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Evaluation of Eurocode 7: Geotechnical design

Evaluation de l'Eurocode 7: Calcul géotechnique

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ABSTRACT

This paper presents the outcome of an International Workshop on Evaluation of Eurocode 7 that was held in Trinity College, Dublin in March/April 2005. The background to the Workshop was the publication in November 2004 of Eurocode 7: Geotechnical design – Part 1: General rules. Prior to the Workshop, ten geotechnical design examples were distributed to European engineers to prepare solutions to Eurocode 7. A large number of solutions with a range of values were received. The authors reviewed these solutions to evaluate Eurocode 7 and identify the reasons for the scatter in the results which included: different assumptions and calculation models, different interpretations of Eurocode 7, different Design Approaches, and calculation errors. Other aspects of Eurocode 7 and limit state design, such as the use of finite element analyses and serviceability limit state design, were also considered during the Workshop.

RÉSUMÉ

Cet article présente les résultats d'un Atelier International pour l'Evaluation de l'Eurocode 7 tenu à Trinity College, Dublin en mars-avril 2005. La base de cet Atelier fut la publication, en novembre 2004, de l'Eurocode 7: Calcul géotechnique – Partie 1 : Règles générales. Avant la tenue de l'Atelier, 10 exemples de calcul avaient été distribués aux ingénieurs européens, afin qu'ils préparent des solutions conformes à l'Eurocode 7. De nombreuses réponses furent reçues. Les auteurs examinèrent les réponses pour identifier les raisons de la dispersion des résultats. Ces raisons sont des hypothèses et des modèles de calcul différents, différentes interprétations de l'Eurocode 7, différentes Approches de Calcul et des erreurs de calcul. D'autres aspects de l'Eurocode 7 et du calcul aux états limites, tels les calculs aux éléments finis et les états limites de service furent également abordés durant cet Atelier.

Keywords: Eurocode 7, design examples, foundations, piles, retaining walls, uplift, heave, embankment

1 INTRODUCTION

This paper presents the outcome of an International Workshop to evaluate Eurocode 7 that was held in Trinity College, Dublin in March/April 2005 (Orr, 2005a). The Workshop was organised by European Regional Technical Committee 10 – Evaluation of the Application of Eurocode 7 of the International Society for Soil Mechanics and Geotechnical Engineering in association with Work Package 2 of GeoTechNet, the European Geotechnical Network for Research and Development, and Technical Committee 23 – Limit State Design of the ISSMGE.

The background to the Workshop was the publication in November 2004 by CEN, the European Committee for Standardization, of the European Standard EN 1997-1: Eurocode 7 Geotechnical design – Part 1: General rules. In order to implement EN 1997-1, each CEN member state has two years following publication of the EN to publish the national standard version with a national annex giving the values of the nationally determined parameter values to be used with the Eurocode in that country; for example Eurocode 7 Part 1 is published in Ireland as IS EN 1997-1 with the Irish National Annex. Schuppener and Frank (2006) have explained this implementation process.

2 EUROCODE 7 DESIGN METHOD

Eurocode 7, like the other Eurocodes for structural design using materials such as concrete or steel, is based on the limit state design method set out in EN 1990 - Eurocode: Basis of Structural Design. This design method involves checking that the occurrence of all ultimate limit states (ULSs) and serviceability limit states (SLSs) is sufficiently unlikely. ULSs are checked using calculations involving the application of partial factors to characteristic parameter values or resistances and to characteristic actions or action effects so as to achieve a certain target reliability.

Due to different ground conditions and design traditions throughout Europe, different ground investigation techniques and calculation models have evolved that embody much experience of local conditions. In order to have Eurocode 7 accepted by all CEN members as a European standard for geotechnical design it was necessary to accommodate this valuable local experience within the framework of Eurocode 7 (Orr, 2006). Eurocode 7 achieves this in two ways: firstly, Eurocode 7 is not a prescriptive code with regard to the calculation model for any design situation but provides the design principles and factors to be considered, and secondly, different ways of applying partial factors in geotechnical design are accommodated by adopting the following three Design Approaches (DAs) for ULSs in permanent and transient design situations:

- DA 1, which has two combinations: Combination 1, where partial factors are applied to the actions while soil strength parameters are not factored, and Combination 2, where partial factors are applied to the ground strength parameters while permanent actions are not factored and a smaller partial factor than in Combination 1 is applied to variable actions. In principle both combinations need to be checked and hence two calculations are required, but in practice it is often obvious which combination controls the design
- DA 2, which has partial factors applied both to the actions or effects of the actions and to ground resistances
- DA 3, which is similar to DA 1, Combinations 1 and 2 combined since, as in Combination 1, partial actors are applied to the actions or action effects and, as in Combination 2, partial factors are applied to the ground strength parameters. Thus only one calculation is required. Also, there are two sets of partial factors for actions, depending on whether actions are structural or geotechnical.

Note that in DA 1, Combination 2, for the design of piles and anchorages, partial factors are applied to pile and anchor resistances and sometimes to ground strength parameters. In DA 2, in the design of slopes and overall stability analyses, partial factors are applied to the resulting effect of the actions on the failure surface. In DA 3, in the design of slopes and overall stability analyses, actions on the soil, for example structural loads and traffic, are treated as geotechnical rather than structural actions, making it the same as DA 1, Combination 2.

3 DESIGN EXAMPLES

In order to evaluate Eurocode 7, 10 geotechnical design examples, involving 5 different areas of geotechnical design, were distributed, prior to the Workshop, to the members of ERTC 10, GeoTechNet WP2 and TC23 to design according to Eurocode 7. The design examples, presented by Orr (2005a) in the Workshop Proceedings, include 2 spread foundations, 2 pile foundations, 3 retaining walls, 2 designs against hydraulic failure and a road embankment on soft ground. The parameters required in these examples are listed in Table 1 and six of the examples are shown in Figures 1(a) – 1(f).

Altogether 98 sets of solutions were received for these design examples, which were prepared by geotechnical engineers from 12 countries, and it was found that there was generally a considerable scatter in the solutions. The ranges in the ULS solutions received for the examples are presented in Table 1, based on Orr (2005b), in the form of the difference between the maximum and minimum values from their mean, expressed as a percentage of their mean. Note that the ranges of values presented in Table 1 are based solely on the extreme values. The average values of all the solutions received are shown in brackets. The least range of solutions, 5%, was obtained for the pile designed from pile load test results, a medium range of 24% was obtained for the two spread foundation examples, while the greatest range, 62%, was obtained for the pile designed from soil parameter values with high values also obtained for the anchored retaining wall (51%) and the heave design examples (45%).

4 REVIEWS OF THE SOLUTIONS

4.1 *Reviewers*

The solutions received were sent to five reviewers who were asked to review them and identify the reasons for the ranges in the solutions and determine if this scatter was due to different interpretations of Eurocode 7 or other reasons. The reviewers who prepared reports on the examples were: G. Scarpelli

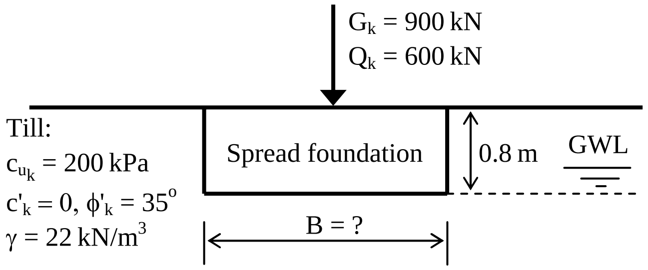

Figure 1(a) Example 1 – Spread foundation with vertical load

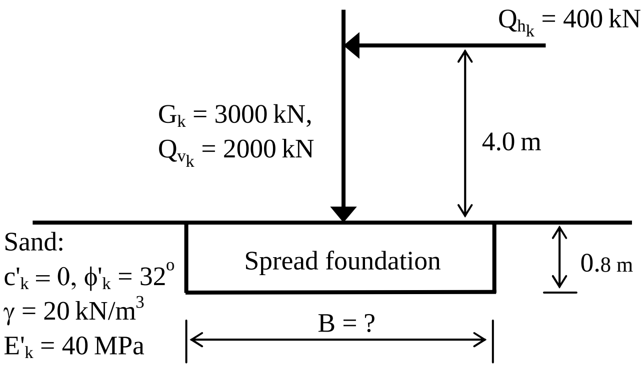

Figure 1(b) Example 2 – Spread foundation with an inclined and eccentric load

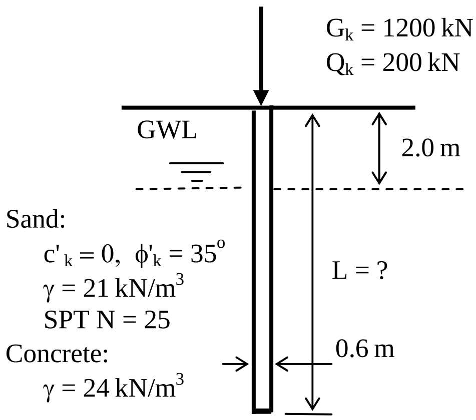

Figure 1(c) Example 3 – Bored concrete pile

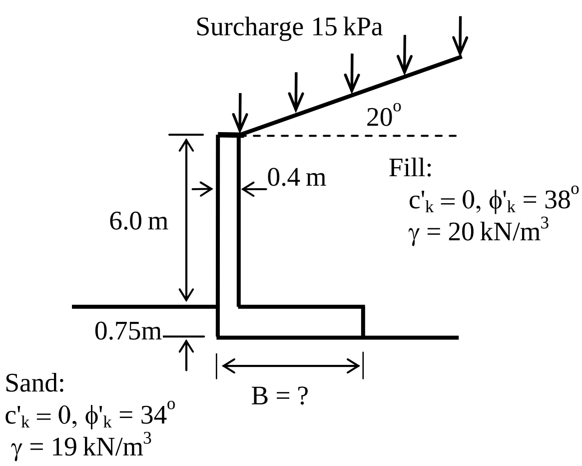

Figure 1(d) Example 4 – Cantilever gravity retaining wall

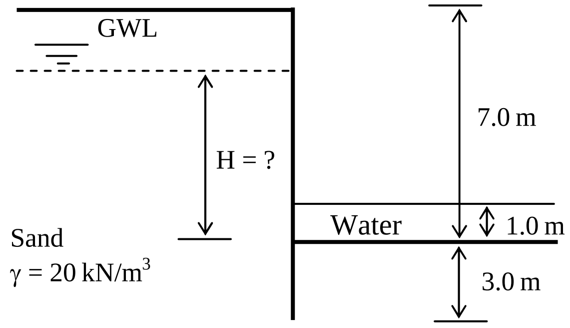

Figure 1(e) Example 9 – Failure by hydraulic heave

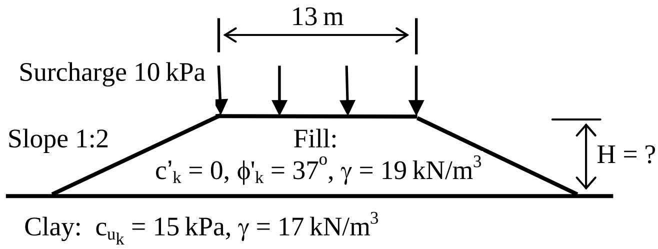

Figure 1(f) Example 10 – Road embankment on soft clay

Table 1: Ranges and averages of solutions for design examples

Example	Required parameter	Range of solutions (average value)	% Range
1	B - foundation width	1.4 – 2.3m (1.9m)	± 24%
2	B - foundation width	3.4 – 5.6m (3.9m)	± 24%
3	L - pile length	10.0 – 42.8m (20.5m)	± 62%
4	N – number of piles	9 – 10 (9)	± 5%
5	B – base width	3.1 – 5.2m (4.3m)	± 37%
6	D - embedment depth	3.9 – 6.9m (5.4m)	± 28%
7	D - embedment depth	2.3 – 7.0m (4.7m)	± 51%
8	T - slab thickness	0.42 – 0.85m (0.60m)	± 33%
9	H - hydraulic head	3.3 – 8.8m (4.9m)	± 45%
10	H - embankment height	1.6 – 3.4m (2.4m)	± 36%

and V.M.E. Fruzzetti on shallow foundations, R. Frank on pile foundations, B. Simpson on retaining structures, T. Orr on designs against hydraulic failure and U. Bergdahl on an embankment on soft ground. The main findings in these reports and the reasons for the ranges in the solutions received are presented in the following sections.

4.2 Examples 1 and 2 - Spread foundations

Scarpelli and Fruzzetti (2005) found that the spread in the solutions for Example 1 could be explained partly by the different Design Approaches adopted to analyse the ULS condition, and, it appeared, partly by some minor misinterpretations of the problem itself. In the case of Example 2, the following reasons were identified for the spread of the solutions in this example:

- only a few of the solutions received could be described as being fully in agreement with Euro-

code 7 as some solutions were obtained by following national codes;
- only a few solutions considered both the ULS and SLS conditions;
- the adopted calculation models were not all completely in agreement with the given problem.

In view of the way Eurocode 7 was used for the design of spread foundations in these examples, Scarpelli and Fruzzetti concluded that there is still a great need for experience in the use of the code by engineers in the member countries in order to achieve uniformity in geotechnical designs in Europe. A major difficulty that arose when these solutions were being prepared was the lack of a common experience basis that would have resulted in a single interpretation of the code and then the only differences in the solutions would be those due to the calculation models adopted.

According to Scarpelli and Fruzzetti, one of the biggest and somewhat worrying source of the differences in the solutions occurred as a consequence of the particular stage when the partial factors were introduced during the design calculations for the different Design Approaches. Normally partial factors are introduced at the start of the calculation, but in the Design Approach called DA 2* by Frank et al. (2004), the partial factors are introduced at the very end of the calculation. In this approach, characteristic resistances are evaluated using characteristic loads to calculate the load eccentricity and load inclination, whereas in DA 2 and the other Design Approaches the design resistance is evaluated using factored loads to calculate the load eccentricity and load inclination. The result is that in the case of a permanent vertical load and a variable horizontal load, for example, both the tangent of the inclination of the resulting load and the eccentricity are 11% greater for DA 2 than DA 2* and hence the design foundation size is larger for DA 2 than DA 2*.

Another difference in the solutions occurred due to the combination of loads chosen to analyse sliding stability. For this failure mechanism, the most unfavourable load combination should be chosen, which arises when there are independent vertical and horizontal variable loads, as in Example 2. In this situation, the variable loads should not be combined but the design against sliding stability should be for the situation when there is only the horizontal variable load and no vertical variable load.

4.3 Examples 3 and 4 - Pile foundations

Frank (2005), in his review of the two pile foundation examples, noted that the received solutions followed Eurocode 7 - Part 1 as well as a number of national codes.

Frank found that, in the case of Example 3, a bored pile designed from soil parameter values, the range of the solutions received was very large. He found that the scatter in the solutions was due to the different models used to calculate the base and shaft resistances from the soil parameter values rather than being due to either the ULS design procedure given in Eurocode 7 or the values of the partial factors used. In the case of Example 4, a driven pile design from pile load test results, he found that the solutions were remarkably consistent for both the ULS and SLS designs. This was attributed to the precise guidelines given in Eurocode 7 for ULS design in this situation and the straightforward analysis of the load-settlement curves for SLS design.

4.4 Examples 5, 6 and 7 - Retaining structures

Simpson (2005) reviewed the solutions received to the three retaining wall design examples, Example 5 being a gravity stem retaining wall, Example 6 an embedded sheet pile retaining wall and Example 7 an anchored sheet pile retaining wall. He found that the results showed considerable scatter, even when calculated by different contributors using nominally the same method. He noted that they clearly represent a considerable range of safety and hence it seems likely that some of the less conservative designs would actually be unsafe if used in practice. If this were not so, then most of the designs would be grossly uneconomic. When the received solutions were first presented to the Workshop, the range was even greater. This was subsequently reduced, partly by providing some additional specifications and partly by the contributors correcting their own calculations. Simpson stated it is apparent that misunderstandings and calculation errors can have significant effects on engineering designs.

In Simpson’s opinion, factors of safety have an important role in covering a certain degree of human error and, for this reason, the results of new design processes should be checked against traditional methods. His view is that reductions in overall safety levels, and related economies in designs adopted for construction, should only be accepted in small increments and should be tested in extensive practice before further reductions are considered.

As a result of the solutions received to the retaining wall examples, Simpson concluded:

1. In general, the results of the Eurocode 7 calculations were within the range of results from national methods. The variation due, for example, to differences of approach to earth pressure coefficients were as great as those due to the differing Design Approaches of Eurocode 7.
2. The method of calculating eccentricity and inclination of loading used in deriving bearing capacity of spread foundations has a significant effect and, as noted above by Scarpelli and Fruzzetti, agreement is needed.
3. Allowance for overdig has a large effect on the resultant design.

4. Eurocode 7 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 the design examples. In principle, this could control the geometry of the designs, but this was unlikely in the particular design examples.
6. It is necessary that factors of safety are large enough to provide some protection against human error.

Following the Workshop, Simpson obtained more information about the solutions received and, concentrating on those solutions which were intended to be in accordance with Eurocode 7 and for which details had been provided, he found many solutions included additional requirements which were not from Eurocode 7, such as δ/ϕ values lower than required by Eurocode 7 and extra penetration beyond the requirements of the equilibrium calculation. He concluded that when the requirements of Eurocode 7 alone are applied, the range of results obtained by the different authors would probably be fairly small.

4.5 Examples 8 and 9 - Uplift and Heave Failure

Orr (2005c) reviewed the solutions received for the two design examples involving hydraulic failure, Example 8 being uplift of a basement and Example 9 being heave of an excavation. As in the case of the other design examples, a range of solutions was received for both examples; Table 1 shows a smaller range of 33% for the uplift example and a greater range of 45% for the heave example. These ranges were found to be due to the use of different calculation models and design assumptions and also due to some differences in the interpretation of Eurocode 7.

In the case of Example 8, the differences in the solutions received were mainly because there was little agreement on how the friction force between the ground and the side wall should be calculated, i.e. whether to use the active or at-rest earth pressure, and how it should be treated, whether as a favourable action or as a resistance, and hence how the partial factors should be applied to it.

In the case of Example 9, the differences in the solutions received were due to the assumptions made regarding the dissipation of hydraulic head around the wall and how the partial action factors were used in the two equilibrium equations for heave given in EN 1997-1. In most of the received solutions, Equation 2.9b was used, with partial factors applied to the characteristic seepage force and effective weight, rather than Equation 2.9a, with the partial factors applied to the total stress and total pore water pressure. In actual fact, the two equations are physically equivalent and the confusion probably arises because Eurocode 7 does not state to what parameters the factors are to be applied. During the Workshop, there was a lively discussion concerning whether Equation 2.9a or 2.9b should be used, with some speakers arguing strongly for the use of total stresses and total pore water pressures when designing against heave.

Investigating the use of Equations 2.9a and 2.9b in designs against heave, Orr concluded that the application of partial factors to the characteristic total stress and total pore water pressure in Equation 2.9a results in very conservative designs and is illogical because the hydrostatic component of the total pore water pressure is increased on one side of the equilibrium equation but decreased on the other. However, if partial factors are only applied to the characteristic excess pore water pressure and the effective stress and not to the hydrostatic pore water pressure in Equation 2.9a, then the same design is obtained as when partial factors are applied to the characteristic seepage force and effective weight in Equation 2.9b. Such a design provides an overall factor of safety of 1.5, calculated as the ratio of the characteristic hydraulic gradient to the critical hydraulic gradient, which is equal to the factor of safety commonly used in designs against heave.

Examining the solutions received for Example 9, it was found that the value obtained for H, the design height of the groundwater level behind the wall, is very sensitive to the assumption made regarding the hydraulic head dissipation around the wall. For ideal uniform, isotropic soil, the assumption of a uniform gradient around the wall, which is commonly made, is very unconservative and leads to a significant overestimate of H.

4.6 Example 10 - Embankment on soft ground

Bergdahl (2005) reviewed the solutions received for Example 10, the design of a road embankment on soft ground and noted that the design embankment heights had a great scatter, ranging from 1.6m to 3.4m. Having examined the solutions, Bergdahl concluded that this range is explained by differences in:

- The Design Approaches or design code used,
- The values used for the partial factors on actions, materials and resistances
- The design model or computer program used.

Bergdahl noted that no direct guidance is provided in Section 12 Embankments of Eurocode 7 for the design of embankments on soft ground. There are only references to Section 11 Overall Stability, which gives in the Annex recommended partial factor values for designs using the three Design Approaches. The differences in the partial factors used for the different Design Approaches, including the use of nationally determined values in one case, partly explain the differences in the design solutions. The other main reason for the differences is that different calculation models were used, with overall stability calculations giving higher embankment heights than bearing capacity calculations. This is due partly to the inclusion of the shearing resistance in the embankment itself in the overall stability calculations but not in the bearing capacity calculations. Bergdahl's opinion is that the bearing capacity model is the correct one to use in this design situation.

Another issue raised by Bergdahl, but one that did not arise in Example 10 as presented, is that the design height of the embankment depends partly on how the characteristic soil strength value, c_{uk} is assessed. He stated that if the characteristic value is 5-10% lower than the mean value, then the overall safety factor will be about 1.5 which is acceptable for embankments but will result in some shear deformations in the ground with time, while an overall safety factor of 1.6 to 1.8 is necessary to avoid creep deformations for structural foundations. Consequently, Bergdahl's view is that the higher design embankment heights ought not to be accepted as these correspond to too low factors of safety and could lead to failure or unacceptable deformations in the ground and embankment. He questioned if the partial factor value γ_{cu} = 1.4 is high enough to give a stable foundation on clay without creep deformations and if there should not be a rule in Eurocode 7 that the bearing capacity model should be used for embankment design.

5 CONCLUSIONS

The main conclusion that has been drawn from the reviews of the solutions received for the examples designed to Eurocode 7 is that a wide range of solutions was obtained. However, these differences are generally within the ranges of solutions obtained using national standards and are due more to different calculation models and design assumptions, which are not specified in Eurocode 7, than to differences in the interpretation of Eurocode 7 or to use of the different Design Approaches. The Workshop did not investigate how variations in ϕ'_k affect the solutions obtained using the different Design Approaches since ϕ'_k was constant for each example. For the ϕ'_k values in the examples, it was found that DA 3 gave the most conservative designs, DA 2* the least conservative designs, and DA 1 and DA 2 generally gave designs between these.

In addition to the reports on the solutions to the examples, a number of papers were presented at the Workshop on general aspects of designs to Eurocode 7 and limit state design including papers on the Eurocode 7 design philosophy, the treatment of favourable and unfavourable loads, the use of finite elements in limit state design, seismic design of retaining structures, reliability analyses and serviceability limit state design.

As with the introduction of any new code, it will take some time for geotechnical engineers to develop experience with using Eurocode 7 so as to fully understand it and become comfortable with using it, particularly since Eurocode 7, being based on limit state concepts, differs from most existing geotechnical codes.

ACKNOWLEDGEMENTS

The authors would like to thank all those geotechnical engineers, representing 12 countries, who prepared the 92 solutions for the design examples that were reviewed for the Workshop.

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Model Solutions for Eurocode 7 Workshop Examples

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ABSTRACT
This paper includes model solutions for the 10 geotechnical design examples circulated prior to and discussed at the International Workshop on the Evaluation of Eurocode 7 held in Trinity College, Dublin on 31st March and 1st April, 2005. A summary of the model solutions for the design examples received from engineers throughout Europe is presented. A range of solutions was received for each example. The model solutions, carried out using the three Design Approaches, generally lie close to the middle of the range of solutions received for the design examples. For the conditions in these design examples, it was found that Design Approach 2 usually provided the least conservative solutions, Design Approach 3 the most conservative solutions, with Design Approach 1 providing solutions lying between these. The ranges of model solutions for the examples obtained by the author using his preferred calculations models and design assumptions were much less than the ranges of the received solutions.

1 DESIGN EXAMPLES

Prior to the International Workshop on Eurocode 7 held in Dublin in 2005, 10 geotechnical design examples involving 2 spread foundations, 2 pile foundations, 3 retaining walls, 2 designs against hydraulic failure and a road embankment on soft ground were distributed to geotechnical engineers who were members of the three committees, ERTC 10, Geotechnet and TC 23, involved in organising the Workshop. A large number of solutions to these examples were received from engineers from many countries, including some solutions carried out using Japanese codes. The solutions obtained from those European geotechnical engineers who carried out the designs according to Eurocode 7 are presented in Table 1. The ranges of the solutions received are also presented in Table 1 in the form of how much the maximum and minimum values for the requested parameters were greater or less than the mean of the maximum and minimum values, expressed as a percentage of the mean. The ranges of the solutions are based solely on the extreme values and do not reflect the average values of all the solutions received. The least range of solutions (5%) was obtained for the pile designed from pile load test results, and a relatively low range (21%) was obtained for the spread foundation with just a vertical central load, while the greatest range was obtained for the pile designed from soil parameter values (61%), with high values for the anchored retaining wall (51%), heave design (45%) and the spread foundation with an inclined eccentric load (42%).

The solutions were sent to five reporters who were asked to examine the solutions received and identify the reasons for the scatter and determine if this was due to different interpretations of Eurocode 7 or due to other reasons. The reporters were: G. Scarpelli and V.M.E. Fruzzetti on shallow foundations, R. Frank on pile foundations, B. Simpson on retaining walls, T. Orr on hydraulic failure and U. Bergdahl on the embankment. Their reports are included in the Workshop Proceedings. The reporters concluded that the scatter in the solutions when using Eurocode 7 was generally within the range of scatter obtained when using the different national standards and was due more to using different calculation models and design assumptions, which are not specified in Eurocode 7, than due to different interpretations of Eurocode 7 or using different Design Approaches. Some differences occurred because certain aspects were not defined sufficiently precisely initially and these were clarified after the Workshop.

Table 1: Ranges of solutions received to design examples using Eurocode 7

Example	Type	Parameter	Range of Solutions	% Range
1	Spread Foundation, vertical central load	B – foundation width	1.4 – 2.3m	± 24%
2	Spread foundation, inclined eccentric load	B - foundation width	3.4 – 5.6m	± 24%
3	Pile foundation from parameter values	L – pile length	10.0 – 42.8 m	± 62%
4	Pile foundation from load test results	N – number of piles	9 - 10	± 5%
5	Gravity retaining wall	B – wall base width	3.1 – 5.2 m	± 37%
6	Embedded retaining wall	D – embedment depth	3.9 – 6.9 m	± 28%
7	Anchored retaining wall	D – embedment depth	2.3 – 7.0 m	± 51%
8	Uplift	T – slab thickness	0.42 – 0.85 m	± 33%
9	Heave	H – hydraulic head	3.3 – 8.8m	± 45%
10	Embankment on soft ground	H – embankment height	1.6 – 3.4 m	± 36%

Table 2: Summary of Model Solutions

Example	Parameter	ULS				SLS	Range of Solutions (% Range)
		DA1.1	DA1.2	DA2	DA3		
1. Spread Foundation, vertical central load	B	*1.62*	**2.08**	**1.87**	**2.29**	< 1.87	1.87 - 2.29m (± 10%) (no DA3 ± 5%)
2. Spread foundation, inclined eccentric load	B	*3.46*	**3.98**	**3.77**	**4.23**	7.0	3.77 - 4.23m (± 6%) (no DA3 ± 3%)
3. Pile foundation from parameter values	L	**14.9**	*14.6*	**14.0**	**16.7**	-	14.0 - 16.7m (± 9%) (no DA3 ± 2%)
4. Pile foundation from load test results	N	9	**9**	**10**	-	10	9 - 10 (± 5%)
5. Gravity retaining wall	B	*3.85*	**5.03**	**4.21**	**5.03**	-	4.21 - 5.03m (± 9%)
6. Embedded retaining wall	D	*3.14*	**4.73**	**4.69**	**4.73**	-	4.35- 4.38m (± 1.1%)
7. Anchored retaining wall	D	*2.60*	**3.64**	**3.67**	**3.64**	-	3.64 - 3.74m(± 0.4%)
8. Uplift	T	**0.60**				-	–
9. Heave	H	**6.84**				-	–
10. Embankment on soft ground	H	*2.90*	**2.40**	**2.15**	**2.40**	-	2.15 - 2.40m (± 5%)

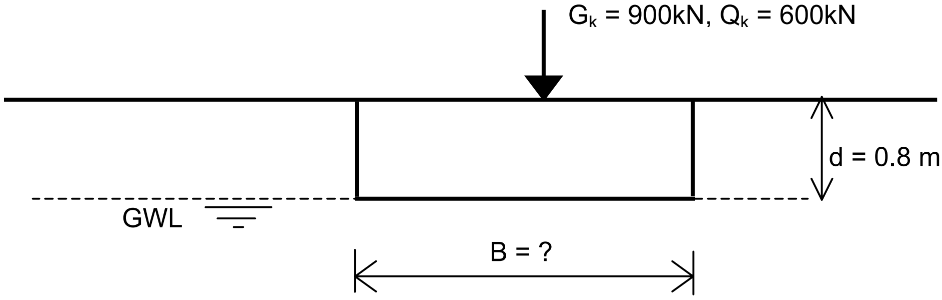
2 MODEL SOLUTIONS

The author has prepared a set of fully worked model solutions, using his preferred calculation models and design assumptions, which are included in this paper and summarised in Table 2. The results of the model solutions for ULS design, using the three Design Approaches where relevant, are presented in bold in Table 2. These solutions show that, for the conditions in the design examples, DA3 gave the most conservative designs, DA2 the least conservative designs, and DA1 gave designs between these, except for the anchored retaining wall, where DA1 was slightly more conservative than DA2. The more conservative designs with DA3 are expected because of the additional partial factor on structural loads when using DA3.

The ranges of the model solutions due to the different Design Approaches are much smaller than the ranges of received solutions in Table 1, being less than 10% for all except the spread foundation with the vertical central load due to the partial factor on the structural load in DA3. Furthermore, the ranges of all the model solutions lie within the ranges of the received solutions in Table 1, except for Example 1. This was because of the assumption in the model solution that the groundwater level could rise to the surface, in accordance with *2.4.6.1(11),* whereas the groundwater level was assumed to remain at foundation level in the solutions received. If DA3 is not included in the ranges for the first three examples, which involve structural loads, then the ranges of the model solutions do not exceed 5%, except for the gravity retaining wall, where the range is 9% with a DA2 solution of 4.21m compared to 5.03m for DA1.

Model Solution for Example 1 - Pad Foundation with a Vertical Central Load

1. Description of the problem



Soil: Stiff till - c_{uk} = 200kPa, c'_k = 0kPa, $\varphi'_k = 35^o$, $\gamma = 22kN/m^3$, SPT N = 40, $m_{vk} = 0.015m^2/MN$

- **Design Situation:**
  - Square pad foundation for a building, 0.8m embedment depth; groundwater level at base of foundation. The allowable settlement is 25mm.
- **Ground Properties:**
  - Overconsolidated glacial till, c_{uk} = 200kPa, c'_k = 0kPa, $\varphi'_k = 35^o$, $\gamma = 22kN/m^3$, SPT N = 40, $m_{vk} = 0.015m^2/MN$.
- **Characteristic values of actions:**
  - Permanent vertical load = 900kN + weight of foundation;
  - Variable vertical load = 600kN;
  - Concrete weight density = 24 kN/m^3.
- **Require foundation width, B to satisfy both ULS and SLS.**

2. Ultimate Limit State Designs

Bearing resistance failure, which is a GEO ultimate limit state, is the relevant ultimate limit state to be considered for the design of the pad foundation in this example. When using calculations to design for this ultimate limit state, the following inequality needs to be satisfied *(6.5.2.1(1)P)*:

$$V_d \leq R_d$$

where V_d is the design vertical load and R_d is the design bearing resistance. The minimum footing size is checked for each *Design Approach* and for both short term and long term design situations *(2.2(1)P)*, i.e. for both a) undrained conditions and b) drained conditions.

The value of the design vertical bearing resistance, R_d is calculated using the equations given in *Annex D* of Eurocode 7 and using the partial factors on actions (γ_F) or the effects of actions (γ_E) from *Table A.3*, the partial factors for soil parameters (γ_M) from *Table A.4* and the partial factors for resistances factors (γ_R) from *Table A.5*.

a) Undrained Conditions

For undrained conditions, the value of the vertical bearing resistance is calculated using *Equation D.1* of *Annex D*.

$$R_d/A = ((\pi + 2)\, c_{u,d}\, b_c\, s_c\, i_c + q_d)/\gamma_R = ((\pi + 2)\, c_{u,d}\, s_c + q_d)/\gamma_R$$

where s_c = 1.2 for a square foundation and q_d is the design total vertical stress at the founding level. Since the load is applied at the centre of the pad and no moment acts on it, the effective area A' is equal to the nominal area A and i_c = 1.0, and since the base of the foundation is horizontal, b_c = 1.0.

Design Approach 1 – Combination 1

The requirement $V_d \le R_d$ is checked for a 1.32 m x 1.32 m pad.

– Design value of the vertical action

$$V_d = \gamma_G (G_k + G_{pad,k}) + \gamma_Q Q_k = \gamma_G (G_k + A \gamma_c d) + \gamma_Q Q_k$$

where $G_{pad,k}$ is the characteristic weight of the concrete pad, γ_c is the weight density of concrete and d is the depth of the pad. Substituting the values for these parameters gives:

$$V_d = 1.35 (900 + 1.32^2 \times 24.0 \times 0.8) + 1.5 \times 600 = \underline{2160.2 \text{ kN}}$$

– Design value of the bearing resistance:

$$R_d = A ((\pi + 2) c_{u,d} s_c + q_d) / \gamma_R = A ((\pi + 2)(c_{u,k} / \gamma_{cu}) s_c + \gamma d) / \gamma_R$$

where γ is the total weight density of the soil, γ_R is the partial resistance factor, and $\gamma_R = 1.0$ for DA1. Hence:

$$R_d = 1.32^2 ((\pi + 2) (200/1.0) 1.2 + 22.0 \times 0.8)) /1.0 = \underline{2180.8 \text{ kN}}$$

The ULS design requirement $V_d \le R_d$ is fulfilled as 2160.2 kN < 2180.8 kN.

Design Approach 1 – Combination 2

The requirement $V_d \le R_d$ is checked for a 1.39 m x 1.39 m pad.

– Design value of the vertical action

$$V_d = \gamma_G (G_k + A \gamma_c d) + \gamma_Q Q_k = 1.0 (900 + 1.39^2 \times 24.0 \times 0.8) + 1.3 \times 600 = \underline{1717.1 \text{ kN}}$$

– Design value of the bearing resistance:

$$R_d = 1.39^2 ((\pi + 2) (200/1.4) 1.2 + 22.0 \times 0.8) / 1.0 = \underline{1737.0 \text{ kN}}$$

The ULS design requirement $V_d \le R_d$ is fulfilled as 1717.1 kN < 1737.0 kN.

Design Approach 2

The requirement $V_d \le R_d$ is checked for a 1.57 m x 1.57 m pad.

– Design value of the vertical action

$$V_d = \gamma_G (G_k + A \gamma_c d) + \gamma_Q Q_k = 1.35 (900 + 1.57^2 \times 24.0 \times 0.8) + 1.5 \times 600 = \underline{2178.9 \text{ kN}}$$

– Design value of the bearing resistance:

$$R_d = 1.57^2 ((\pi + 2) (200/1.0) 1.2 + 22.0 \times 0.8) / 1.4 = \underline{2203.6 \text{ kN}}$$

The ULS design requirement $V_d \le R_d$ is fulfilled as 2178.9 kN < 2203.6 kN.

Design Approach 3

The requirement $V_d \le R_d$ is checked for a 1.56 m x 1.56 m pad.

– Design value of the vertical action, treating the pad weight as a structural load:

$$V_d = \gamma_G (G_k + A \gamma_c d) + \gamma_Q Q_k = 1.35 (900 + 1.56^2 \times 24.0 \times 0.8) + 1.5 \times 600 = \underline{2178.1 \text{ kN}}$$

– Design value of the bearing resistance:

$$R_d = 1.56^2 ((\pi + 2) (200/1.4) 1.2 + 22.0 \times 0.8) / 1.0 = \underline{2187.8 \text{ kN}}$$

The ULS design requirement $V_d \le R_d$ is fulfilled as 2178.1 kN < 2187.8 kN.

b) Drained Conditions

For drained conditions the value of the vertical bearing resistance is calculated using *Equation D.2* of *Annex D*.

$$R_d/A' = c' N_c b_c s_c i_c + q' N_q b_q s_q i_q + 0.5 \gamma' B' N_\gamma b_\gamma s_\gamma i_\gamma$$

As $c' = 0$, the c terms are not required. If the weight density of water, γ_w is 9.81 kN/m^3 and as the weight density of the soil, $\gamma = 22$ kN/m^3, the effective weight density of the soil is 22.0 – 9.81 = 12.19kN/m^3. Although the groundwater level is given as being at the foundation level, the design is checked for the maximum possible ground water level, which is the ground surface *(2.4.6.1(11))*, so $q' = q = 0.8 * 12.19 = 9.75$ kPa. Since the load is applied at the centre of the pad and no moment acts on it, the effective area A' is equal to the nominal area A and i_c, i_q and i_γ are all unity, and since the base of the foundation is horizontal, the values of b_c, b_q and b_γ are also all unity. For DA1(C1) and DA2, $\phi'_d = \phi'_k = 35^o$ as $\gamma_M = 1.0$, while for DA1(C2) and DA3, $\phi'_d = \tan^{-1}(\tan \phi'_k)/\gamma_M$ = tan-1(tan35/1.25) = 29.3^o.

Design Approach 1 – Combination 1

The requirement $V_d \le R_d$ is checked for a 1.62m x 1.62 m pad.

– Design value of the vertical action

$$V_d = \gamma_G (G_k + \gamma_c' A\, d) + \gamma_Q Q_k = \gamma_G (G_k + (\gamma_c - \gamma_w) A\, d)) + \gamma_Q Q_k$$
$$= 1.35 (900 + (24 - 9.81) \times 0.8 \times 1.62^2) + 1.5 \times 600 = \underline{2155.2 \text{ kN}}$$

– Bearing resistance factors:

- $N_q = e^{\pi \tan\phi'} \tan^2(\pi/4 + \phi'/2) = e^{\pi \tan 35} \tan^2(\pi/4 + 35.0/2) = 33.30$
- $N_\gamma = 2(N_q - 1) \tan\phi' = 2(33.3 – 1) \tan 35 = 45.23$

- $s_q = 1 + \sin\phi' = 1 + \sin 35 = 1.57$
- $s_\gamma = 0.7$

  $R_d = A\,(q'\,N_q s_q + 0.5\gamma' B' N_\gamma\, s_\gamma)/\gamma_R = 1.62^2(9.75x33.3x1.57 + 0.5x12.19x1.62x45.23x0.7)/1.0$
  = <u>2158.2 kN</u>

The ULS design requirement $V_d \leq R_d$ is fulfilled as 2155.2 kN < 2158.2 kN

Design Approach 1 – Combination 2
The requirement $V_d \leq R_d$ is checked for a 2.08 m x 2.08 m pad.

– Design value of the vertical action

  V_d = 1.0 (900 + (24 - 9.81) x 0.8 x 2.08^2) + 1.3 x 600 = <u>1729.1 kN</u>

– Bearing resistance factors:

- $N_q = e^{\pi\tan\phi'}\tan^2(\pi/4 + \phi'/2) = e^{\pi\tan 29.3}\tan^2(\pi/4 + 29.3/2) = 16.92$
- $N_\gamma = 2(N_q - 1)\tan\phi' = 2(17.0 – 1)\tan 29.3 = 17.84$
- $s_q = 1 + \sin\phi' = 1 + \sin 29.3 = 1.49$
- $s_\gamma = 0.7$

  $R_d = A\,(q'N_q s_q + 0.5\gamma' B' N_\gamma s_\gamma)/\gamma_R = 2.08^2(9.75x16.92x1.49 + 0.5x12.19x2.08x17.84x0.7)/1.0$
  = <u>1748.4 kN</u>

The ULS design requirement $V_d \leq R_d$ is fulfilled as 1729.1 kN < 1748.4 kN

Design Approach 2
The requirement $V_d \leq R_d$ is checked for a 1.87 m x 1.87 m pad.

– Design value of the vertical action

  V_d = 1.35 (900 + (24 – 9.81)x 0.8 x 1.87^2) + 1.5 x 600 = <u>2168.9 kN</u>

– Bearing resistance factors:

- $N_q = e^{\pi\tan\phi'}\tan^2(\pi/4 + \phi'/2) = e^{\pi\tan 35}\tan^2(\pi/4 + 35.0/2) = 33.30$
- $N_\gamma = 2(N_q - 1)\tan\phi' = 2(33.3 – 1)\tan 35 = 45.23$
- $s_q = 1 + \sin\phi' = 1 + \sin 35 = 1.57$
- $s_\gamma = 0.7$

  $R_d = A\,(q'N_q s_q + 0.5\gamma' B' N_\gamma s_\gamma)/\gamma_R = 1.87^2(9.75x33.3x1.57 + 0.5x12.19x1.87x45.23x0.7)/1.4$
  = <u>2174.6 kN</u>

The ULS design requirement $V_d \leq R_d$ is fulfilled as 2168.9 kN < 2174.6 kN

Design Approach 3
The requirement $V_d \leq R_d$ is checked for a 2.29 m x 2.29 m pad.

– Design value of the vertical action

  V_d = 1.35 (900 + (24 – 9.81) x 0.8 x 2.29^2) + 1.5 x 600 = <u>2195.4 kN</u>

– Bearing resistance factors:

- $N_q = e^{\pi\tan\phi'}\tan^2(\pi/4 + \phi'/2) = e^{\pi\tan 29.3}\tan^2(\pi/4 + 29.3/2) = 16.92$
- $N_\gamma = 2(N_q - 1)\tan\phi' = 2(17.0 – 1)\tan 29.3 = 17.84$
- $s_q = 1 + \sin\phi' = 1 + \sin 29.3 = 1.49$
- $s_\gamma = 0.7$

  $R_d = q'\,N_q\,s_q + 0.5\,\gamma'\,B'\,N_\gamma\,s_\gamma = 2.29^2(9.75x16.92x1.49 + 0.5x12.19x2.29x17.84x0.7)/1.0$
  = <u>2203.1 kN</u>

The ULS design requirement $V_d \leq R_d$ is fulfilled as 2203.1 kN < 2195.4 kN

3. Summary of the Ultimate Limit State Designs

The design foundation widths for the three design approaches for undrained and drained and drained conditions calculated above are presented in Table 1. The results in Table 1 show that, for the given loading and soil conditions, the foundation widths for drained conditions are always greater than for undrained and therefore control the designs. The controlling design widths are shown in bold type. In the case of Design Approach 1, the foundation widths for Combination 1 for both undrained and drained conditions are greater than those for Combination 2 and the design width for Design Approach 1 is 1.75m. Design Approach 3 gives the largest foundation width at 2.39m while Design Approach 2 gives the smallest design width at 1.87m.

Assuming the till is clay soil, then in accordance with *6.6.2(16)*, the ratio of the bearing capacity of the ground at its initial undrained shear strength, R_{uk} to the applied serviceability loading, V_k is calculated and the values presented in Table 1. These values are 2.3, 2.0 and 2.9

Table 1: Design widths for different Design Approaches and design conditions

Design Approach	Foundation width (m)		OFS = R_{uk}/V_k
	Undrained conditions	Drained	
DA1 (Combination 1)	1.32	1.62	
DA1 (Combination 2)	1.39	**2.08**	2.3
DA2	1.57	**1.87**	2.0
DA3	1.56	**2.29**	2.9

and, as they are less than 3, settlement calculations should be carried out, but, since they are not less than 2, it is not necessary to take non-linear stiffness effects in the ground into account

4. Serviceability Limit State Design

To check the serviceability limit state the following inequality must be satisfied:

$$E_d \le C_d$$

where E_d is the calculated design value of the effect of the actions, i.e. the settlement due to the SLS loads, and C_d is the limiting design value of the effect of the actions, i.e. the maximum allowable settlement which is 25mm. The SLS loads are obtained from the characteristic loads as the SLS partial factors are all equal to unity. In this example, as the direct method is used, a settlement calculation is used to check the serviceability limit states *(6.4(5)P)*.

The following two components of settlement are considered in this calculation:

- s_0: immediate settlement;
- s_1: consolidation settlement.

The immediate settlement is evaluated using elasticity theory (*Annex F.2*):

$$s_o = p\,(1-\nu_\upsilon^2)\,B\,f\,/E_u$$

where p is the net SLS bearing pressure induced at the base level of the foundation, B is the foundation width, E_u is the Young's modulus, ν_u is Poisson's ratio (equals 0.5 for undrained conditions), and f is a settlement coefficient whose value depends on the nature of the foundation and its stiffness (for the settlement at the centre of a flexible square pad f = 1.12). Since the soil is overconsolidated, assume $E_u = 750c_u$ = 750 x 200 = 150 MPa.

The consolidation settlement is calculated by dividing the ground below the foundation into layers and summing the settlement of each layer using the following equation:

$$s_1 = \Sigma\, m_v\, h\, \Delta\sigma'$$

where m_v is the coefficient of volume compressibility = 0.015 m^2/MN (assumed constant in each layer for the relevant stress level), h is the thickness of the layer and $\Delta\sigma'$ is the increment in vertical effective stress in the layer due to the net bearing pressure.

<u>SLS check</u>

The serviceability limit state is checked for the smallest ultimate limit state design foundation width as this gives the largest settlement. The smallest ULS design foundation width was obtained for D*esign Approach 2,* i.e. for B = 1.87m.

Net bearing pressure: $p = (G'_{pad,k} + G_k + Q_k)\,/\,A$

$= (1.87^2 \times 0.8 \times (24 - 22) + 900 + 600) / 1.87^2 = 430.6$ kPa

Immediate settlement: $s_o = p\,(1-\nu_\upsilon^2)\,B\,f\,/E_u = 430.6 \times (1 - 0.5^2)\ 1.87 \times 1.2 / 150 = \underline{4.8\ \text{mm}}$

Consolidation settlement: Soil below foundation is divided into 4 layers, 1.0m thick. Increases in vertical stress, $\Delta\sigma'$ at centres of layers due to foundation are estimated from Fadum Chart to be: 362, 258, 86 and 52 kPa. Hence:

Consolidation settlement: $s_o = \Sigma\, m_v\, h\, \Delta\sigma' = 0.015 \times 1.0 \times (362 + 258 + 86 + 52) = \underline{11.4\ \text{mm}}$

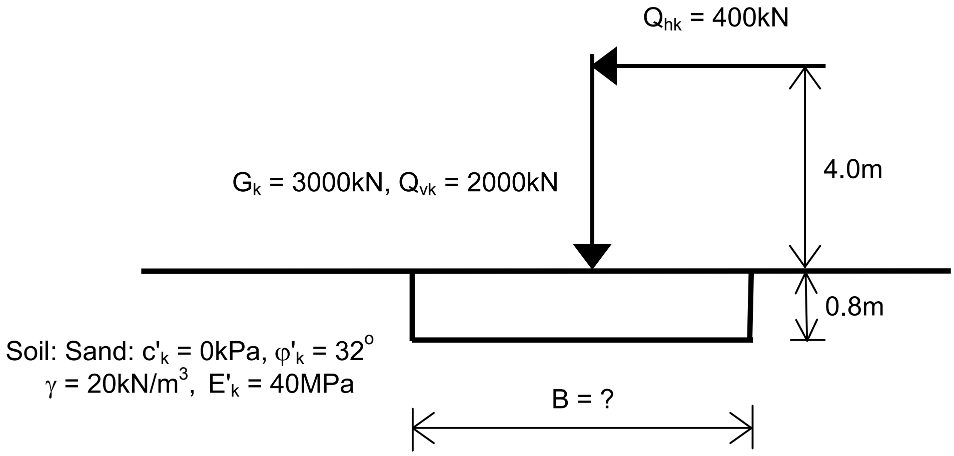
Total settlement $s = s_o + s_c = 4.8 + 11.4 \approx \underline{16\text{mm}} < 25\text{mm}$, therefore SLS requirement is satisfied.

5. Conclusion

The largest settlement for the design widths is ≈ 19mm for the DA2 width, which is less than the allowable settlement of 25mm, therefore, for this example, the design is controlled by the ULS and not the SLS requirement.

Model Solution for Example 2 – Pad Foundation with an Inclined Eccentric Load

1. Description of the problem



- **Design situation:**
  - Square foundation for a building, 0.8m embedment depth, groundwater level at great depth. Allowable settlement is 25mm and maximum rotation is 1/2000.
- **Soil conditions:**
  - Cohesionless sand, $c'_k = 0$, $\varphi'_k = 32^o$, $\gamma = 20kN/m^3$, $E'_k = 40MPa$.
- **Characteristic values of actions:**
  - Permanent vertical load $G_k = 3000kN$ plus weight of pad foundation;
  - Variable vertical load $Q_{vk} = 2000kN$ (at top of foundation);
  - Permanent horizontal load = 0;
  - Variable horizontal load $Q_{hk} = 400kN$ at a height of 4m above the ground surface;
  - Variable loads are independent of each other.
- **Require foundation width, B.**

2. Ultimate limit state design using an analytical method

2.1 – Bearing resistance

Bearing resistance failure is one ultimate limit state considered for the design of the pad. The following inequality needs to be satisfied *(6.5.2.1(1)P)*:

$$V_d \leq R_d$$

The minimum footing size is assessed for each *Design Approach* and just for drained conditions as $c'_k = 0$.

In this example, as the horizontal variable load is applied 4m above the ground surface, a moment *M* acts on the foundation, introducing an eccentricity *e*, and the effective area is different from the nominal area (i.e. $A' \neq A$). Furthermore, as the variable vertical and horizontal loads are independent of each other, according to EN 1990 *(6.4.3.2 Eqn. 6.10)* it is necessary to combine the leading and accompanying variable loads, with a combination factor of $\Psi_o = 0.7$ applied to the accompanying variable action, in order to determine the least favourable combination. The partial and combination factors are applied to loads at the start of the calculation in order to determine the load eccentricity and this eccentricity is then used to calculate the design resistance.

For each Design Approach, it is necessary to check if treating the vertical design load, V_d as a favourable action, which increases the eccentricity but reduces the vertical load, or as an unfavourable action, which reduces the eccentricity but increases the vertical load, is the most severe condition. Treating the Q_h as the leading variable load is the most severe condition.

$$V_d = G_d + G_{pad,d} + Q_{vd} = \gamma_G(Gk + B \times B \times h \times \gamma_{concrete}) + \gamma_Q \times \psi_o \times Q_{vk}$$

where G and Q_v are the permanent and variable vertical loads, G_{pad} is the weight of the pad foundation, B and h are the width and height of the foundation, $\gamma_{concrete}$ is the weight density of the foundation and the subscripts k and d indicate characteristic and design values. If the horizontal variable load is Q_h and this acts at a height h above the top of the foundation and the foundation is t thick, then the moment on the base of foundation is $M = Q(h + h)$. The eccentricity $e = M/V$ and the effective width $B' = B-2e$. To ensure that special allowances are not necessary, the eccentricity is checked to ensure that it does not exceed *B/3 (6.5.4(1)P)*.

The value of the design drained bearing resistance, R_d is calculated using *Eqn. D.2* of *Annex D:*

$$R_d/A' = c'\, N_c\, b_c\, s_c\, i_c + q'\, N_q\, b_q\, s_q\, i_q + 0.5\, \gamma'\, B'\, N_\gamma\, b_\gamma\, s_\gamma\, i_\gamma$$

where the bearing factors are:

$N_q = e^{\pi \tan \phi'} \tan^2(45 + \phi'/2)$	
$N_c = (N_q - 1) \cot\phi'$	
$N_\gamma = 2 (N_q - 1) \tan\phi'$	*when $\delta > \phi'/2$ (rough base)*
$s_q = 1 + \sin\phi'$	*for a square or circular shape*
$s_\gamma = 1 - 0.3(B'/L')$	*for a rectangular shape*
$s_\gamma = 0.7$	*for a square or circular shape*
$s_c = (s_q N_q - 1)/(N_q - 1)$	*for rectangular, square or circular shape*
$i_c = i_q - (1 - i_q)/N_c \tan\phi'$	
$i_q = [1 - H/(V + A'c'\cot\phi')]^m$	
$i_\gamma = [1 - H/(V + A'c'\cot\phi')]^{m+1}$	
$m = [2 + (B'/L')]/[1+(B'/L')]$	*when H acts in the direction of B'*

The *b* factors are all zero as the foundation base is horizontal. For DA1 and DA3, the R_d values are calculated by only applying partial factors to $\tan\phi'$ and the loads, while for DA2, R_d is calculated by applying partial factors of unity to $\tan\phi'$ and dividing the resistance obtained by γ_R. The values used in the design calculations for different design approaches and loading conditions are presented in Table 1.

The B values presented in Table 1 are the critical foundation widths when, for each design approach and loading condition, the design vertical load is equal to the design bearing resistance. The results show that, for the design conditions in this particular example, treating the vertical load as unfavourable is the critical loading condition for DA1 and DA3, while treating the vertical load as favourable is the critical loading condition for DA2. In the case of DA1, the design width is determined by Combination 2. Comparing the different design approaches, DA3 gives the largest width of 4.23m, DA2 gives the smallest width of 3.77m and DA1 gives an intermediate width of 3.98m. On overall factor of safety, OFS, is obtained by dividing the unfactored bearing resistance, R_k by the unfactored vertical load (with $\psi = 1.0$), V_k. For B = 3.98m, OFS = R_k / V_k = 2.5.

2.2 – Sliding resistance

The second ultimate limit state considered is sliding failure, satisfying the inequality *(6.5.3(2)P)*:

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

where H_d is the design horizontal load on the foundation, and R_d is the design horizontal resistance and $R_{p,d}$ is the passive earth resistance in front of the wall. As $R_{p,d}$ cannot be relied upon, it is assumed that $R_{p,d}$ = 0. The design shear resistance between the base of the wall and the soil, in front of the wall, R_d is given by:

$$H_d = (V_d' \tan\delta_d) / \gamma_R$$

Table 1: Ultimate limit state design of pad foundation

Parameter	DA1(C1)		DA1(C2)		DA2		DA3	
	$V_{fav.}$	$V_{unfav.}$	$V_{fav.}$	$V_{unfav.}$	$V_{fav.}$	$V_{unfav.}$	$V_{fav.}$	$V_{unfav.}$
B - pad width	3.46	3.26	3.97	**3.98**	**3.77**	3.65	4.09	**4.23**
γ_G-Structural (unfav.)	1.35	1.35	1.00	1.00	1.35	1.35	1.35	1.35
γ_G-Geotech (unfav.)	1.35	1.35	1.00	1.00	1.35	1.35	1.00	1.00
γ_G-Structural (fav.)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
γ_G-Geotech.(fav.)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
γ_Q-Structural (unfav.)	1.50	1.50	1.30	1.30	1.50	1.50	1.50	1.50
γ_Q-Geotech (unfav.)	1.50	1.50	1.30	1.30	1.50	1.50	1.35	1.35
Ψ_0-Comb. factor	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
G_{vk}	3000.00	3000.00	3000.00	3000.00	3000.00	3000.00	3000.00	3000.00
Q_{vk}	2000.00	2000.00	2000.00	2000.00	2000.00	2000.00	2000.00	2000.00
Q_{hk}	400.00	400.00	400.00	400.00	400.00	400.00	400.00	400.00
h - height of Q_h	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00
d - pad depth	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80
γ_c – conc. wt density	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00
G_{padk} = pad weight	230.49	203.97	302.99	303.94	272.18	255.26	320.63	344.02
V_d	3230.49	6425.36	3302.99	5123.94	3272.18	6494.6	3320.63	6614.43
Q_{hd}	600	600	520	520	600	600	600	600
Bearing Resistance								
M_d	2880	2880	2496	2496	2880	2880	2880	2880
e	0.892	0.448	0.756	0.487	0.880	0.443	0.867	0.435
Check B/3 – e > 0	0.26	0.64	0.57	0.84	0.37	0.77	0.49	0.98
B' = B-2e	1.68	2.36	2.46	3.00	2.00	2.76	2.35	3.36
L' = B	3.46	3.26	3.97	3.98	3.77	3.65	4.09	4.23
A' = B' x L	5.83	7.70	9.78	11.95	7.55	10.06	9.61	14.23
γ	20	20	20	20	20	20	20	20
q at foundation level	16	16	16	16	16	16	16	16
γ_ϕ	1.00	1.00	1.25	1.25	1.00	1.00	1.25	1.25
γ_c	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25
γ_R	1.00	1.00	1.00	1.00	1.40	1.40	1.00	1.00
ϕ'_k	32	32	32	32	32	32	32	32
$\phi'd$	32.00	32.00	26.56	26.56	32.00	32.00	26.56	26.56
N_q	23.177	23.177	12.588	12.588	23.177	23.177	12.588	12.588
N_γ	27.715	27.715	11.585	11.585	27.715	27.715	11.585	11.585
s_q	1.257	1.384	1.277	1.338	1.282	1.401	1.257	1.355
s_γ	0.854	0.783	0.814	0.773	0.840	0.773	0.827	0.762
m	1.673	1.580	1.617	1.570	1.653	1.569	1.635	1.557
i_q	0.709	0.857	0.758	0.845	0.716	0.859	0.851	0.862
i_γ	0.577	0.777	0.639	0.760	0.584	0.780	0.591	0.784
R_d	3230.49	6425.36	3302.99	5123.94	3272.18	6494.60	3320.63	6614.43
Vd	3230.49	6425.36	3302.99	5123.94	3272.18	6494.60	3320.63	6614.43
Check R_d-V_d > 0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Sliding Resistance								
$V_d = \gamma G_G k$	3230.491	3203.972	3302.99	3303.94	3272.18	3255.26	3320.63	3344.03
$H_d = \gamma_Q Q_h$	600	600	520	520	600	600	600	600
$\delta_d = \phi'_d$	32.0	32.0	26.6	26.6	32.0	32.0	26.6	26.6
$R_{hd} = V\tan\delta/\gamma_R$	2018.64	2002.06	1651.15	1651.62	1460.49	1452.94	1659.97	1671.66
R_d-V_d > 0	1418.64	1402.06	1131.15	1131.62	860.49	852.94	1059.97	1071.66

where δ_d is the design friction angle between the base of the foundation. According to *6.5.3(10)*, δ_d may be assumed equal to the design value of the effective critical state angle of shearing resistance $\phi'_{cv;d}$, for cast-in-situ concrete foundations and equal to 2/3 $\phi'_{cv;d}$ for smooth precast foundations". In this example it is assumed the foundation was cast-in-situ, so that δ_d=2/3 $\phi'_{cv;d}$. and it was assumed that $\phi'_{cv,d} = \phi'_d$.

For DA1 and DA3, γ_R = 1.0 so that the partial factors greater than zero are applied to *tanϕ'* and the loads, while for DA2, the partial factor on *tanϕ'* is unity so that partial factors greater than unity are applied to the loads and the calculated resistance is divided by γ_R.

The parameters values to check the sliding resistance are presented in Table 1 and the results show the foundation widths to ensure stability against bearing failure also to ensure the foundation is stable against sliding failure for all the design approaches and loading conditions.

3. Serviceability limit state design

To check that serviceability limit states are not exceeded, the following inequality must be satisfied:

$$E_d \leq C_d$$

where E_d is the calculated design value of the effect of the actions, i.e. the settlement or tilt, and C_d is the limiting design value of the effect of the actions, i.e. the maximum allowable settlement which is 25mm or the maximum allowable tilt, which is 1/2000 = 0.0005. In this example, as the direct method is used, a settlement calculation and a tilt calculation are used to check the serviceability limit states *(6.4(5)P)*.

3.1 – Settlement

As the soil is sand and only the Young's modulus, E is given, the settlement is calculated using the following elastic equation (Poulos and Davis, 1974):

$$s = \frac{\mathrm{V_k}(1-\nu^2)}{\mathrm{E}\beta_z\sqrt{\mathrm{BL}}}$$

where V_k is the net characteristic bearing pressure on the foundation due to the vertical loads, ν is Poisson's ratio and is assumed to be 0.3 and β_z is a coefficient whose value is 1.1 for a square foundation. Substituting the relevant values in the above equation for the smallest ULS design width of 3.77m for DA2 gives:

$$s = \frac{5272.9(1-0.3^2)}{40000 \text{ x } 1.1\sqrt{3.77 \text{ x } 3.77}} \text{ = 28.4mm } > 25\text{mm}$$

As s = 28.4mm > 25mm, the settlement SLS condition is not satisfied for any of the ULS design widths.

3.2 – Tilt

According to *6.6.2(15)*, (15) the tilting of an eccentrically loaded foundation should be estimated by assuming a linear bearing pressure distribution and then calculating the settlement at the corner points of the foundation, using the vertical stress distribution in the ground beneath each corner point As an alternative to this application rule, the tilt is calculated using the elastic equation (Poulos and Davis, 1974) so that this design solution is not claimed to be wholly in accordance with EC7 (*1.4(4)*):

$$\theta = \frac{\mathrm{M_k}(1-\nu^2)}{\mathrm{EB^2L}} I_t \text{ = 27.6mm } > 25\text{mm}$$

where M_k is the unfactored moment on the foundation base and I_t is a factor whose value is 3.7 for a square foundation when B = L. For B = 7.0m, the tilt is:

$$\theta = \frac{400 \text{ x } 4.8 \text{ x } (1-0.3^2)}{40000 \text{ x } 7.0^2 \text{ x } 7.0} 3.7 = 0.000471 = 1/2122 < 1/2000$$

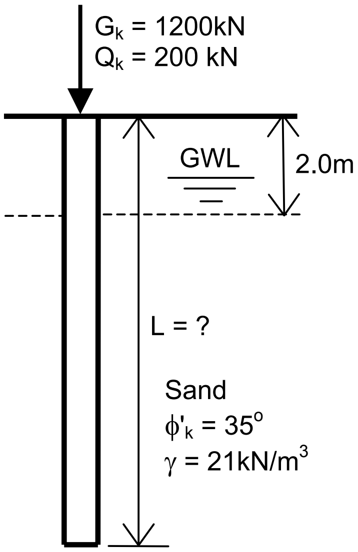
Since θ = 1/2122 < 1/2000, the tilt SLS condition is satisfied when B = 7.0m. As B = 7.0m is larger than the ULS and settlement design widths, the foundation design in this example is controlled by the severe SLS requirement that the tilt should not exceed 1/2000.

References

Poulos H.G. & Davies E.H. (1974) Elastic solutions for soil and rock mechanics, Wiley

Model Solution for Example 3 - Pile Foundation Designed from Soil Parameter Values

1. Description of the problem



- **Design situation**
  - Bored pile for a building, 600mm diameter
  - Groundwater level at depth of 2m below the ground surface
- **Soil conditions**
  - Sand: $c'_k = 0$, $\phi'_k = 35^o$, $\gamma = 21$kN/m^3
    SPT N = 25
- **Actions**
  - Characteristic permanent load G_k = 1200kN
  - Characteristic variable load Q_k = 200kN
  - Weight density of concrete = 24kN/m^3
- **Require**
  - Pile length, L

2. Ultimate limit state design

2.1 – General

Compressive, i.e. bearing, resistance failure is the ultimate limit state failure mechanism considered for the design of the pile for the building. To demonstrate that the pile will support the design load, the following inequality must be satisfied *(7.6.2.1 (P))*:

$$F_{c,d} \leq R_{c,d}$$

where $F_{c,d}$ is the design action (load) and $R_{c,d}$ is the design resistance of the pile. The design pile length is determined for Design Approaches 1, 2 and 3.

2.2 – Design Action

The design vertical action, $F_{c,d}$ is given by:

$$F_{c,d} = \gamma_G G_k + \gamma_Q Q_k$$

where G_k = 1200kN, Q_k = 200kN and the values of γ_G and γ_Q depend on the Design Approach being used. The weight of the pile is ignored as it is assumed that the pressure at the toe due to the pile weight is similar to the overburden pressure at that depth *(7.6.2.1(2).*

2.2 – Ultimate compressive resistance from ground test results

Using Equation 7.6 in Clause *7.6.2.3 (3),* the design compressive resistance is given by:

$$R_{c,d} = R_{b,d} + R_{s,d} = R_{b,k}/\gamma_b + R_{s,k}/\gamma_s$$

where $R_{b;d}$ and $R_{s;d}$ are the design base and shaft resistances, $R_{b;k}$ and $R_{s;k}$ are the characteristic base and shaft resistances, and γ_b and γ_b are the relevant partial resistance factors. The characteristic resistances may be obtained either from Equation 7.8, which involves correlation factors x whose values depend on the number of profiles of tests, or from Equation 7.9, which is $R_{bk} = A_b q_{bk}$ and $R_{sk} = A_s q_{sk}$, where q_{bk} and q_{sk} are the characteristic base resistances and shaft friction obtained from the ground parameters and A_b and A_s are the areas of the pile base and pile shaft. Since no profiles of tests are provided but the characteristic ϕ' value is provided, the characteristic resistances are calculated using Equation 7.9. As stated in the Note to *Clause 9.6.2.3(8)*, when the alternative approach is used, the values of the partial factors γ_b and γ_b may need to be corrected by a model factor, γ_R to be set by the National Annex. In this example, the value of γ_R = 1.5 is used.

Since only one ϕ'_k value, 35^o, is provided, it is assumed that this is the appropriate characteristic value for calculating both the base and the shaft resistances. Since the partial resistance factors for Design Approach 3 are equal to 1.0, the design resistance for DA3 is obtained by applying the partial material factor γ_M = 1.25 to tanϕ'_k to obtain ϕ'_d = 29.3^o *(7.6.2.3(9)P)* which is used to obtain the design resistances as shown below. It is assumed the base bearing resistance is given by $q_{bk} = \sigma_{v0}'N_q$, where σ_{v0}' is the vertical effective stress at the pile base and using the Berezantzev et al. (1961) relationship between N_q and ϕ' with N_q = 50 for ϕ'_k = 35^o and N_q = 17 for ϕ'_k = 29.3^o, and the shaft resistance is given by $q_{sk} = \Sigma\sigma_h'\tan\delta = \Sigma K_0\sigma_v'\tan\delta = 0.5(1-\sin\phi')\sigma_{v0}'\tan\phi'$, when $\delta = \phi'$ and there is a single layer of soil. In this example, $\sigma_{vo}' = 2\gamma + (L-2)(\gamma-\gamma_w)$ = 2x21.0 + (L-2)(21.0-9.91) = 19.62+11.19L.

Design Approach 1 – Combination 1

The requirement $F_{c,d} \leq R_{c,d}$ is checked for a 14.9m long pile. σ_{v0}' = (19.62 + 11.19x14.9) = 186.4 kN

$F_{c,d}$ = $\gamma_G G_k + \gamma_Q Q_k$ = 1.35 x 1200 + 1.5 x 200 = **1920 kN**

$R_{c,d}$ = $R_{b,k}/(\gamma_b \times \gamma_R) + R_{s,k}/(\gamma_b \times \gamma_R) = A_b\sigma_{v0}'N_q/(\gamma_b \times \gamma_R) + A_s0.5(1-\sin\phi')\sigma_{v0}'\tan\phi'/(\gamma_b \times \gamma_R)$

(πx0.6^2/4)x186.4x50/(1.25x1.5) + (πx0.6x14.9)x0.5x(1-sin35)x186.4xtan35/(1.0x1.5)

$R_{c,d}$ = 1405.1 + 529.1 = **1926.2** kN

Since 1920 < 1926, therefore $F_{c,d} \leq R_{c,d}$

Design Approach 1 – Combination 2

The requirement $F_{c,d} \leq R_{c,d}$ is checked for a 14.6m long pile. σ_{v0}' = (19.62 + 11.19x14.6) = 183.0 kN.

$F_{c,d}$ = $\gamma_G G_k + \gamma_Q Q_k$ = 1.0 x 1200 + 1.3 x 200 = **1460** kN

$R_{c,d}$ = $R_{b,k}/(\gamma_b \times \gamma_R) + R_{s,k}/(\gamma_b \times \gamma_R) = A_b\sigma_{v0}'N_q/(\gamma_b \times \gamma_R) + A_s0.5(1-\sin\phi')\sigma_{v0}'\tan\phi'/(\gamma_b \times \gamma_R)$

(πx0.6^2/4)x183.0x50/(1.6x1.5) + (πx0.6x14.6)x0.5(1-sin35)x183.0xtan35)/(1.3x1.5)

$R_{c,d}$ = 1077.9 + 385.6 = .**1463.5** kN

Since 1460 < 1463, therefore $F_{c,d} \leq R_{c,d}$

Design Approach 2

The requirement $F_{c,d} \leq R_{c,d}$ is checked for a 14.0m long pile. σ_{v0}' = (19.62 + 11.19x14.0) = 176.3 kN.

$F_{c,d}$ = $\gamma_G G_k + \gamma_Q Q_k$ = 1.35 x 1200 + 1.5 x 200 = **1920** kN

$R_{c,d}$ = $R_{b,k}/(\gamma_b \times \gamma_R) + R_{s,k}/(\gamma_b \times \gamma_R) = A_b\sigma_{v0}'N_q/(\gamma_b \times \gamma_R) + A_s0.5(1-\sin\phi')\sigma_{v0}'\tan\phi'/(\gamma_b \times \gamma_R)$

(πx0.6^2/4)x176.3x50/(1.1x1.5) + (πx0.6x14.0)x0.5x(1-sin35)x176.3xtan35/(1.1x1.5)

= 1510.4 + 420.9 = .**1931.**3 kN

Since 1920 < 1931, therefore $F_{c,d} \leq R_{c,d}$

Design Approach 3

The requirement $F_{c,d} \leq R_{c,d}$ is checked for a 16.7m long pile. σ_{v0}' = (19.62 + 11.19x16.7) = 205.5 kN.

$F_{c,d}$ = $\gamma_G G_k + \gamma_Q Q_k$ = 1.35 x 1200 + 1.5 x 200 = **1920** kN

$R_{c,d}$ = $R_{b,d} + R_{s,d} = A_b\sigma_{v0}'N_{qd} + A_s0.5(1-\sin\phi'_d)\sigma_{v0}'\tan\phi'_d$

(πx0.6^2/4)x205.5x17 + (πx0.6x16.7)x0.5x(1-sin29.3)x205.5xtan29.3)

$R_{c,d}$ = 992.5 + 931.3 = .**1923.8** kN   OFS = $R_{c;k}/F_k$ =

Since 1920 < 1924, therefore $F_{c,d} \leq R_{c,d}$

3. Conclusions

The ULS design pile lengths, L using ϕ'_k with the three Design Approaches are given in Table 1 in bold, together with the overall factors of safety, OFS defined as $R_{c;k}/F_k$. These show that DA1.C1 is critical for DA1; DA2 gives the least conservative design (least L and lowest OFS) while DA3 gives the most conservative design. A ULS design using the given N value with a correlation between N and the pressuremeter limit pressure, p_L and the French design rules is given by Frank (2004). Since no information is given about the pile settlement or ground stiffness, no SLS check has been carried out.

Table 1: ULS design of pile using ϕ'

Design Approach	L	OFS
DA1.C1	**14.9**	2.4
DA!.C2	14.6	2.4
DA2	**14.0**	2.3
DA3	**16.7**	2.7

References

Berezanysev V.G., Kristoforov V.S. and Golubkov V.N. (1961) Load bearing capacity and deformation of piled foundations, *Proceedings V International Conference on Soil mechanics and Foundation Engineering*, Paris, 2, 11-15

Frank, R. (2005) Evaluation of Eurocode 7 – Two pile foundation design examples, *Proceedings of International Workshop on Evaluation of Eurocode 7*, Dublin, March-April 2005, Department of Civil, Structural and Environmental Engineering, Trinity College Dublin

Model Solution for Example 4 – Pile Foundation Designed from Pile Load Tests

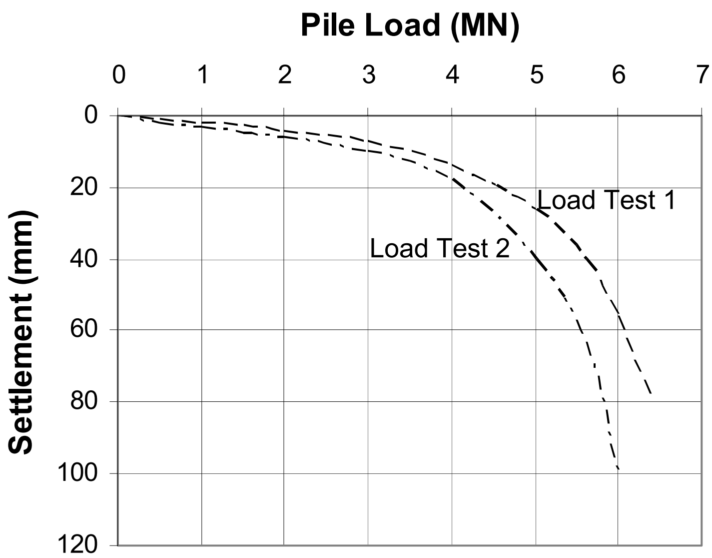
1. Description of the problem

- **Design Situation**
  - Pile group foundation, driven piles, pile diameter D = 0.4m and length = 15m. The building supported by the piles does not have the capacity to transfer the load from weak to strong piles. The allowable pile settlement is 10mm.

- **Pile Resistance**
  - 2 static pile load test results provided on driven piles of same diameter and length as design piles. Piles were loaded beyond a settlement of 0.1D = 40mm to give the limit load.

- **Characteristic values of actions**
  - Permanent vertical load G_k = 20000kN;
  - Variable vertical load Q_k = 5000kN.

- **Require number of piles needed to satisfy both ULS and SLS.**



Pile Load Test Results

Load (MN)	Settlement Pile 1(mm)	Settlement Pile 2 (mm)
0	0	0
0.5	2.1	1.2
1.0	3.6	2.1
1.5	5.0	2.9
2.0	6.2	4.1
3.0	10.0	7.0
4.0	18.0	14.0
5.0	40.0	26.0
5.6	63.0	40.0
6.0	100.0	56.0
6.4		80.0

2. Ultimate Limit State Design

2.1 – General

Compressive, i.e. bearing, resistance failure is the ultimate limit state failure mechanism considered for the design of the piles for the building. To demonstrate that the piles will support the design load, the following inequality must be satisfied *(7.6.2.1 (P))*:

$$F_{c,d} \leq R_{c,d}$$

where $F_{c,d}$ is the design action (load) and $R_{c,d}$ is the design resistance of all the piles.

The number of piles is determined for *Design Approach 1 (Combinations 1 and 2)* and *Design Approach 2*. *Design Approach 3* is not used because it is only relevant when designing using soil strength parameters, not measured resistances such as the pile resistance, because γ_R = 1 for DA3.

2.2 – Design Action

The design vertical action, $F_{c,d}$ is given by:

$$F_{c,d} = \gamma_G G_k + \gamma_Q Q_k$$

where G_k = 20000kN, Q_k = 5000kN and the values of γ_G and γ_Q depend on the Design Approach being used.

2.3 – Ultimate compressive resistance from static load test

The results of two static pile load tests are plotted in the figure above. The characteristic compressive resistance of the piles, $R_{c;k}$ is obtained using the following equation (*7.6.2.2 (8)P*):

$$R_{c;k} = \min\left\{\frac{(R_{c;m})_{mean}}{\xi_1};\frac{(R_{c;m})_{\min}}{\xi_2}\right\}$$

where $(R_{c;m})_{mean}$ and $(R_{c;m})_{min}$ are the mean and the lowest values of the measured compressive resistance, respectively and the ξ factors are correlation factors whose values depend on the number of tested piles. The measured limit loads are obtained from the pile load test results at s = 40mm, giving P_1 = 5,000 kN and P_2 = 5,600 kN. Hence the mean and lowest measured pile loads are:

$$(R_{c,m})_{mean} = (5{,}000 + 5{,}600)/2 = 5{,}300 \text{ kN}$$

$$(R_{c,m})_{min} = 5{,}000 \text{ kN}$$

Since two piles were tested, n = 2 and hence ξ_1 = 1.3 and ξ_2 = 1.2 from Table A.9 of EC7 so that

$$(R_{c,m})_{mean} / \xi_1 = 5300/1.3 = 4{,}077\text{kN}$$

$$(R_{c,m})_{min} / \xi_2 = 5000/1.2 = 4{,}167\text{kN}$$

Thus the characteristic compressive resistance is the minimum of these two values, i.e.:

$$R_{c,k} = 4{,}077\text{kN}$$

Since the piles do not transfer load from strong to weak piles, they act independently and there is no group effect. As the piles are driven piles, the design resistance of the piles is obtained by applying the partial resistance factor, γ_R from Table A.6 to $R_{c,k}$. If there are N piles, the design ultimate compressive resistance is:

$$R_{c,d} = N R_{c,k}/\gamma_R$$

2.4 – Design Approach Calculations

Design Approach 1 – Combination 1

Checking the requirement $F_{c;d} \leq R_{c;d}$ for 9 piles.

- Design vertical action:

$$F_{c,d} = \gamma_G G_k + \gamma_Q Q_k = 1.35 \times 20{,}000 + 1.5 \times 5{,}000 = 34{,}500 \text{ kN}$$

- Design compressive resistance:

$$R_{c,d} = N R_{c,k}/\gamma_R = 9 \times 4{,}077 / 1.0 = 35{,}693 \text{ kN}$$

The requirement $F_{c;d} \leq R_{c;d}$ is fulfilled as 34,500 kN ≤ 36,693 kN.

Design Approach 1 – Combination 2

Checking the requirement $F_{c;d} \le R_{c;d}$ for 9 piles.

- Design vertical action:

  $F_{c,d} = \gamma_G G_k + \gamma_Q Q_k = 1.0 \times 20{,}000 + 1.3 \times 5{,}000 = 26{,}500$ kN
- Design compressive resistance:

  $R_{c,d} = N R_{c,k}/\gamma_R = 9 \times 4{,}077 / 1.3 = 28{,}225$ kN

The requirement $F_{c;d} \le R_{c;d}$ is fulfilled as 26,500 kN ≤ 28,225 kN.

Design Approach 2

Checking the requirement $F_{c;d} \le R_{c;d}$ for 10 piles.

- Design vertical action:

  $F_{c,d} = \gamma_G G_k + \gamma_Q Q_k = 1.35 \times 20{,}000 + 1.5 \times 5{,}000 = 34{,}500$ kN
- Design compressive resistance:

  $R_{c,d} = N R_{c,k}/\gamma_R = 10 \times 4{,}077 / 1.1 = 37{,}064$ kN

The requirement $F_{c;d} \le R_{c;d}$ is fulfilled as 34,500 kN ≤ 37,064 kN.

Note:
To satisfy the ultimate limit state requirement, 10 piles are required according to Design Approach 2 and 9 piles according to Design Approach 1.

3. Serviceability Limit State Design

To check that serviceability limit state requirement is not exceeded, the following inequality must be satisfied:

$$E_d \le C_d$$

where E_d is the design value of the effect of the actions, i.e. the settlement, and C_d is the limiting design value of the effect of the action, i.e. the maximum allowable settlement which is given as 10mm.

The design value of the effect of the actions is the settlement of each pile under the characteristic load. If there are N piles, the characteristic load in each pile is found from:

$$P_k = (G_k + Q_k) /N = (5{,}000 + 20{,}000) /N$$

The SLS design settlement due to this characteristic load is obtained from the load settlement curve in the reverse of the way that the design load is determined for the ultimate limit state design; i.e. by multiplying the characteristic load, P_k by the appropriate correlation factor, ξ to obtain the design SLS load, $P_{SLS,d}$ (no partial factors are involved since in SLS the partial factor is unity):

$$P_{SLS,d} = P_{mean,d} = P_k \xi_1$$

The design SLS settlement is then obtained, in this example, from the mean pile load-settlement graph as the settlement at $P_{SLS,d}$.

For 9 piles, $P_k = (5{,}000 + 20{,}000) /9 = 2778$ kN

$$P_{SLS,d} = P_k \xi_1 = 2778 \times 1.3 = 3611 \text{ kN}$$

From the mean load-settlement curve, s_d = 11mm > 10mm, hence SLS condition is not satisfied.

For 10 piles, P_k = (5,000 + 20,000) /10 = 2500 kN

$P_{av} = P_k \xi_1$ = 2500 x 1.3 = 3250 kN

From the mean load-settlement curve, s_d = 9mm < 10mm, hence SLS condition is satisfied.

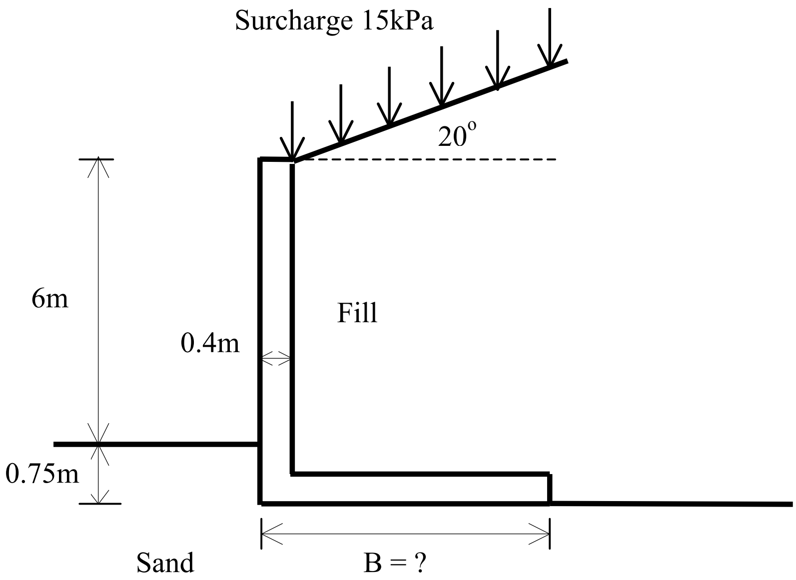
4. Conclusions

To satisfy the ultimate limit state requirement, according to Design Approach 1, 9 piles are required, while according to Design Approach 2, 10 piles are required.

For the serviceability test requirement, 10 piles are required. Thus the SLS controls the design if Design Approach 1 is used while with Design Approach 2, the ULS and SLS designs are the same.

Model Solution for Example 5 – Gravity Retaining Wall

1. Description of the problem



- **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°
  - $\gamma_{concrete}$ = 25 kN/m³ (additional information provided after the Workshop)

- **Soil Conditions:**
  - Sand beneath wall: $c'_k = 0$, $\phi'_k = 34^o$, $\gamma = 19$ kN/m³
  - Fill behind wall: $c'_k = 0$, $\phi'_k = 38^o$, $\gamma = 20$ kN/m³
  - Friction angle between base of wall and soil: $\phi'_k = 30^o$ (additional information provided after the Workshop

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

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

2. Ultimate limit state design using analytical calculations

2.1 – General

For the design of a gravity retaining wall, the following ultimate limit states need to be considered (*9.2(2)P*):

- bearing resistance failure of the soil below the base;

- failure by sliding on the base;
- failure by toppling.

Failure by toppling, i.e. overturning, is not considered to be relevant because, for a retaining wall on soil, bearing failure of the soil beneath the base will occur before overturning can occur.

The following inequality needs to be satisfied (*2.4.7.3.1(1)P*):

$$E_d \leq R_d$$

where E_d is the design value of the effect of actions and R_d is the design value of the resistance to the actions. The width of base of the retaining wall, B and the design shear force, S and bending moment, M are assessed for the three Design Approaches, using the following partial factors from EN 1997-1:

- *Table A.3* – Partial factors on actions (γ_F) or the effects of actions (γ_E);
- *Table A.4* – Partial factors for soil parameters (γ_M);
- *Table A.13* – Partial resistance factors (γ_R) for retaining structures.

2.2 – Bearing and Shearing Resistance

When checking against bearing resistance failure of the soil below the base or failure by sliding on the base, the earth pressure behind the wall is assumed to act on the vertical effective back of the wall above the back of the base, at an angle δ equal to β, the slope behind the retaining wall. Hence $\delta = \beta = 20^o$. Since for unfactored $\phi' = 38^o$, β/ϕ' = 20/38 = 0.53 and for factored $\phi' = 32^o$, β/ϕ' = 20/32 = 0.63, the values of the coefficient of the horizontal component of the active earth pressure, K_a are obtained from the graph in Figure C.1.3 of EN 1997-1 for β/ϕ' = 0.66 which for $\phi' = 38^o$ and β/ϕ' = 0.53 gives K_a = 0.26, while for $\phi' = 32^o$ and β/ϕ' = 0.63 gives K_a = 0.35. The passive pressure in front of the wall is ignored in analysing the stability of the retaining wall.

The data used to analyse the gravity retaining wall and the results obtained are presented in Tables 5.1 and 5.2. Since there is no structural loading, DA3 is the same as DA1(C2) and so is not included separately in the Table. For each Design Approach, the bearing resistance failure is checked for two positions of the variable surcharge load on the surface (loading conditions):

1) when the variable load on the surface is not above the heel but only extends as far as the effective back of the wall
2) when the variable load is on the ground surface above the wall heel.

For the first loading condition, when the variable load is not above the heel, all the loads due to the weight of the retaining wall and the fill above the wall are treated as favourable loads while the earth pressure and surcharge loads on the virtual back of the wall are treated as unfavourable loads. For the second loading condition, with the variable load above the heel, the loads due to the wall, the fill, the surcharge and the earth pressure are all treated as unfavourable loads. These loading conditions are presented in the left and right hand columns respectively of Tables 5.1 and 5.2 for each Design Approach. While the partial factors on favourable unfavourable loads are different in DA1(C1) and DA2, they are the same in DA1(C2) and hence the difference between the two columns for DA1(C2) is only due to the inclusion on the surcharge load in the right column.

The loads due to the weight of the retaining wall, the fill above the heel, the variable surcharge load and the earth pressure on the effective back of the wall are calculated for the chosen foundation widths. Then, taking moments of the design (factored) values of these forces about the centre of the base, the eccentricity of the load on the base of the wall is calculated.

In Table 5.2, the bearing resistance is calculated for each Design Approach and for both loading conditions. As when calculating the eccentricity of the load on the base, the inclination factors are calculated using design (factored) loads and soil parameters. The minimum base width has been calculated for each Design Approach and it is found that they are:

Design Approaches 1(C2) and 3:	**5.03m** with no surcharge above the heel
Design Approach 2:	**4.21m**, again for no surcharge above the toe.

Thus DA1 is more conservative than DA2 for this design situation.

The eccentricity of the load for DA1(C2) is 0.69m, so that e/B = 0.14, and for DA2 is 0.79m so that e/B = 0.19, hence, as in both Design Approaches e/B < 0.33, no special precautions are required

Table 5.1: Values used to calculate the load eccentricity and effective foundation width

Parameter	**Description/Calculation**	**DA1(C1)**		**DA1(C2) = DA3**		**DA2**	
		no q above heel W fav.	q above heel W unfav.	no q above heel W fav.	q above heel W unfav.	no q above heel W fav.	q above heel W unfav.
B	Base width	3.85	3.30	**5.03**	4.83	**4.21**	3.70
γ_G-unfav.		1.35	1.35	1.00	1.00	1.35	1.35
γ_G-fav.		1.00	1.00	1.00	1.00	1.00	1.00
γ_Q-unfav.		1.50	1.50	1.30	1.30	1.50	1.50
h_w	Wall stem height	6.35	6.35	6.35	6.35	6.35	6.35
w	Wall stem width	0.40	0.40	0.40	0.40	0.40	0.40
D	Base depth	0.40	0.40	0.40	0.40	0.40	0.40
β	Slope of ground (^{o})	20.00	20.00	20.00	20.00	20.00	20.00
H	Height of effective back: h_w+D+(B-w)tanβ	8.00	7.80	8.43	8.36	8.14	7.95
d	Soil depth before wall	0.75	0.75	0.75	0.75	0.75	0.75
γ_c	Concrete weight density	25	25	25	25	25	25
γ_f	Fill weight density	20	20	20	20	20	20
γ	Soil weight density	19	19	19	19	19	19
q_k	Characteristic surcharge	15	15	15	15	15	16
γ_ϕ		1.00	1.00	1.25	1.25	1.00	1.00
γ_c		1.00	1.00	1.25	1.25	1.00	1.00
γ_R		1.00	1.00	1.00	1.00	1.40	1.40
$\phi'_{f,k}$	Char. friction angle for fill	38	38	38	38	38	38
$\phi'_{s,k}$	Char. friction angle for sand	34	34	34	34	34	34
$\phi'_{f,d}$	Design friction angle for fill	38.00	38.00	32.01	32.01	38.00	38.00
$\phi'_{s,d}$	Design friction angle for sand	34.00	34.00	28.35	28.35	34.00	34.00
δ/ϕ	β/ϕ	0.53	0.53	0.62	0.62	0.53	0.53
K_{ad}		0.26	0.26	0.35	0.35	0.26	0.26
	Actions (Loads)						
$W_{pad,k}$	W_1 = Base characteristic weight	38.46	36.30	50.26	48.33	42.14	40.19
$W_{wall,k}$	W_2 = Wall characteristic weight	63.50	63.50	63.50	63.50	63.50	63.50
$W_{fillf,k}$ =	W_3 = Rect. part of fill char. weight	437.60	410.18	587.50	562.94	484.37	459.58
$W_{fillf,k}$ =	W_4 = Sloping fill char. weight	43.21	37.97	77.89	71.51	52.94	47.66
W_{qk}	W_5 = Characteristic surcharge load	0.00	43.48	0.00	66.49	0.00	52.72
E_h	$0.5*K_{ad}*\gamma_f*H^2$	166.57	158.38	248.95	244.81	172.20	164.30
E_{vk}	E_{hd}*tanδ	60.63	57.65	90.61	89.10	62.67	59.80
$Q_{h\text{-}vb}$	Hor surcharge load on virtual .back	31.22	30.44	44.28	43.91	31.74	33.07
$Q_{v\text{-}vb}$	Vert surcharge load on virtual back	11.36	11.08	16.12	15.98	11.55	12.04
V_k	= ΣW	654.76	607.36	885.88	917.86	717.18	682.99
V_d	$(W_1+W_2+W_3+W_4)*\gamma_G+W_5*\gamma_Q$	**681.66**	**828.12**	**890.71**	**942.60**	**744.89**	**931.75**
H	$E_h + Q_{h\text{-}vb}$	197.79	188.82	293.22	288.72	203.94	197.37
H_d	$E_h*\gamma_G + Q_{h\text{-}vb}*\gamma_Q$	**271.70**	**259.48**	**306.51**	**301.89**	**280.07**	**271.40**
H_d/V_d		**0.40**	**0.31**	**0.34**	**0.32**	**0.38**	**0.29**
	Moments about centre of base						
Mwall	$\gamma_G*W_1*(B - w)/2$	109.40	124.23	146.88	140.74	121.09	141.23
Mfill rect	$\gamma_G*W_3*w/2$	-87.52	-99.39	-117.50	-112.59	-96.87	-112.99
Mfill sloping	$\gamma_G*W_4*(B/6 + w/3)$	-33.46	-28.20	-75.63	-67.14	-44.24	-39.97
M E_h	$\gamma_G*E_h*H/3$	599.97	556.28	699.85	682.48	630.61	587.72
M E_v	$\gamma_G*E_v*B/2$	-157.38	-128.35	-227.70	-215.30	-178.27	-149.15
M q	$\gamma_G*W_5*w/2$	0.00	-13.04	0.00	-17.29	0.00	-15.82
M $Q_{h\text{-}vb}$	$\gamma_Q*Q_{hvb,k}*H/3$	187.39	178.18	242.72	238.69	193.72	197.16
M $Q_{v\text{-}vb}$	$\gamma_Q*Q_{vvb,k}*B/2$	-32.77	-27.41	-52.65	-50.20	-36.51	-33.36
M_d	ΣM	585.63	562.32	615.97	599.39	589.53	574.84
e	Load eccentricity	**0.86**	**0.68**	**0.69**	**0.64**	**0.79**	**0.62**
e/B		0.22	0.21	0.14	0.13	0.19	0.17
B' = B-2e	Effective width	2.13	1.94	3.64	3.56	2.63	2.46

Table 5.2: Values used to calculate resistances, shear force and bending moment

Parameter	Description/Calculation	DA1(C1)		DA1(C2) = DA3		DA2	
		no q above heel W fav.	q above heel W unfav.	no q above heel W fav.	q above heel W unfav.	no q above heel W fav.	q above heel W unfav.
B	Base width	3.85	3.30	**5.03**	4.83	**4.21**	3.70
	Bearing resistance						
q_k	Overburden pressure	14.25	14.25	14.25	14.25	14.25	14.25
c'_k		0	0	0	0	0	0
c'_d		0	0	0	0	0	0
Nq		29.44	29.44	15.30	15.30	29.44	29.44
Nc		42.16	42.16	26.50	26.50	42.16	42.16
Nγ		38.37	38.37	15.43	15.43	38.37	38.37
s_q		1.00	1.00	1.00	1.00	1.00	1.00
s_γ		1.00	1.00	1.00	1.00	1.00	1.00
s_c		1.00	1.00	1.00	1.00	1.00	1.00
m_B		2.0	2.0	2.0	2.0	2.0	2.0
iq		0.362	0.472	0.430	0.462	0.389	0.502
iγ		0.218	0.324	0.282	0.314	0.243	0.356
ic		0.339	0.453	0.390	0.424	0.368	0.485
R_d	Design bearing resistance	681.66	828.12	890.71	942.60	744.89	931.75
Vd	Design vertical load	681.66	828.12	890.71	942.60	744.89	931.75
Check R_d-V_d		0.362	0.472	0.430	0.462	0.389	0.502
	Sliding resistance						
$\delta_\kappa = \phi'_{s,k}$ -	base/sand friction angle	30.00	30.00	30.00	30.00	30.00	30.00
δ_δ		30.00	30.00	24.79	24.79	30.00	30.00
R_{hd}	$V_d\tan\delta/\gamma_R$	393.56	478.12	411.40	435.37	307.19	384.25
H_d		271.70	259.48	306.51	301.89	280.07	271.40
R_{hd}-H_d > 0		121.86	218.64	104.89	133.48	27.11	112.84
Maximum SF and BM in wall							
S_{ax}	$\gamma_G 0.5K_{ad}\gamma_f h_w^2 + \gamma_Q K_{ad} q_d h_w$		178.7		184.5		181.2
M_{max}	$\gamma_G(1/6)K_{ad}\gamma_f h_w^3 + \gamma_Q 0.5K_{ad} q_d h_w^2$		417.5		436.3		425.4

(*C6.5.4(1)P*). However, as e/B in DA2 is greater than 0.17, the load lies outside the middle third of the foundation and hence it is advisable to check the settlement.

The calculations to check the sliding resistance are also shown in Table 5.2. These show that, for the foundation widths obtained when designing against bearing failure, sufficient resistance is provided against sliding failure as all the R_h values well exceed the H_d values, even when assuming no passive resistance in front of the wall. As for bearing resistance, the critical condition for sliding is when there is no surcharge above the heel.

2.3 – Shear Force (S) and Bending Moment (M) in the wall

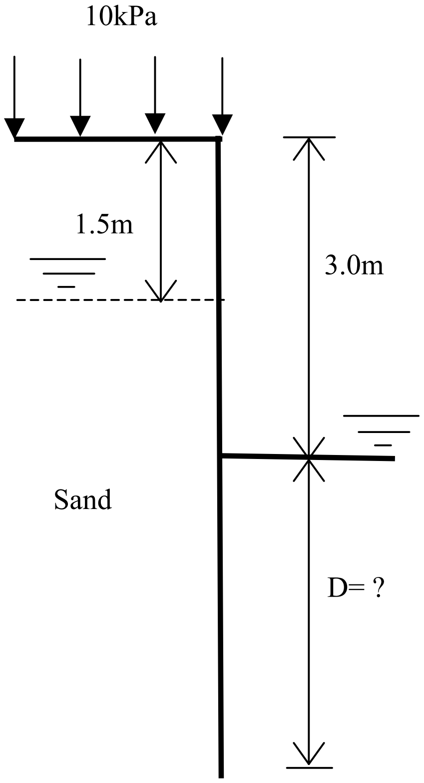
The maximum shear force, S and bending moment, M occur at the base of the wall. The greatest values occur when there is surcharge on the surface above the heel. It assumed that the maximum earth pressure for the design of the wall is the active earth pressure used to design against sliding and bearing resistance. A higher value may be chosen if the fill behind the wall is compacted and if the wall does not move sufficiently to mobilise the active pressure. Assuming active earth pressure, the maximum shear force and bending moments are calculated to be:

Design Approaches 1(C2) and 3: S_{max} = 184.5kN M_{max} = 436.3kNm
Design Approach 2: S_{max} = 181.2kN M_{max} = 425.0kNm

Thus, as in the case of the foundation width, DA1 is more conservative than DA2 for this design situation.

Model Solution for Example 6 – Embedded Retaining Wall

1. Description of the problem



- **Design situation**
  - Embedded sheet pile retaining wall for a 3m deep excavation with a surcharge on the surface behind the wall
- **Soil conditions**
  - Sand: $c'_k = 0$, $\phi'_k = 37^o$, $\gamma = 20kN/m^3$
- **Actions**
  - Characteristic variable 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

2. Ultimate Limit State Design

2.1 – General

For the design of an embedded retaining wall, the following ultimate limit states need to be considered (*9.2(2)P*):
- failure by rotation of the wall;
- failure by lack of vertical equilibrium.

Failure by rotation of the wall about the toe is considered first and then the wall is checked for vertical equilibrium.

For each failure mechanism, the following inequality needs to be satisfied (*2.4.7.3.1(1)P*):

$$E_d \leq R_d$$

where E_d is the design value of the effect of actions and R_d is the design value of the resistance to the actions. The design depth of embedment of the retaining wall, D and the design bending moment, M in the wall are assessed for Design Approaches 1, 2 and 3, using the following partial factors from EN 1997-1:
- *Table A.3* – Partial factors on actions (γ_F) or the effects of actions (γ_E);
- *Table A.4* – Partial factors for soil parameters (γ_M);
- *Table A.13* – Partial resistance factors (γ_R) for retaining structures.

Since no structural loads are involved in this example, Design Approach 3 is the same as Design Approach 1, Combination 2.

2.2 – Failure by Rotation about Toe

The depth of embedment, *d* of the wall is calculated for stability against rotation about the toe due to the moments of the horizontal earth and groundwater pressures on the wall. Then, the additional embedment depth, Δd required to provide horizontal equilibrium is calculated so that the required embedment depth is $D = d + \Delta d$. Since the characteristic friction angle for the sand, $\phi'_k = 37^o$, the design value for Design Approach 1, Combination 2 is $\phi'_d = \tan^{-1}(\tan 35/1.25) = 31.1^o$. It is assumed that, on the back and front sides of the wall, the earth pressure is at an angle $\delta = (2/3)\phi'$ so that the horizontal components of the coefficients of active and passive earth pressure, obtained from the relevant graphs in *Figures C.1.1* and *C2.1* of EN 1997-1, are: $K_a = 0.21$ and $K_p = 8.6$ for $\phi' = 37^o$, and $K_a = 0.27$ and $K_p = 5.5$ for $\phi'_d = 31.1^o$.

The data used to determine the depth of embedment for moment equilibrium are presented in Tables 6.1 and the data used to assess the horizontal and vertical stability and to determine the design maximum bending moment are presented in Table 6.2. An overdig, OD of 0.5m is assumed on the excavation side of the wall so that the ground and water levels are reduced by 0.5m on this side. Moment equilibrium was established by taking moments about the toe of the triangular and rectangular pressure diagrams due to the horizontal active and passive earth pressures, the surcharge pressure and the water pressures on the wall. The water pressure is assumed to dissipate uniformly about around the wall so that the hydraulic head at the toe, h_t is given by 2(d+h)(d-OD)/L, where d and OD are defined above, h is the depth of the excavation and L is the length of the seepage path around the wall. The depths, d required for stability with regard to moments about the toe were calculated to be:

Design Approaches 1(C1):	3.14m
Design Approaches 1(C2) and 3:	4.38m
Design Approach 2:	4.35m.

The additional depth, Δd required for horizontal equilibrium in each case was determined assuming that the active pressure behind the wall and the passive pressure in front of the wall suddenly reverse and become full passive and full active pressures, respectively for embedment depths greater than d. The calculated additional depths, shown in Table 6.2, were 0.26m for DA1(C1) and 0.34m for both DA1(C2) and DA2 so that, taking account of rounding errors, the following total design depths, D are:

Design Approaches 1(C1):	3.40m
Design Approaches 1(C2) and 3:	**4.73m**
Design Approach 2:	**4.69m**.

Thus, for the conditions in this example, Combination 2 (shown in bold) controls for DA1, and DA1 is very slightly more conservative than DA2.

Checking vertical equilibrium, the results in Table 6.2 show that for each Design Approach, the vertical resistance exceeds the vertical loading on the wall.

The maximum design bending moment is where the shear force on the wall is zero. The depth at which S = 0 is obtained for each Design Approach, as shown in Table 6.2. For Design Approach 1, the depth where S= 0 is calculated using the design Combination 2 depth of 4.73m for both Combinations 1 and 2. The maximum bending moments are:

Design Approaches 1(C1):	128 kNm
Design Approaches 1(C2) and 3:	**163 kNm**
Design Approach 2:	**177 kNm**.

The maximum bending moment for DA1(C1) for the embedment length of 3.40m is not relevant since the design embedment length of the wall for DA1 is 4.73m given by DA1(C2),. Since the wall is not at limiting equilibrium for DA1(C1) when the wall has the DA1(C2) design embedment length of 4.73m, an interaction analysis using a spring model for the soil or a finite element analysis to redistribute the earth pressures is required to determine the maximum bending moment in the wall for DA1(C1). This has not been carried out in this example

Table 6.1: Values used to calculate the embedment depth for moment stability

Parameter	Description/Calculation	DA1(C1)	DA1(C1) with DA1(C2)_ length	DA1(C2) = DA3	DA2
Geometry, Weight Densities, Surcharge Loading and Head at Toe					
d	Depth required for moment equilibrium	3.14	4.38	4.38	4.35
h	Height of excavation	3	3	3	3
OD	Overdig allowance in front of wall	0.3	0.3	0.3	0.3
j	height of GWL behind wall above excavation	1.50	1.50	1.50	1.50
d_{gwl}	depth of GWL below surface behind wall	1.50	1.50	1.50	1.50
γ_1	Weight density soil above GWL	20.0	20.0	20.0	20.0
γ_2	Weight density soil below GWL	20.0	20.0	20.0	20.0
γ_w	Weight density water	9.81	9.81	9.81	9.81
q_k	Characteristic surcharge	10	10	10	10
L	Seepage path length L = 2d+j-OD	7.49	9.97	9.97	9.91
h_t	Head at toe (assume linear dissipation) h_t = 2(d+j)(d-OD)/L	3.53	4.82	4.82	4.79
Partial Factors					
$\gamma_{G,unfav}$		1.35	1.35	1	1.35
$\gamma_{G,fav}$		1	1	1	1
$\gamma_{Q,unfav}$		1.5	1.5	1.3	1.5
γ_{ϕ}		1.0	1.0	1.25	1.0
γ_R		1	1	1	1.4
Friction angles and Coefficients of Horizontal Earth Pressure					
ϕ'_k	Characteristic soil friction angle	37	37	37	37
ϕ'_d	Design soil friction angle	37	37	31.1	37
δ_{ad}	(2/3)ϕ'_d	24.7	24.7	20.7	24.7
δ_{pd}	(2/3)ϕ'_d	24.7	24.7	20.7	24.7
K_{ad}	Design horizontal coeffcient of active earth pressure	0.21	0.21	0.27	0.21
K_{pd}	Design horizontal coeffcient of passive earth pressure	8.6	8.6	5.5	8.6
Actions - Horizontal Earth and Water Pressure Forces on Wall					
Pah-soil-1	$P1 = \gamma_{Gunfav}*0.5*K_{ad}*\gamma_1*d_{gwl}^2$	6.38	6.38	6.08	6.38
Pah-soil-2	$P2 = \gamma_{Gunfav}*K_{ad}*\gamma_1*d_{gwl}*(d+j)$	39.49	50.05	47.66	49.79
Pah-soil-3	$P3 = \gamma_{Gunfav}0.5*K_{ad}*(\gamma2'*(d+j)-\gamma_w*h_t)*(d+j)$	38.34	58.71	55.92	58.16
Pqhd	$P4 = \gamma_{Q,unfav}*K_{ad}*qk*(h+d)$	19.35	23.26	25.92	23.16
Pawd	$P5 = \gamma_{Gunfav}*0.5*\gamma w*ht*(d+j)$	108.41	187.90	139.18	185.69
Pphd:DA1&3	$P6:DA13 = \gamma_{Gunfav}*0.5*K_{pd}*(\gamma2*(d-OD)-\gamma_w*h_t)*(d-OD)$	367.37	815.34	386.25	
Pphd:DA2	$P6:DA2 = 0.5*K_{pd}*(\gamma_2*(d-OD)-\gamma_w*h_t)*(d-OD)/\gamma_R$				424.39
Ppwd	$P7 = \gamma_{Gunfav}*0.5*\gamma_w*h_t*(D-OD)$	66.38	130.42	96.61	128.59
Lever Arms					
l1	l1 = d+h-2*dgwl/3	5.14	6.38	6.38	6.35
l2	l2 = 0.5*(d+j)	2.32	2.94	2.94	2.93
l3	l3 = (d+j)/3	1.55	1.96	1.96	1.95
l4	l4 = 0.5*(d+h)	3.07	3.69	3.69	3.68
l5	l5= (d+j)/3	1.55	1.96	1.96	1.95
l6	l6 = (d-OD)/3	0.95	1.36	1.36	1.35
l7	l7 = (d-OD)/3	0.95	1.36	1.36	1.35
Moments about Toe					
Ma	P1*l1+P2*l2+P3*l3	183.81	303.15	288.71	299.73
Mq	P4*l4	59.43	85.89	95.70	85.18
Mp:DA1&3	P6:13*l6	348.11	1110.10	525.89	
Mp:DA2	P6:2*l6				573.48
Mw	P5*l5-P7*l7	104.87	190.99	141.48	188.57
Moment equilibrium					
ΣM-DA13	Mp:DA13-(Ma+Mq+Mw)	0.00	530.08	0.00	
ΣM-DA2	Mp:DA2-(Ma+Mq+Mw)				0.00
d	*Embedment depth required for moment equilibrium*	*3.14*	*4.38*	*4.38*	*4.35*

Table 6.2: Values used to calculate design embedment depth and maximum bending moment

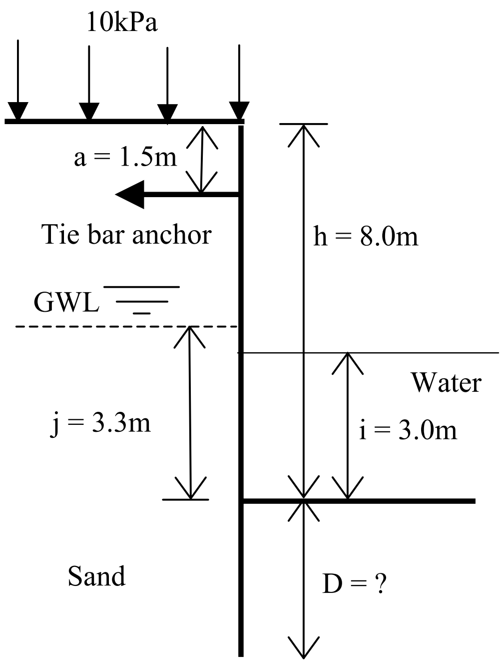
Parameter	Description/Calculation	DA1(C1)	DA1(C1) with DA1(C2)_ length	DA1(C2) = DA3	DA2
Horizontal Equilibrium					
Δd	Additional embedment required for horizontal equilibium	0.26	0.34	0.34	0.34
	Additional depth as percentage of d = Δd*100/d	8.33	7.84	7.84	7.82
$h_{t-d+\Delta d}$	Head at d+Δd – assuming linear dissipation	3.80	5.18	5.18	5.14
h_{t-mida}	Head at middle of add. depth on active side (behind wall)	3.70	5.03	5.03	5.00
h_{t-midp}	Head at middle of add. depth on passive side (front of wall)	3.64	4.98	4.98	4.94
ΔPpsoil:DA13	P81:DA13 = $\gamma_{G,fav}K_{pd}(\gamma_1 d_{gwl}+\gamma_2*(d+j+\Delta d/2)-\gamma_{w*}h_{t-mida})\Delta d$	200.82	300.80	192.37	296.95
ΔPpq:DA13	P82:DA13 = $\gamma_{Q,fav}*K_{pd}*q_k*\Delta d$	22.52	29.56	18.91	29.28
ΔPpsoil:DA2	P81:DA2 = $K_{pd}*(\gamma_1*d_{gwl}+g2*(d+j+\Delta d/2)-\gamma_w*h_{t-mida})*\Delta d/\gamma_R$	200.82	300.80	192.37	212.11
ΔPpq:DA2	P82:DA2 = $K_{pd}*q_k*\Delta d/\gamma_R$	22.52	29.56	18.91	20.91
ΔPa soil front	P9 = $\gamma_{G,unfav}*K_{pa}(\gamma_2'*(d-OD+\Delta d/2)-\gamma_w*h_{t-midp})*\Delta d$	1.76	3.54	3.37	3.47
ΔPawd-ext	P10 = $\gamma_{G,unfav}*0.5*\gamma_w*h_{t-mide}*\Delta d$	12.83	22.91	16.97	22.54
ΔPpwd-ext	P11= $\gamma_{G,unfav}*0.5*\gamma_w*h_{t-midp}*\Delta d$	12.63	22.65	16.78	22.28
Check Horizontal Equilibrium					
ΣH = 0:DA13	Σ(P1:P5)-P6:DA13-P7+P81:DA13+P82:DA13+P10-P9-P11	0.00	-292.38	0.00	
ΣH = 0:DA2	Σ(P1:P5)-P6:DA2-P7+P81:DA2+P82:DA2+P10-P9-P11				0.00
D	**Design Embedment Depth D** = d + Δd	3.40	4.73	**4.73**	**4.69**
Check Vertical Equilibrium					
ΣV = 0:DA13	(P6+P81+P82):DA13* $\tan\delta_{pd}$ – (Σ(P1 to P4)+P9)*$\tan\delta_{ad}$ -	222.91	460.97	173.49	
ΣV = 0:DA2	(P6+P81+P82):DA2* $\tan\delta_{pd}$ – (Σ(P1 to P4)+P9)*$\tan\delta_{ad}$				237.18
Find where S =0 to locate maximum bending moment, M_{max}					
d_s	Depth where shear force = 0	1.83	1.80	2.61	2.61
Pah-soil-1	$P1_M = \gamma_{G,unfav}*0.5*K_{ad}*\gamma_1*d_{gwl}^2$	6.38	6.38	6.08	6.38
Pah-soil-2	$P2_M = \gamma_{G,unfav}*K_{ad}*\gamma_1*d_{gwl}*(d_s+j)$	28.30	28.03	33.27	34.92
Pah-soil-3	$p3_M = \gamma_{G,unfav}*0.5*K_{ad}*(\gamma_2*(d_s+j)-\gamma_w*(d_s+j)*h_t/(d+j))*(d_s+j)$	19.70	18.42	27.24	28.61
Pqh	$P4_M = \gamma_{Q,unfav}*K_{ad}*qk*(h+d_s)$	15.21	15.11	19.68	17.66
Paw	$P5_M = \gamma_{G,unfav}*0.5*\gamma_w*h_t*(d_s+j)^2/(d+j)$	55.70	58.95	67.80	91.35
Pph:DA13	$P6{:}DA13_M = \gamma_{G,unfav}0.5K_{pd}(\gamma_2(d_s-OD)-\gamma_w(d_s-OD)h_t/(d-OD))(d_s-OD)$	106.11	109.40	123.24	259.53
Pph:DA2	$P6{:}DA2_M = 0.5K_{pd}(\gamma_2(d_s-OD)-\gamma_w(d_s-OD)h_t/(d-OD))(d_s-OD)/\gamma_R$	78.60	81.03	123.24	137.32
Ppw	$P7_M = \gamma_{G,unfav}*0.5*\gamma_w*h_t*(d_s-OD)^2/(d-OD)$	19.17	17.50	30.83	41.61
ΣH = 0:DA13	$P1_M + P2_M + P3_M + P4_M + P5_M$ - P6:$DA13_M$ - $P7_M$	0.00	0.00	0.00	
ΣH = 0:DA2	$P1_M + P2_M + P3_M + P4_M + P5_M$ - P6:$DA2_M$ - $P7_M$				0.00
Lever Arms for M_{max}					
l1	$l1=d_s+h-2*d_{gwl}/3$	3.83	3.80	4.61	4.61
l2	$l2 = 0.5*(d_s+j)$	1.66	1.65	2.05	2.05
l3	$l3 = (d_s+j)/3$	1.11	1.10	1.37	1.37
l4	$l4 = 0.5*(d_s+h)$	2.41	2.40	2.80	2.80
l5	$l5= (d_s+j)/3$	1.11	1.10	1.37	1.37
l6	$l6 = (d_s-OD)/3$	0.51	0.50	0.77	0.77
l7	$l7 = (d_s-OD)/3$	0.51	0.50	0.77	0.77
Maximum Bending Moment - M_{max}					
Ma +Mq	Ma = $P1_M$ *l1+ $P2_M$ *l2+$P3_M$ *l3+$P4_M$ *l4	130.07	126.89	188.78	189.73
Mp:DA13+pw	Mp = $P6_M$ -13*l6	54.04	54.56	94.79	199.48
Mp:DA2	Mp = $P6_M$ -2*l6	40.03	40.41	94.79	105.55
Mw	$P5_M$ *l5-P_M 7*l7	52.02	56.05	69.12	93.05
Mmax:DA13	Ma+Mq-Mp:DA13	128.05	128.37	**163.12**	
Mmax:DA2	Ma+Mq-Mp:DA2				**177.23**

3. Conclusions

Thus, for the conditions in this example, DA1(C2) Combination 2 gives the design length for DA1, which is 4.73m, and the design maximum bending moment is 163 kNm. The design length for DA2 is 4.69m, which is slightly less than the DA1 length, i.e less conservative, while the maximum design bending moment is 177 kNm, which is slightly greater, i.e. slightly more conservative.

Model Solution for Example 7 – Anchored Retaining Wall

1. Description of the problem



- **Design situation**
  - Anchored sheet pile retaining wall for an 8m high quay using a horizontal tie bar anchor.
- **Soil conditions**
  - Gravelly sand - $\phi'_k = 35^o$, $\gamma = 18kN/m^3$ (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

2. Ultimate Limit State Design

2.1 – General

For the design of an embedded retaining wall, the following ultimate limit states need to be considered (*9.2(2)P*):

- failure by rotation of the wall;
- failure by lack of vertical equilibrium.

Failure by rotation of the wall about the anchor is considered first and then the wall is checked for vertical equilibrium.

For each failure mechanism, the following inequality needs to be satisfied (*2.4.7.3.1(1)P*):

$$E_d \leq R_d$$

where E_d is the design value of the effect of actions and R_d is the design value of the resistance to the actions. The design depth of embedment of the retaining wall, D and the design bending moment, M in the wall are assessed for Design Approaches 1, 2 and 3, using the following partial factors from EN 1997-1:

- *Table A.3* – Partial factors on actions (γ_F) or the effects of actions (γ_E);
- *Table A.4* – Partial factors for soil parameters (γ_M);

– *Table A.13* – Partial resistance factors (γ_R) for retaining structures.

Since no structural loads are involved in this example, Design Approach 3 is the same as Design Approach 1, Combination 2.

2.2 – Failure by Rotation

The depth of embedment, *D* of the wall is calculated for stability against rotation is examined by considering moments due to the earth, surcharge, groundwater and free water pressures on the wall about the anchor. A limiting equilibrium model is used, assuming free earth support at the toe with full active pressure behind the wall and full passive pressure in front of the wall. An overdig, OD of 0.5m is assumed on the excavation side of the wall so that the ground and water levels are reduced by 0.5m on this side. The groundwater level is at a height, j above the excavation level behind the wall and in front of the wall the water is at a height i above the excavation level. The difference in hydraulic head between the two sides of the wall is assumed to dissipate uniformly about around the wall so that the hydraulic head at the toe, h_t is given by (2D+j)(D+i-OD)/L, where L is the length of the seepage path around the wall. It is assumed that the earth pressure is at an angle $\delta = 0.5\ \phi'$ to the normal on the rear of the wall and = (2/3) ϕ' on the front of the wall. Since $\phi'_k = 35^o$, for Design Approach 1, Combination 2, $\phi'_d = \tan^{-1}(\tan35/1.25) = 29.3^o$. Hence, the horizontal components of the coefficients of active and passive earth pressure, obtained from the relevant graphs in *Figures C.1.1* and *C2.1* of EN 1997-1, are: $K_a = 0.24$ and $K_p = 7.1$ for $\phi' = 35^o$, and $K_a = 0.30$ and $K_p = 4.8$ for $\phi'_d = 29.3^o$.

The data used to determine the depth of embedment for moment equilibrium are presented in Table 6.1 and the data used to assess the vertical stability and calculate the depth at which the shear force is zero, and hence determine the design maximum bending moment, are presented in Table 6.2. Moment equilibrium was established by taking moments about the anchor of the triangular and rectangular pressure diagrams due to the horizontal active and passive earth pressures, the surcharge pressure and the water pressures on the wall. The depths, D required for stability against rotation were calculated to be:

Design Approaches 1(C1):	2.60m
Design Approaches 1(C2) and 3:	**3.64**m
Design Approach 2:	**3.67**m.

Thus, for the conditions in this example, Combination 2 (shown in bold) controls for DA1, and DA2 is very slightly more conservative than DA2, i.e. gives a slightly longer wall.

2.2 – Vertical Equilibrium

Checking vertical equilibrium, it can be seen from the figures in Table 6.2 that the vertical downward earth pressure force on the wall exceeds the upward resisting force by 10.3kN, i.e. 12%, in the case of DA1(C2) and by 10.8kN, i.e. 10%, in the case of DA2. However, vertical equilibrium may be ensured either by taking account of the end bearing of the sheet piles or by reducing the angle δ on the active side of the wall slightly more below $0.5\phi'$ as noted in *9.4.1(3)*.

The maximum design bending moment in the wall occurs where the shear force, S is zero. The depth at which S = 0 is obtained for each Design Approach, as shown in Table 6.2. The maximum bending moments for the design embedment lengths obtained for the Design Approaches are:

Design Approaches 1(C2) and 3:	**341 kNm**
Design Approach 2:	**370 kNm**.

Thus, for the conditions in this example, The DA2 maximum bending moment is slightly greater than the DA1 design bending moment.

The maximum bending moment for DA1(C1) for the embedment length of 2.60m is not relevant since the design embedment length of the wall for DA1 is 3.64m given by DA1(C2),. Since the wall is not at limiting equilibrium for DA1(C1) when the wall has the DA1(C2) design embedment length of 3.64m, an interaction analysis using a spring model for the soil or a finite element analysis to redistribute the earth pressures is required to determine the maximum bending moment in the wall for DA1(C1). This has not been carried out in this example.

The design anchor forces are calculated to be 134kN for DA1 and 143kN for DA2.

Table 6.1: Values used to calculate the embedment depth for moment stability

Parameter	Description/Calculation	DA1(C1)	DA1(C2) = DA3	DA2
Geometry, Weight Densities, Surcharge Loading and Head at Toe				
D	Embedment Depth	2.60	**3.64**	**3.67**
h	Height of excavation	8	8	8
OD	Overdig allowance in front of wall	0.5	0.5	0.5
j	Height of free water above excavation	3.00	3.00	3.00
j	Height of GWL behind wall above excavation	3.30	3.30	3.30
d_{gwl}	Depth of GWL below surface behind wall	4.70	4.70	4.70
a	Depth of anchor below gound level behind wall	1.50	1.50	1.50
γ_1	Weight density soil above GWL	20.0	20.0	20.0
γ_2	Weight density soil below GWL	20.0	20.0	20.0
γ_w	Weight density water	9.81	9.81	9.81
q_k	Characteristic surcharge	10	10	10
L	Seepage path length L = 2d+j-OD	8.00	10.08	10.15
h_t	Head at toe (assume linear dissipation) h_t = 2(D+j)(D-OD)/L	5.68	6.73	6.77
Partial Factors				
$\gamma_{G,unfav}$		1.35	1	1.35
$\gamma_{G,fav}$		1	1	1
$\gamma_{Q,unfav}$		1.5	1.3	1.5
γ_{ϕ}		1.0	1.25	1.0
γ_R		1	1	1.4
Friction angles and Coefficients of Horizontal Earth Pressure				
ϕ'_k	Characteristic soil friction angle	35	35	35
ϕ'_d	Design soil friction angle	35.00	29.26	35.00
δ_{ad}	(2/3)ϕ'_d	17.5	14.6	17.5
δ_{pd}	(2/3)ϕ'_d	23.3	19.5	23.3
K_{ad}	Design horizontal coeffcient of active earth pressure	0.24	0.3	0.24
K_{pd}	Design horizontal coeffcient of passive earth pressure	7.1	4.8	7.1
Actions - Horizontal Earth and Water Pressure Forces on Wall				
Pah-soil-1	$P1 = \gamma_{Gunfav}*0.5*K_{ad}*\gamma_1*d_{gwl}^2$	64.41	59.64	64.41
Pah-soil-2	$P2 = \gamma_{Gunfav}*K_{ad}*\gamma_1*d_{gwl}*(D+j)$	161.66	176.08	191.14
Pah-soil-3	$P3 = \gamma_{Gunfav}0.5*K_{ad}*(\gamma2'*(D+j)-\gamma_w*h_t)*(D+j)$	59.50	75.68	82.55
Pqhd	$P4 = \gamma_{Q,unfav}*K_{ad}*qk*(h+D)$	38.15	45.39	42.02
Pawd	$P5 = \gamma_{Gunfav}*0.5*\gamma w*ht*(D+j)$	221.69	229.07	312.46
Pphd:DA1&3	$P6:DA13 = \gamma_{Gunfav}*0.5*K_{pd}*(\gamma2*(D-OD)-\gamma_w*h_t)*(D-OD)$	207.16	233.90	
Pphd:DA2	$P6:DA2 = 0.5*K_{pd}*(\gamma_2*(D-OD)-\gamma_w*h_t)*(D-OD)/\gamma_R$			252.76
Pphwater1	$P71= \gamma Gunfav*0.5*\gamma w*(i+OD)^{2}$	81.12	60.09	81.12
Pphwater2	$P72= \gamma Gunfav*\gamma w*(i+OD)*(D-OD)$	97.24	107.74	147.08
Pphwater3	$P73= \gamma Gunfav*0.5*\gamma w*(ht-i-OD)*(D-OD)$	30.23	49.73	68.64
Lever Arms				
l1	l1=(2/3)dgwl-a	1.63	1.63	1.63
l2	l2 = 0.5*(D+j)+dgwl-a	6.15	6.67	6.69
l3	l3 =(2/3)(D+j)+dgwl-a	7.13	7.83	7.85
l4	l4 = 0.5*(D+h)-a	3.80	4.32	4.34
l5	l5= (2/3)(D+j)+dgwl-a	7.13	7.83	7.85
l6	l6 = (2/3)(D-OD)+OD+h-a	8.40	9.09	9.12
l71	l71 = (i+OD)/3+h-i-a	5.83	5.83	5.83
l72	l72 = 0.5*(D-OD)+OD+h-a	8.05	8.57	8.59
l73	l73 = (2/3)(D-OD)+OD+h-a	8.40	9.09	9.12
Moments about Anchor				
Ma+Mq	P1*l1+P2*l2+P3*l3+P4*l4	1668.50	2059.97	2213.44
Mp:DA1&3	P6:13*l6	1739.80	2126.59	
Mp:DA2	P6:2*l6			2304.03
Mw	P5*l5-P7*l7	71.30	66.63	90.59
Moment equilibrium				
ΣM:DA13	Mp:DA13-(Ma+Mq+Mw)	0.00	0.00	
ΣM:DA2	Mp:DA2-(Ma+Mq+Mw)			0.00

Table 6.2: Values used to check vertical equilibrium and determine maximum design bending moment

Parameter	Description/Calculation	DA1(C1)	DA1(C2) = DA3	DA2
D	Embedment Depth	2.60	**3.64**	**3.67**
Check Vertical Equilibrium		-12.71	-10.28	
$\Sigma V = 0$:DA13	P6:DA13* $\tan\delta_{pd}$ – Σ(P1 to P4)* $\tan\delta_{ad}$	-12.71	-10.28	
$\Sigma V = 0$:DA2	P6:DA2* $\tan\delta_{pd}$ –Σ(P1 to P4)* $\tan\delta_{ad}$			-10.82
Horizontal Equilibrium for Anchor Force				
Pa:DA13	Anchor force = P1+P2+P3+P4+P5-P6:DA13-P71-P72-P73	129.67	**134.41**	
Pa:DA2	Anchor force = P1+P2+P3+P4+P5-P6:DA13-P71-P72-P73			**142.98**
Find where S =0 to locate maximum bending moment, M_{max}				
d_s	Depth where shear force = 0	6.03	6.35	6.36
Pah-soil-1	$P1_M$ = γGunfav*0.5*Kad*γ1*dgwl^2	1.28	1.60	1.61
Pah-soil-2	$P2_M$ = γGunfav*Kad*γ1*dgwl*(ds-dgwl)	1.03	1.35	3.60
Pah-soil-3	$P3_M$ = γGunfav*0.5*Kad*(γ2*(ds-dgwl)-gw*hsb)*(ds-dgwl)	64.41	59.64	64.41
Pqh	$P4_M$ = γQunfav*Kad*qk*ds	36.33	41.76	45.52
Paw	$P5_M$ = γGunfav*0.5*γw*ht*(ds-dgwl)	3.00	4.26	4.68
Pphwater1	$P71_M$ = γGunfav*0.5*γw*(ds-dgwl-(j-i))^2	6.96	8.88	12.26
S	S:DA13=P1+P2+P3+P4+P5-P6:DA13-Pa:DA13	0.00	0.00	
	S-DA2			0.00
Lever Arms for M_{max}				
l1	ds-2*dgwl/3	2.89	3.21	3.23
l2	(ds-dgwl)/2	0.66	0.82	0.83
l3	(ds-dgwl)/3	0.44	0.55	0.55
l4	ds/2	3.01	3.17	3.18
l5	(ds-dgwl)/3	0.44	0.55	0.55
l7	(ds-dgwl-(j-i))/3	0.34	0.45	0.45
la	ds-a	4.53	4.85	4.86
Maximum Bending Moment - M_{max}				
Mmax:DA13	$P1_M$*l1+$P2_M$*l2+$P3_M$*l3+$P4_M$*l4+$P5_M$ *l5-$P7_M$*l7-Pa:DA13*la	-307.2	**-341.4**	
Mmax:DA2	$P1_M$*l1+$P2_M$*l2+$P3_M$*l3+$P4_M$*l4+$P5_M$ *l5-$P7_M$*l7-Pa:DA13*la			**-369.6**

3. Conclusions

Thus, for the conditions in this example, the design embedment lengths are:

(Design Approaches 1(C1): 2.60m)
Design Approaches 1(C2) and 3: **3.64**m
Design Approach 2: **3.67**m.

The design maximum bending moments are:

Design Approaches 1(C2) and 3: **341 kNm**
Design Approach 2: **370 kNm**.

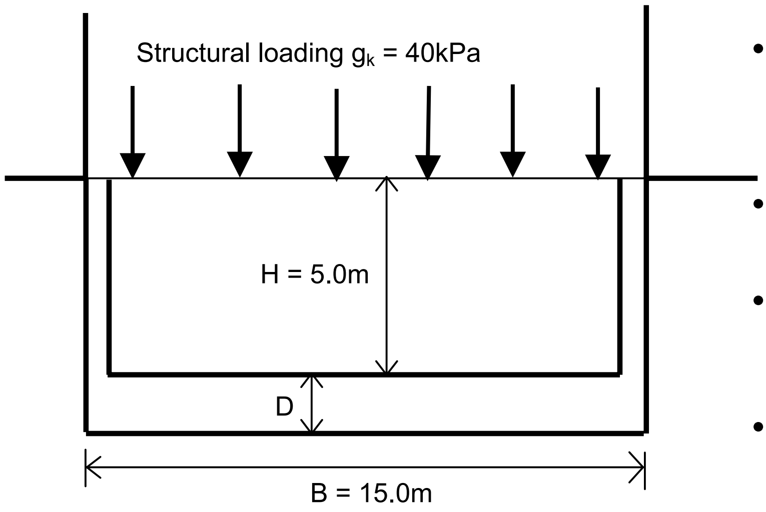
The design anchor forces are:

Design Approaches 1(C2) and 3: **134 kN**
Design Approach 2: **143 kN**.

In this design example, the design embedment length of 3.67m obtained using Design Approach 2 is slightly greater than the design embedment length of 3.64m obtained using Design Approaches 1 and 3. For the design conditions in the GEO examples, this is the only example where Design Approach 2 is more conservative than Design Approach 1.

Model Solution for Example 8 – Uplift of a Deep Basement

1. Description of the problem



- **Design situation**
  - Long structure, 15m wide, with a 5m deep basement
  - Groundwater level can rise to the ground surface
- **Soil Conditions**
  - Sand – $c'_k = 0$, $\varphi'_k = 35^o$, $\gamma = 20 kN/m^3$ (below water table)
- **Actions**
  - Characteristic structural loading g_k = 40kPa
  - Concrete weight density $\gamma = 24 kN/m^3$
  - Wall thickness = 0.3m
- **Require**
  - Thickness of base slab, D for safety against uplift

2. Ultimate Limit State Design

According to Eurocode 7, designing against the ultimate limit state of uplift (UPL) is carried out by checking that the design value of the combination of destabilising permanent and variable vertical actions, $V_{dst;d}$ is less than or equal to the sum of the design value of the stabilising permanent vertical actions, $G_{stb;d}$ and the design value of any additional resistance to uplift, R_d (*2.4.7.4(1)*):

$$V_{dst,d} \leq G_{stb,d} + R_d \qquad (2.8)$$

where

$$V_{dst,d} \leq G_{dst,d} + Q_{dst,d}$$

This general inequality is applied when checking the stability of submerged structures against uplift failure and the stability against uplift of impermeable layers in excavations.

According to *Clause 2.4.7.4(2)*, the additional resistance, R_d may also be treated as a stabilising permanent vertical action, $G_{stb;d}$. The required thickness of the base slab, D is assumed to be 0.60m.

3. Destabilising Vertical Action

Assuming the groundwater level is at the ground surface and treating the upward water pressure on the bottom of the basement as a permanent action *(Clause 2.4.2(16)P)*, and, as there is no variable destabilising action so that $Q_{dst,d} = 0$, the design value of the destabilising vertical action per metre length of the basement for D = 0.56 m is given by:

$$V_{dst,d} = G_{dst,d} = \gamma_{G,dst}\, \gamma_w\, (H + D)\, B = 1.0 \times 9.81 \times (5 + 0.60) \times 15 = \mathbf{824.0\ kN/m}$$

where the weight density of the water = 9.81 kN/m^3.

4. Value of the Stabilising Vertical Action

The design value of the stabilising permanent vertical action is given by the sum of the design value of the stabilising permanent vertical action, $G_{stb;d}$ and the additional resistance to uplift, R_d. due to friction on the sides of the basement. The stabilising permanent vertical action is given by:

$$G_{stb,d} = \gamma_{G,stb}\, (\gamma_{c,k}\, (2\, t\, H + B\, D) + g_k\, B)$$

where $\gamma_{c,k}$ is the characteristic weight density of the concrete and *t* is the thickness of the basement walls. Hence, since the partial factor on weight density is unity:

$$G_{stb,d} = 0.9 \times (24 \times (2 \times 0.3 \times 5 + 15 \times 0.60) + 40 \times 15) = 0.9 \times (72 + 216 + 600) = \mathbf{799.2\ kN/m}$$

The additional resistance between the soil and basement side walls, R_k may be obtained by assuming that $R = 2 (H + D) K\sigma_v' \tan\delta$, where *K* is the lateral earth pressure, $\sigma_v' = 0.5(H + D)(\gamma - \gamma_w) = 0.5(5 + 0.60)(20 - 9.81) = 28.5$ kPa is the vertical effective stress at the mid-height of the side walls and δ is the angle of shearing resistance between the soil and the side walls. Taking a conservative approach, it is assumed that $K_k = K_{ak}$. Also assuming that $\delta = (2/3)\phi'_k$, the value of K_{ak} for 35^o from Figure C1.1 is = 0.24. Hence the characteristic resistance, R_k is:

$$R_k = 2 (H + D) K_{ak}\, \sigma_v' \tan\delta_k, = 2 (5 + 0.60)\ 0.24 \times 28.5 \times \tan 23.3 = 33.0 \text{ kN/m}$$

Applying the partial factor γ_M to reduce $\tan\phi'_k$ gives $\phi'_d = \tan^{-1}(\tan 35/1.25) = 29.3^o$, $K_{ad} = 0.30$ and $\delta_d = (2/3)29.3 = 19.5^o$. Using these ϕ'_d, K_{ad} and δ_d values to obtain R_d *gives:*

$$R_d = 2 (H + D) K_d\, \sigma_v' \tan\delta_d, = 2 (5 + 0.6)\ 0.30 \times 28.5 \tan 19.5 = 33.9 \text{ kN/m}$$

Hence, for this design situation, applying the partial factor γ_M to reduce $\tan\phi'_k$ provides no margin of safety on R_d because, although $\tan\delta$ is reduced, *K* is increased with the result that R_d is slightly greater than R_k. For this reason, as recommended by Frank et al. (2004), R_k has been treated as a favourable action, in accordance with *Clause 2.4.7.4(2)*, and, using $1/\gamma_M$ as a partial action factor, R_d *is* calculated as follows:

$$R_d = R_k/\gamma_M = 33.0/1.25 = \mathbf{26.4\ kN/m}$$

5. Stability against Uplift

The stability of the basement against uplift is checked by substituting the values above in Equation 2.8:

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

$$V_{dst,d} = 824.0 \ \mathbf{\leq}\ G_{stb,d} + R_d = 799.2 + 26.4 = 825.6$$

Since $V_{dst,d} \leq G_{stb,d} + R_d$, the basement is stable against uplift.

Hence for the requirement for stability against uplift is satisfied if **D ≥ 0.60m**.

Note

1. As R_d = 26.4 kN/m while the destabilising vertical action $V_{dsb,d}$ = 824.0 kN/m, in this example, R_d provides only 3.2 % of the total design stabilising action. Thus the stability is almost entirely determined by the weight of the basement.

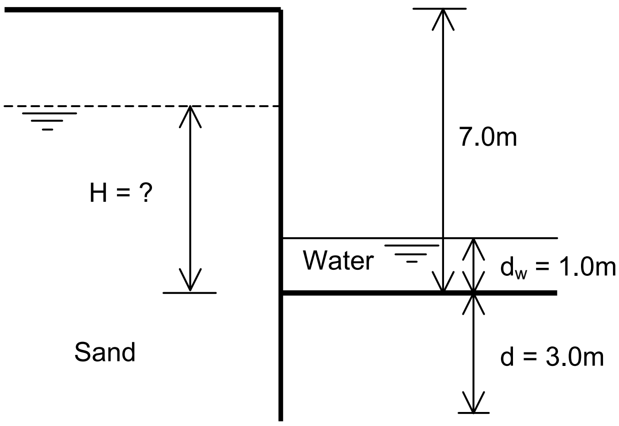
2. If the side resistance is ignored, then the thickness of the base needs to be increased to 0.74m, i.e. it needs to be increased by 23.4% more than the thickness obtained when the side resistance is included. This indicates that, in this example, the thickness of the base is sensitive with regard to whether the soil resistance on the side walls is ignored or included.

References

Frank, R. et al. (2004) *Designers' Guide to EN 1997 -1 Eurocode 7: Geotechnical Design - General Rules,* Thomas Telford Ltd, London,

Model Solution for Example 9 – Failure by Hydraulic Heave

1. Description of the problem



- **Design situation**
  Seepage around an embedded sheet pile retaining wall
- **Soil conditions**
  Sand: $\gamma = 20$ kN/m^3
- **Actions**
  Groundwater level 1,0 m above ground surface in front of wall
- **Require**
  Maximum height *H* of water behind the wall, above ground surface in front of the wall, to ensure safety against hydraulic heave

2. Ultimate Limit State Design

The objective in this example is to determine H, the height of the groundwater level behind the wall to avoid heave failure in front of the wall. It is assumed that the ground consists of ideal uniform sand with uniform permeability. In practice, however, when heave failure is a real danger, this will not be the case and the designer will need to assess the most unfavourable combination of soil layers and pore pressures that could occur *(2.4.6.1(6)P)* and hence assess the appropriate design values for the hydraulic gradient, pore-water pressures and seepage forces *(10.1(3)P).*

According to Eurocode 7, design against hydraulic heave failure (HYD) may be carried out using either *Equation 2.9a* or *Equation 2.9b*. Considering first *Equation 2.9b*, which states that the design destabilising seepage force, $S_{dst,d}$ on the column of soil in front of the wall must not exceed the design stabilising submerged weight, $G'_{stb,d}$ of the column of soil in front of the wall, i.e.:

$$S_{dst,d} \leq G'_{stb,d} \qquad (2.9b)$$

Using this equation with the partial action factors $\gamma_{G,dst}$ and $\gamma_{G,stb}$ in *Table A.17* of EN 1997-1 for a column of soil of depth d and volume V gives $S_{dst,d} = \gamma_{G,dst}\gamma_w i_k V = \gamma_{G,dst}\gamma_w(h_k/d)V$, where $i_k = h_k/d$ is the characteristic hydraulic gradient in front of the wall, h_k = characteristic hydraulic head at the wall toe and d = wall embedment depth, and $G' = \gamma_{G,stb}(\gamma - \gamma_w)V = \gamma_{G,stb}\gamma'V$. Hence:

$$\gamma_{G,dst}\gamma_w(h_k/d)V \leq \gamma_{G,stb}\gamma'V \quad \text{or} \quad \gamma_{G,dst}\gamma_w h_k/d \leq \gamma_{G,stb}\gamma'$$

Equation 2.9a states that the design destabilising total pore water pressure, $u_{dst,d}$ at the base of the column of soil in front of the wall must not exceed the design stabilising total stress, $\sigma_{stb,d}$, i.e.

$$u_{dst,d} \leq \sigma_{stb,d} \qquad (2.9a)$$

Using this equation with $\gamma_{G,dst}$ and $\gamma_{G,stb}$ in *Table A.17* applied to the characteristic total pore water pressure and the total stress, it becomes $u_{dst,d} = \gamma_{G,dst}u_k = \gamma_w(d + d_w + h_k)$ and $\sigma_{stb,d} = \gamma_{G,stb}\sigma_k = (\gamma d + \gamma_w d_w) = \gamma_{G,stb}((\gamma' + \gamma_w)d + \gamma_w d_w)$, where, d_w = depth of water above ground surface in front of wall, , γ and γ' are the soil total and effective weight densities and γ_w = 9.81kN/m^3 is the water weight density. Hence:

$$\gamma_{G,dst}\gamma_w(d + d_w + h_k) \leq \gamma_{G,stb}(\gamma' d + \gamma_w(d + \gamma_w d_w))$$

As can be seen from this equation, when *Equation 2.9a* is used in this way the hydrostatic pore water pressure, $\gamma_w(d + d_w)$, is multiplied by different factors on either side of the equation, which is not logical. If the partial factors are just applied to the excess pore water pressure $u_{h,k} = \gamma_w h_k$, which is destabilising action, and to the effective stress, $\gamma' d$, the hydrostatic pore water pressures on either side of the equation cancel and *Equation 2.9a* becomes the same as *Equation 2.9b*.

It is assumed that the ground surface level in front of the wall is reliably controlled so that no allowance is required for overdig *(9.3.2.2).*

Hydraulic head at toe of wall

In order to determine the value of H, it is necessary to assess the value of h_k, taking account of the hydraulic gradients around the wall and particularly in front of the wall. For clay soils and low seepage gradients, a linear loss of hydraulic head around the wall is often assumed when designing the embedment depth of embedded retaining walls. This results in an underestimate of h_k and hence an overprediction of H and thus an unconservative design, as noted by Orr (2005). The hydraulic gradient inside a cofferdam is often much higher than outside, even for uniform soils. A higher estimate of the hydraulic head at the toe of the wall may be obtained from the following equation in EAU (2004):

$$h_k = \frac{\left((d + d_w)\sqrt{d + H} + (d + H)\sqrt{d}\right)}{\left(\sqrt{d + H} + \sqrt{d}\right)} - (d + d_w)$$

Design using Equation 2.9a with HYD partial action factors applied to $u_{dst,k}$ and $\sigma_{stb,k}$

Assuming H = 2.78m, and for d = 3.0m and d_w = 1.0m in this example, the value of h_k calculated using the above equation is:

$$h_k = \frac{((3.0 + 1.0)\sqrt{3.0 + 2.78} + (3.0 + 2.78)\sqrt{3.0}}{(\sqrt{3.0 + 2.78)} + \sqrt{3.0)})} - (3.0 + 1.0) = 0.74m$$

Hence, as γ = 20kN/m^3,

$u_{dst,d} = \gamma_{G,dst}\gamma_w(d + d_w + h_k) = 1.35 \times 9.81\ (3.0 + 1.0 + 0.74) = 62.8$ kPa

$\sigma_{stb,d} = \gamma_{G,stb}(\gamma d + \gamma_w d_w) = 0.9 \times (20.0 \times 3.0 + 9.81 \times 1.0) = 62.8$ kPa

Thus $u_{dst,d} = \sigma_{stb,d}$ and hence Equation 2.9a is satisfied for H ≤.2.78m.

Design using Equation 2.9b with HYD partial action factors applied to $S_{dst,k}$ and $G'_{stb,k}$

Assuming *H* = 6.84m, the value of h_k at the toe is calculated using the above equation as:

$$h = \frac{((3.0 + 1.0)\sqrt{3.0 + 6.84} + (3.0 + 6.84)\sqrt{3.0}}{(\sqrt{3.0 + 6.84)} + \sqrt{3.0)})} - (3.0 + 1.0) = 2.08m$$

Hence, for a unit volume of soil:

$S_{dst,d} = \gamma_{G,dst}\gamma_w h_k/d = 1.35 \times 9.81 \times 2.08/3.0 = 9.2$ kN

$G' = \gamma_{G,stb}(\gamma - \gamma_w) = 0.9 \times (20.0 – 9.81) = 9.2$ kN

Thus $S_{dst,d} = G'$ and hence *Equation 2.9b* is satisfied for H ≤.6.84m.

3. Conclusions

When designing against heave failure it is important not to assume uniform conditions but to determine the most unfavourable combination of soil layers and pore pressures that could occur. However, assuming uniform soil conditions in this example and using the equation in EAU (2004) for the hydraulic head at the toe of the wall, the maximum design height of the groundwater level behind the wall for design against hydraulic heave was found to be 2.78m when $\gamma_{G,dst}$ and $\gamma_{G,stb}$ are applied to $u_{dst,k}$ and $\sigma_{stb,k}$ in *Equation 2.9a* and 6.84m when $\gamma_{G,dst}$ and $\gamma_{G,stb}$ are applied to $S_{dst,d}$ and G'_{stb} in *Equation 2.9b*. Thus applying the partial factors in this way in *Equation 2.9a* results in a much more conservative design using *Equation 2.9a* than using *Equation 2.9b*.

Applying $\gamma_{G,dst}$ and $\gamma_{G,stb}$ to $u_{dst,k}$ and $\sigma_{stb,k}$ in *Equation 2.9a* not only gives a more conservative result but it is not logical since different partial factors are applied to the hydrostatic pore water pressure on each side of the equation with the result that, as shown by Orr (2005), it predicts that h_k decreases as the depth of the water above the ground in front of the wall increases. To obtain the same result using $\gamma_{G,dst}$ and $\gamma_{G,stb}$ with *Equations 2.9a* and *2.9b*, these partial factors need to be applied to just the hydraulic head, $u_{h,k}$ and the effective stress, $\sigma' = \gamma' d$ in *Equation 2.9a* and the hydrostatic pore water pressure should not be factored.

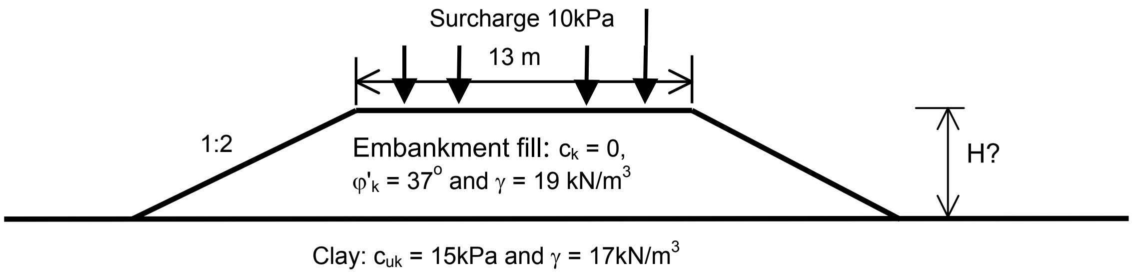
References

EAU (2004) *Recommendations of the Committee for Waterfront Structures - Harbours and Waterways*, 8th edition, Berlin: Ernst & Sohn

Orr T.L.L. (2005) Evaluation of uplift and heave designs to Eurocode 7, *Proceedings of International Workshop on Evaluation of Eurocode 7*, Dublin March-April 2005, Department of Civil, Structural and Environmental Engineering, Trinity College Dublin.

Model Solution for Example 10 – Road Embankment on Soft Ground

1. Description of the problem



- **Design Situation**
  - A road embankment is to be constructed over soft clay. Embankment is 13m wide at the top and has side slopes at 1:2 (26.6°)
- **Soil conditions**
  - Fill for embankment: Granular soil $c'_k = 0$, $\varphi'_k = 37^o$, $\gamma = 19kN/m^3$
  - Soil beneath embankment: Clay $c_{uk} = 15kPa$, $\gamma = 17kN/m^3$
- **Characteristic values of actions**
  - Traffic load on embankment: $q_k = 10kPa$
- **Require**
  - Maximum height, H, of embankment

2. Ultimate Limit State Design

The maximum height is assessed for each *Design Approach* and for undrained conditions for the ultimate limit state of overall stability (C*12.2(2)*) using the partial factors for actions, γ_F or the effects of actions, γ_E in Table A.3, the partial factors for soil parameters, γ_M in Table A.4 and the partial resistance factors, γ_R for slopes and overall stability in Table A.14 in Annex A of Eurocode 7.

The ultimate limit state analysis of the overall stability is performed using the Bishop Simplified method of slices. As the ground is homogeneous, circular failure surfaces are assumed (*C10.5.1(5)*). Stability analyses were carried out with different circular slip surfaces to determine the slip circle giving the critical slip circle, i.e. the circle giving the minimum F value (factor of safety).

The characteristic and design values of actions and ground strength parameters used for the three Design Approaches (DA) are presented in Table 1. In slope stability analyses, factoring the actions due to the weight of the ground in DA1-C1 and DA2 by the partial factor γ_M = 1.35 or factoring the resistances by γ_R = 1.1 is difficult because neither the actions nor the resistances are calculated explicitly and it is not appropriate to factor the weight density as the partial factor on weight density given in Eurocode 7 is unity. To overcome this difficulty, the procedure recommended by Frank et al.

Table 1: Partial factors, actions, soil parameters and results in the embankment designs

Parameter	DA1-C1	DA1-C2	DA2	DA3
Partial factors				
γ_G	1.35	1.0	1.35	1.0
γ_Q	1.5	1.3	1.5	1.3
γ_G used in analyses = γ_G/γ_Q = 1.0	1.0	1.0	1.0	1.0
γ_Q used in analyses = (γ_Q/γ_Q)	1.11	1,3	1,11	1,3
$\gamma_{\phi'}$	1.0	1.25	1.0	1.25
γ_{cu}	1.0	1.4	1.0	1.4
γ_{cu}	1.0	1.0	1.1	1.0
Embankment				
Characteristic surcharge, q_k (kPa)	10	10	10	10
Design surcharge, q_d (kPa)	11.1	13	11.1	13
Fill characteristic weight density, γ_k (kN/m^3)	19	19	19	19
Fill design weight density, γ_d (kN/m^3)	19	19	19	19
ϕ'_k	37.0	37.0	37.0	37.0
ϕ'_d	37.0	31.1	37.0	31.1
Clay Soil				
Clay characteristic weight density, γ_k (kN/m^3)	17	17	17	17
Clay design weight density, γ_d (kN/m^3)	17	17	17	19
c_{uk}	15.0	15.0	15.0	15.0
c_{ud}	15.0	10.7	15	10.7
Embankment Design				
Target F value in analyses = $\gamma_G \gamma_R$	1.35	1.0	1.49	1.0
Design embankment height, H	2.90	**2.40**	**2.15**	**2.40**
Overall safety factor using characteristic values	-	1.47	1.53	1.47

(2004) is adopted whereby the variable unfavourable action (traffic) load) is factored by γ_Q/γ_G at the start of the analysis and the analysis is then carried out to achieve a calculated factor of safety F = $\gamma_G \gamma_R$. Thus in DA1-C1 and DA2, the variable load is multiplied by 1.5/1.35 = 1.11 at the start of the analysis and in the DA1-C1 analysis, the aim is to achieve F = 1.35 * 1.0 = 1.35, while in the DA2 analysis, the aim is select an embankment height that will provide F = 1.35 * 1.1 = 1.49, as shown in Table 1.

3. Conclusions

From the results in table, it is seen that the height obtained using DA1-C1, 2.9m, is greater than that obtained using DA1-C2, 2.4m, therefore DA1-C2 determines the design for DA1. The design height for DA3, 2.4m is the same as for DA3 because the variable traffic loading is treated as a geotechnical action rather a structural action and hence the partial factors for DA3 are the same as those for DA1-C2. The design height for DA2 is 2.15m, which is less than the height obtained using DA1 and DA3. Thus, in this example, DA2 is more conservative than DA1 or DA3, giving an overall factor of 1.53, compared to 1.47 using DA1 and DA3.

References

Frank, R., Bauduin, C., Driscoll, R., Kavvadas, M., Krebs Ovesen, N., Orr, T. and Schuppener, B. (2004) - *Designers' Guide to EN 1997-1*, Thomas Telford, London

Evaluation of Uplift and Heave Designs to Eurocode 7

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ABSTRACT
The equations in Eurocode 7 to design against the ultimate limit states of uplift and heave are presented and examined. The application of the HYD partial action factors in Eurocode 7 to the characteristic total pore water pressure and total stress in Equation 2.9a is shown to be very conservative and not logical. However, if the factors are applied to the excess pore water pressure and effective stress in Equations 2.9a, the same design is obtained as when the factors are applied in Equation 2.9b. Alternatively, if just the excess pore water pressure in Equation 2.9a and the hydraulic gradient in Equation 2.9b are factored, or if the hydraulic head is treated as a geometrical property and increased by an appropriate margin, then the same design is obtained using both Equations 2.9a and 2.9b. The solutions received to the uplift and heave design examples are compared and it is found that, in the uplift example, there was little agreement regarding how the side wall resistance should be treated, whereas, in the heave example, there was a general preference to use Eurocode 7's Equation 2.9b rather than Equation 2.9a. Much of the difference between the solutions received for the examples was due to the use of different calculation models and design assumptions, rather than due to different interpretations of Eurocode 7. The solution to the heave example was found to be particularly sensitive to the assumption regarding the hydraulic gradient around the wall.

1 GENERAL

1.1 *Ultimate Limit States*

The principal ultimate limit states with which Eurocode 7 is concerned are those where "*the strength of the ground is significant in providing resistance*", for example the bearing failure of shallow foundations and piles; these are known as GEO ultimate limit states. However, Eurocode 7 is also concerned with two other ultimate limit states where the strength of the ground is not significant in providing resistance. These are the ultimate limit states where the forces or pressures due to pore water cause failure, either by excessive pore water pressure causing uplift, termed UPL, or excessive seepage forces causing heave failure, termed HYD. In these ultimate limit states, the resistance to the destabilising action is principally provided by the self-weight of the structure or the ground. The principles for designing against these ultimate limit states are provided in Section 11 of EN 1997-1 on hydraulic failure.

The UPL ultimate limit state is "*loss of equilibrium of a structure or the ground due to uplift by water pressure (buoyancy) or other vertical actions*". The most common UPL ultimate limit state is when static water pressure causes uplift of a structure with a deep basement. In this situation the static water pressure causes a destabilising uplift force on the structure, which is resisted mainly by the self-weight of the structure, but there may also be some resistance from the ground around the structure due to the strength of the ground. UPL is also relevant when the uplift force on the structure is not due to water pressure but due to some other force, for example the tensile force from the cable for a suspended structure acting on an anchor block foundation.

The HYD ultimate limit state is "*hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients*". Thus while UPL is for static groundwater at

Table 1 Partial factors for UPL and HYD ultimate limit states

Partial factors	UPL	HYD
Actions		
$\gamma_{G,dst}$	1.0	1.35
$\gamma_{G,stb}$	0.9	0.9
$\gamma_{Q,dst}$	1.5	1.5
Soil Parameters and Resistances		
$\gamma_{\phi'}$	1.25	-
$\gamma_{c'}$	1.25	-
γ_{cu}	1.4	-
$\gamma_{\sigma;t}$	1.4	-
γ_{a}	1.4	-

hydrostatic pressure, HYD is when hydraulic gradients cause failure by the force of the upward seeping water exceeding the weight of the soil. This situation is also known as boiling *(Clause 10.1(1)P)*. In this situation, the resistance to the disturbing force is provided entirely by the weight of the soil and the strength of the ground is not considered to be involved at all in resisting the force of the seeping water.

1.2 *Equilibrium Equations for Design Against Uplift and Heave Failure*

In designs against uplift failure or UPL ultimate limit states, EN 1997-1 states in *2.4.7.4(1)* that the equilibrium equation to be satisfied is:

$$V_{dst,d} \leq G_{stb,d} + R_d \qquad (2.8)$$

where the design destabilising action, $V_{dst,d} = G_{dst,d} + Q_{dst,d}$ can be a permanent and/or a variable uplift force and the equation number in italic is the number of the equation in EN 1997-1. In the case of uplift due to groundwater pressure, this may be interpreted as the groundwater pressure having a permanent and a variable component. The design stabilising action is the design self-weight of the structure, $G_{stb,d}$, which is a permanent load, and there may also be some additional design resistance, R_d between the soil and the structure.

In designs against heave failure or HYD ultimate limit states, EN 1997-1 states in *2.4.7.5(1)P* that either of the following two equilibrium equations may be used:

$$u_{dst,d} \leq \sigma_{stb,d} \qquad (2.9a)$$

where $u_{dst,d}$ is the design total pore water pressure at the base of the relevant column of soil whose equilibrium is being analysed and $\sigma_{stb,d}$ is the design total vertical stress at the base of the column, or

$$S_{dst,d} \leq G'_{stb,d} \qquad (2.9b)$$

where $S_{dst,d}$ is the design seepage force due to the seeping water on the relevant column of soil and $G'_{stb,d}$ is the design effective weight of the column. It is noted that *Equations 2.9a* and *2.9b* only involve actions and no soil resistance and hence no soil strength. Also *Equation 2.9a* is the only equilibrium equation in Eurocode 7 presented in terms of stress; all the other equilibrium equations are presented in terms of actions and resistances that are forces.

1.3 *Partial Factors Provided for Designs against Uplift and Heave*

For designs against heave failure, EN 1997-1 provides UPL partial factors on stabilising and destabilising actions, plus partial factors on material parameters and resistances, while for designs against heave failure, EN 1997-1 only provides HYD partial factors on stabilising favourable and destabilising unfavourable actions. These partial factors are presented in Table 1. The partial factor on stabilising favourable permanent actions is 0.9 for both uplift and heave. However, the partial factor on destabilising unfavourable permanent actions is 1.0 in the case of uplift and 1.35 in the case of heave, while the partial factor on destabilising variable actions is 1.5 for both uplift and heave. It should be noted that partial factors for ground strength parameters and resistances are only provided in the case of uplift and not for heave since heave does not involve any strength of the ground.

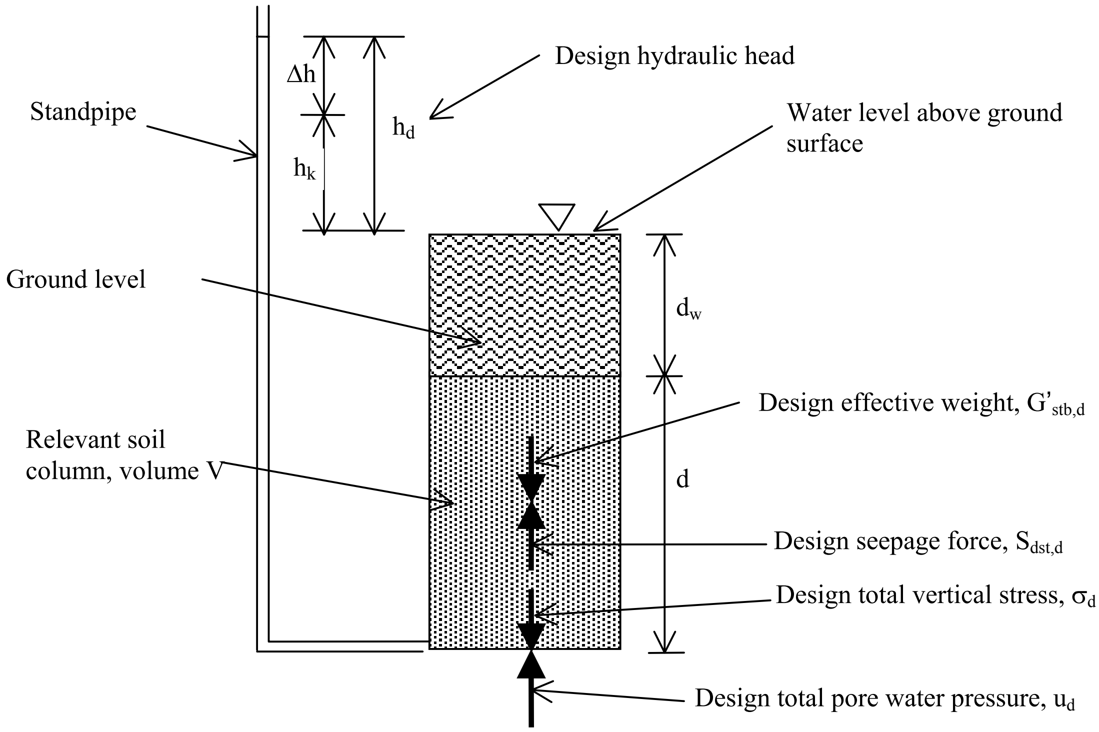


Figure 1: Design forces and pressures on soil column subjected to seepage

1.4 *Use of Partial Factors in Designs Against Uplift*

Ignoring the side resistance on a buried structure, the design destabilising and stabilising actions in *Equation 2.8* when designing against uplift are obtained by applying the UPL partial action factors in Table 1 to the characteristic stabilising and destabilising actions as follows:

$$V_{dst,d} \leq G_{stb,d} \quad \Rightarrow \quad \gamma_{G,dst}G_{dst,k} + \gamma_{Q,dst}Q_{dst,k} \;\leq\; \gamma_{G,stb}G_{stb,k} \tag{2.8}$$

For an uplift situation where there are only permanent stabilising actions and no additional ground resistance, the overall factor of safety, OFS is the ratio of the characteristic permanent stabilising action, $G_{stb,k}$ to the characteristic permanent destabilising action, $V_{dst,k}$, which, from *Equation 2.8*, is given by:

$$OFS = G_{stb,k}/V_{dst,k} = \gamma_{G,dst}/\gamma_{G,sbt} = 1.0/0.9 = 1.11 \tag{1}$$

In practice there will normally be some additional ground resistance and this will increase the OFS value above 1.11.

In the case of uplift, the destabilising action is the uplift force due to the hydrostatic water pressure acting on the buried structure.

1.5 *Use of Partial Factors $\gamma_{G,dst}$ and $\gamma_{G,stb}$ in Designs Against Heave*

When designing against heave, *2.4.7.5(2)P* states that the design destabilising and stabilising actions in *Equations 2.9a and 2.9b* shall obtained using the HYD partial action factors $\gamma_{G,dst}$ *and* $\gamma_{G,stb}$ in Table 1. However, no examples are given in EN 1997-1 showing the use of $\gamma_{G,dst}$ *and* $\gamma_{G,stb}$ in *Equations 2.9a* and *2.9b*.

Taking *Equation 2.9b* first, and considering a soil column of depth d and volume V with a depth d_w of water above the ground, and with a characteristic hydraulic head at the base of the column equal to h_k, as shown in Figure 1, the characteristic average hydraulic gradient through the column is $i_k = h_k/d$. The characteristic seepage force is $S_{dst,k} = \gamma_w i_k V$. Applying the partial factors in Table 1 to the characteristic stabilising and destabilising actions in *Equation 2.9b* gives:

$$S_{dst,d} \leq G'_{stb,d} \quad \Rightarrow \quad \gamma_{G,dst}S_{dst,k} = \gamma_{G,stb}G'_{stb,k} \tag{2}$$

$$\Rightarrow \quad \gamma_{G,dst}\,\gamma_w\, i_k V \;\leq\; \gamma_{G,stb}\,\gamma' V \tag{3}$$

Rearranging Equation 2 as the characteristic stabilising force over the characteristic destabilising force, gives the overall factor of safety against heave:

$$OFS = G_{stb,k}/G_{dst,k} = \gamma_{G,dst}/\gamma_{G,stb} = 1.35/0.9 = 1.5$$

While rearranging Equation 3, gives the following equation for OFS:

$$OFS = i_c/i_k = 1.35/0.9 = 1.5 \quad (4)$$

as the ratio of the critical hydraulic gradient $i_c = \gamma'/\gamma_w$, at which the characteristic upward seepage force just balances the effective weight of the soil and boiling occurs, to the average hydraulic gradient through the column i_k. Equation 4, derived from *Equation 2.9b*, is the equation often used in traditional designs against heave failure when OFS is normally in the range 1.5 to 2.0. If the column of soil represents the ground in front of a retaining wall, then, from Equation 4 and substituting $i_k = h_k/d$, the characteristic excess hydraulic head, h_k at the toe of the retaining wall to avoid hydraulic failure for a particular embedment depth d, is given by:

$$h_k = i_c d/1.5 = \gamma' d/(1.5\gamma_w) \quad (5)$$

Equation 2.9b was the equation chosen by most of those who submitted solutions to Design Example 9. However a number of those at the Workshop were of the view that it was not appropriate to analyse heave failure using the *Equation 2.9b* and the effective weight density of the soil, but that it should be analysed using *Equation 2.9a* and the total weight density of the soil.

Considering the same column of soil as analysed using *Equation 2.9b*, and applying the HYD partial action factors in *Table 1* to the characteristic total pore water pressure at the base of the column, assuming this is a permanent destabilising action, and the characteristic total stress at the base of the column to obtain the design destabilising and stabilising actions in *Equation 2.9a* gives the overall factor of safety:

$$u_{dst,d} \leq \sigma_{stb,d} \;\Rightarrow\; \gamma_{G,dst} u_{dst,k} \leq \gamma_{G,stb}\sigma_{stb,k} \;\Rightarrow\; \gamma_{G,dst}(u_s + u_{h,k}) \leq \gamma_{G,stb}(\sigma' + u_s) \quad (6)$$

where $u_s = \gamma_w(d + d_w)$ is the hydrostatic pore water pressure, $u_{h,k} = \gamma_w h_k$ is the excess pore water pressure and $\sigma' = \gamma' d$ is the effective stress. Substituting for these terms in Equation 6 gives:

$$\gamma_{G,dst}(\gamma_w(d + d_w) + \gamma_w h_k) \leq \gamma_{G,stb}(\gamma' d + \gamma_w(d + d_w)) \quad (7)$$

Rearranging Equation 6 as the characteristic total stress over the characteristic total pore water pressure and substituting for the terms and using the partial factor values from Table 1 gives:

$$OFS = \frac{\sigma_{stb,k}}{\sigma_{dst,k}} = \frac{\gamma_{G,dst}}{\gamma_{G,stb}} = \frac{1.35}{0.9} = 1.5 = \frac{\gamma' d + \gamma_w(d + d_w)}{\gamma_w h_k + \gamma_w(d + d_w)} = \frac{i_c/i_k + (d + d_w)/h_k}{1 + (d + d_s)/h_k} \quad (8)$$

Rearranging Equation 8 to obtain the characteristic hydraulic head at the toe of the wall gives:

$$h_k = \gamma' d/(1.5\gamma_w) - (1.35\text{-}0.9)(d+d_w)/0.9 = \gamma' d/(1.5\gamma_w) - (d+d_w)/3 \quad (9)$$

Thus, for a given d value, the characteristic hydraulic head at the toe of the wall when $\gamma_{G,dst}$ and $\gamma_{G,stb}$ are applied to $u_{dst,k}$ and $\sigma_{stb,k}$ in *Equation 2.9a* is $(d+d_w)/3$ less than the value obtained when $\gamma_{G,dst}$ and $\gamma_{G,stb}$ are applied to $S_{dst,d}$ and $G'_{stb,d}$ in *Equation 2.9b*; i.e. applying $\gamma_{G,dst}$ and $\gamma_{G,stb}$ in this way, *Equation 2.9a* is much more conservative than *Equation 2.9b*.

As may be seen from Equation 8, the OFS value obtained when $\gamma_{G,dst}$ and $\gamma_{G,stb}$ in Table 1 are applied to $u_{dst,k}$ and $\sigma_{stb,k}$ to obtain the design destabilising and stabilising actions in *Equation 2.9a* depends on the values of d and d_w, as well as on i_k As d_w is increased with d kept constant, h_k decreases according to Equation 8, which is not logical. Eventually, when d_w increased to $(2\gamma'/\gamma\text{-}1)d$, then:

$$h_k = \gamma'd/(1.5\gamma_w) - (d + 2\gamma'd/\gamma\text{-}d)/3 = \gamma'd/(1.5\gamma_w) - \gamma'd/(1.5\gamma_w) = 0$$

and hence the ground is predicted to be unstable against heave for no hydraulic head! If $\gamma' = \gamma_w = 10$ kPa, this occurs when $d_w \geq d$.

Multiplying out Equation 7 gives:

$$\boldsymbol{\gamma_{G,dst}\gamma_w(d + d_w)} + \gamma_{G,dst}\gamma_w h_k \leq \gamma_{G,stb}\gamma' d + \boldsymbol{\gamma_{G,stb}\gamma_w(d + d_w)} \quad (10)$$

This equation has the hydrostatic static water pressure, $\gamma_w(d + d_w)$, shown in bold, on both sides of the equation, but multiplied by the partial factor $\gamma_{G,dst} = 1.35$ on one side and by $\gamma_{G,stb}$

= 0.9 on the other side. This is contrary to the single source principle of applying the same factor to actions from the same source.

The problems noted above occur because $\gamma_{G,dst}$ and $\gamma_{G,stb}$ in Table 1 are applied to the full characteristic pore water pressure and the full characteristic total stress in *Equation 2.9a*. However, as shown by Equation 6, the characteristic pore water pressure and characteristic total stress both consist of two components: the total pore water pressure $u_{dst,k}$ consists of the hydrostatic pore water pressure, u_s plus the excess pore water pressure $u_{h,k}$, while the total stress, $\sigma_{dst,k}$ consists of the effective stress, σ' plus the hydrostatic pore water pressure, u_s. The hydrostatic pore water pressure component of $u_{dst,k}$ and $\sigma_{dst,k}$ is neither a stabilising nor a destabilising action with regard to heave, but a neutral action and hence the partial factors $\gamma_{G,stb}$ and $\gamma_{G,dst}$ should not be applied to this component. The stabilising and destabilising components of $\sigma_{dst,k}$ and $u_{dst,k}$ are the excess pore water pressure and the effective stress, and therefore $\gamma_{G,stb}$ and $\gamma_{G,dst}$ should only be applied to these components when using *Equation 2.9a* as follows:

$$u_{dst,d} \leq \sigma_{stb,d} \Rightarrow u_s + \gamma_{G,dst}u_{h,k} \leq \gamma_{G,stb}\sigma' + u_s$$

Cancelling the hydrostatic pore water pressure, this equation reduces to:

$$\gamma_{G,dst}u_{h,k} \leq \gamma_{G,stb}\sigma' \quad (11)$$

Substituting $u_{h,k} = \gamma_w h_k$, $\sigma' = \gamma' d$, multiplying by V and substituting $i_k = h_k/d$, gives:

$$\gamma_{G,ds}\gamma_w i_k V \leq \gamma_{G,stb}\gamma' V$$

which is the same as *Equation 2.9b*. Hence, applying the partial factors $\gamma_{G,stb}$ and $\gamma_{G,dst}$ only to the excess pore water pressure and the effective stress components of $u_{dst,d}$ and $\sigma_{stb,d}$ results in the same designs against heave as when applying these partial factors to $S_{dst,k}$ and $G'_{stb,k}$ in *Equation 2.9b*.

The application of the destabilising partial action factor, $\gamma_{G,dst}$ to the total characteristic pore water pressure in *Equation 2.9a*, not just to the excess pore water pressure, $u_{h,k}$, i.e. the destabilising component causing heave, is similar to the application of the partial resistance factor, γ_R to the total resistance in the case of Design Approach 2 in GEO ultimate limit states rather than applying the partial material factor, γ_M to the soil strength, as in the case of Design Approach 1. Also, the application of different partial action factors to the characteristic actions due to the weight of the water coming from the same source on opposite sides of Equation 2.9a is similar to Design Approach 2.

1.6 *Use of a Single Partial Factor γ_F in Designs Against Heave*

Instead of applying the partial action factors in Table 1 to both the stabilising and destabilising characteristic actions in *Equations 2.9a* and *2.9b*, a single partial action factor, γ_F could be applied to just the destabilising components of $u_{dst,k}$ and $S_{dst,k}$ causing the heave, while applying no partial factor to actions due to the total or effective weights of the soil on the stabilising side. As noted in the previous section, the component of $u_{dst,k}$ causing the heave is the excess pore pressure $u_{h,k}$,. Since $S_{dst,k} = \gamma_w i_k V$, the component of this action causing the heave may be considered to be i_k, the hydraulic gradient. However, the hydraulic gradient does not conform to the definition of an action given in *2.4.1(1)P*, nor is it listed as an action in *2.4.2(4)*, although *2.4.7.1(1)P* refers to "*heave…caused by hydraulic gradients*".

Starting again with *Equation 2.9b* and applying γ_F to i_k, treating i_k as the destabilising action, and applying no partial factor to the action due to the effective weight, gives:

$$S_{dst,d} \leq G'_{stb,d} \Rightarrow \gamma_w i_d V \leq \gamma' V \Rightarrow \gamma_w \gamma_F i_k V \leq \gamma' V \Rightarrow \gamma_w \gamma_F h_k/d \leq \gamma'$$

$$\Rightarrow h_k = \gamma' d/\gamma_F\gamma_w \quad (12)$$

By comparing Equation 12 with Equation 5 it can be seen that to obtain the same design when using γ_F as a partial action factor on the characteristic hydraulic gradient as when using the partial action factors $\gamma_{G,stb}$ and $\gamma_{G,dst}$ in Table 1 on the stabilising and destabilising actions in *Equation 2.9b*, then γ_F must equal 1.5.

Using *Equation 2.9a* and applying the partial action factor, γ_F to the characteristic excess pore water pressure, $u_{h,k}$ and applying no partial factor to the actions due to the soil effective stress or the hydrostatic pore water pressure, gives:

$u_{dst,d} \leq \sigma_{stb,d} \Rightarrow u_s + u_{h,d} \leq \sigma' + u_s \Rightarrow u_s + \gamma_F u_{h,k} \leq \sigma' + u_s$

$\Rightarrow \gamma_w(d + d_w) + \gamma_F \gamma_w h_k \leq \gamma' d + \gamma_w(d + d_w)$

Hence, cancelling the term $\gamma_w(d + d_w)$, which occurs on both sides of this equation, gives:

$\Rightarrow h_k \leq \gamma' d/\gamma_F \gamma_w$

which is the same as Equation 12 for h_k obtained using *Equation 2.9b*. Thus, applying the partial action factor γ_F to the characteristic excess pore water pressure or the characteristic hydraulic gradient and not applying any partial factor to actions due to the effective soil weight or due to the soil effective stress or the hydrostatic pore water pressure, designs against heave may be carried out using either *Equation 2.9a* or *2.9b*, with both giving the same result. However, no partial factor γ_F is given in EN 1997-1 at present.

1.7 *Use of an Additional Margin, Δh on the Hydraulic Head in Equations 2.9a and 2.9b*

Since the actions $u_{h,k} = \gamma_w h_k$ and $S_{dst,k} = \gamma_w i_k V = \gamma_w(h_k/d)V$, to which the partial factor γ_F was applied in the previous section, are both functions of h_k, an alternative approach when designing against heave is to apply the safety element to h_k, as the source of the action, treated as a geometrical parameter as proposed by Simpson (2005). It is stated in *2.4.4(1)P* that "*the level and slope of the ground surface, water levels, levels of interfaces between strata and the dimensions of the geotechnical structure shall be treated as geometrical data*", and in *2.4.6.3(2)P*, it is stated that "*design values of geometric data (a_d) shall either be assessed directly or be derived from nominal values using the following equation $a_d = a_{nom} \pm \Delta a$*". Hence, treating the hydraulic head (piezometric level) as a geometric parameter and adopting this approach, the characteristic hydraulic head, h_k, taken as the nominal value, is increased by an additional margin, Δh to obtain the design head, h_d, as shown in Figure 1; i.e.

$$h_d = h_k + \Delta h \tag{13}$$

Using *Equation 2.9a* and applying an additive margin Δh to obtain the design hydraulic head, but not applying any partial factor to the actions due to the weight of the soil or the hydrostatic pore water pressure, gives:

$u_{dst,d} \leq \sigma_{stb,d} \Rightarrow u_s + u_{h,d} \leq \sigma' + u_s \Rightarrow \gamma_w(d + d_w) + \gamma_w h_d \leq \gamma' d + \gamma_w(d + d_w)$

$\Rightarrow \gamma_w(d + d_w) + \gamma_w(h_k + \Delta h) \leq \gamma' d + \gamma_w(d + d_w)$

$$\Rightarrow \gamma_w (h_k + \Delta h) \leq \gamma' d \tag{14}$$

Adopting this approach and using *Equation 2.9b* gives:

$S_{dst,d} \leq G'_{stb,d} \Rightarrow \gamma_w i_d V \leq \gamma' V \Rightarrow \gamma_w (h_d/d) \leq \gamma' \Rightarrow \gamma_w(h_k + \Delta h)/d \leq \gamma'$

$$\Rightarrow \gamma_w (h_k + \Delta h) \leq \gamma' d \tag{15}$$

Hence, as Equations 14 and 15 are the same, adopting this approach with an additive margin Δh on h_k to obtain h_d, designs against heave failure can be carried out using either *Equation 2.9a* or *2.9b*. Rearranging Equation 14, the characteristic hydraulic head is given by:

$$h_k = \frac{\gamma' d}{\gamma_w} - \Delta h \tag{16}$$

Thus h_k is a function of γ', γ_w, and d as well as being a function of Δh, and h_k increases as d increases. To obtain the same design when using Equation *9* with Δh as when using the Table 1 partial action factors and *Equation 2.9b*, the values for h_k in Equations 5 and 16 are equated giving:

$$\frac{\gamma' d}{1.5\gamma_w} = \frac{\gamma' d}{\gamma_w} - \Delta h$$

$$\Delta h = \frac{\gamma' d}{3\gamma_w} \tag{17}$$

If γ' is equal to γ_w, Equation 17 becomes:

$$\Delta h = \frac{d}{3} \tag{18}$$

Designing against heave failure using $\Delta h = d/3$ from Equation 18 only involves a geometric term, d and not the soil effective weight density, γ'. Equation 18 is similar to the allowance for the ground level in front of retaining walls given in *9.3.2.2(2)*, which states that, in the case of cantilever retaining walls, "*an allowance Δa, equal to 10% of the wall height above the excavation level, subject to a maximum of 0.5m, should be made*". However, if the weight density of the soil is low, for example if $\gamma = 15.7kN/m^3$, and Equation 17 is used, then $\Delta h = \gamma' d/(3\gamma_w) = (15 - 9.81)d/(3x9.81) = d/5$. Thus, for this situation, Equation 17 provides a much more cautious value for Δh than d/3, resulting in a lower h_k value and hence a more cautious design that takes account of the soil density. Thus Equation 17 is recommended for use in design. The design hydraulic head is given by:

$$h_d = h_k + \frac{\gamma' d}{3\gamma_w} \tag{19}$$

The design values of γ' or γ, which are normally assessed directly *(2.4.6.2(1)P),* should be appropriately assessed, taking account of the uncertainty and variability in γ' or γ, and the presence of any soil layers with low weight densities.

The advantage of using Δa in designs against heave, rather than $\gamma_{G,stb}$ and $\gamma_{G,dst}$ or γ_F, is that the safety element is applied to the source of the uncertainty at the start of the calculation and it is not necessary first to calculate the excess pore water pressure or to apply partial factors to any terms involving the weight densities of either the soil or the water, since, as shown above, this causes problems when the $\gamma_{G,stb}$ and $\gamma_{G,dst}$ are applied to $u_{dst,k}$ and $\sigma_{stb,k}$ in *Equation 2.9a*.

It should be noted that Eurocode 7 only provides the partial factors $\gamma_{G,stb}$ and $\gamma_{G,dst}$ to obtain $u_{dst,d}$, $\sigma_{stb,d}$, $S_{dst,d}$ and $G'_{dst,k}$ for designs against heave using Equations 2.9a and 2.9b. However design values of actions may also be determined directly rather than by the application of partial action factors *(2.4.6.1(1)P)*. When designing against heave, it is necessary to take account of all unfavourable conditions *(10.3(2)P)*, including variations in permeability, and it may be appropriate to select the values of the actions directly rather than to rely on prescribed factors since it is difficult for code drafters to foresee what sort of factors or margins are relevant and realistic in practical design cases.

2 UPLIFT DESIGN EXAMPLE

2.1. Design Situation and Solutions Obtained

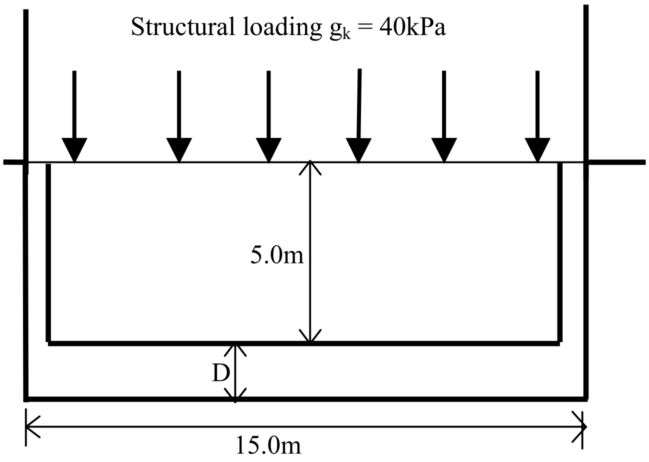
The design situation in Example 8 for the Eurocode 7 Workshop consisted of a structure with a 5m deep basement, as shown in Figure 2. The design involving determining the thickness of the concrete base slab, D to ensure safety against uplift failure. 12 solutions were received to this example based on Eurocode 7 and 2 solutions were received based on National Standards. The ranges in the values of D in these solutions received were as follows:

- Range of D values from Eurocode 7: 0.41 to 1.11m = average value ± 46%
- Range of D values from National Standards: 0.57 to 1.35m = average value ± 41%.

The ranges of values of D when using Eurocode 7 and the National Standards are similar. There are a number of reasons for the range of D values obtained using Eurocode 7 and these are discussed in the following sections.

2.2. Partial factors on the Uplift Force

In most of the solutions to Example 8, the uplift force from the groundwater pressure was treated as a permanent action and thus the partial factor $\gamma_{G,dst} = 1.0$ was used. However, in some solutions the uplift force was treated as a variable action with $\gamma_{Q,dst} = 1.5$, giving rise to very large base thicknesses of 3.47m and 5.26m. One solution stated out that, if the mean groundwater level were known, the uplift force due to the groundwater at this mean level could be treated as a permanent action and the uplift force due to the difference between the mean groundwater level and the higher nominal groundwater level could be treated as a variable action. In another solution, the uplift force was treated as an accidental action, with $\gamma_{G,dst} = 1.0$, which is equivalent to treating the uplift force as a permanent action.



- **Design situation**
  - Long structure, 15m wide, with a 5m deep basement
  - Groundwater level can rise to the ground surface
- **Soil Conditions**
  - Sand – $c'_k = 0$, $\phi'_k = 35^o$, γ = 20kN/m^3 (below water table)
- **Actions**
  - Characteristic structural loading g_k = 40kPa
  - Concrete weight density γ = 24kN/m^3
  - Wall thickness = 0.3m
- **Require**
  - Thickness of base slab, D for safety against uplift

Figure 2: Example 8 - Uplift of a deep basement

2.3. Ground Resistance on Buried Structure

The main differences between the solutions arose due to different assumptions regarding the resistance between the sand and the side walls of the basement. In all the solutions based on Eurocode 7, some resistance was assumed and this was determined from the shear stress, τ given by an equation of the form $\tau = K\sigma_v'\tan\delta$, where K is the coefficient of horizontal earth pressure and is a function of ϕ', σ_v' is the vertical effective stress, and δ is the friction angle against between the soil and wall. Since σ_v' is calculated from the weight density of the soil, no assumptions were required to calculate it and no partial factors were applied to it. However, different assumptions were made in the solutions regarding the values of K and δ and regarding what partial factors should be applied to obtain the design values of the combined parameter K$\tan\delta$.

The wall friction angle, δ was set equal to $(2/3)\phi'$ in the majority of the solutions, in accordance with the maximum value stated in *9.5.1(6)* for precast concrete walls, while the others used $\delta = \phi'$, which is the value that may be assumed for concrete cast against soil.

Regarding the coefficient of earth pressure, K the solutions were evenly divided between those who chose $K_0 = (1 - \sin\phi')$ and those who chose the more conservative K_a value, mostly obtained from *Figure C1.1* of EN 1997-1 taking account of the wall friction angle.

Regarding the application of partial factors to $\tan\phi'$ and $\tan\delta$ or to $K\tan\delta$, there are 5 different possible ways this can be done, as set out in Table 2, all of which may be considered to be in accordance with EN 1997-1. The first row in this table is when no factors are applied and provides the characteristic $K\tan\delta$ value, assuming $K = K_a$. The main features of the different ways of applying the factors and the resulting overall factor of safety, OFS, which is the ratio of the characteristic $K_a\tan\delta$ value to the design $K_a\tan\delta$ value, i.e. $K_{a,k}\tan\delta_k/(K_a\tan\delta)_d$, may be summarised as follows:

1. Applying the factor $\gamma_{\phi'}$ to reduce ϕ' and reduce δ, causes δ to reduce but causes K_a to increase, which is unconservative. This results in an overall factor of safety for $K_a\tan\delta$ of 0.96; i.e. less than 1.0 and hence no safety because the design $K_a\tan\delta$ value using this method is 0.103, which is greater than the characteristic value.
2. Applying the factor $\gamma_{\phi'}$ to reduce ϕ' and δ and then treating the resulting side friction force, which is an additional resistance to uplift, as a stabilising permanent vertical action in accordance with *2.4.7.4(2)* and applying the action factor $\gamma_{G,stb}$ to it as an action, results in an overall factor of safety of 1.08. The problem with this method is that there is double factoring, with R being treated as both a resistance and an action, and the overall factor of safety is low
3. Treating the characteristic side friction force as an action and applying $\gamma_{G,stb}$ resulting in an overall factor of safety of 1.11. The problem with this method is that it treats what is

Table 2: Different methods for obtaining the design resistance on a buried structure

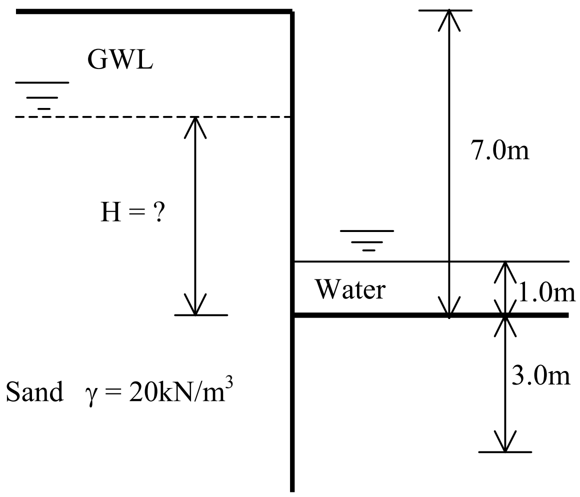
Method No. and assumptions regarding Ktanδ	$\gamma_{\phi'}$	γ_{δ}	$\gamma_{G,stb}$	γ_R	ϕ'	δ	K_a	tanδ	$(K_a\tan\delta)_d$	OFS
0. No factors	-	-	-	-	35.0	23.3	0.23	0.431	0.099	
1. Only $\gamma_{\phi'}$ applied to reduce ϕ' and δ	1.25	1.25	1.0	1.0	29.3	19.5	0.29	0.354	0.103	0.96
2. $\gamma_{\phi'}$ applied to reduce ϕ' and δ and also $\gamma_{G,stb}$ applied	1.25	1.25	0.9	1.0	29.3	19.5	0.29	0.354	0.092	1.08
3. $K_k\tan\delta_k$ treated as an action and only $\gamma_{G,stb}$ applied	1.0	1.0	0.9	1.0	35.0	23.3	0.23	0.431	0.089	1.11
4. Only $\gamma_{\phi'}$ applied to increase ϕ' and reduce δ	1/1.25	1.25	1.0	1.0	41.2	19.5	0.17	0.354	0.060	1.65
5. $K_k\tan\delta_k$ treated as a resistance and only $\gamma_{\phi'}$ applied as a resistance factor	1.0	1.0	1.0	1.25	35.0	23.3	0.23	0.431	0.079	1.25

clearly a resistance as an action and, although permitted, does not appear to be logical. Also the overall factor of safety is low.

4. Applying the factor $\gamma_{\phi'}$ to reduce δ but to increase ϕ', so as to reduce K_a and hence provide a cautious estimate of σ_h', results in an overall factor of safety of 1.65, which is the largest obtained by all the methods. With regard to this method, it should be noted that, although δ is originally considered to be a function of ϕ', the partial factor is applied differently to δ and ϕ'. This may be justified on the basis that the soil close to the wall providing the δ value is different from the general mass of the soil providing the K_a value and hence it is appropriate to treat δ and ϕ' differently. The problem with regard to EN 1997-1 is that there is no provision in *2.4.6.2(1)P* allowing for a partial material factor γ_M less than 1.0 to be applied to a characteristic soil strength value so as to increase the soil strength, ϕ' when it acts unfavourably, as it does in this example when ϕ' is used to calculate K_a. In the ENV version of Eurocode 7, however, there was a clause *2.4.3(10)P* which stated that *"For ultimate limit states in which soil strength acts in an unfavourable manner, the value of γ_M adopted shall be less than 1.0".*
5. Another fifth method, not specifically mentioned in EN 1997-1 but proposed in the case of the side resistance on a pile in the *Designers' Guide to Eurocode 7* by Frank et al.(2004), is to treat $\gamma_{\phi'}$ as a resistance factor and apply it to the characteristic $K_a\tan\delta$ value. Naturally this results in an overall factor of safety of 1.25.

2.4. Conclusions Regarding Uplift

The examination of the solutions the uplift design example show that most of the variations in the solutions were due to assumptions made regarding the soil resistance on the side walls of



- **Design situation**
  - Seepage around an embedded sheet pile retaining wall
- **Soil Conditions**
  - $\gamma = 20kN/m^3$
- **Actions**
  - Groundwater level 1.0m above ground surface in front of wall
- **Require**
  - Maximum height, H of water behind wall above ground surface in front of the wall to ensure safety against hydraulic heave

Figure 3: Example 9 – Failure by hydraulic heave

the basement. As shown in Table 2, if the resistance is calculated assuming the friction on the side walls is calculated from Ktanδ, where δ and K are both functions of the same ϕ', then applying a factor γ_M to reduce ϕ' (Method 1), does not provide any safety as the factored resistance is greater than the characteristic resistance. Treating γ_M as a resistance factor and applying it to the characteristic value of Ktanδ (Method 5) provides a reasonable factor of safety of 1.25 on the resistance, whereas applying γ_M to reduce δ and increase ϕ', and hence reduce K_a,(Method 4), which is consistent with the ENV version of Eurocode 7, provides a greater factor of safety of 1.65.

If Method 5 is used for this design example and the K is assumed equal to K_a obtained from *Annex C1.1* of EN 1997-1, then as shown in the Model Solution for Example 8 (Orr, 2005), the resistance on the side walls is calculated to be only 3.2% of the design stabilising action and the thickness of the base slab is calculated to be 0.60m. If the side resistance is ignored, the required thickness of the base slab is calculated to be 0.74m. These values lie within the range of values of 0.41m to 1.11m for the submitted solutions.

3 HEAVE EXAMPLE

2.1. Design Situation

The design situation in Example 9 for the Eurocode 7 Workshop, which consists of an embedded sheet pile retaining wall supporting a 7m deep excavation in sand, is shown in Figure 3. The wall penetrates 3m into the ground beneath the excavation and the design involves determining the maximum height, H to which the water level in the ground behind the wall may rise above the level of the ground in front of the wall to avoid heave failure when there is 1m of water above the ground surface in front of the wall.

2.2. Choice of Equilibrium Equation and Range of Solutions Obtained

Most of the solutions received for this example were based on *Equation 2.9b*, with the calculated design heights of the water level behind the wall ranging from 4.7 to 8.1m, which is equal to the average of these values ± 26% and with one solution stated as being an unspecified height greater than 7m, which is the height of the retaining wall. Thus the solution with the water height of 8.1m exceeds the height of the retaining wall.

2.3. Choice of Partial Factors

In the submitted solutions, the seepage force was generally treated as a permanent action with a partial factor $\gamma_{G,dst} = 1.35$ applied to it, although in one example, the seepage force was treated as a variable action and a partial factor, $\gamma_{Q,dst} = 1.5$ was used, while in another solution, the seepage force was treated as an accidental action with a partial factor $\gamma_{w,dst} = 1.0$ applied to it. Other solutions referred to the use of different $\gamma_{G,dst}$ values in their national standards that are related to the type of soil.

2.4. Choice of Excess Hydraulic Head at Toe of Wall

One of the main reasons for the differences between the solutions was due to value chosen for the hydraulic head at the toe of the wall, h_k, which is based on the assumption regarding the gradient in the hydraulic head around the retaining wall. In several solutions, h_k was calculated from the equation in EAU (2004), which gives h_k in terms of the required height of water behind the wall, H, the embedment depth, d and the depth of water above the ground level in front of the wall, d_w as follows:

$$h_k = \frac{\left((d + d_w)\sqrt{d + H} + (d + H)\sqrt{d}\right)}{\left(\sqrt{d + H} + \sqrt{d}\right)} - (d + d_w) \tag{20}$$

This equation is derived from the observation that the rate of loss of hydraulic head around the wall is not constant, but is greater in front of than behind the wall. When this equation is used with *Equation 2.9b*, if H = 6.84m, as adopted in the model solution for this example (Orr, 2005), and assuming $\gamma_w = 9.81$ kPa, then, from Equation 20, $h_k = 2.08$m so that the hydraulic gradient in front of the wall, $i_{f,k} = h_k/d = 0.69$ while the hydraulic gradient behind the wall, $i_{b,k} = (H\text{-}d_w\text{-}\ h_k)/(H+d) = 0.38$, as shown in Table 3. Hence the design seepage force on the column of soil in front of the wall with V = 1.0 is $S_{dst,d} = \gamma_{G,dst}\gamma_w i_{f,d} V = \gamma_{G,dst}\gamma_w h_k/d$ = 1.35x9.81x2.08/3.0 = 9.2kN. The design buoyant weight of soil in front of the wall is $G'_{G,stb} = \gamma_{G,stb}\gamma' V$ = 0.9x(20-9.81) = 9.2kN. Thus $S_{dst,d} = G'_{G,stb}$ and hence for stability, H ≦ 6.84m.

In two solutions it was assumed that the hydraulic head dissipated uniformly along the length of the wall. However, other assumptions were also made in these solutions. In one solution it was assumed that the water level in front of the wall was at the ground level, which gave H = 4.9m assuming that $\gamma_w = 9.81\text{kN/m}^3$. In the other solution, it was assumed that the hydraulic head dissipated uniformly only along a length of the wall equal to twice the embedment depth, i.e. dissipation behind the wall only started at a depth behind the wall equal the ground level in front of the wall, which gave H = 5.0m with $\gamma_w = 10\text{kN/m}^3$.

If the additional assumptions used in the two solutions reported above are not made when the hydraulic head is assumed to dissipate uniformly along the length of the wall, then although the h_k value is still equal to 2.08m, the value of H is found to be very large, being equal to 16.76m, as shown in Table 3, rather than the values of 4.9m or 5.0m obtained using the additional assumptions. Thus, whether intentionally or fortuitously, the additional assumptions made when it was assumed that the excess hydraulic head dissipated uniformly, provided conservative solutions that were closer to the design H value of 6.84m obtained when the hydraulic head at the toe was calculated assuming a non-uniform dissipation of hydraulic head around the wall and using Equation 20.

When a non-uniform dissipation of hydraulic head around the wall is assumed and Equation 20 is used with *Equation 2.9a*, i.e. with HYD partial factors applied to the total stress and total pore water pressures, the results in Table 3 show that the design H = 2.78m and $h_k = 0.74$m, whereas, if a uniform dissipation is assumed, then H = 3.31m and $h_k = 0.74$m. Thus, as explained above, applying the HYD partial factors to $u_{dst,k}$ and $\sigma_{dst,k}$ in Equation 2.9a gives solutions that are much more conservative than the solutions of 6.84m and 16.76m obtained by applying the HYD partial factors to $S_{dst,k}$ and $G'_{dst,k}$ in *Equation 2.9b*.

Instead of applying $\gamma_{G,dst}$ and $\gamma_{G,stb}$ to $u_{dst,k}$ and $\sigma_{dst,k}$ or $S_{dst,d}$ and $G'_{G,stb}$, an additional margin $\Delta h = \gamma' d/(3\gamma_w)$ may be added to h_k, to obtain h_d and hence $S_{dst,d}$ as explained above. Using Equation 17 and the design conditions in this example: $\Delta h = \gamma' d/(3\gamma_w)$ = (20.0 - 9.81)x3.0/(3x9.81) = 1.04m. Assuming H = 6.84m, which gives $h_k = 2.08$m from Equation 20, so that $h_d = h_k + \Delta h = 2.08 + 1.04 = 3.12$m and hence $i_d = h_d/d = 3.12/3.0 = 1.04$. For V = 1.0, $S_{dst,d} = \gamma_w i_d V$ = 9.81x1.04x1.0 = 10.2kN. $G'_{G,stb} = \gamma' V$ = (20 – 9.81) x 1.0 = 10.2kN. Thus $S_{dst,d} = G'_{G,stb}$ and hence for stability, H ≦ 6.84m; i.e. the same result that has been obtained above using $\gamma_{G,dst}$ and $\gamma_{G,stb}$ with Equation 2.9b. The same result would also have been obtained using Δh with Equation 2.9a.

Table 3: Results of using Equations 2.9a and 2.9b and different hydraulic heads for heave design

Excess head at toe	Equation 2.9a					Equation 2.9b				
	Δh	$i_{f,k}$	$i_{b,k}$	**H**	H/h_k	Δh	$I_{f,k}$	$i_{b,k}$	**H**	H/h_k
EAU equation.	0.74	0.25	0.18	**2.78**	3.73	2.08	0.69	0.38	**6.84**	3.29
Uniform dissipation along wall	0.74	0.25	0.25	**3.31**	4.45	2.08	0.69	0.69	**16.76**	8.07

2.5. Sensitivity of Heave Designs to Equation used and Assumed Hydraulic Head

As shown in the previous section, another factor contributing to the differences between the solutions obtained in this example is the sensitivity of the equations to the assumptions regarding the dissipation of the hydraulic head around the retaining wall. The results in Table 3 show that, using *Equation 2.9b*, the ratio H/h_k is 3.29 when using the EAU equation for the hydraulic head at the toe of the wall and 8.07 when assuming a uniform dissipation of hydraulic head around the wall. Thus, if the object is to determine H, the design height of the water in the ground behind the wall, assuming a uniform dissipation of hydraulic head around the wall results in a very significant overestimate in H compared to the value obtained using the EAU equation for h_k at the toe. However, if *Equation 2.9a* is used with partial factors on the total stress and total pore water pressure, then the H value is not very sensitive to the assumption made to determine the hydraulic head at the toe since the ratio H/h_k is 3.73 when using the EAU equation for the hydraulic head at the toe and 4.45 when assuming a uniform dissipation of hydraulic head around the wall.

4 CONCLUSIONS

Investigation of the Eurocode 7 equilibrium equations and partial factors for designs against uplift and heave has clarified the design principles for these situations. It has been shown that the use of the HYD partial action factors on the characteristic total stress and total pore water pressure in *Equation 2.9a* results in very conservative designs and is illogical. The partial action factors $\gamma_{G,dst}$ and $\gamma_{G,stb}$ should only be applied to the characteristic destabilising action causing heave, which is the excess pore water pressure, and the stabilising effective stress, not the hydrostatic pore water pressure. Alternatively, although not in the current version of Eurocode 7, applying a partial action factor to the excess pore water pressure or an additional margin to the hydraulic head, with no factor applied to terms involving the weight of the soil or the water gives the same result as applying the HYD partial action factors to the seepage force and effective weight in *Equation 2.9b*.

The main differences between the solutions received for the uplift example were found to be due to the assumptions made regarding the earth pressure on the side walls and how to obtain the design value of the side wall resistance against uplift. In the case of the example involving heave in front of a retaining wall, there was a general preference to use *Equation 2.9b* with the HYD partial action factors. However, it is found that the design height of the groundwater level behind the wall is very sensitive to the assumption made regarding the dissipation of the hydraulic head around the wall. The assumption of a uniform gradient around the wall, which is commonly made when designing the length of the wall, results in a significant overestimate of the design height of the groundwater level behind the wall.

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