see EC7 2.4.7.3.2(2)).

2.3 Reasons for the range of results
The calculated base widths shown in Figure 2 suggest that smaller bases have been obtained using DA2 than with DA1, at least for the cases which can be checked in detail (bold symbols). This is thought to be mainly a result of the way eccentricity and inclination are calculated, as discussed above. Values for bending moment and shear force in the wall are
relatively consistent. Some contributors have used a coefficient of earth pressure equivalent to $\frac{1}{2}(K_a+K_o)$ for calculation of action effects in the wall. In the author’s view, this is appropriate for the SLS check on the wall, checking deflections and crack widths based on earth pressures using unfactored parameters, but probably not to the ULS strength check. This could well mean that the SLS check governs the thickness of the wall and its reinforcement.

Some contributors used Coulomb’s equation for derivation of coefficients of earth pressure, in contrast to the methods proposed by EN1997-1 Annex C. In this example, this made little difference since only coefficients of active pressure are important. Some contributors used the passive resistance in front of the wall whilst others ignored it; this made little difference.

In general, the results of calculations using EC7 are within the range of results obtained using other national methods.

2.4 Serviceability limit states

Some contributors checked that the resultant force $F$ for the characteristic state passed through the middle third of the footing. This was the only serviceability check noted; no contributors attempted to calculate displacement – no targets were set in the instructions.

3 EXAMPLE 6 – EMBEDDED SHEET PILE RETAINING WALL

The design problem set as Example 6 is shown in Figure 7, and the results submitted are summarised in Figures 8 and 9. All contributors used simple active and passive earth pressure diagrams, of the type illustrated in Figure 10.

Results for the various design approaches of EN1997-1 are fairly consistent in this case: the differences between contributors using, nominally, the same method were greater than those of single contributors using several methods. The majority of contributors who provided details used $\delta = \frac{2}{3}\phi'$, consistently with EN1997-1, 9.5.1(6) and allowed for 0.3m overdig, following EN1997-1, 9.3.2.2(2), but there were exceptions to both of these. Some contributors considered vertical equilibrium and as a result reduced the available angle of wall friction substantially on the passive side; the author has not been able to confirm the need for this.

Some variations in result were caused by differing methods of deriving coefficients of earth pressure, including the difference between the charts and formulae of EN1997-1 Annex C. As noted by Simpson and Driscoll (1998), for high values of $\phi'$ and $\delta/\phi'$ the charts give higher values for coefficient of passive pressure than do the formulae. The charts are based on the work of Kerisel and Absi (1990), whereas the formulae give results consistent with Lancellotta (2002).

According to the author’s calculations (Contributor “0” on the plots), the effect of 0.3m overdig is to increase the required embedment by 0.76m. This is shown by the first two results for DA1 at the left hand side of Figure 8: the point marked “1*” has no allowance for overdig. It has a similarly significant effect on calculated bending moment, as shown in Figure 9.

Overall, the results obtained using EC7 appear to be similar to the results of calculations using other national methods.

4 EXAMPLE 7 – ANCHORED SHEET PILE QUAY WALL

The design problem set as Example 7 is shown in Figure 11, and the results submitted are summarised in Figures 12 and 14. In this case, calculations could use either simple active-passive diagrams, as illustrated in Figure 15, or a method of soil-structure interaction which allows for arching and redistribution of earth pressures.

Similar comments to those of Example 6 apply to this example, and again the point marked “1*” shows the effect of no allowance for overdig.
Figures 16 and 17 show two methods used to represent redistribution of earth pressures. Figure 16 shows the results of a calculation of soil-structure interaction by the Oasys FREW program, which models the soil as an elastic continuum limited by active and passive pressures, which are allowed to redistribute within certain theoretical limits. Figure 17 shows redistribution rules typical of German practice (EAU 1996).

These methods tend to give lower bending moments, possibly shorter walls, but higher anchor forces than are obtained by simple active-passive diagrams, depending on the stiffness assumed for the anchor. In this context it is relevant that the additional specifications given after the workshop include the requirement “the length of the wall is to be the minimum allowable”. It would have been equally possible to specify “the length of the wall is to be x metres”, “the anchor force is to be the minimum allowable” or “the bending moment is to be the minimum allowable”, each of which could have led to different designs.

5 STRUCTURAL DESIGN

Examples 6 and 7 required the ULS design bending moments to be calculated and compared with national methods. However, the most relevant comparisons between methods would be based on the final outcome: in this case the lengths of the walls and the steel sections to be adopted or, more generally, the cost of construction. Designs to EC7 should be completed by reference to EC3 Part 5, which allows for plastic design of most robust sheet pile sections, hence providing a saving compared with some traditional methods, even for the same design bending moment. This is particularly significant for the propped cantilever since some limited rotation is allowed to occur at the first hinge to form, allowing further redistribution of earth pressures.

EN1997-1 notes the importance of potential forms of brittle behaviour (“reduction in strength with deformation”) in retaining wall design at 9.4.1(4) and 9.7.6(4). Neither of these paragraphs refer specifically to struts or anchors, but it is because of fear of brittle behaviour in strutting, in particular, that some national codes apply large factors of safety to calculated strut forces. A distinction might be made between forces calculated for struts and anchors or between brittle and ductile situations, but, as in EC7, this is sometimes not done. (An example of this is shown by the national result from Contributor A in Figure 14 which had a factor of 1.85 on the anchor force.) In the author’s opinion, designers should be particularly aware of the use of brittle components in support of retaining walls, especially if simple active-passive diagrams, like that of Figure 15, are adopted, since these tend to underestimate strut or anchor forces.

6 HUMAN ERROR

The results show considerable scatter, even when calculated by different contributors using nominally the same method. They clearly represent a considerable range of safety, and it seems likely that some of the less conservative designs would actually be unsafe if used in practice. If this is not so, then most of the designs are grossly uneconomic. When the results were first presented to the workshop, the range was even greater. It has been reduced partly by the additional specifications noted on Figures 1, 7 and 11, and partly by contributors correcting their own calculations.

It is apparent that misunderstandings and calculation errors can have significant effects on engineering designs. In the author’s opinion, factors of safety have an important role in covering a certain degree of human error. For this reason, the results of new design processes should be checked against traditional methods. Reductions in overall safety levels, and related economies in designs adopted for construction, should only be accepted in small increments, and tested in extensive practice before further reductions are considered.

7 CONCLUSIONS

The following conclusions are drawn.
1. In general, the results of the EC7 calculations were within the range of results from national methods. The variation due to differences of approach to earth pressure coefficients, for example, are as great as those due to the differing design approaches of EC7.

2. The method of calculating eccentricity and inclination of loading to be used in deriving bearing capacity of spread foundation has a significant effect; agreement is needed.

3. Allowance for overdig has a big effect on the resultant design.

4. EC7 should perhaps differentiate, in terms of methods or safety factors, between those struts or anchors which are ductile and those which fail in a brittle manner.

5. Virtually no attention was paid to serviceability. In principle, this could control the geometry of the designs, but this is unlikely in these cases.

6. It is necessary that factors of safety are large enough to provide some protection against human error.

REFERENCES


**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: $c'k = 0, \phi'k = 34^\circ, \gamma = 19kN/m^3$
- Fill behind wall: $c'k = 0, \phi'k = 38^\circ, \gamma = 20kN/m^3$

**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).

**Figure 1  Cantilever Gravity Retaining Wall**
Example 5 - Gravity wall

Figure 2 Gravity retaining wall - calculated required base width

Figure 3 Gravity retaining wall – calculated ULS design bending movements
Example 5 - Gravity wall

Figure 4 Gravity retaining wall – calculated ULS design shear force

Figure 5 Gravity retaining wall – calculation to DA1
Data
\[
\begin{align*}
\gamma (kN/m^3) & \quad 19 \\
\phi_k (^\circ) & \quad \phi_k' (^\circ) (c_k' = 0) & \quad 34 \\
\text{Overburden depth (m)} & \quad 0.75 \\
\text{Overburden pressure (kPa)} & \quad 14.25
\end{align*}
\]

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<th>4</th>
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<td>Base width</td>
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<td>3.75</td>
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<tr>
<td>Eccentricity (m)</td>
<td>0.57</td>
<td>0.57</td>
<td>0.57</td>
<td>0.79</td>
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<tr>
<td>Effective width B’ (m)</td>
<td>2.61</td>
<td>2.61</td>
<td>2.61</td>
<td>2.17</td>
<td>2.17</td>
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\[
\begin{align*}
\text{Vertical force kN/m} & \quad 690 \quad 941 \quad 690 \quad 941 \quad 690 \\
\text{Horizontal force kN/m} & \quad 207 \quad 285 \quad 285 \quad 285 \quad 285 \\
\text{Inclination H/V} & \quad 0.30 \quad 0.30 \quad 0.41 \quad \text{note} \quad 0.41 \\
Nq & \quad 29.4 \quad 29.4 \quad 29.4 \quad 29.4 \quad 29.4 \\
Nc & \quad 42.2 \quad 42.2 \quad 42.2 \quad 42.2 \quad 42.2 \\
Ng & \quad 38.4 \quad 38.4 \quad 38.4 \quad 38.4 \quad 38.4 \\
iq & \quad 0.49 \quad 0.49 \quad 0.34 \quad 0.34 \quad 0.34 \\
ig & \quad 0.34 \quad 0.34 \quad 0.20 \quad 0.20 \quad 0.20 \\
ic & \quad 0.47 \quad 0.47 \quad 0.32 \quad 0.32 \quad 0.32 \\
R (kN/m) & \quad 1392 \quad 1373 \quad 879 \quad 659 \quad 659 \\
\gamma (R) & \quad 1 \quad 1.4 \quad 1.4 \quad 1.4 \quad 1.4 \\
Rd (kN/m) & \quad 1392 \quad 981 \quad 628 \quad 471 \quad 471 \\
Rd/Vd & \quad 2.02 \quad 1.04 \quad 0.91 \quad 0.50 \quad 0.68
\end{align*}
\]

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 for 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.

Figure 6. Base of gravity wall – alternative approaches to eccentricity and inclination
Additional specifications provided after the workshop:

1. The surcharge is a variable load.
2. The wall is a permanent structure.

**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, $M$

---

Figure 7 Example 6 – Embedded cantilever

Figure 8 Embedded cantilever – calculated required embedment
Figure 9  Embedded cantilever – calculated ULS bending moment

Figure 10  Embedded cantilever – calculation to DA1, Combination 2

**Sand**

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<th>Value</th>
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<td>$\phi_k$</td>
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<tr>
<td>$\phi_\alpha$</td>
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<tr>
<td>$\phi_\beta$</td>
<td>31.1</td>
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<td>$S/\phi$ active</td>
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<td>$S/\phi$ passive</td>
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<td>$K_a$</td>
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<td>$K_a(d)$</td>
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<td>$K_p(d)$</td>
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<tr>
<td>Overhang (m)</td>
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<td>$y$ (surchage)</td>
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<td><strong>Embedment (m)</strong></td>
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<td>Bending moment (kNm/m)</td>
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<tr>
<td><strong>Design bending moment (kNm/m)</strong></td>
<td>176.1</td>
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- **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, $D$
  - Design bending moment, $M$ in the wall

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.

Figure 11  Example 7 – Anchored sheet pile quay wall
Figure 12  Example 7 – Anchored sheet pile quay wall

Figure 13  Anchored sheet pile quay wall – calculated ULS bending moment
Figure 14  Anchored sheet pile quay wall – calculated ULS anchor force

Figure 15  Anchored sheet pile quay wall – design without redistribution
Figure 16   Redistribution of earth pressures – Oasys FREW

Figure 17   German practice for sheet pile design – EAU (1996)