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Findings from the 2nd Set of Eurocode 7 Design Examples

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ABSTRACT: In April 2010 the 2nd International Workshop on the Evaluation of Eurocode 7 was held in Pavia, Italy. This Workshop was organised by ETC 10 and the SC7 Maintenance Group. In preparation for the Workshop, a set of six design examples was prepared and published on a website together with on-line questionnaires for each example. These examples were completed by geotechnical engineers from different European countries using the partial factors in their own National Annexes and submitted on-line. Whereas the design examples for the 1st International Workshop held in Dublin in 2005 provided the characteristic parameter values, the design examples for the 2nd International Workshop held in Pavia did not but provided instead the results of the geotechnical investigations for each example. These included field and laboratory tests and required the characteristic values to be selected from this geotechnical information. Reviewers were appointed to evaluate the designs submitted for each example and to report to the Pavia Workshop on the designs received. This paper presents an overview of the findings from the second set of Eurocode 7 design examples. These findings are compared with the findings from the first set of design examples for the Dublin Workshop and assessed in the light of the implementation of Eurocode 7 in Europe in 2010.

Keywords: Eurocode 7, Pavia Workshop, design examples, limit states, confidence

1 INTRODUCTION

1.1 *Eurocode 7 Workshops*

The 1st International Workshop on the Evaluation of Eurocode 7, organised by European Technical Committee 10 (ETC 10) of the International Society for Soil Mechanics and Geotechnical Engineering and the European Geotechnical Thematic Network, Geotechnet, was held in Dublin in 2005 and a volume of Workshop Proceedings was published (Orr, 2005). Since in April 2010, the suite of Eurocodes, with Eurocode 7 for Geotechnical Design, superseded the existing national standards for structural and geotechnical design in the 26 CEN (European Standardization Committee) member countries it was appropriate that the 2nd International Workshop was held in the EUCENTRE in Pavia, Italy in April 2010. This Workshop was organized by ETC 10 together with the Maintenance Group of the CEN committee for Eurocode 7, TC 250/SC7 – Geotechnical Design. The main findings from the examples presented at this Workshop are reviewed in this paper based on the draft Proceedings (due to be published later).

1.2 *Dublin Workshop*

Prior to the Dublin Workshop in 2005, a set of 10 geotechnical design examples involving the design situations shown in Table 1 were circulated by email to engineers in Europe and worldwide. The characteristic values of the parameters were provided for the engineers to obtain solutions for the examples in accordance with Eurocode 7. A total of 90 solutions were received from engineers from 11 countries, including some solutions from Japanese engineers, who carried out the designs using Japanese codes and reliability analyses. The finding from the reliability analyses are not discussed in this paper but a paper on them will be included in the Workshop Proceedings.

Table 1. Details of Eurocode 7 design examples

Examples	Design situation	Required parameter	Reporter(s)
1[st] Set			
1	Spread foundation, vertical central load	*B* – foundation width	G. Scarpelli & V. Fruzzetti
2	Spread foundation, inclined eccentric load	*B* – foundation width	G. Scarpelli & V. Fruzzetti
3	Pile foundation from parameter values	*L* – pile length	R. Frank
4	Pile foundation from load test results	*N* – number of piles	R. Frank
5	Gravity retaining wall	*B* – wall base width	B. Simpson
6	Embedded retaining wall	*D* – embedment depth	B. Simpson
7	Anchored retaining wall	*D* – embedment depth	B. Simpson
8	Uplift of a deep basement below GWL	*T* – slab thickness	T. Orr
9	Heave of an excavation due to seepage	*H* – hydraulic head	T. Orr
10	Embankment on soft ground	*H* – embankment height	U. Bergdahl
2[nd] Set			
2.1	Spread foundation, vertical central load	*B* – foundation width	J. Brito & C.S. Sorensen
2.2	Spread foundation, inclined eccentric load	*B* – foundation width	N. Vogt
2.3	Pile in clay	*L* – pile length	A. van Seters
2.4	Earth and water pressures on basement wall	*d* – depth of groundwater behind wall	H.R. Schneider
2.5	Embankment on soft peat	*H* – embankment height (initial stage)	E.R. Farrell
2.6	Pile in sand (from parameter values)	*L* – pile length	B. Kłosiński

Reports on the solutions submitted to the first set of examples were prepared by the reporters listed in Table 1 and are included in the Proceedings of the Dublin Workshop (Orr, 2005). A large scatter was obtained for some of the examples, particularly for the eccentrically loaded foundation, the pile designed from soil parameters, and the uplift example. However, 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 more due to using different calculation models and design assumptions, which are not specified in Eurocode 7, than to different interpretations of Eurocode 7 or using the different Design Approaches.

1.3 *Pavia Workshop*

In geotechnical designs, there are three main components that affect the resulting design: the geotechnical parameter values, the calculation model, and the safety factors. In practice, these factors are often moderated by the designer's experience. In the first set of examples, the characteristic values were provided, so the variation in the designs received were due to the calculation models used and the partial factors chosen, which in the case of designs to Eurocode 7 means the particular Design Approach, and how it is applied. In the second set of design examples, the raw geotechnical data was provided rather than the characteristic values and hence the authors first had to determine the characteristic parameter values before calculating design values. This made these examples more realistic and also made it possible to investigate how much of the scatter in the designs received was due to the selection of characteristic values and how much was due to the choice of calculation model and adoption of a particular Design Approach and set of partial factors.

The second set of 6 geotechnical design examples, prepared for Pavia, are listed in Table 1. Besides providing raw data rather than characteristic parameter values, the second set of design examples differed from the first set in another way; the examples were placed on a website (www.eurocode7.com/etc10) and engineers were invited to submit their solutions via an online questionnaire comprising about 20 questions. The questions were circulated widely in Europe and also worldwide, and while almost 100 solutions were received, it was disappointing that 78% came from just four countries – Poland, UK, Germany and Italy – and the remaining 22% came from only six countries – Greece, Netherlands, France, Japan, Ireland, and Portugal. As in the case of the first set of examples, the solutions received for the second set were reviewed by the reporters listed in Table 1, who made presentations on their findings during the Pavia Workshop (these will be reported in the Workshop Proceedings).

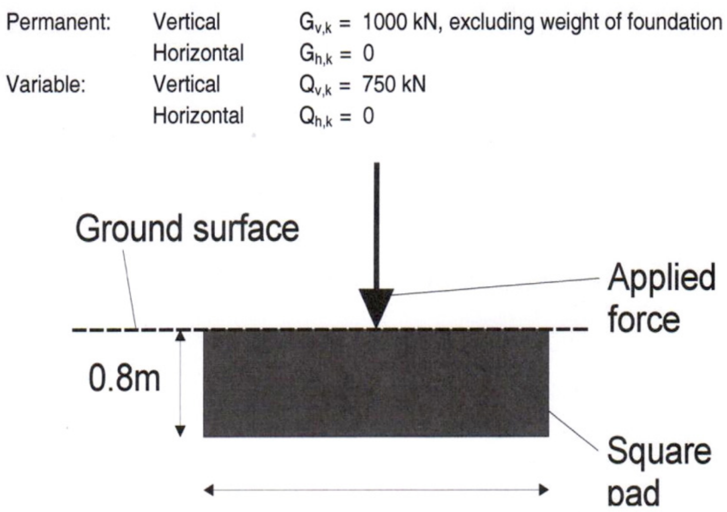

Figure 1. Example 2.1 Pad foundation on dense sand design situation

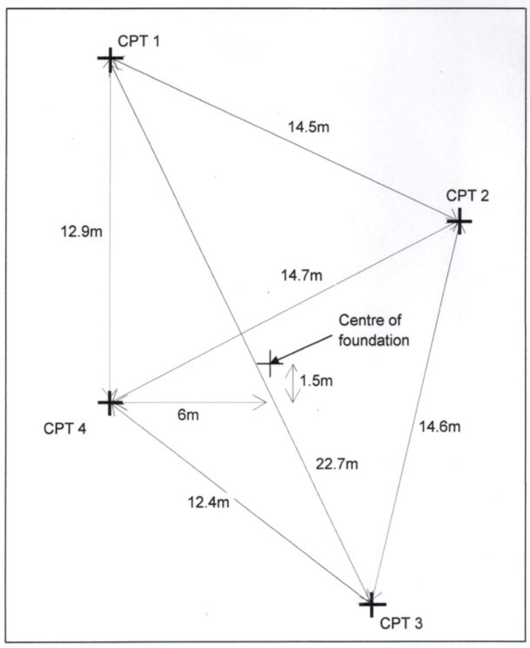

Figure 2. Example 2.1 site plan and borehole locations

2 SECOND SET OF DESIGN EXAMPLES

2.1 *Example 2.1 – Pad Foundation with Vertical Central Load on Dense Sand*

The first of the second set of examples was to determine the width of the square pad foundation shown in Figure 1 supporting a vertical central permanent load of 1000kN and a vertical variable load of 750kN, no horizontal load, and founded on a dense sand stratum. The geotechnical data provided were obtained from CPT tests carried out in four boreholes located on the site with respect to the centre of the foundation as shown in Figure 2. The q_c values measured in the CPT tests are plotted in Figure 3.

Brito and Sorensen (2010), in their presentation on this example, noted that the respondents gave no special weighing to any particular borehole or set of CPT results. They also noted that there two are main interdependent tasks to be considered in most geotechnical design problems when selecting geotechnical parameter values: one is to divide the soil into a few well defined homogeneous layers and the other is to select appropriate geotechnical parameter values for each layer, which for designs to Eurocode 7 are characteristic values. The characteristic values selected by the respondents are plotted in Figure 3, showing that the respondents selected a wide range of $q_{c,k}$ values from close to the mean of the test results down to below a lower bound value. When selecting the characteristic E values, the respondents selected an even greater range of values.

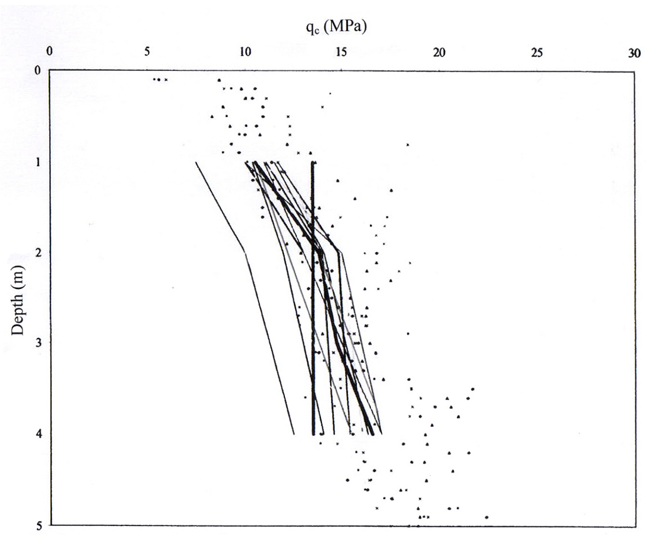

Figure 3. Example 2.1 measured q_c and selected $q_{c,k}$ v. depth

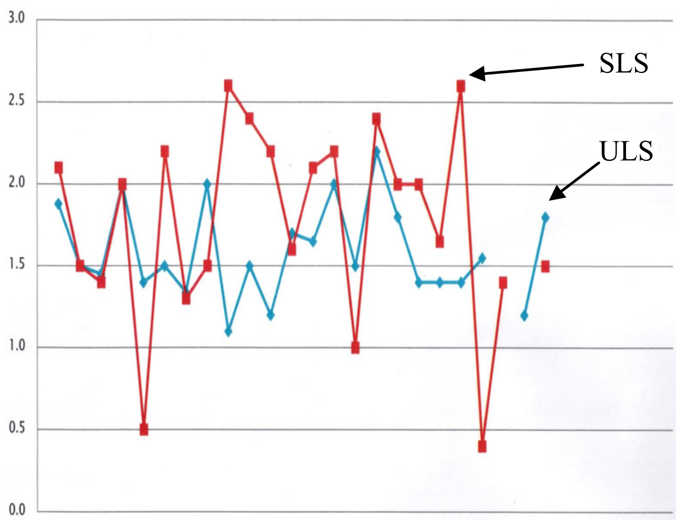

Figure 4. Example 2.1 ULS and SLS design foundation widths

With regard to the calculation model for ULS design, about half (48%) of the respondents used the bearing resistance equation in Annex D of EN 1997-1, a quarter (24%) used other equations from their National Annexes and the other quarter (28%) used other bearing resistance equations, such as Brinch Hansen's.

With regard to the method to estimate settlement, over half (59%) used either or Annex D.3 of EN 1997-2Annex F.1 of EN 1997-1. The Design Approach chosen by the respondents reflects the DA adopted in their National Annex with the result that those from Portugal, Italy and the UK chose DA1; those from Greece, France, Germany, and Poland chose DA2; those from Denmark and the Netherlands chose DA3; and of the two results from Ireland, one used DA1 and the other DA2.

The design foundation widths for ULS and SLS conditions obtained by the respondents are plotted in Figure 4. The ULS widths ranged from 1.1m to 2.3m with an average value of 1.6m, while the SLS widths ranged from 0.5 to 2.6m with an average value of 1.8m. Thus there was much more variability in the SLS design widths than in the ULS widths reflecting the greater number of calculation models used and the wide range of E_k values selected.

The variability in the ULS and SLS design widths is particularly significant in this example because, depending on the parameter values selected and calculation model and Design Approach adopted, the results in Figure 5 show that 56% of the respondents found that the design was controlled by the SLS while 35% found it was controlled by the ULS with the remainder finding that the ULS and SLS designs were the same.

This demonstrates the sensitivity of this particular design to the SLS requirement and the need for reliable methods to estimate the settlement of a foundation.

2.2 *Example 2.2 – Pad Foundation with Inclined Eccentric Load on Boulder Clay*

The second example was to determine the width of a square pad foundation shown in Figure 5 with a vertical central permanent load of 1000kN and a variable load of 750kN at a height of 2m and resting on stiff to very stiff boulder clay. The geotechnical data provided consisted of the results of SPT tests, carried out in four boreholes around the proposed location foundation, as shown in Figure 6, and water content and index tests. The SPT *N* values are plotted in Figure 7 and show considerable scatter.

Vogt (2010), in his presentation on this example, noted that, when selecting the data from the different boreholes for the design, the majority of the respondents (73%) either chose the average of the data from all the boreholes or did not consider the borehole location, while 20% considered the trend of the boreholes, biased towards the nearest.

One respondent, who was familiar with this particular soil, commented that experience of this soil has shown it can vary in an apparently random manner across the site.

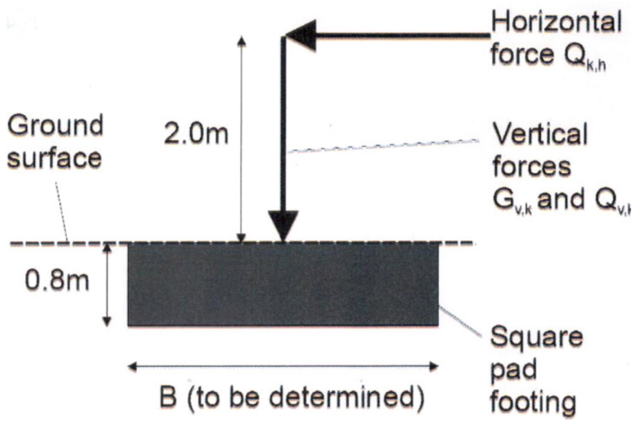

Figure 5. Example 2.2 – Pad foundation with inclined load design situation

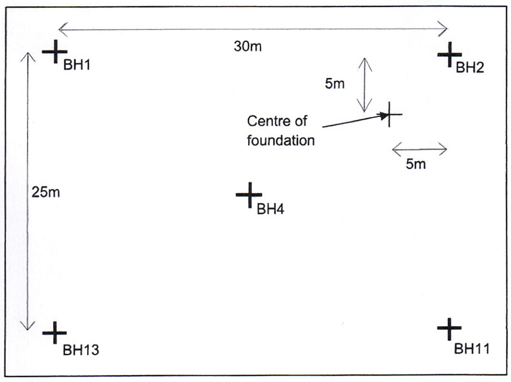

Figure 6. Location of boreholes and centre of foundation

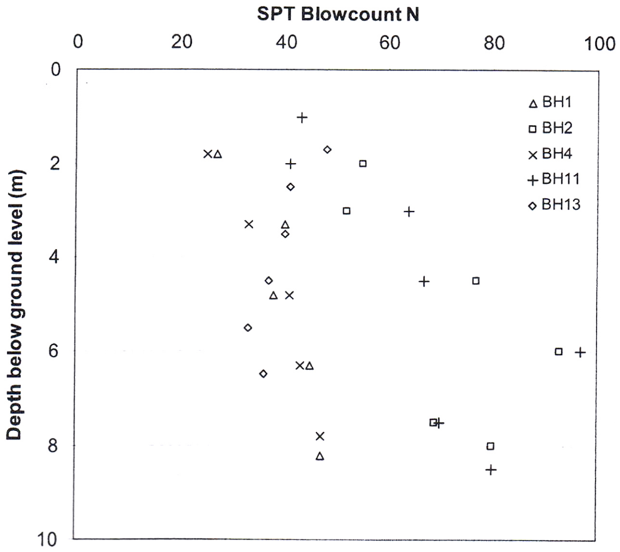

Figure 7. SPT N values v. depth

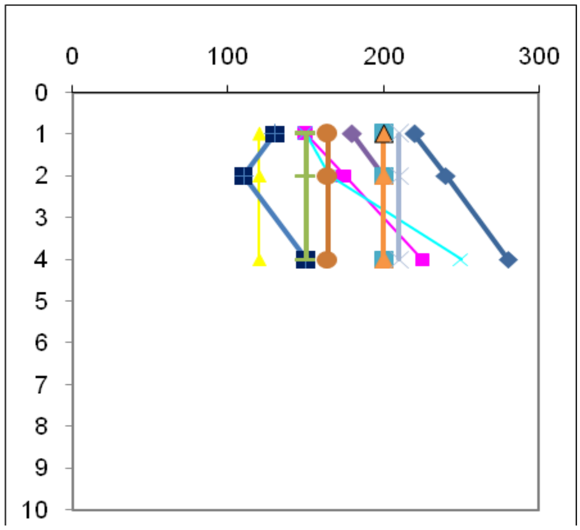

Figure 8. Selected characteristic $c_{u,k}$ values v. depth

Table 2. Design widths of foundation with an inclined and eccentric load

	Stage 1	
	ULS	SLS
Range of design widths (m)	3.1 – 4.66	1.2 – 4.5
Average (m)	4.08	3.46
Coefficient of variation (COV)	0.14	0.29
	Stage 2*	
	ULS using own selected $c_{u,k}$	ULS using given benchmark $c_{u,k}$
Range of design widths (m)	3.5 – 4.63	3.5 – 4.63
Average (m)	4.55	4.19
Coefficient of variation (COV)	0.03	0.12

* Only four respondents submitted Stage 2 design widths. Two were same as the original widths and two were much reduced

Regarding the parameters used to determine the ULS design foundation width, 93% used c_u, 67% used φ' and 33% used c'. Therefore, while almost all the respondents checked for undrained conditions, only two-thirds checked drained conditions as well and half of those assumed that $c' = 0$. With regard to SLS design, a variety of methods were used to calculate the foundation settlement with 33% using one of the methods in Annexes F1, F2 and F3 of EN 1997-1. One respondent noted that, in accordance with paragraph 6.6.2(16) of EN 1997-1, it was not necessary to calculate the settlement since the ratio of the bearing resistance of the ground at its initial undrained shear strength to the serviceability loading was less than 3, while another respondent noted that the tilt condition was more critical than settlement. Various parameters were used to calculate the settlement including E_u, E' and m_v.

A variety of correlations were used by the respondents to derive the c_u value; most involving correlations with the N value based on the index properties. In view of the considerable scatter in the N values, the respondents gave a large number of methods for how they selected the characteristic value, which included by eye (20%) and by statistical analysis (13%). The characteristic $c_{u,k}$ values selected by the respondents are plotted in Figure 8. As in Example 2.1, the respondents selected a wide range of characteristic values.

With regards to which calculation method was used to check against bearing failure, the majority (71%) of the respondents used the bearing resistance equation in Annex D of EN 1997-1. Others used alternative models given in their National Annexes or standards and one respondent used the Brinch Hansen bearing resistance equation. Regarding the Design Approach adopted, 64% used DA1 and 29% used DA2 or DA2*. No respondent used DA3.

The design widths to avoid a ULS calculated by the respondents are shown in Table 2 and ranged from 3.1 to 4.66m, with an average value of 4.08 and a coefficient of variation (COV) of 0.14. All the respondents found that the SLS design width of this foundation on stiff soil was less than the ULS design width so that the design was controlled by the ULS. The COV of the SLS widths at 0.29 was greater than the value of 0.14 for the ULS widths due to different calculations models used to estimate the settlement. While only four respondents submitted Stage 2 designs using benchmark $c_{u,k}$ values, two of these were the same as the original values and two were smaller with the result that their average design width reduced but the COV of their design widths increased significantly from 0.03 to 0.12. This indicates that, in his example, the variation in the design widths is more due to how $c_{u,k}$ is selected than to the calculation model used or the Design Approach adopted.

2.3 *Example 2.3 – Pile in Clay*

Example 2.3 was the design of a 450mm diameter pile in clay to support a permanent load of 300kN and a variable load of 150kN as shown in Figure 9. The ground consisted of 0 - 3 to 4m of made ground over London Clay with sand at a depth of 34m. The geotechnical data provided consisted of the results of CPT, SPT and pressuremeter field tests and laboratory undrained triaxial (UU) tests. The results of the UU tests are shown in Figure 10.

Van Seters (2010), in his presentation on this example, noted that all the ULS designs were based on the c_u values. A number of different correlations were used to determine the c_u value from the field tests, some of which were taken from existing standards. When determining the c_u value, almost the same number (53%) used an average of the tests from all the boreholes as those who took the location of the boreholes into account and used the nearest borehole (47%).

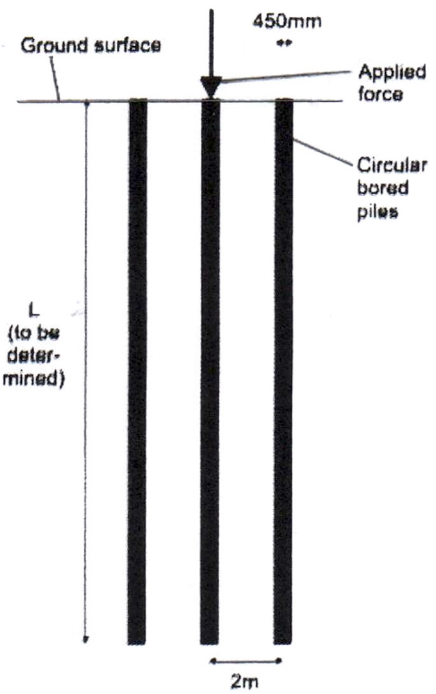

Figure 9. Example 2.3 - Pile in clay design situation

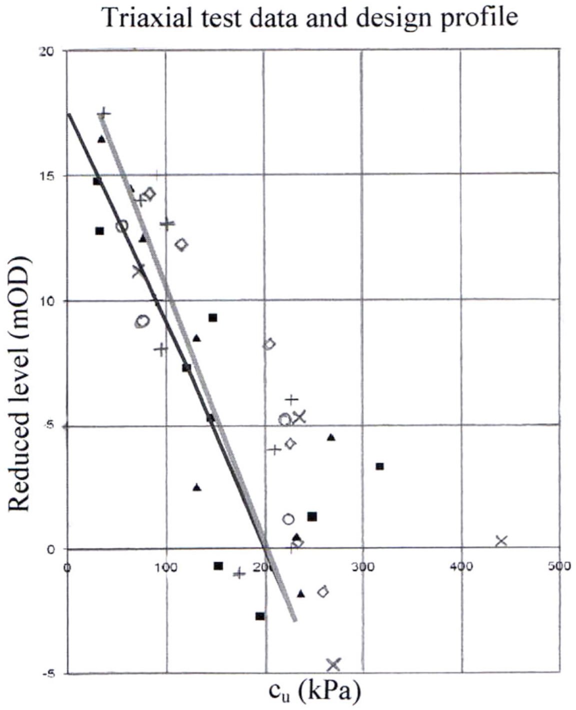

Figure 10. Laboratory c_u v. depth

The most popular method to select the characteristic $c_{u,k}$ value was "by eye", which was used by 53%; linear regression was used by 18% and other methods were used by the remainder. In spite of the different geotechnical data sources used and the different methods adopted to select the characteristic $c_{u,k}$ value, the COV of the $c_{u,k}$ values was less than 0.10 below a depth of 7m. The UK and Portuguese respondents used model factors of 1.4 and 1.5 respectively on c_u. The respondents from these countries also used the alternative design method based on the $c_{u,k}$ value with the model factor applied whereas the other respondents used the model pile method and a c_{uk} value selected from field test results. A majority, 69%, of the respondents used DA1 while the remainder, used DA2.

The average design pile length was found to be 15.1m for the ULS and 14.0m for the SLS so that the ULS controlled the design. The COV of the chosen pile lengths was 0.20 for SLS and 0.28 for ULS. The reporter makes the interesting observation that the average pile length chosen by the UK respondents was 12.5m, which is significantly less than the average for all the other respondents. This probably reflects the fact that the UK designers have used their "local experience" of the performance of piles in London Clay to obtain a more economic design.

2.4 *Example 2.4: Earth and Pore Water Pressures on a Basement Wall*

Example 2.4 differed from the other design examples in that it did not ask the respondent to determine the design dimensions of a particular structure; instead it asked them to address the realistic design situation of assessing design water levels behind and earth pressures acting on the retaining wall shown in Figure 11, which has fill directly behind the wall, with no drainage provided, and natural ground beyond the fill. Water depths measured in boreholes at three distances of 10, 25, and 50m from the wall were 2.2, 1.5, and 3.1m, so that the average water depth was 2.3m. The respondents were asked to give, for both ULS and SLS design situations, the characteristic and design water depths at the back of the wall for the following three design situations with different combinations of fill and soil types: A) Clay soil and clay fill, B) Clay soil and granular fill, and C) Gravel soil and granular fill; and to state how they would calculate the ULS earth pressures.

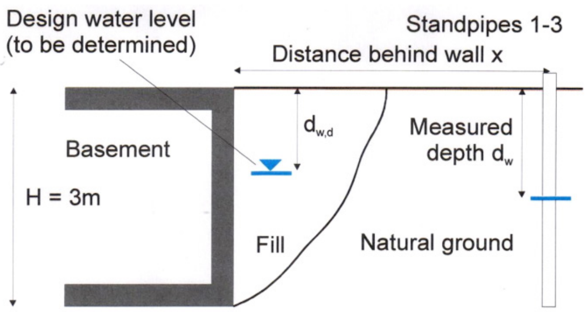

Figure 11: Example 2.4 - Basement wall design situation

Table 3. Water depths and thrusts on basement wall

Design Situation	Natural ground and Fill material	Average water depth (m)	Average characteristic water depth, d_k	SLS depth, d_{SLS}	Water thrust $P_{w,SLS}$ (kN/m)	ULS depth, d_{ULS}
A	Clay soil and clay fill	2.3	0.7	0.7	28	All situations
B	Clay soil and granular fill	2.3	0.7	0.7	28	56%: $d_{w,ULS} = d_k$
C	Gravel soil and granular fill	2.3	1.3	1.3	16	Others: $d_{w,ULS} = 0$

Schneider (2010), in his presentation on this example, gives the average of characteristic, SLS and ULS water depths chosen by the respondents and the average SLS water thrust on the wall for the three design situations, which are shown in Table 3.

Comparing the solutions received for all the examples, the greatest variability in the results, and hence the largest COV values, occurred in the case of this example, with COV = 0.57 for SLS and 0.40 for ULS. For SLS design of the basement wall, all the respondents except 2 stated that the design water depth was equal to the characteristic water depth. For Design Situation A, with clay soil and clay fill, the average of the given characteristic water depths was 0.7m. For Design Situation B, with clay soil and granular fill, the average of the given characteristic water depths was 0.66m, while for Design Situation C, with gravel soil and granular fill, the average of the given characteristic water depth was 1.3m. The thrust on the wall from the water pressure is non-linear and hence is very sensitive to the chosen design water depth. In summarising the responses to this example, Schneider (2010) noted that:

- The deepest average characteristic water depth of 1.3m, which is 1.0m higher than the average measured water depth, was obtained for Design Situation C with gravel soil and granular fill; while a shallower average characteristic water depth of 0.7m was obtained for both Design Situation A and B with the clay soil and granular fill and the clay soil and granular fill
- The SLS water depth was chosen as the characteristic water depth by all respondents
- The water thrust was calculated assuming a triangular water pressure distribution
- 56% of the respondents chose the characteristic water depth for the ULS water depth while, of the remaining respondents, most chose the characteristic ground water level at the surface, i.e. $d_{k,ULS} = 0$
- To calculate the earth pressure, 22% used K_a, 50% used K_0, 11% used $(K_a + K_0)/2$, 6% including compaction pressure and it was unclear how 6% calculated the earth pressure
- With regard to Design Approach, 24% used DA1 with both Combinations 1 and 2, 18% used DA1 and just Combination 1, while 58% used DA2
- With regard to factoring the characteristic water pressure, 50% factored it by 1.35 but when the characteristic water level was chosen at the ground surface, a factor of 1.0 was often applied.

In conclusion, Schneider noted that more guidance was needed in EN 1997-1 concerning the selection of the characteristic water depth and the selection of the depths for ULS and SLS design situations. He posed the following questions:

- Should a partial factor greater than 1.0 be applied when the characteristic water level is at the ground surface?
- Does a partial factor greater than 1.0 on the characteristic water force make sense on physical grounds or should a partial factor only be applied to the water depth?

2.5 *Example 2.5: Embankment on Soft Peat*

Example 2.5 was to determine the height for the initial stage of an embankment to be constructed on pseudo-fibrous to amorphous peat resting on sand at a depth of 7m. The geotechnical information provided consisted of 5 borehole logs spaced at 40m to 50m along the centreline of the embankment and 5 field vane tests giving the measured $c_{u,vane}$ values shown in Figure 12. It was stated that the topsoil in this example was not to be removed, there was to be no hydraulic fill behind the embankment, no construction traffic on the embankment and no serviceability requirements or accidental design situations.
Farrell (2010), in his presentation on this example, noted that to derive c_u for this example, a majority of the respondents, 83%, used the measured $c_{u,vane}$ values directly and only 17% applied a correction factor to account for the field test conditions including a factor of 0.5 to account for the fibrous nature of the peat. With regard to accounting for the location of the boreholes and field vane tests, since no allocation plan for the embankment was given, 50% of the respondents used the average of the results from all the boreholes and 17% made a pessimistic choice of borehole. The characteristic $c_{u,k}$ values selected by the respondents have been plotted in Figure 13 and are very different from each other: 58% selected the $c_{u,k}$ value by eye while the remainder used a statistical approach. Some of the selected $c_{u,k}$ values are constant with depth while others decrease at first and then increases.

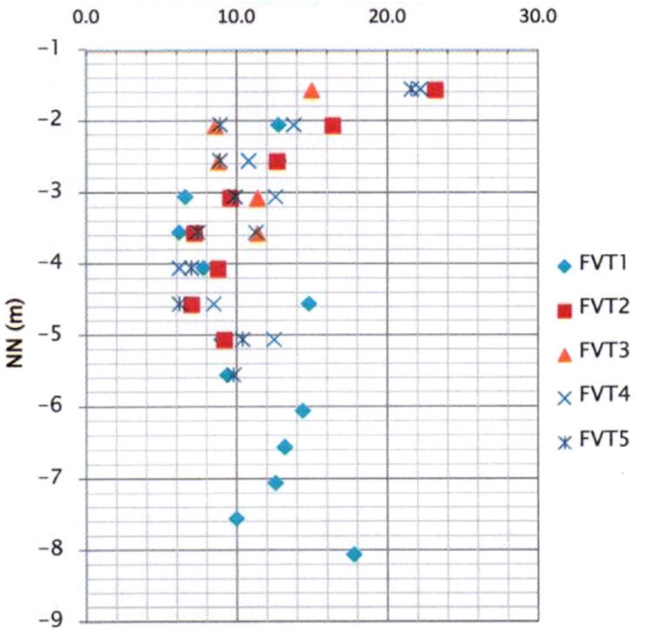

Figure 13. Example 2.5 measured $c_{u,vane}$ v. depth

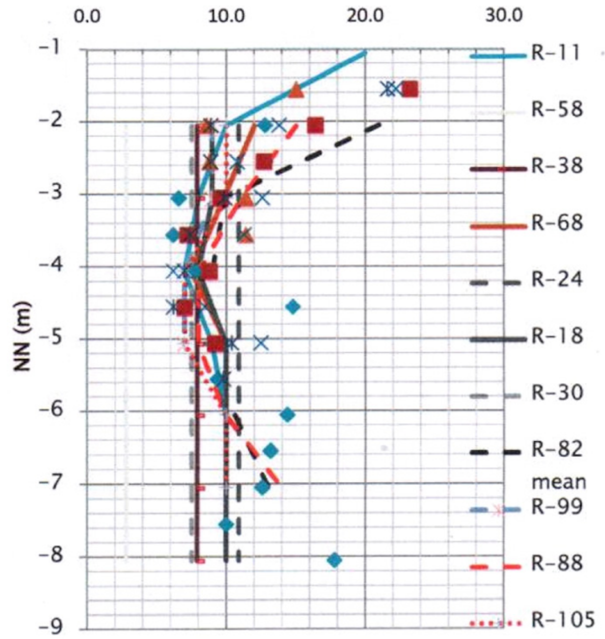

Figure 14. Example 2.5 characteristic $c_{u,k}$ values v. depth

Farrell (2010) noted that only two responses were obtained concerning the type of calculation model used to determine the maximum height of the embankment. However two models were mentioned: a slip circle analysis using Bishop's variable interslice forces, and a bearing resistance failure model. With regard to the Design Approach adopted, 58% used DA1, and 33% used DA2 and/or DA3 and 8% used a purely statistical method. The range of design heights obtained by the respondents was very large, ranging from 0.6m to 2.35m, with an average height of 1.67m and a COV of 0.32. This range was reduced in the second stage, when the respondents were given benchmark $c_{u,k}$ values to use. Only 4 respondents submitted designs based on the benchmark $c_{u,k}$ values. These design height values had very similar COV values to their designs based on the selected $c_{u,k}$ values, 0.28 compared to 0.30, but the embankments heights with the benchmark values ranged from 1.0m to 2.0m, with an average value of 1.53m compared to their original heights which ranged from 1.1m to 2.35m with an average value of 1.63m. This indicates that, unlike the spread foundation in Example 2.2, the differences between the designs is not principally due to how

$c_{u,k}$ was selected but due to the other choices made during the design process, such as the calculation model, Design Approach and partial factor values.

In his report on this example, Farrell (2010) listed the following issues, raised by the respondents, that either require consideration when designing an embankment to Eurocode 7 or should be taken into account when revising the current version of EN 1997-1:

- How to account for local experience and whether to use correction factors for c_u from field vane tests
- Whether it is appropriate to use the bearing resistance model in Eurocode 7 for design of embankments
- The effect of using different calculation models
- How to apply the partial factor on earth resistance in slope stability analyses using DA2
- The merging of DA1 and DA3 for the analysis of slopes
- The different way partial factors may be applied in slope stability analyses
- Whether to account for tension cracks in the analysis of an embankment.

2.6 *Example 2.6: Pile in Sand*

Example 2.6 was to determine the design length of 450mm diameter bored piles supporting a building on clay with peat seams over fine sand. The piles were required to support a vertical compressive permanent load of 300kN and variable load of 150kN. The geotechnical data provided were a borehole log and a CPT test result showing a cone resistance value varying around a mean value of about 3MPa in the clay, increasing to about 17MPa in the top of the sand at a depth of 18m and then slowly decreasing in the sand to about 11MPa at a depth of 28m. The piles were being used to transfer the loading from the building to the lower sand stratum. It was stated that settlements would not control the design and since it is a small project load, no load testing was to be carried out.

Table 4. Average characteristic cone, pile shaft and pile base resistances and average pile design length

Depth (m)	av. $q_{c,k}$ (MPa)	COV $q_{c,k}$	av. $q_{s,k}$ (MPa)	COV $q_{s,k}$	av. $q_{b,k}$ (MPa)	COV $q_{b,k}$	av. L	COV L
2.5	4.2	0.70	16.0	1.33				
7.5	3.0	0.46	20.4	0.79				
12.5	2.5	0.43	22.6	1.41			18.73	0.08
17.5	13.5	0.24	84.4	0.44	3564	0.60		
22.5	13.8	0.08	97.8	0.43	3846	0.68		

The questionnaire for this design example asked the respondents to select the characteristic cone resistance $q_{c,k}$, characteristic unit pile shaft resistance $q_{s,k}$ and characteristic unit base resistance $q_{b,k}$ at the selected depths of 2.5, 7.5, 12.5, 17.5 and 22.5m. The way the respondents selected their characteristic values was: by eye – 50%, by statistical analysis – 23% with the others using a variety of different methods including previous design experience. Annex D of EN 1997-2 provides two models for calculating the resistance of a pile from CPT tests results and, while 38% of the respondents used these models, the majority used alternative calculation models to obtain q_s and q_b.

The average of the characteristic values selected by the respondents and their COVs are given in Table 4 together with the average design pile length, L and the COV of the L values. As Kłosiński (2010) has noted in his report on this example, there was a large scatter in the $q_{c,k}$ values chosen by the respondents for the upper clay stratum, with many respondents selecting $q_{c,k} = 0$ while others selected high values of 4, 5 and even 8MPa. This large scatter is reflected in the high COV values for $q_{c,k}$, which range from 0.43 to 0.70. There was less scatter in the $q_{c,k}$ values chosen for the sand stratum, which had a COV of 0.24 at a depth of 17.5m and a COV of only 0.08 at a depth of 22.5m. The large scatter in the $q_{c,k}$ values in the clay resulted is the very large scatter in the $q_{s,k}$ values as shown by the COV values for $q_{s,k}$ in Table 2.4 which range from 0.79 to 1.41 in the clay stratum. This large COV value for the clay arises because many respondents chose $q_{s,k} = 0$ while others chose $q_{s,k}$ values of 74kPa at 2.5m, 52kPa at 7.5m and 111kPa at 12.5m. Although there was less scatter in the $q_{c,k}$ values selected for the sand stratum, there was still a great scatter in the respondents' $q_{b,k}$ values, which ranged from 56 (!) to 6600kPa at 17.5m depth, with a COV of 0.60.

With regard to the Design Approaches adopted and the partial factors chosen to calculate the design length of the pile, Kłosiński reported that 46% used DA1, 38% used DA2, 8% used DA3 and 8% used a reliability based design. However, when adopting these Design Approaches, Kłosiński noted that the partial factor values used by some of the respondents, which were taken from their National Annexes, are larger than the recommended values in EN 1997-1. The design pile lengths were found to range from 16.5 to 21.0m and had a COV value of only 0.08. Thus, in spite of the large scatter in both $q_{s,k}$ and $q_{b,k}$, there was little scatter in the design pile lengths. Indeed, this COV value is the lowest for all the six design examples. The reason for this appears to be that, when carrying out their pile designs, the respondents have made use of their experience regarding the performance of a pile and the fact that it needs to be founded in the sand stratum, so that they have selected their characteristic pile resistances and chosen their calculation method and partial factor values, together with correlation and model factors, in such a way that, in this design example, the different decisions made during the design process tend to compensate and the pile designs tend to converge.

In reviewing this example, Kłosiński expressed disappointment with regard to the level of harmonisation that has taken place in the design of piles following the introduction of Eurocode 7. Indeed he states that it is difficult to say if a method of designing piles to Eurocode 7 exists since Eurocode 7 allows so much freedom with regard to the calculation methods for the design of piles. If the use of Eurocode 7 does not lead to uniformity in the calculation methods, he says it should at least achieve a comparable level of safety and economy for pile designs.

3 OVERVIEW OF SECOND SET OF DESIGN EXAMPLES

3.1 *Comparison between first and second set of examples*

Bond (2010), in his presentation at Pavia, compared the two sets of design examples by looking at the interquartile range (in which 50% of values lie) normalised by the mean. The results of the 1st and 2nd set of examples presented in Dublin and Pavia are presented in Figures 15 and 16. These figures show that, in spite of providing the raw data rather than the characteristic parameter values for design, the scatter in the results for the spread and pile foundations was generally less in the second set than in the first, particularly in the case of the piles; however the scatter for the earth/ water pressure and embankment exampleswas greater than for the first set. The reduction in scatter for the spread and pile foundations reflects, to some extent, the passage of five years and experience gained in the use of Eurocode 7 since the first set of examples. It also indicates that the selection of characteristic parameters from raw data does not significantly affect the scatter obtained in the designs. However, in the case of the earth/water pressure example, the selection of characteristic water level significantly affects the design and this is an aspect on which Eurocode 7 provides little guidance and which needs to be addressed.

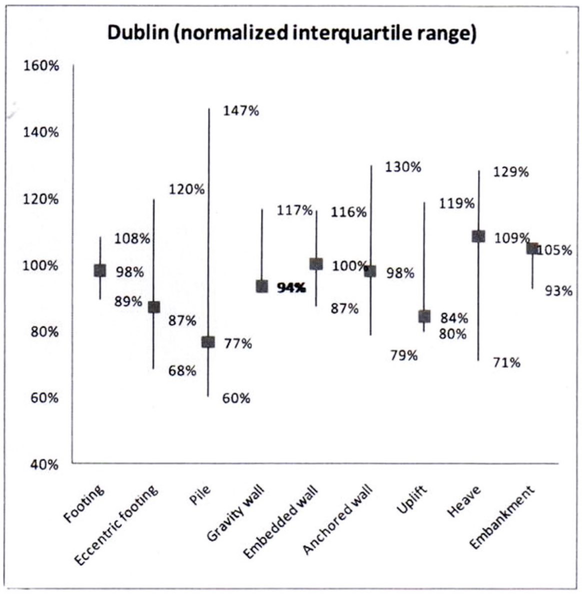

Figure 15. Normalised results for 1st Set of examples

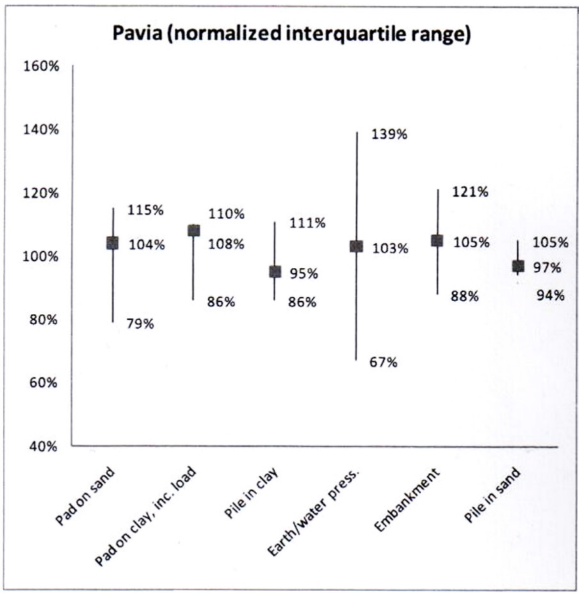

Figure 16. Normalised results for 2nd Set of examples

There was great variability in the embankment designs and a high COV value was obtained for the designs in the case of this example. When examining the designs for this example, particularly when comparing the initial designs based on the raw data with those based on the benchmark characteristic $c_{u,k}$ values, it was concluded that the variability was mainly due to the calculation model chosen and the Design Approach and partial factor values adopted rather than the how the $c_{u,k}$ value was selected.

3.2 *Confidence in the designs to Eurocode 7*

The questionnaires for the different examples asked the respondents to assess their designs to Eurocode 7, whether they thought designs to Eurocode 7 were sound, how conservative they thought their designs were and how they compared to their previous national practice. A summary of the responses to these questions for Example 2.2, 2.3, 2.5 and 2.6, expressed in terms of the percentage of the total number of respondents, is provided in Table 5. These results show that, for all these examples, which involving different design situations, the vast majority (82%) of the respondents were confident that their designs to Eurocode 7 were sound and, with regard to the conservatism of their designs to Eurocode 7, 60% on average considered to them be about right and 35% considered them to be conservative or very conservative.

Table 5. Assessment of Designs to Eurocode 7

Example	Confident / very confident Eurocode 7 designs are sound (%)	Conservatism of designs to Eurocode 7 (%)				Eurocode 7 conservatism compared to previous national practice (%)		
		Very cons.	Cons,	About right	Uncons,	More cons.	About Same	Less cons.
2.1	86	0	27	72	0	5	72	22
2.2	73	0	25	67	8	0	67	33
2.3	94	8	23	62	8	17	67	17
2.4	78	0	57	43	0	31	62	8
2.5	83	14	14	71	0	38	50	12
2.6	69	0	54	36	9	18	55	27
Averages	82	3	32	60	4	16	64	20

When comparing the Eurocode 7 designs with those to existing national practice, they were considered to about right by 64%, less conservative by 20% and more conservative by 16%. Hence the respondents' assessments of the designs to Eurocode 7 were favourable. They generally were confident in their designs to Eurocode 7 and the majority considering their designs to be about the same as previous designs and to be about right or conservative.

4 CONCLUSIONS

The 2nd set of Eurocode 7 design examples presented at the Pavia Workshop in 2010 have provided some interesting information about Eurocode 7 and its application in practice. They have shown that, since the 1st set of examples were presented at the Dublin Workshop in 2005, geotechnical engineers have developed confidence and consistency in the use of Eurocode 7 for the design of spread and pile foundations. However, there is still great variability in how characteristic values are chosen. This is particularly so in the case of water levels for basement wall and retaining structures and hence this is an area that requires to be addressed in a future revision of Eurocode 7.

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