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A Proposal for Some Modifications of EN 1997-1 Design Approaches

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Abstract. All Eurocodes are currently under a critical review, while the work for a second generation of codes are about to start in 2015. For the geotechnical design EN 1997-1 is facing high demands for harmonization and simplification of the present code. The paper presents some proposals for improving the code regarding ultimate limit state (ULS) design. The goal is to make the code better in accounting for uncertainties involved in the design and possible consequences of an ultimate limit state. When applying a material factor approach (MFA), the partial safety factors are suggested to depend on both the uncertainty of the material and the consequence of failure. Such an approach is well suited for slope stability analysis. However, the authors suggest that also the uncertainties involved with loads should be placed on the material factors. For retaining wall design and load factor approach (LFA) is suggested in addition to MFA similar to present Design Approach (DA) 1 in the Eurocode. This approach gave the most consistent design for all cases in a comprehensive study preformed. Some modifications are though suggested also for DA1 to make the design simpler and even more consistent.

Keywords. safety, consequence of failure, Eurocode, stability, retaining wall

1. Introduction

All Eurocodes are currently under a critical review, while the work for a second generation of codes are about to start in 2015. For the geotechnical design EN 1997-1 is facing high demands for harmonization and simplification of the present code. At present, the European countries seem to agree on how to apply safety into slope stability calculations (Bond 2013). Still it has been pointed out, that i.e. the present differentiation of consequence classes in the Eurocode is not working for slope stability (Länsivaara and Poutanen 2013). For retaining wall design all three design approaches (DA) in EN 1997-1 are in use (Bond 2013). It is thus fair to ask what is the most consistent way to apply safety in retaining wall design?

In the paper the application of safety is considered for these two cases. The general aim is to propose how the safety should be applied so that it would be transparent, result in a more constant reliability index, and that the differentiation of consequence classes would influence on the probability of failure for all cases.

2. Slope Stability Analysis

2.1. Application of Safety in EN 1997-1

According to Bond (Bond 2013) the European countries seems to be very much in line on how to apply safety in slope stability. In all countries either DA1 or DA3 can be applied. For slope stability these two are analogous, while safety is applied on strength and on actions. The recommended values for partial safety factors on soils strength are γ_{ϕ}’ = γ_c’ = 1.25 for effective stress analysis and γ_{cu} = 1.4 for total stress analysis, where the subscripts ϕ’, c’ and cu refer to effective friction angle, cohesion and undrained shear strength, and where γ stands for the partial safety factor. Regarding actions, only actions from variable loads are factored, while the partial safety factor for permanent loads is γ_G = 1.0. The recommended value in EN-1997 for variable actions is γ_Q = 1.3.

In the Eurocodes, the differentiation of consequence of failure is addressed by three different consequence classes. A multiplication factor K_{FI} is applied to unfavorable loads and its value depends on the consequence of failure/reliability class (RC).

2.2. Uncertainty of Loads

As given above, permanent loads are not factored in slope stability analysis according to EN 1997-1. This is logical as most of the permanent load comes from soil weight. However, factoring the variable loads results according to the authors to incoherence, as for an example a higher safety level is required for a slope with a railway line where iron ore is transported compared to a slope where a living block is situated. Länsivaara and Poutanen (2013) have showed that if variable loads are factored, then to receive constant probability of failure, the material factor should be vary depending on the magnitude of variable load. Alternatively, the uncertainty in loads could be applied into the material factors.

2.3. Consequence of Failure

Länsivaara and Poutanen (2013) have shown that the multiplication factor K_{FI} has a rather random effect on slope stability. They suggested that the differentiation of consequence of failure should in stability analysis be done by changing the material factor based on the required probability of failure/reliability index. In their paper, they presented partial safety factors that where calculated based on reliability theory to achieve require reliability index values. The target β values for a 50 year reference period were chosen according to EN 1990 as β_{50} = 4.3 (for RC3), β_{50} = 3.8 (RC2) and β_{50} = 3.2 (RC1).

2.4. Proposal for Stability

A full description of the Reliability Based Design (RBD) based partial safety factors for slope stability can be found in the paper by Länsivaara and Poutanen (2013). The proposal presented herein is based on that framework and includes the following main points;

1. All safety are placed on the material factor (to achieve a more constant β).
2. The partial safety factor for strength should depend on the uncertainty of the strength.
3. The consequence of failure should influence the partial safety factor on strength (not load).

In Figure 1 a graph of partial safety factors for different target reliability index values is presented based on the work by Länsivaara and Poutanen (2013). The values are calculated with the following assumptions. Although soil weight is usually left unfactored, the uncertainty involved can be accounted for by including it in the material factor. The coefficient of variation (COV) was set equal to 0.1 for the permanent load. For the variable load, a COV of 0.25 has been used. A normal probability density function (PDF) was used for both the permanent and variable loads and the loads are combined dependently.

The material property distribution was assumed as lognormal and the calculations where done for three different values of coefficient of variation.

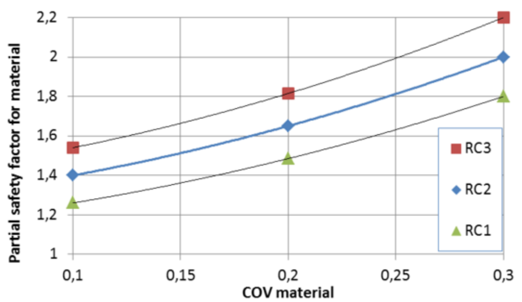

Figure 1. Proposed partial safety factors for different consequence classes. All load factors are equal to 1.0.

The authors don't think it is realistic to assume that in a regular geotechnical project a true variation of the strength could be determined based on performed soil investigations. The standards should define basic requirements and values that should be applied, if no better information is available. This could though be based on the extent of performed soil investigations. If effective stress parameters would be determined based on triaxial tests, values corresponding to COV =0.1 could be used. For values based solely on soundings higher values could be required depending on the method of sounding.

For undrained shear strength the classification could be based on type of tests used. The authors view is, that especially for undrained shear strength the whole scale of COV = 0.1…0.3 could be used. As an example vane test with torque measurement from the ground level and no casing could require a COV = 0.3. If the torque would be applied and measured right above the vane, and in addition cone penetration testing with pore pressure measurement (CPTU) with sensitive cone would be done, a safety factor corresponding to COV = 0.2 could be used. If this would be accompanied with block sampling and extensive laboratory work for full active-direct-passive (ADP) analysis, a safety factor corresponding to COV = 0.1 might be used. Such approach would encourage to do better soil investigations as there would be a clear benefit also to the stake holders.

Instead of presenting graphs for different consequence classes material factors K_{MI} could be introduced. In table 1 the proposed partial safety factors are presented using K_{MI} factors.

Table 1. Proposed partial safety factors for slope stability. All load factors are equal to 1.0.

Uncertainty	COV 0.1	COV 0.2	COV 0.3
γ_M	1.4 K_{MI}	1.65 K_{MI}	2.0 K_{MI}
Consequence	RC1	RC2	RC3
K_{MI}	0.9	1.0	1.1

3. Designing of Retaining Walls

3.1. Application of Safety in EN 1997-1

As stated earlier, all the three design approaches given in EN 1997-1 are in frequent use in Europe in the design of retaining walls. In DA 1, two combinations must be checked. The first combination aims to govern uncertainties related to actions, or their effects, from their characteristic values (γ_G=1.35; $\gamma_{G,inf}$=1.0; γ_Q=1.5), whereas the design values of strength parameters are equal to their characteristic values ($\gamma_{\phi'}$=$\gamma_{c'}$=γ_{cu}=1.0). In DA1 combination 2, the partial factors are mainly applied to strength parameters, governing unfavorable deviations in these ($\gamma_{\phi'}$=$\gamma_{c'}$=1.25; γ_{cu}=1.4), whereas only variable actions are factored (γ_G=1.0; γ_Q=1.3).

DA 2 requires only one verification, where the same values of partial factors are used for geotechnical and structural actions (γ_G=1.35; $\gamma_{G,inf}$=1.0; γ_Q=1.5). The partial factors have been taken from structural engineering. DA 2 can be checked in two ways; depending on when the partial factors are applied. If referred to DA 2, the partial factors are applied to the characteristic actions at the beginning of the calculation, and if referred to DA2*, the calculation is done with characteristic values and the partial factors are applied to action effects at the final state of the calculation. In both ways, for the resistance side of the limit state equation, the partial factors are applied to ground resistance ($\gamma_{R,e}$=1.4).

DA3 is similar to DA1 combination 2. Partial factors are applied to strength parameters ($\gamma_{\phi'}$=$\gamma_{c'}$=1.25; γ_{cu}=1.4 and variable load (γ_G=1.0; γ_Q=1.3) at the start of the calculation. However, actions coming from the structure are multiplied by the partial factors of DA2 (γ_G=1.35; $\gamma_{G,inf}$=1.0; γ_Q=1.5). Only one verification is necessary.

The values for the partial safety factors given above refer to the recommended values. In Finland, load combination equations 6.10 a) and b) in EN 1990 have been chosen for DA2. Then according to Finnish national annex (NA), safety on actions or effect of actions are applied according to one combination with only permanent actions included (γ_G=1.35) and another including both permanent and variable actions (γ_G=1.15; γ_Q=1.5). To compensate the reduced safety from actions, the partial coefficient for resistance has been raised to $\gamma_{R,e}$=1.5 in Finnish NA.

3.2. Basis of the Study

A detailed study has been performed to study the different design approaches in retaining wall design (Knuuti 2015, Knuuti and Länsivaara 2015). The initiative to the study came from concerns that the Finnish NA would give unsafe design in some situations. Three different calculation examples were created in order to compare how the three DA's can match up with the assumed variations in ground properties and variable actions. All cases were analyses with program GeoCalc using beam on springs model with non-linear springs (Vianova 2012).

3.2.1. Case 1

The first calculation example is an embedded sheet pile wall retaining a 4 m deep excavation in sand, Figure 2. The characteristic unit weight of the sand is γ_k=18kN/m^3, friction angle ϕ_k=40° and effective cohesion c'$_k$=0 kPa. The wall is supported by a single row of struts installed at one meter below ground level. A variable imposed surcharge of 10kPa acts at the top of the wall. Groundwater level is assumed to be deep.

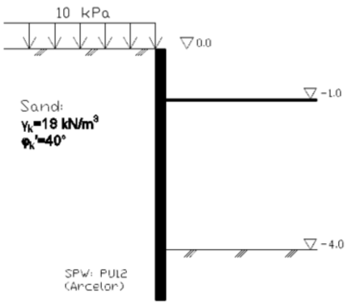

Figure 2. Case 1, excavation in dense sand.

3.2.2. Case 2

The second calculation example is an anchored sheet pile wall in soft clay, Figure 3. The excavation is 10 meters deep, reaching the rock surface. The sheet pile wall is supported by three rows of pre-stressed rock anchors placed at 1, 3.5 and 7 meters below ground level. Installation angle of the anchors is 45 degrees. The toe of the wall is anchored to rock with rock bolts in the excavation phase. For final condition a concrete beam is made at the toe of the wall.

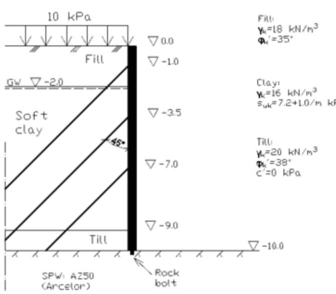

Figure 3. Case 2, excavation in soft clay.

The soil consists of two meter thick fill layer, followed by seven meters of soft clay and a one meter thick till layer above bedrock. Jet grouting is performed in the till back of the wall to prevent water flow. However, the jet grouting is not included in the calculation. Characteristic undrained shear strength of the clay is s_{uk}=7kPa, increasing with depth with Δs_{uk}=1.2kPa/m. Unit weight is assumed to be γ_k=16 kN/m^3. A variable imposed surcharge of 10kPa is acting at ground level of the retained side. Groundwater level is below the fill layer. The wall is made watertight.

3.2.3. Case 3

The third calculation example is a two-story underground car park where the sheet pile wall forms a permanent wall structure, Figure 4. The wall is supported by three reinforced concrete slabs at levels 3.8, 0.7 and -2m. Soil consists of two meters of fill followed by 20 meter thick, stiff clay layer. Depth dependent characteristic undrained shear strength of the clay is s_{uk}=35+1.2kPa/m and unit weight γ_k=15 kN/m^3. A variable imposed surcharge of 20kPa acts at the head of the wall. Groundwater level is at ground level on the retained side and maintained at formation level on the restraining side. The excavation is made using top-down method with step wise exaction for temporary support of the wall.

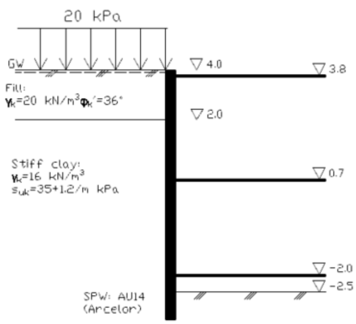

Figure 4. Case 3, excavation in stiff clay.

3.2.4. Variation of the input parameters

In order to investigate the sensitivity of the DA's for misinterprets in characteristic soil strength, variation of the determining soil parameters was allowed. Suitable COV- values were chosen according to Phoon et al. (1995), for friction angle and undrained shear strength. Chosen values are 0.1 and 0.2 respectively. Based on these COV- values and formerly presented corresponding "mean" strength values, standard deviations were calculated. For comparison purposes, reduced strength values were chosen one and two deviations away from the mean value, see Table 2.

Table 2. Variation of the strength parameters of determining soil layers in calculation examples.

Case	property	mean (μ)	-σ	-2σ
Case1	ϕ_k' [°]	40	37.1	33.9
Case2	s_{uk} [kPa]	7.2 +1.2/m	5.6+1.0/m	4.2+0.7/m
Case3	s_{uk} [kPa]	32+1.2/m	28+1.0/m	21+0.7/m

In addition the variable load was given values of 0, 10 and 20 kPa in all cases.

3.3. Conclusion of the Study

In case 1, differences between obtained design bending moments and prop loads were rather negligible between design approaches. Moreover, regardless of the used DA, design values (calculated with ϕ'=40°) were on the safe side even if the angle of shearing resistance had been reduced to its lowest value (40°→33.9°). What proved to be more important is the overall stability of the wall. For example, if the angle of shearing resistance had been taken to be 40^o and in reality would be 33.9^o, the overall factor of safety would reduce to near 1.0 for all DA's or even below for some. It addresses the importance to also lower the level of resisting soil in serviceability limit state (SLS) calculation below expected level when the stability depends on ground resistance.

The excavation type given in case 2, is quite typical in Finland. In DA2, partial factor applied to ground resistance are now useless, because there is no earth on passive side of the wall. Then the whole safety comes from load combinations if the material factor for steel is equal to 1.0. Using the load combinations 6.10 a) and b) given in Finnish NA gives then a quite low overall safety. When variable actions are about 10 % from the total actions, the total factor of safety for the design is near 1.2. This can be avoided by using separate model factors.

The highest safety for case 2 was obtained with DA2 (recommended values) and DA1 combination 1, which in this case are analogous. DA1 combination 2 and DA3 are not well suited for this design case. That is, because when the strength of the clay is low, the influence of the partial factor applied to it is also small. For DA3, the design bending moments and anchor forces were much smaller than those obtained by DA 1 combination 1 and DA2. For DA3 the design bending moment was only 17% higher than the characteristic bending moment from SLS calculation. And even more worrying, the anchor forces for the third anchor row were in practice the same for ULS and SLS, i.e. the application of safety to strength did not increase the design anchor forces.

Case 3 is rather difficult to make comparisons with high variability in strength, as that would influence much on the design. To be able to compare the results reasonably, the embedment depth was taken as 1 m for all cases.

Opposite to the results obtained for case 2, in case 3, DA1 combination 2 and DA3 gave the largest design bending moments and prop loads. Now the effects of the partial factors applied to the ground properties are of high importance. Although none of the DA's cannot take account of the increases in design moments and prop loads if the clays undrained shear strength drops from 35 kPa to 21 kPa, but the differences are substantial when comparing DA1-2 and DA3 to DA1-1 and DA2. For DA1-2 and DA3 the ULS bending moments (s_u=35kPa) are only 12% lower than SLS moments (s_u=21kPa), whereas the ULS moments obtained with DA1-1 and DA2 are over 60% lower than SLS moments. It is also interesting to note that for support level 2 material factoring would govern the design (as for bending moments) while for support levels 1 and 3 load factoring governs and material factoring gave only a very moderate increase to the prop loads.

Based on these design cases, the DA 1, with its two combinations, seems to produce the most consistent design for retaining walls. The study supports thus the conclusions made by Simpson (2007). Also DA 2 can govern uncertainties related to ground properties rather well, but when the unfavorable deviations in strength are high, problems may occur. Moreover, in some design cases the partial factor for ground resistance can be useless. Considering given examples, the DA 3 is seen as problematic for cases where the soil strength is low.

3.4. Uncertainty of Loads

In the Eurocode's the applied load factors origins from structural design. One should then bear in mind, that the uncertainties involved might be quite different in geotechnical engineering. For example, in DA2 the load in retaining wall design is active earth pressure, and the uncertainty includes also uncertainties in strength, earth pressure theory and calculation model, likewise for permanent and transient conditions. Obviously a load factor of 1.15 (Finnish NA) will not cover these uncertainties. It can also be discussed if it is justified to require a higher load factor for a variable load than for a load coming from i.e. an historic building. The consequence of different load factors is that the calculations become more complicated. Also for DA3 and DA1-2 it may be asked if it is justified to factor a load coming from a variable load, but leave the load from an existing building unfactored.

3.5. Proposal for Retaining Wall Design

Based on the performed study, the authors recommend the use of DA1 as the best alternative out of the three. It works consistently for both cases where soil strength dominates the behavior and in cases where factoring strength has no impact. However, the authors suggest the following changes.

- Permanent and transient loads should be treated equally
- When material factor approach is used (DA1-2) all uncertainty should be place on strength and the influence of consequence should also be in soil strength.

This would then give the following two combinations.

DA1-1: $\gamma_G = \gamma_Q = 1.5\ K_{FI,}\ (\gamma_M = \gamma_R = 1{,}0)$

DA1-2: $\gamma_M = f(COV_M)K_{MI,}\ (\gamma_G = \gamma_Q = \gamma_R = 1{,}0)$

where the material factor would be taken as proposed for stability in chapter 2.4.

If DA2 is applied the authors recommend that only load combination 6.10 should be used. In addition care should be taken in situation where soil strength is high but factoring the resistance has low impact.

If DA3 is used, a similar combination to Danish NA (DS/EN 1997-1 DK NA, 2013) load combination 5 is suggested to be included. Therein low influence of factoring soil strength is compensated by increasing the material factor for the structural component.

4. Conclusions

The authors suggest that more focus should be placed into the true uncertainties involved in geotechnical engineering, and that the partial safety factors would be based on them.

When differentiating the consequences of failure in different classes and requiring different target reliability index values for those, the impact should be placed not only to loads, but alternatively to material strength.

After a comprehensive study of three retaining wall examples the authors concluded that DA1 in the Eurocode performed in most consistent way out of the three design approaches for all situations. Small changes are suggested to make the design simpler and more consistent with respect to uncertainties in loads and consequence of failure.

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