Ultimate Limit State design to Eurocode 7 using numerical methods Part II: proposed design procedure and application

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Summary

In the UK, assessment of the Ultimate Limit State (ULS) is to be carried out using Eurocode 7 Design Approach 1. In most cases this involves two calculations, one which primarily involves an "action factor" approach (Design Approach 1, Combination 1), the other which primarily involves a "material factor" approach (Design Approach 1, Combination 2). In Part I of this paper a general methodology for undertaking ULS design using numerical methods for both material factor and action factor (including action/resistance factor) approaches was presented.

In Part II, the practical application of this methodology will be described as part of a proposed general purpose "problem agnostic" design procedure. In particular the challenges of applying partial factors in the presence of potential non-linear relationships within the problem will be addressed (for example a bearing resistance derived from a frictional material property).

A range of worked examples are presented to highlight key issues encountered when applying the procedure. Familiarity with the core principles of the Eurocode, as summarised in Part I (see *GE* October), is assumed.

Introduction

In Part I of the present paper (Smith & Gilbert 2011), the Eurocode 7 (BSI 2004) Ultimate Limit State (ULS) design process was examined in the context of numerical analysis procedures. This included examination of the material factor and action/resistance factor analysis approaches described in the Eurocode (where the action/resistance factor approach is taken to include the action factor approach as a special case). Key conclusions from this paper were that:

- In a material factor approach, the material parameters involved are *pre-factored* prior to performing a mechanical analysis, and the approach can therefore readily be used in conjunction with a general numerical analysis procedure to automatically identify the critical collapse mode.
- In an action/resistance factor approach, a numerical analysis must be carried out in a pre-specified "equilibrium direction" (appropriate to a given collapse mode, for example horizontal to replicate sliding failure of a retaining wall), where the ULS collapse state is induced by applying an additional disturbing force in the equilibrium direction. It is in this direction that the fundamental Eurocode stability equation is evaluated:

$$E_d \leq R_d \qquad (1)$$

Where E_d represents the design (ie factored) actions, or action effects, and R_d represents the design resistance. In this approach the relevant parameters involved are *post-factored* following the mechanical analysis. Therefore this approach cannot be used to automatically identify the critical collapse mode. Instead, each mode must be examined separately, as has been standard practice for many years.
- Internal structural stability may be assessed directly using the action/resistance factor design approach. However in many cases (for example when bending of a sheet pile is involved) an equivalent "inverse-factoring" method is shown to be more convenient. This makes the problem to be solved similar in form to that encountered when using the material factor approach.
- It is essential to adopt an unambiguous interpretation of *actions*, *action effects* and *resistances*.

The following interpretations have been proposed for the geotechnical components of a design check: an action is a load that is independent of the collapse mechanism, and can be *favourable* or *unfavourable*. An *action effect* and a *resistance* are dependent on the form of the collapse mechanism. The former is always *unfavourable* (otherwise it would be termed a resistance) while the latter is always *favourable*.

Having established a general methodology which allows material factor and action/resistance factor design approaches to be used in conjunction with general numerical analysis procedures, in Part II a consistent design procedure designed to ensure a "safe" overall design of a geotechnical construction in accordance with Eurocode 7 is described. Specifically, the design procedure developed addresses the possibility that "non-linearity" exists in the system. Appropriate examples are then used to illustrate its application.

Finally, it is assumed that the reader is familiar with the principles and notation used in the Eurocodes, as summarised in Part I of this paper. The following abbreviated forms are also used throughout: DA1 (Design Approach 1), DA1/1 (Design Approach 1, Design Combination 1) etc. As in Part I of the paper, only Eurocode limit states GEO and STR are considered.

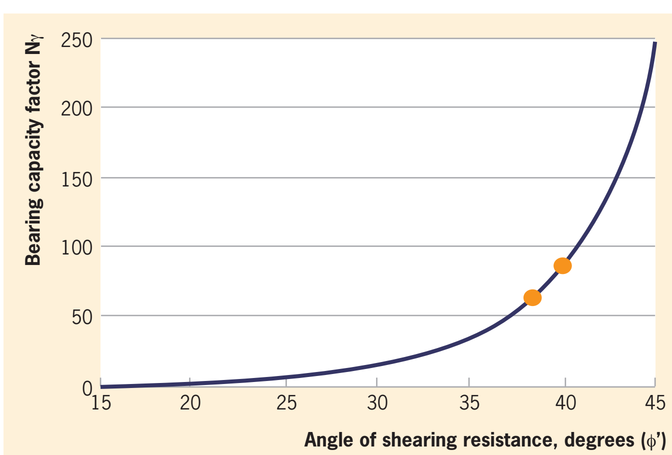


Figure 1: The variation in bearing capacity factor (N_γ) with angle of shearing resistance Φ′ for a strip footing illustrates a highly non-linear relationship between the bearing resistance and the material property (Φ′). Markers indicate values for Φ′ = 38.4º and Φ′ = 40º.

The challenge of non-linearity

Non-linearity and partial factors

The use of partial factors, as employed in the Eurocodes, is sometimes criticised because it can lead to apparent problems when the relationship between actions and/or material properties and their corresponding action effects and/or resistances is non-linear, for example in the case of bearing capacity on cohesionless soils, as illustrated in Figure 1.

Consider the vertical stability of a concrete spread footing subject to a permanent load and founded on the surface of a uniform sand layer with angle of shearing resistance ϕ' = 40º. The theoretical bearing capacity of the footing, factored by 1.35, is in this case equivalent to the bearing capacity of an identical footing on a sand layer with ϕ' = 38.4º (as shown in Figure 1). Thus the uncertainty in the soil strength is 1.6º, or a factor of 1.06 applied to tan ϕ'. But 1.06 is clearly considerably less than the value of 1.25 suggested by DA1/2, which at first sight appears anomalous.

Alternatively, consider the design of the foundation against, for example, punching shear of the column through its base. In this case the shear stresses in the concrete foundation are almost entirely a function of the applied loads (suitably factored). Here the DA1/2 partial factor values stipulate a margin of safety of 1.00 on the predicted design shear stresses, whereas DA1/1 partial factor values stipulate a margin of safety of 1.35.

Driscoll & Simpson (2001) note that EN1990 (BSI 2002) does address this issue by recommending that the following simple rules are followed when a single action dominates [EN1990, 6.3.2(4)]:

- When the *action effect* increases more than the *action*, the partial factor γ_F should be applied to the representative value of the *action*.
- When the *action effect* increases less than the *action*, the partial factor γ_F should be applied to the *action effect* of the representative value of the *action*.

However, if the above cannot easily be evaluated in advance, or if there are multiple interacting actions, it is necessary to adopt an approach which can address the issue of non-linearity in a more general way. One means of doing this is to carry out design checks sequentially in stages, applying partial factors which are appropriate to a given stage, and also verifying that the design is safe at all stages. This is not necessarily as onerous as it sounds, and will be considered further in the next section.

Addressing non-linearity through the use of "analysis levels" Examination of the literature on Eurocode 7 (for example, Frank et al. (2004); Bond & Harris (2008)) indicates three common stages of a design calculation at which an engineer might wish to identify *actions*, *action effects* and *resistances*, prior to the application of partial factors. These stages correspond to the following analysis levels (ie levels at which partial factors are applied in the calculations):

- Analysis Level 1: At source ie applied to the originating *action* such as soil self weight or external load. This permits global stability to be assessed.
- Analysis Level 2: At a structure and at soil/structure interfaces ie applied to an *action effect* or *resistance* such as an earth pressure. This permits the overall stability of a structure to be assessed.
- Analysis Level 3: Within the structure ie applied to an *action effect* such as a bending moment. This permits internal structural stability to be assessed. (This is sometimes referred to as the * approach in the literature, for example DA1* or DA2*).

Methods for undertaking the appropriate calculation type for each analysis level were described in Part I of the paper (though without specific reference to the concept of analysis levels).

It should also be noted that although these "analysis levels" are often implied in current literature on Eurocode 7, it is useful to identify them explicitly, as this should ensure that factors appropriate to different levels are not inadvertently applied in a single calculation (this can sometimes lead to fundamental mechanical principles being violated in the problem being analysed).

In this context, it is also necessary to consider an appropriate methodology for handling variable actions at Analysis Level 2 or Analysis Level 3. This is addressed in Appendix I.

It is proposed that a general design procedure should, in principle, involve design checks at all three analysis levels, using three separate calculations to address the potential issue of non-linearity. It should also be noted that Analysis Level 1 checks will, in many cases, also implicitly cover checks subsequently carried out at Analysis Level 2 and Analysis Level 3 (for example the bending moments in a sheet pile wall can be checked during an Analysis Level 1 calculation, as well as in a subsequent Analysis Level 3 calculation).

Similarly, checks carried out at Analysis Level 2 will often implicitly also cover checks subsequently carried out at Analysis Level 3. However, because of non-linearity and the application of differing partial factor values, this does not necessarily mean that the same elements within a problem will be identified as being critical when considered at the different analysis levels (for example, in the strip footing example referred to in section 2.1, the shear stresses in the concrete footing are less likely to be identified as being critical when undertaking a Level 1 (DA1/2) check than when undertaking a specific Level 3 (DA1/1) check). Thus, in principle, all analysis levels should be checked separately, applying factors appropriate to the analysis level under consideration, and the most critical of these in due course identified.

Proposed general design procedure

Eurocode 7 provides the engineer with the flexibility to perform design checks at any given analysis level. In the general design procedure described here it is proposed that all three analysis levels defined previously are considered in turn, with the resulting design being governed by the most onerous of these, thereby addressing the issue of non-linearity. The proposed design procedure is summarised in the flowchart shown in Figure 2 (see overleaf), with associated notes.

A review of the design examples described in the existing literature indicates that the checks shown in columns (i) and (ii) of Table 1 have been recommended for design problems which involve both soil and structural elements (note that the quoted analysis levels have been inferred from the descriptions given).

However, to address fully the issue of non-linearity it is proposed here that the checks indicated in either column (iii) or (iv) of Table 1 are undertaken (though where structural elements are not present an Analysis Level 1 check will be sufficient). The design checks listed in columns (iii) and (iv) will typically involve the use of general numerical analyses at Analysis Level 1 and Analysis Level 3, where the collapse mode can be automatically identified.

Analysis Level	Usage – from the literature[1] (i)	(ii)	Proposed usage (iii)	(iv)	Typical purpose (eg for footing design)
1	DA1/2	–	DA1/2	DA3	sizing of footing[3]
2	DA1/1[2]	DA2	DA1/1	DA2	sizing of footing[3]
3	–	–	DA1/1	DA2	internal footing design

[1]Analysis Level 3 checks are sometimes implicitly performed as part of Analysis Level 1 or Analysis Level 2 calculations.
[2]In some cases DA1/1 checks can be performed at Analysis Level 1
[3] for (iii) DA1/2 typically controls footing width, while for (iv) DA3 is not normally used for foundation design

Table 1: Design checks to be performed for soil-structure interaction problems at different analysis levels, either (i), (ii) inferred from current literature, or (iii), (iv) proposed here. (NB checks are expected to be performed at all analysis levels indicated in a given column)

It is suggested that a small number of checks at Analysis Level 2 are also carried out, typically using the critical collapse modes identified at Analysis Level 1. With the increasing availability of general purpose numerical analysis procedures, such checks are not too onerous to undertake. Finally, if it is obvious that design checks at one particular level are unnecessary, then these do not need to be undertaken.

Illustrative examples of the design checks to be undertaken at each analysis level will now be presented.

Worked examples

Analysis Level 1 example: strip footing design (DA1/1 pre-factoring approach)

Problem statement

It is required that Analysis Level 1 DA1/1 checks of the vertical bearing stability of a strip footing are undertaken; full details of the problem are given in Figure 3 (see overleaf).

This sort of simple problem does not generally involve *action effects*, and is therefore amenable to the DA1/1 "pre-factoring approach", in which factors are applied to *actions* but not *resistances*.

In this case, the actions are: the external load applied to the footing, the weight of the footing itself and the surcharge applied to the surrounding soil. The only *resistance* is that provided by the soil itself, experienced by the footing at the soil/footing interface.

As part of the design check, an overdesign factor with respect to external applied load will be sought. When using the DLO-based numerical limit analysis software LimitState:GEO (LimitState 2009) this means that the adequacy factor λ_A applied to the external applied load is computed, together with an associated collapse mechanism (for example, as shown in Figure 3).

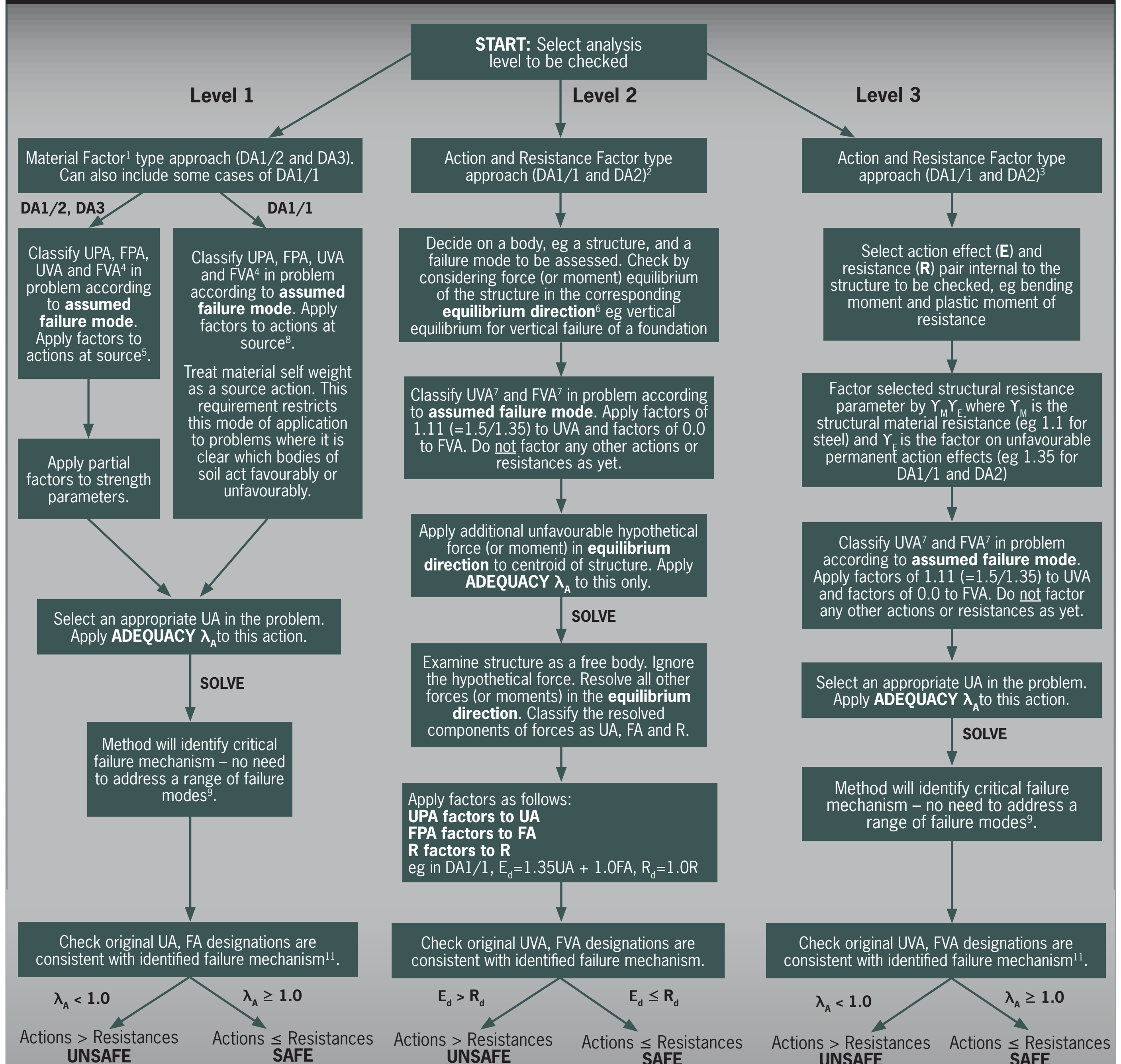
Numerical analysis

The values of the parameters used in the pre-factoring analysis are given in Table 2 (using the unfactored soil properties given in Figure 3), together with results from a numerical limit analysis model of the problem undertaken using LimitState:GEO and a "fine" numerical discretisation comprising approx. 1,000 nodes (LimitState 2009). In this problem the soil self-weight has no effect (ie is neutral). The result indicates that the design is safe and is overdesigned over and above the Eurocode requirements by a factor of 1.05 on the applied load.

Analytical check

The collapse soil resistance »

Figure 2: Proposed design procedure for performing ULS design to Eurocode 7 in conjunction with numerical methods



Definitions/abbreviations (actions[11] and resistances):
UPA: Unfavourable permanent action
FPA: Favourable permanent action
UVA: Unfavourable variable action
FVA: Favourable variable action
UA: Unfavourable action (either permanent or variable)
FA: Favourable action (either permanent or variable)
R: Resistance

A **Favourable Action** arises from a pure force (e.g. self weight).

A **Resistance** arises from (significant) material strength, but may also be influenced by an action (such as a frictional resistance).
Water Pressures should be treated as **Permanent Actions**. They should be based on characteristic hydraulic conditions.
Actions arising from a **single source** should be classified on the basis of the net effect of the single source[12].

Notes:
1 These methods include some factors on actions.
2 The methodology described here is also applicable to, for example, the LRFD approach used in the US, but different factors will be used.
3 This approach essentially models the action/resistance factor approach using an equivalent material factor approach which is less cumbersome. It is considered equivalent if a single plastic hinge forms in the mechanism, otherwise it will normally be conservative compared to the equivalent action/resistance factor approach.
4 The factor on resistance in these design approaches is 1.0 so resistance does not require designation.
5 Only actions that are normally externally applied receive factors > 1.0 ie UVA in DA1/2 and DA3 and actions arising from a structural source in DA3. All other actions receive factors of 1.0. Thus self weight is factored by 1.0.
6 In many cases, several directions should be checked. An indication of the likely critical direction could be obtained from an Analysis Level 1 type calculation.
7 These will normally be externally applied actions.
8 It may be necessary to treat water pressure as an action and apply partial factors directly to it.
9 Note that the failure mode and adequacy factor produced will depend on where the adequacy factor has been applied. However, the result will be independent of the point of application when the adequacy factor λ_A = 1.0, which is the transition point from a safe to unsafe design.
10 It should also be checked that the selected characteristic parameters are consistent with the identified failure mechanism.
11 This chart does not consider accidental actions. Since they receive factors of 1.0, it would seem logical to treat them in the same way as FPA.
12 In an Analysis Level 2 calculation only resolved forces are factored, potentially leading to non-uniform factoring of single source effects. However it can be argued that this is the most logical approach.

» R beneath the footing may be calculated from the Terzaghi bearing capacity equation:

$$R/B = cN_c + qN_q + {}^1/_2\gamma BN_\gamma \qquad (2)$$

For an undrained failure with no surcharge, N_c = 5.14 and:

$$R/B = c_uN_c \qquad (3)$$

Hence, for this problem, R = 617kN/m. Details of a "hand" analysis check are summarised in Table 3. The over-design factor with respect to the applied load is (617 – 54)/540 = 1.04. The numerical result (in terms of adequacy, or over-design factor) is thus ≈1% different from the exact theoretical answer.

Analysis Level 2 example: gravity wall design (DA2 post-factoring approach)

Problem statement

It is required that an Analysis Level 2, DA2, design check of the short term horizontal stability of the gravity wall shown in Figure 4 is performed (ie a check against horizontal sliding failure). The problem requires a "post-factoring approach" and a hypothetical horizontal force H acting in the postulated direction of failure (ie equilibrium direction) is to be used, as shown in Figure 4. The "adequacy factor" λ_A is applied to this load, H, while the degree of overdesign can be determined by computing the ratio of resistance to actions.

Numerical analysis

The relevant Eurocode 7 design parameters are given in Table 4 (using the unfactored soil properties given in Figure 4), together with results from the analysis of the problem obtained using LimitState:GEO and a "medium" numerical discretisation comprising approximately 500 nodes (LimitState 2009). In this example a modified partial factor γ_Q/γ_G is applied to the surcharge; refer to Appendix I for justification of this. The result indicates that an additional horizontal force H = 38.5kN/m is required to cause failure.

Table 5 lists the key action effects and resistances identified after undertaking the numerical analysis. In this case the effective active pressure resultant is an *unfavourable action effect* and is therefore factored by 1.35. The water pressure is an *action* since its value is not modified by the collapse mechanism. Its net effect in the equilibrium direction is *unfavourable* and so it is also factored by 1.35.

The data indicates that the factored resistances are less than the factored action effects. The wall is therefore under-designed when sliding failure is involved by a factor of (resistances/actions) 123.2/162.6 = 0.76 (ie the design is unsafe).

Analytical check

In a conventional factored load and resistance approach it is implicitly assumed that the soil is yielding either side of the wall. In this case active and passive Rankine pressure distributions are typically taken to act on each side of the wall. This is valid in this case but is not necessarily correct in all situations.

In a DA2 check soil strengths are unfactored and on the active side the design angle of drained shearing resistance Φ'_d = 30º. Thus the active force A' due to the soil only (ie based on the effective stresses) is given by:

$$A' = {}^1/_2K_a(\gamma_{a,sat} - \gamma_w)4^2 + 1.11qK_a4 = 41.9\text{kN/m} \qquad (4)$$

The resultant water force U_a on the retained side is given by:

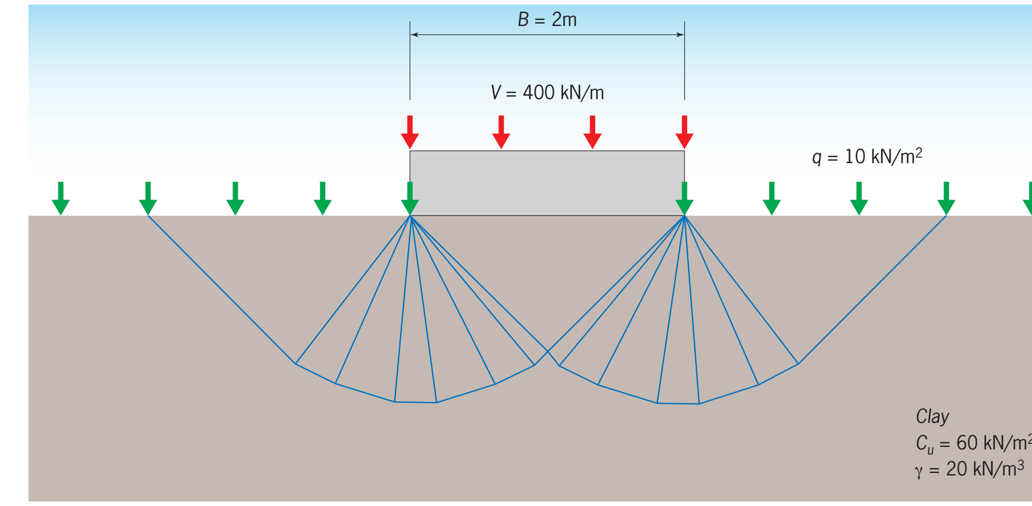


Figure 3: Analysis Level 1 (strip footing) design example: details of problem, also showing predicted collapse mechanism. (Additional parameters: footing weight (exerted at soil-footing interface) = 40kN/m. Surcharge q is taken as variable.)

Quantity	Type	Partial factor
Pre-analysis		
Applied load (V)	Permanent unfavourable	1.35
Footing weight	Permanent unfavourable	1.35
Surcharge (q)	Variable favourable	0.0
Soil unit weight (ϒ)	(Neutral)	1.0
Adequacy factor λ_A on:	*applied load*	
Numerical analysis		
Adequacy factor λ_A		1.05
$\lambda_A \geq 1.0 \rightarrow E_d \leq R_d$?	true ➔ **safe**	

Table 2: Analysis Level 1 (strip footing) design example: Eurocode 7 parameters and outcome

Quantity	Characteristic value	Partial factor	Design value
Soil properties			
Undrained shear strength c_u (kN/m²)	60	1.0	60
Actions (F)			
Applied load	400	1.35	540
Footing weight	40	1.35	54
Sum	440		594
Resistances (R)			
Applied load	617	(1.0)	617
F ≤ R?	true **safe**		true **safe**

Table 3: Analysis Level 1 (strip footing) design example: analytical determination of actions and resistances (all actions and resistances in kN/m)

$$U_a = {}^1/_2\gamma_w4^2 = 78.5\text{kN/m} \qquad (5)$$

The passive force P (short term total stress), based on the undrained shear strength c_u and the soil unit weight γ_p is given by:

$$P = {}^1/_2\gamma_p1^2 + 2c_u = 110\text{kN/m} \qquad (6)$$

which matches the result obtained using the numerical analysis procedure (see Table 5).

Since the walls are smooth, the normal force on the base of the wall is equal to the wall weight W less the upthrust U_u. The base friction T is therefore given by:

$$(W - U_u)\tan\phi'_d = 1.5 \times 4 \times (24 - 9.81) \times \tan30 = 49.2\text{ kN/m} \qquad (7)$$

Once partial factors are applied, the over-design factor is therefore the same as computed previously, ie (110.1/1.4 + 49.2/1.1)/(120.4 × 1.35) = 0.76.

Note that in this case the calculated shear resistance at the base of the wall is based on the unfactored weight of the wall, not because the wall weight is classed as a *favourable action*, but because the calculation is carried out using characteristic values, with the factors subsequently being applied. This is an important distinction.

For similar reasons the shear resistance at the base of the wall and the active earth pressures are based on the unfactored water pressure, despite the fact that the effect of the water pressure in the equilibrium direction is *unfavourable*.

Analysis Level 3 example: embedded cantilever design (DA1/1 inverse-factor approach)

Problem statement

It is required that an Analysis Level 3, DA1/1 check on the bending »

Quantity	Type	Partial factor
Pre-analysis		
Retained soil	(Neutral)	1.0
Soil on excavated side	(Neutral)	1.0
Wall weight	(Neutral)	1.0
Water pressure resultants	(Neutral)	1.0
Soil surcharge	Variable unfavourable	1.11
Unit load H	(Neutral)	1.0
Adequacy Factor λ_A on:	*load H*	
Numerical analysis		38.8
Adequacy factor λ_A		
Post-analysis stability check (horizontal equilibrium)		
Active force from soil	Unfavourable permanent	1.35
Active force from water	Unfavourable permanent	1.35
Passive force	Resistance (passive)	1.4
Base friction	Resistance (sliding)	1.1
Load *H*	*ignore*	*0.0*

Table 4: Analysis Level 2 (gravity wall) design example: Eurocode 7 parameters and outcome.

Quantity	Characteristic value	Partial factor	Design value
Action effects (E)			
Active force (soil)	42.0	1.35	56.7
Active force (water)	78.5	1.35	105.9
Total	120.4		162.6
Resistances (R)			
Passive force	110.1	1.4	78.6
Base Friction	49.2	1.1	44.7
Total	159.2		123.3
Outcome	$E_k \leq R_k$ **safe**		$E_d > R_d$ **unsafe**

Table 5: Analysis Level 2 (gravity wall) design example: post analysis stability check (horizontal equilibrium). Characteristic values taken from numerical analysis (all forces in kN/m).

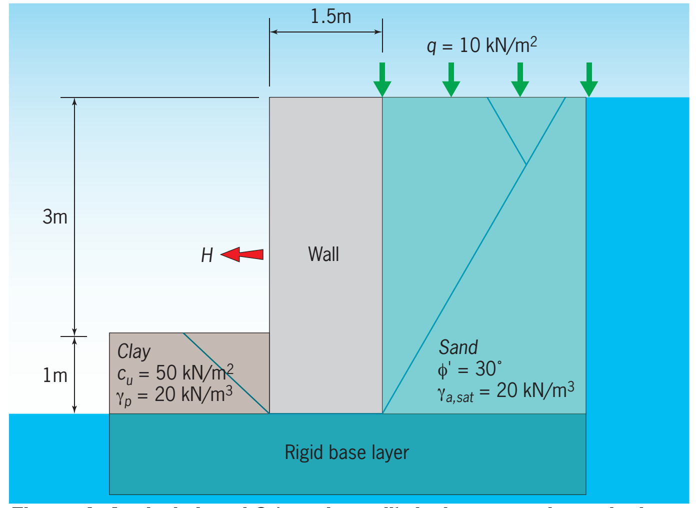


Figure 4: Analysis Level 2 (gravity wall) design example: typical pattern of slip-lines at collapse. (Surcharge q is taken as variable; wall vertical faces are taken as smooth; wall base angle of drained shearing resistance is taken as Φ' = 30°; retained fill is saturated with water table at soil surface; water pressures are assumed to act beneath the wall, but in the short term not in the clay.)

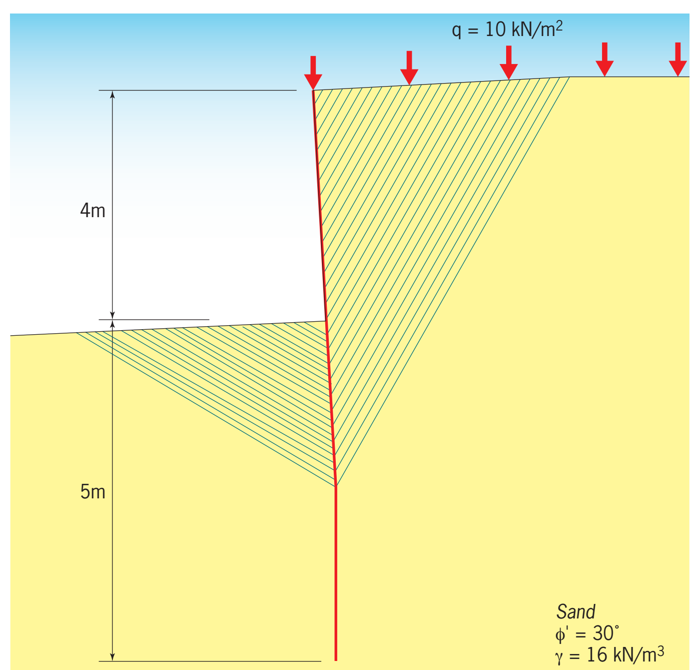


Figure 5: Analysis Level 3 (embedded wall) design example: typical pattern of slip-lines at collapse. (Surcharge q is taken as variable; wall bending strength M_p = 409kNm/m; wall is taken as smooth.)

» strength of the embedded wall problem shown in Figure 5 is undertaken. In this example the action effect (*E*) is considered to be the bending moment in the wall, while the resistance (*R*) is considered to be the plastic moment of resistance of the wall. This problem is most conveniently analysed using a pre-factoring ("inverse factoring") approach (Smith & Gilbert 2011). All parameters enter the calculation unfactored, apart from variable loads.

The resistance is factored down by the product of the action effect factor and the wall strength factor, $\gamma_G\gamma_M$. The adequacy factor is applied to any unfavourable action – in this case the surcharge applied to the surface of the retained soil is chosen. The numerical analysis procedure will automatically find the point in the wall at which bending failure occurs.

The method should provide equivalent results to using a direct action/resistance factor approach (as typically used at Analysis Level 2) for problems which involve formation of a single plastic hinge.

■ *Numerical analysis*

The values of the parameters used in the *pre-factoring* analysis are given in Table 6 (unfactored soil properties are shown in Figure 5), together with results from the LimitState:GEO analysis of the problem obtained using a "medium" numerical discretisation comprising approximately 500 nodes, with the nodes concentrated along the wall and the soil surface. A one-dimensional "Engineered Element" (LimitState 2009) was used to model the wall, with locations of potential plastic hinges spaced at 0.1m intervals along the embedded portion.

The result indicates that the design is safe ($\lambda_A > 1$). To determine the overdesign factor (over and above the Eurocode factors) on the bending strength, it is necessary to reduce the bending strength in the analysis until $\lambda_A = 1$. This occurs for $M_{p,k}$ = 263kNm/m (implying a factored value in the analysis of M_p = 195kNm/m), with the hinge forming 6.3m below the top of the wall. The overdesign factor is thus 409/263 = 1.6.

■ *Analytical check*

In a conventional factored load and resistance approach it is implicitly assumed that the soil is yielding either side of the wall. In this case active and passive Rankine pressure distributions are typically taken to act on each side of the wall. This is valid in this case but not necessarily correct in all situations. For a DA1/1 analysis soil strengths are unfactored and the design drained angle of shearing resistance $\Phi'_d = 30°$. Let the depth to the hinge be *d*, and thus the resultant active force *A* is given by:

$$A = A_1 + A_2 = {}^1/_2 K_a\gamma(4 + d)^2 + 1.11qK_a(4 + d) \quad (8)$$

and the resultant passive force *P* is given by:

$$P = {}^1/_2 K_p\gamma d^2 \quad (9)$$

Taking moments about the hinge

$$M = A_1\frac{4 + d}{3} + A_2\frac{4 + d}{2} - P\frac{d}{3} \quad (10)$$

This reaches a maximum value of *M* = 198kNm/m when *d* = 6.3m. The value of the moment and its position is within ~ 1% of the value obtained using the numerical analysis (*M* = 195kNm/m; *d* = 6.3m).

Discussion

In Part I of this paper (*GE* October), it was noted that Eurocode 7 provides the engineer with a significant degree of flexibility in how to apply each of the three design approaches (DA) described. This allows application of partial factors at the analysis level(s) selected by the engineer, to provide an appropriate safety margin. This also allows "non-linearities" of the sort referred to previously to be appropriately accounted for by experienced users the code. Unfortunately, this flexibility also means there is potential for the code to be applied inconsistently, in turn leading to confusion, especially amongst inexperienced users.

The goal here has been to marry a design code of general applicability (ie Eurocode 7) with analysis procedures of general applicability (for example based on numerical limit analysis), leading to a "one size fits all" methodology. By requiring all three of the analysis levels described to be addressed, it is anticipated that most non-linear effects will be handled appropriately. The advantage of general numerical analysis procedures is that otherwise repetitive checks can be automated.

If the proposed procedure is to be used with DA1, in principle, all three analysis levels should be checked using DA1/1 and DA1/2, implying six calculations in all. However, in practice, as listed in Table 1, three checks will typically suffice:

■ A DA1/2 calculation is typically viewed as a check that corresponds to Analysis Level 1. However, it implicitly covers all three analysis levels simultaneously.

■ In the case of DA1/1, only certain problems are amenable to Analysis Level 1 checks since it is often challenging to distinguish between favourable and unfavourable actions arising from soil self weight. Therefore these checks need only be carried out at Analysis Level 2 and Analysis Level 3 (for footings, for example, Analysis Level 1 and 2 checks are essentially equivalent).

Furthermore, since DA1/2 is normally found to govern over DA1/1 for checks at Analysis Level 1 and Analysis Level 2, it could be argued that in most cases DA1/1 checks at Analysis Level 2 could be omitted, requiring a DA1/1 check only at Analysis Level 3.

This situation is ideal as far as users of general purpose numerical analysis procedures are concerned since both cases allow the use of the much more straightforward pre-factoring approaches discussed in Smith & Gilbert (2011), which can identify automatically the collapse mechanism without any further user input.

Alternatively, if the proposed procedure is to be used with DA2, only Analysis Level 2 and Analysis Level 3 can realistically be checked, since DA2 is not generally amenable to an Analysis Level 1 check. Thus it has been proposed that DA3 is used to perform the Analysis Level 1 check, if required. Implicit in the use of DA2 with general purpose numerical analysis procedures is the use of post-factoring approaches which are less convenient than the pre-factoring approaches that can be used with DA1.

Quantity	Type	Partial factor
Pre-analysis		
Surcharge	Variable unfavourable	1.11
Soil unit weight	(Neutral)	1.0
Wall bending strength	Resistance	1.35
Adequacy Factor λ_A on:	*Surcharge*	
Numerical analysis		
Adequacy factor λ_A		2.38
$\lambda_A \geq 1.0 \rightarrow E_d \leq R_d$?	true ➔ **safe**	
Hinge location (from wall top)	6.3m	

Table 6: Analysis Level 3 (embedded cantilever) design example: Eurocode 7 parameters and outcome. (NB the factor on wall bending strength is calculated as 1.0 × 1.35, where 1.0 is the material partial factor for steel from EN 1993-5.)

Conclusions

1. Eurocode 7 provides a new unified approach to geotechnical design, with partial factors allowing uncertainty to be accounted for at source. The unified approach means the Eurocode can be used in conjunction with general purpose numerical analysis procedures for a broad range of problem types.

2. When considering the ULS it can be established that there are three distinct analysis levels at which design checks can be performed:

■ Analysis Level 1, considering global failure (for example, instability within the soil mass).

■ Analysis Level 2, considering structural failure (corresponding to instability of the structure as a whole, due to factored actions and resistances acting on the boundary of the structure).

■ Analysis Level 3, considering internal failure within a structure.

While these are often implied in the current literature, a clear enumeration of them should assist engineers undertaking design checks.

3. Non-linearity within a problem can render the effects of a partial factor applied at one analysis level insignificant when considered at another level. To address this it is proposed that:

■ Partial factors should only be applied to the quantities (ie actions, action effects and resistances) relevant at the analysis level under consideration.

■ In principle checks at all three analysis levels should be undertaken when verifying a design.

4. A simple and consistent general-purpose Eurocode 7 design procedure which can be used in conjunction with general numerical analysis procedures has been proposed. Three worked examples have been provided to illustrate its application.

5. It is argued that use of Design Approach 1 will pragmatically often only require checks at Analysis Level 1 using DA1/2 and Analysis Level 3 using DA1/1. Both are amenable to pre-factoring approaches and thus benefit from automatic identification of collapse mechanisms.

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All numerical analyses described herein were undertaken using LimitState:GEO version 2.0; see: www.limitstate.com/geo

Appendix I: Variable and permanent actions

Different values of partial factors are stipulated for variable and permanent actions in Eurocode 7. This presents a challenge for Analysis Level 2 and Analysis Level 3 calculations, which generally apply factors to action effects and resistances at locations internal within a problem.

These quantities are usually considered permanent, and corresponding partial factors are used.

A variable action will almost always be an externally applied action, and in general it is not straightforward to quantify directly its individual contribution to an action effect or resistance at Analysis Level 2 and Analysis Level 3. The pragmatic solution to this, as suggested by, for example, Frank et al. (2004) and Bond & Harris (2008), is to use a modified partial factor of γ_Q/γ_G, which is applied to the variable action at its source. The contribution of the effect of this factored action to an action effect (for example earth pressure on a structure) is then factored by γ_G. If the influence of the variable action on the action effect is linear then this process is equivalent to using a factor of γ_Q on the source action.

Since variable actions are small relative to permanent actions in most design problems, it would seem pragmatic also to adopt generally this simple rule for cases where the relationship is non-linear. (However, if a variable action dominates then it would be necessary to adopt a more refined approach in cases where strong non-linearity exists.)

When performing a DA1/1 check the modified partial factor becomes 1.5/1.35 = 1.11 for unfavourable variable actions, and 0.0/1.0 = 0.0 for favourable variable actions. In a rare case where an unfavourable variable action contributes to the reduction of a resistance, the variable action would remain effectively factored by γ_Q/γ_G. However, at Analysis Level 3 this issue does not present itself since the resistance is typically a structural material resistance which is known in advance.

Finally, whether a variable action should be classified as "favourable" or "unfavourable" will generally be obvious to the engineer, but an analysis should be carried out to confirm this.

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