

Approaches to ULS design – The merits of Design Approach 1 in Eurocode 7

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ABSTRACT: Development of Eurocode 7 (EN1997-1) has led to many debates about the way safety should be ensured in design calculations, particularly when considering ultimate limit states. As a result, EN1997-1 allows national choice between three “Design Approaches” which are different ways of combining partial factors. In this paper, some of the basic principles of geotechnical design are reviewed, providing the rationale for the development of Design Approach 1 – DA1.

1 INTRODUCTION

Geotechnical design involves materials – soil – whose strength depends principally on friction, used in combination with conventional structural materials. Much of the loading is derived from the weight of the ground itself, and the strength and loads (or actions) interact in ways which are often complex and non-linear. These features distinguish geotechnical design from structural design, and form the background to the choice of a suitable safety format.

The paper will consider some general principles about ultimate limit state design, using examples related to Eurocode 7. Four aspects will be addressed:

- Consistent geotechnical and structural design. The structure and the ground are all part of the same problem.
- The benefits of using “combinations” of factors.
- The importance of applying factors to parameters before they are combined.
- The use of finite element methods in ULS calculations.

Limit states are states at which the behaviour of the structure is unacceptable and which should therefore be prevented. Ultimate limit states (ULS) are defined in EN1990 as those which concern the safety of people or of the structure. This fundamental definition is based entirely on the practical issues of degrees of danger, damage and, by implication, cost of repair. It does not necessarily dictate the form of calculation required to ensure that the state is prevented. Because ultimate limit states are very serious, their occurrence must be made an *unrealistic* possibility.

2 DESIGN APPROACHES IN EN1997-1

EN1997-1 allows national choice between three “Design Approaches” for ULS, which are differing sets of partial factors - each nation is free to choose which approach or approaches it will allow for construction which will take place on its territory. The main factors proposed by EN1997-1 are summarised in Fig.1, in which green cells represent partial factors of 1.0. This chart is a simplification of 10 pages of factors.

In DA1, the user is required to carry out two separate calculations, with separate sets of partial safety factors. These have come to be known as Combinations 1 and 2, and the combination giving the more adverse results governs the design decisions. The design, as it will be built, must accommodate both combinations. It will often be obvious which combination is more critical, and in those cases users will avoid the trouble of a dual calculation.

			Design approach 1						Design approach 2			Design approach 3					
			Combination 1			Combination 2 - piles & anchors			DA2 - Comb 1			DA3 - Slopes					
			A1	M1	R1	A2	M2	R1	A2	M1 or ... M2	R4	A1	M=R2	A1	A2	M2	R3
Actions	Permanent	unfav	1,35									1,35		1,35			
	Variable	unfav	1,5			1,3			1,3			1,5		1,5	1,5	1,3	
Soil	$\tan \phi'$						1,25			1,25					1,5	1,3	1,25
	Effective cohesion						1,25			1,25					Structur	Geotech	1,25
	Undrained strength						1,4			1,4					actions	actions	1,25
	Unconfined strength						1,4			1,4							1,4
	Weight density										1,4						1,4
Spread footings	Bearing												1,4				
	Sliding												1,1				
Driven piles	Base									1,3			1,1				
	Shaft (compression)									1,3			1,1				
	Total/combined									1,3			1,1				
	Shaft in tension									1,6			1,15				1,1
Bored piles	Base					1,25				1,6			1,1				
	Shaft (compression)					1,0				1,3			1,1				
	Total/combined					1,15				1,5			1,1				
	Shaft in tension					1,25				1,6			1,15				1,1
CFA piles	Base					1,1				1,45			1,1				
	Shaft (compression)					1,0				1,3			1,1				
	Total/combined					1,1				1,4			1,1				
	Shaft in tension					1,25				1,6			1,15				1,1
Anchors	Temporary					1,1				1,1			1,1				
	Permanent					1,1				1,1			1,1				
Retaining walls	Bearing capacity												1,4				
	Sliding resistance												1,1				
	Earth resistance												1,4				
Slopes	Earth resistance													1,1			

indicates partial factor = 1.0

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Fig.1 Factors given in EN1997-1 Annex A for the three Design Approaches

			Design approach 1			Design approach 2			Design approach 3		
			Combination 1			Combination 2			Combination 2 - piles & anchors		
			A1	M1	R1	A2	M2	R1	A2	M1 or ... M2	R4
Actions	Permanent	unfav	1,35								
	Variable	unfav	1,5			1,3			1,3		
Soil	$\tan \phi'$					1,25				1,25	
	Effective cohesion					1,25				1,25	
	Undrained strength					1,4				1,4	
	Unconfined strength					1,4				1,4	
	Weight density										
Spread footings	Bearing										
	Sliding										
Driven piles	Base									1,3	
	Shaft (compression)									1,3	
	Total/combined									1,3	
	Shaft in tension									2,0	
Bored piles	Base									1,6	
	Shaft (compression)									1,3	
	Total/combined									1,6	
	Shaft in tension									2,0	
CFA piles	Base									1,5	
	Shaft (compression)									1,3	
	Total/combined									1,5	
	Shaft in tension									2,0	
Anchors	Temporary									1,1	
	Permanent									1,1	
Retaining walls	Bearing capacity										
	Sliding resistance										
	Earth resistance										
Slopes	Earth resistance										

Fig.2 Factors adopted in the UK National Annex for DA1

Broadly, in Combination 1 factors are applied to actions but not to soil strengths, and in Combination 2 factors are applied to soil strengths or resistances, with only a relatively small factor applied to variable actions. This follows the scheme proposed in the “Vornorm” version of Eurocode 7, ENV1997 (1994), in which Combinations 1 and 2 were called Cases B and C.

In DA2, factors are applied simultaneously to actions and to soil resistances, such as bearing capacity or passive earth forces. This approach does not work for slope stability problems, for which essentially DA1 Combination 2 is adopted in its place. In DA3, factors are applied simultaneously to actions and soil strengths.

The United Kingdom proposes to adopt DA1, and Fig.2 shows the factors in the current draft of the UK National Annex. Some of the factors recommended in EN1997-1 have been modified slightly, in particular adopting unit factors for soils in Combination 1, as was originally proposed in ENV1997.

In most situations, factors in Combination 2 are applied to soil strength rather than resistances, but an exception is made for pile design. The author suggests that what is really factored in pile design is the strength of the pile/soil interface or the soil beneath the base, which is affected by construction, rather than the body of the soil as a whole.

3 CONSISTENT GEOTECHNICAL AND STRUCTURAL DESIGN

The following proposition is considered to be important: “geotechnical and structural design should all be part of one consistent whole, because the ground and related structures are all part of the same problem and have to work together consistently”.

Fig.3 shows a residential block built in Malaysia at the top of a steep slope; an anchored retaining wall can be seen in front of the block. The situation on the slope is shown in Fig.4. Slope stability analyses using characteristic values for the soil strengths gave factors of safety greater than 1, but they were not big enough to be acceptable to any design approach. It was therefore decided to build the anchored retaining wall, seen in section in Fig.4. The wall was required on the basis of slope stability calculations, and the designers intended to calculate the bending moments and anchor forces using characteristic soil strengths, unfactored, then applying factors to the results. However, this led to an anomaly: for unfactored soil strengths, the wall was not necessary, so the calculated bending moments and anchor forces were effectively zero. And applying a factor to the end result, zero, did not add any safety.

The problem was that this design approach required two inconsistent calculations – one for the slope stability (effectively with factored soil strength) and a different one for the structural design, in which the soil strength was not factored, though the action effects were to be factored. In reality, the slope and retaining wall are part of the same problem. The structure and soil must be designed together – consistently.

So in DA1, the whole of the design – soil and structure – are checked for Combination 1, then the whole design is checked again for Combination 2, unless it is obvious in advance which combination will be critical. In contrast, in DA2 the slope stability and wall design are considered by different approaches, giving the type of inconsistency described here.



Fig.3 Residential block at the top of a steep slope

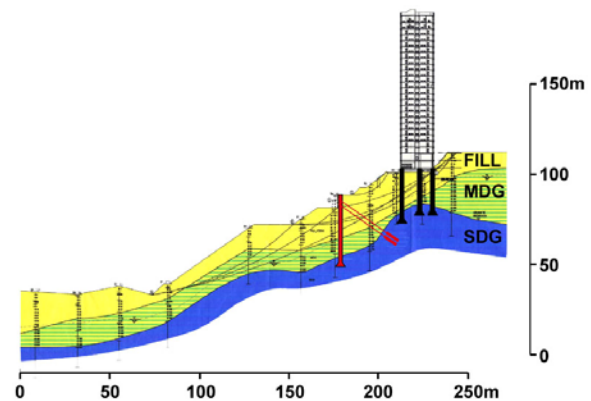


Fig.4 Slope stabilisation measures

4 USE OF TWO “COMBINATIONS” OF FACTORS

DA1 requires that the design is checked against two “combinations” of factors, unless it is obvious which of these is critical. This has the obvious disadvantage that it requires slightly more work on the part of the designer, though in practice three factors minimise this problem: (a) it is frequently the case that the critical combination is obvious by inspection; (b) Combination 1 can often be derived from a

serviceability limit state calculation, which is required by all the design approaches; (c) most computations are carried out by computer and there is very little difficulty in running a second case, if it is needed. Nevertheless, the question arises: why is it necessary to have more than one combination?

One of the aims of design is to achieve roughly constant reliabilities irrespective of how actions, strengths and resistances combine in particular situations. In Annex C of EN1990, reliability is represented by the target reliability index β , which represents the number of standard deviations between the characteristic state and the working state. EN1990 discusses how the values of partial factors might be selected in order to achieve this, proposing that factors could be applied simultaneously to actions and strengths (or action effects and resistances). In effect it proposes that the action effects for ULS design should be 0.7β standard deviations from their characteristic values, and the margin on resistances should be 0.8β . But it places an important limit on this approach: it is only applicable if the ratio of the standard deviations of the action effect and resistance, σ_E/σ_R , lies within the range 0.16 to 7.6. The implication of this is that a different approach is to be used if the uncertainty of one of variables – actions or resistances – is much more important to the design than is the other one. For such a situation, the margin on the more critical variable is required to be 1.0β , with a lower margin, 0.4β , on the less critical variable.

The result of this approach is shown in Figure 5, in which the reliability achieved (in terms of number of standard deviations of the design point from the mean) is plotted against the ratio of the standard deviations expressed as $\sigma_E/(\sigma_E + \sigma_R)$. The result is normalised by dividing by the required reliability, β standard deviations, so that the desired value is 1.0. Over the range in which both σ_E and σ_R are of similar, significant magnitude, the result is reasonably close to the desired value. However, as either σ_E and σ_R becomes small compared to the other one, the reliability achieved drops substantially, indicating an unsafe design with inadequate reliability. This explains why EN1990 limits the range of applicability of the approach to $\sigma_E/\sigma_R = 0.16$ to 7.6.

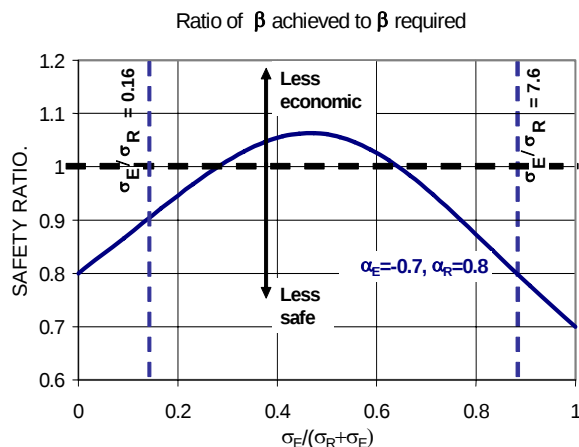


Fig.5 Reliability achieved using (0.7, 0.8) combination for

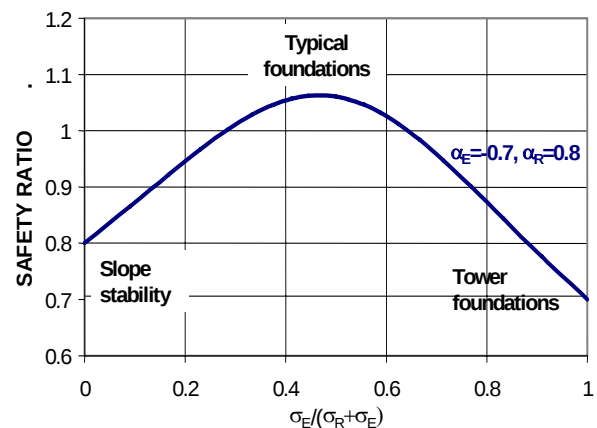


Fig.6 Reliability for some typical geotechnical situations

Figure 6 shows that in geotechnical design it is important to consider the full range of σ_E/σ_R values. Conventional foundations may have σ_E and σ_R of similar magnitude, but other situations are dominated by either σ_E and σ_R . For example, in slope stability problems there is often very little uncertainty about the loading and uncertainty of soil strength is dominant, as shown by the fact that factors of safety are normally applied to soil strength. This leads to the need for a different approach to slope stability in DA2. At the other extreme, designs for foundations of tall towers may have loading as the dominant uncertainty. In geotechnical design, these problems often occur together, so the approach adopted must be able to accommodate the full range of σ_E/σ_R .

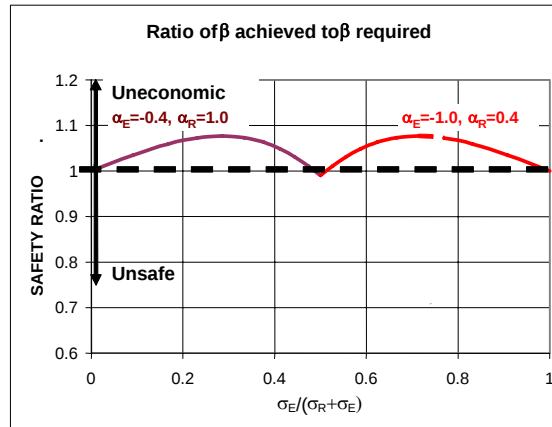


Fig.7 Reliability achieved using (1.0, 0.4) combinations for.

Figure 7 shows the result in terms of reliability of an approach using two “combinations” in which the margin on the more critical variable is required to be 1.0β , with 0.4β on the less critical variable. Much greater consistency is achieved, with none of the resulting values falling substantially lower than required (ie 1.0).

The benefit of the use of two combinations is that a very wide range of design situations can be covered without change in the design approach. In common with other design approaches, the factors used in DA1 have not been deduced by probabilistic calculation. Nevertheless, they do reflect the principles propounded in EN1990 and the lessons which may be learnt by considering a probabilistic framework.

Although the concept of “combinations” is relatively new to geotechnics, first being introduced in the ENV of 1994-5, it is familiar to structural engineers who frequently design for several combinations of actions. The background to DA1 is essentially the same as that of combinations of actions, giving a severe value to the lead variable in combination with less severe values of other variables, but in DA1 the method is extended to include resistances or material strengths, as suggested by EN1990. The fundamental principle of DA1 is that “All designs must comply with both combinations in all respects, both geotechnical and structural”. The “design” meaning “that which will be built”.

5 PARAMETERS MUST BE FACTORED BEFORE THEY ARE COMBINED

EN1990 [6.3.2(4)] refers to “non-linear analysis”, by which it means situations where an action effect changes disproportionately as the action changes. EN1990 does not consider the possibility of a similar disproportionate relationship between material strength and resistance, and this may not be very important in structural engineering. In soils, however, where the strength is essentially frictional, such disproportionality is often significant. For example, passive resistance and bearing capacity both increase disproportionately with angle of shearing resistance ϕ' ; in some cases, when ϕ' is large, a small change in ϕ' has a very large effect on the resistance. The author suggests that similar thinking should therefore be applied to strengths and resistances as to actions and action effects.

Disproportionate effects may occur simply due to the addition of actions which tend to cancel. On a historical note, interest in partial factoring methods in the United Kingdom was encouraged by the study of the collapse of the Ferrybridge cooling towers (for details see http://www.knottingley.org/history/tales_and_events.htm). The stress in the concrete was derived from the addition of compression due to the weight of the towers and tension due to wind loading, which tended to cancel each other. Using a working state approach, this resulting stress was then compared with a factored strength. Unfortunately, the wind loading was underestimated and this led to a disproportionately large increase in the resulting tension, which caused a very serious collapse. The

disaster might have been avoided if the two actions, weight and wind load, had been factored separately before being combined into a single action effect.

Figure 8 shows the requirements of EN1990 when there is a non-linear relationship between actions and action effects. If the changes in the effect are disproportionately large compared with those in the action, then it is important that safety factors are applied to the action, not the action effect.

Similarly, Figure 9 shows situations where there is a non-linear relationship between material strength and resistance. The author submits that if the changes in the resistance are disproportionately large compared with those in the strength, then it is important that safety factors are applied to the strength, not the resistance. This situation commonly occurs in frictional materials and so is relevant to geotechnical design.

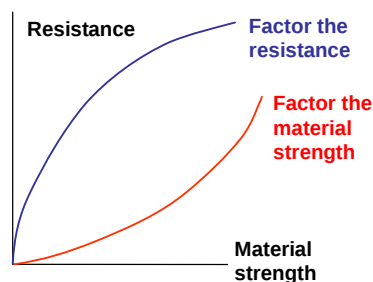


Fig.8 Non-linear relationship between actions and action effects

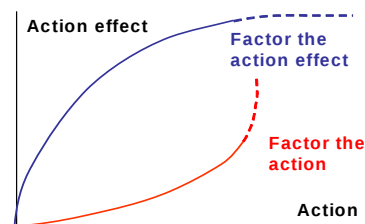


Fig.9 Non-linear relationship between material strengths and resistance

In view of this, parameters are factored before they are combined in DA1, as far as it is reasonably possible, and generally factors are applied to soil strength rather than to “resistances”. Difficult situations for combining actions occur in earth pressure calculations for design of retaining walls, affecting DA1 Combination 1 and also DA2. These are considered in EN1997-1 in 2.4.7.3.2(2) which says “In some design situations, the application of partial factors to actions coming from or through the soil (such as earth or water pressures) could lead to design values which are unreasonable or even physically impossible. In these situations, the factors may be applied directly to the effects of actions derived from representative values of the actions.”

The author submits that this approach is acceptable for DA1 Combination 1, where factors are applied to actions, provided that Combination 2 is also checked, factoring the soil strength before using it to calculate resistances. The absence of an equivalence of this in DA2 is considered by the author to be a shortcoming of DA2. One additional benefit of this approach is that it coincides with conventional structural design, indirectly applying a factor to ground water pressure for the calculation of structural forces and bending moments.

Fig.10 shows the design situation of a simple sheet pile wall. Most simple design methods aim to minimise the wall length, but in the author’s experience this is often not what is really needed. In this case, the wall is required, for reasons outside the designer’s control, to be 12m long. The design requirement is to find the minimum tie force needed to make it safe. Any design approach will show that 12m is not sufficient length for the wall to act safely as a cantilever. However, if calculations are carried out using the unfactored characteristic strength of the ground, it is found that no tie force is needed; in other words, the wall could function as a cantilever, without a tie, based on the characteristic soil strength. If the design method uses this calculation to derive the characteristic tie force, then a ULS design tie force = zero x factor could be obtained. DA1 avoids this problem; its Combination 2 with factored soil strength shows that the minimum tie force acceptable for equilibrium is 75kN/m. (The problem could be avoided in other design approaches by adding other requirements to those of EC7 to specify the method of calculation, but these would take away the flexibility of allowing the designer to choose to minimise the tie force, bending moment or wall length).

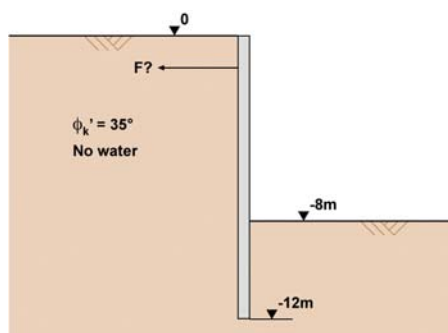


Fig.10 Design situation for a sheet pile wall

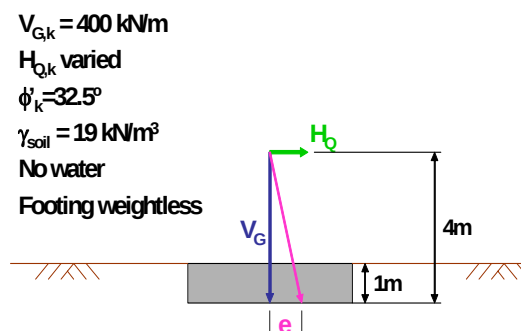


Fig.11 Inclined, eccentric loading on a footing

6 INCLINED, ECCENTRIC LOADING ON A FOOTING

Figure 11 shows an example published by Schuppener (2006) which provides an illustration of the effect of factoring loads before or after they are combined. This involves the calculation of the width of a strip footing subject to inclined eccentric loading, based on ULS bearing capacity calculations. A range of the characteristic value of the horizontal force $H_{Q,k}$ is considered. Figure 12 shows that Combination 1 of DA1 requires two separate action combinations to be considered, applying 1.5 to the variable horizontal action, and 1.35 to the permanent vertical action if it is unfavourable (red dashed arrow), or 1.0 if it is favourable (red solid arrow). This is consistent with EN1990 and the Eurocodes for structural design, and in principle it is also required for DA2 and DA3.

Figure 13 shows similar diagrams for Combination 2 of DA1 and for DA2 and a variant of this referred to as DA2*, which is recommended by Schuppener (2006). In DA2*, for consideration of bearing capacity, the horizontal and vertical loads are combined into a single resultant before the load factors are applied. The value applied is 1.35. The 1.5 factor on the variable action appears to be ignored, but even more importantly, the inclination and eccentricity of the design action effect takes no account of the separate factors on permanent and variable load, or of the possibility that the permanent vertical load is favourable.

The results of calculations of the required footing width for varying ratio H_k/V_k are shown in Figure 14, using the bearing capacity equations in Annex D of EN1997-1. Results for DA1 and DA2 are fairly similar, and DA3 is more conservative, as expected. However, DA2* gives a markedly less conservative, and so less safe design. In Figure 15, the factors of safety available on the single quantities (a) γ_ϕ on $\tan\phi'$ and (b) γ_Q on H_Q have been calculated using the footing widths obtained by DA2*. This shows that DA2* is equivalent to a single factor of safety slightly greater than 1.2 on $\tan\phi'$ or a variable factor on H_Q which falls to about 1.17 at $H_{Q,k}/V_{G,k}$ of 1.4. As the only safety factor in the calculation, a value of 1.17 on the variable load is remarkably low.

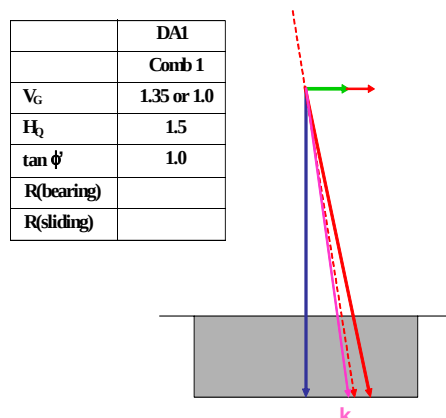


Fig.12 ULS design resultant action derived for DA1

	DA 1		DA2	DA2*	DA2*
	Comb 1	Comb 2		bearing	sliding
V_G	1.35 or 1.0	1.0	1.35 or 1.0	1.35	1.0
H_Q	1.5	1.3	1.5	1.35 (1.5??)	1.5
$\tan \phi'$	1.0	1.25			
R(bearing)			1.4	1.4	
R(sliding)			1.1	Retains characteristic inclination and eccentricity	1.1

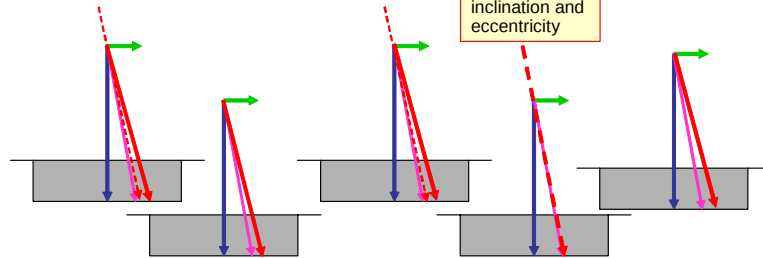


Fig.13 ULS design resultant action derived for DA1, DA2 and DA2*

The proponents of DA2* argue that the results it obtains are in line with previous experience. However, the author submits that: (a) it is unlikely that there is a significant database of footings which have, in practice, been loaded to the high H/V ratios considered here (though it is very important that footings which have to be designed for high ratios are reliable); (b) the approach taken for bearing capacity is inconsistent with that taken for sliding, which is likely to cause confusion; (c) the approach taken for bearing capacity is inconsistent with that taken for structural design, which is also confusing; (d) as noted above, as the only safety factor in the calculation, a value of 1.17 on the variable load is remarkably low.

In practical use, design rules are often pushed to extremes not anticipated by code drafters. For this problem, it is desirable to check what happens if the lever arm of the horizontal load is larger. It is assumed here that structural designers would factor the vertical and horizontal actions independently. Fig.16 shows that at a relatively modest ratio $H_k/V_k = 0.13$ the eccentricity of the resultant load used in structural design exceeds the half width of the footing derived by DA2*, implying that the resultant load passes outside the base of the footing, which cannot give equilibrium. This will clearly cause consternation and confusion to the structural designer, and illustrates the inconsistency of DA2*.

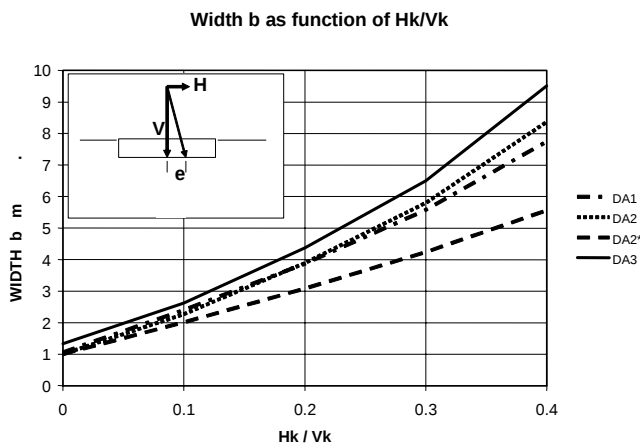


Fig.14 Footing widths calculated for ULS design

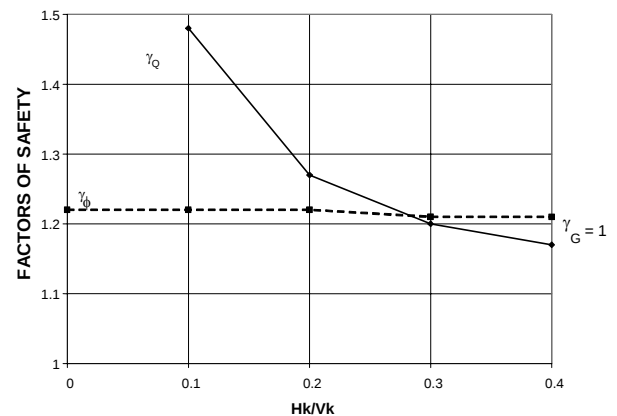


Fig.15 Total factors of safety implied by DA2 on $\tan \phi'$ and H_Q , taken singly.

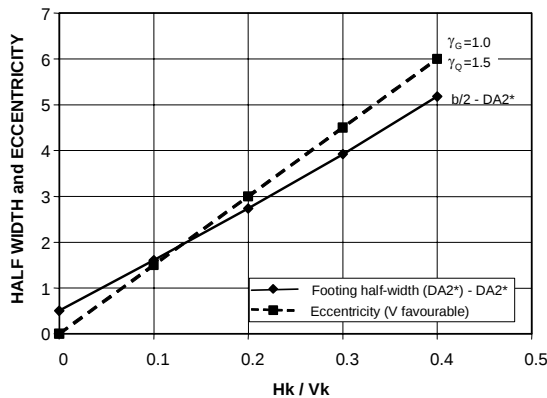


Fig.16 Eccentricities in DA2 and structural design, for 10m arm

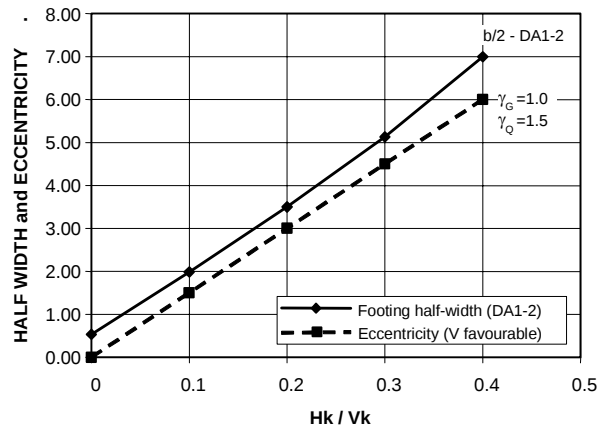


Fig.17 Eccentricities in DA1 and structural design, for 10m lever arm

In contrast, Fig.17 shows that for DA1 the resultant load always lies within the width of the footing; Combination 2 is more critical than Combination 1, and so determines the width. Fig.18 shows that DA1 even accommodates a much more extreme lever arm for the horizontal load of 100m. This is guaranteed since the geotechnical design is checked for the same loading as the structure, consistently, in Combination 1, and for this extreme case it is Combination 1 which determines the footing width. For such an extreme case, much of the width of the footing might be redundant and the structure could be replaced by an A frame, but all the considerations noted above would still apply.

7 A PROBLEM WITH EQU

In carrying out calculations for the example described above, the author noted a problem with the factors given for the EQU limit state in EC7. EQU is defined as “loss of equilibrium of the structure or the ground, considered as a rigid body, in which the strengths of structural materials and the ground are insignificant in providing resistance”; nevertheless, partial factors are provided for strength. For DA1, the critical case above was usually given by Combination 2: $\gamma_G=1.0$, $\gamma_Q=1.5$, $\gamma_\phi=1.25$. But EQU in EC7 requires $\gamma_G=0.9$, for the same values of γ_Q and γ_ϕ , which is bound to be more critical. As EQU is independent of design approach, it will dominate them all, which was not the intention of the drafters of the code for this sort of situation. The author proposes that this problem can be solved by adopting a lower value of γ_ϕ in EQU, such as 1.10 as used in ENV1997 (1994).

8 USE OF FINITE ELEMENT ANALYSIS – COMPARISON OF DA1 AND DA2

Finite element analysis is increasingly powerful and commonly used, so it is essential that new codes of practice provide rules compatible with its use. This can be achieved if factors are to be applied to either (a) the input parameters – actions and material strengths, or (b) the results of the equilibrium established by finite elements, such as bending moments and structural forces. There is no problem in factoring resistances of structural members which are given directly in the input, such as the bending capacity of a plate or shell element, but factoring resistances and action effects which are derived during the finite element analysis is much more difficult. These include passive earth pressures and bearing capacities, for example.

In order to make a comparison between results of DA1 and DA2, a published example of DA2 has been studied. Fig.19 shows the design situation of a sheet pile retaining wall published by Weissenbach et al (2003). The wall is to support a surcharge with a characteristic value of $q=100\text{kPa}$, which Weissenbach et al represented by an additional earth pressure of qK_a spread over a height of 3.32m, giving a force of 74.4kN/m.

The author suspected that this force was too low, and so carried out the finite element analysis

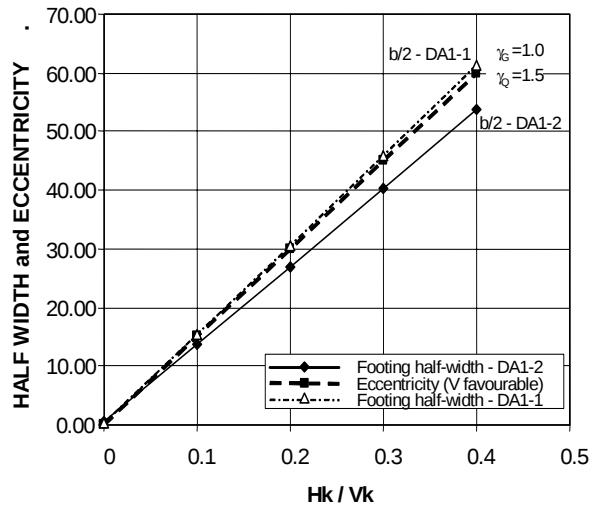


Fig.18 Eccentricities in DA1 and structural design, for 100m lever arm

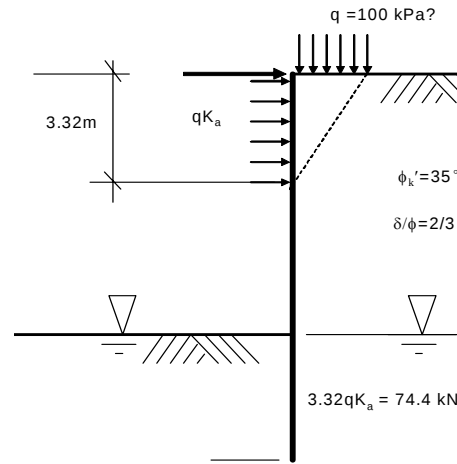
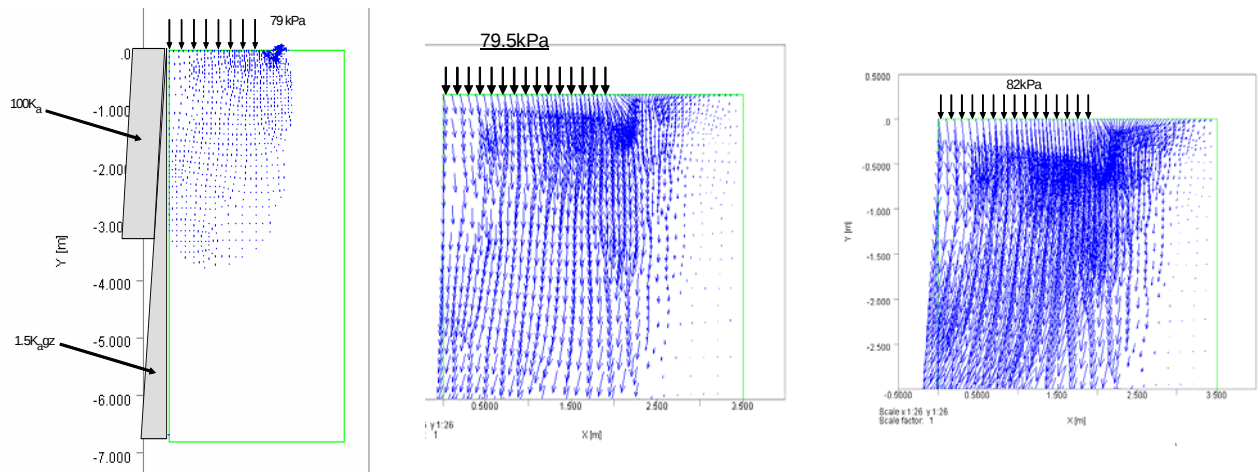


Fig.19 Design situation for sheet pile wall

shown in Fig.20 in which the wall was replaced by applied pressures, inclined at $\frac{2}{3}\phi'$ to the horizontal. The active pressure from the soil weight was more than supported by a pressure of $1.5K_a\gamma_z$ and the additional stress block as proposed by Weissenbach et al. Fig.20a shows that the situation is stable for a surcharge of 79kPa, but Fig.20b and 20c show that failure started at 79.5kPa. The remaining comparisons have therefore been carried out using a characteristic surcharge of 80kPa. All the analyses assumed and justified ARBED AZ 18 sheetpiles, and in each method the shortest wall length giving equilibrium has been found.

Following German practice, the calculations of Weissenbach et al redistribute the active earth pressure behind the propped wall as shown in Fig.21. When the same calculation approach is used with either DA2 or DA1, the results are almost identical, as shown by the first two sets of results in Fig.22. In this figure, the “DA1 FREW” results were obtained using the wall-structure interaction program FREW (Oasys, 2007a), which computed an earth pressure diagram as shown in Fig.23; the surcharge is multiplied by 1.3 as required by DA1 Combination 2. The “DA1 FE” results were computed using the finite element program SAFE (Oasys 2007b), giving the deformed mesh shown in Fig.24. It seems likely that FREW over-estimates the amount of redistribution, but the finite element result gives less than the German approach, leading to a lower prop force and higher wall bending moment.



(a) 79kPa,

(b) 79.5kPa

(c) 82kPa

Fig.20 Finite element check on active pressure caused by surcharge. Surcharge values:

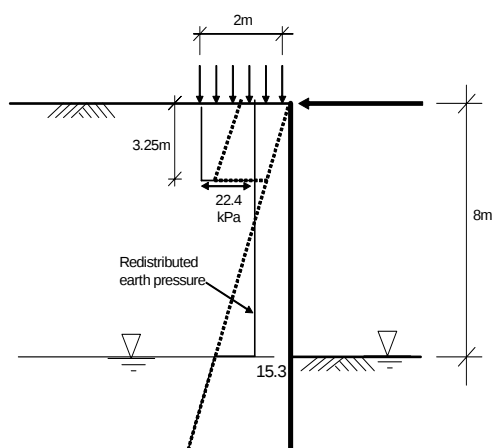


Fig.21 Redistributed earth pressure, as in Weissenbach et al (2003)

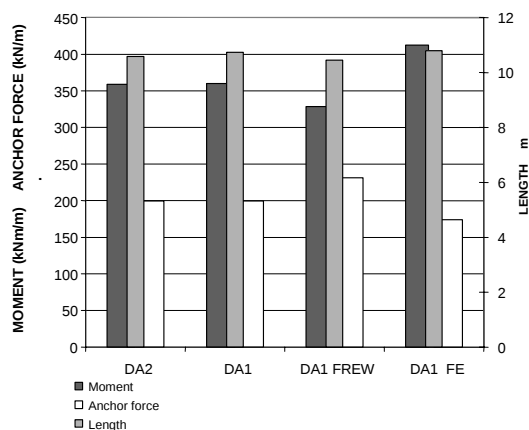


Fig.22 Sheet pile retaining wall - comparison of results

In the author's experience, the results of DA1 and DA2 are often very close, provided the calculations adopted are the same in other respects. Figures 14 and 22 are both examples of this, though it was noted above that the results of DA2* are different. Changing the calculation method, as shown by the FREW and FE results, has a bigger effect. In the author's view, all the results shown in Fig.22 comply with EC7 and would give satisfactory designs; there is a trade-off between prop forces and bending moment.

One advantage of the finite element analysis is that, unlike the other methods, it automatically checks the vertical equilibrium of the wall, provided an adequate mesh is provided at the wall toe. In fact, a soft toe can be provided in the analysis if necessary in order to remove any doubt. It was found in this case that this made little difference, but in other situations with greater vertical loads on the wall it could become important.

It was a simple process to use finite element analysis with DA1, in which input parameters are factored, but the author does not know how it can be done with DA2 in which factors are applied to intermediate quantities such as passive resistance.

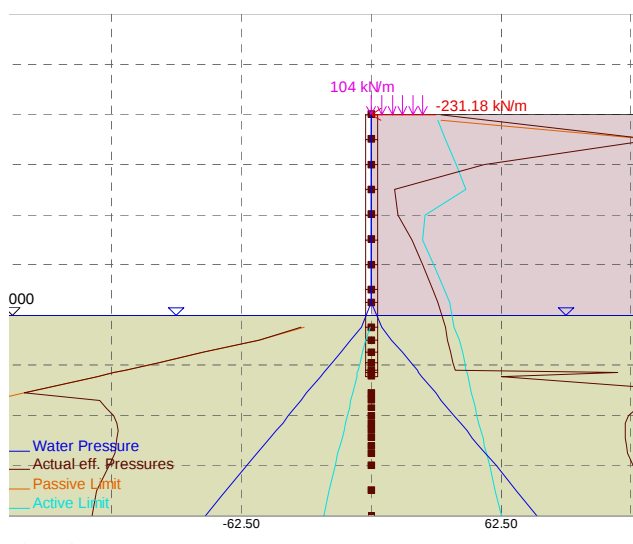


Fig.23 Earth Pressures computed by FREW

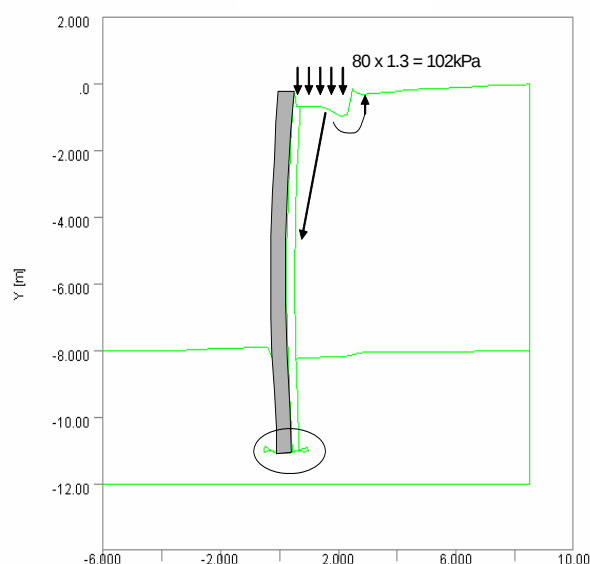


Fig.24 Displacements computed by SAFE Finite element programme

9 CONCLUSIONS

It is suggested that a good design approach, or safety format, should achieve three goals: (a) safety and intelligibility of the approach, (b) economy of the results, and (c) usability with a variety of calculations, from simple methods to finite elements. Whilst perfection is impossible, the author contends that DA1 achieves these goals to a greater extent than the other design approaches of EC7. Some further specific points about geotechnical design, particularly at ULS, are as follows.

1. Geotechnical and structural design should all be part of one consistent whole, because the ground and related structures are all part of the same problem and have to work together consistently. Confusion arises unless geotechnical and structural design are consistent and compatible. This has been achieved in DA1 by adopting the “combinations” approach, so that all structure/ground systems are checked for the normal ULS structural loading and for factored soil strength.

2. The combinations approach of DA1 follows the principles of EN1990, set out to achieve a reasonably consistent reliability index across a wide range of situations. Retaining walls, foundations and slope stability can be designed in a consistent manner by this approach.

3. Because the strength of soil is derived from friction, non-linear, or disproportionate relationships between soil parameters and resistances are common. In these circumstances, it is important to check designs with factors of safety applied to the basic strength parameters of the soil. This is done in DA1 Combination 2.

4. When actions are combined they may tend to cancel each other, or the high significance of variation of individual actions may be lost. It is therefore important to apply factors to actions before they are combined to give resultants. This is achieved by DA1 and DA2, but not by DA2*.

5. The factors suggested in Annex A of EN1997-1 for the EQU limit state lead to anomalous results in some cases. It is proposed that the factor on $\tan\phi'$ be reduced to $\gamma_\phi=1.10$ for EQU.

6. DA1 can readily be used with both simply calculations and finite element methods. It appears to be much more difficult to use DA2 with finite elements methods.

7. In many situations, the results of DA1 and DA2 are closely similar, but results for DA2* are less safe and those for DA3 are more safe.

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