Issues Raised by the Application of Eurocode 7 to the Design of Reinforced Soil Structures

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MALAYSIA is adopting Eurocode 7 (EN 1997-1:2004) as its national standard for geotechnical design, referred to here as EC7. Currently, EC7 does not cover the detailed design of reinforced fill structures and the values of the partial factors given in EN 1997-1 have not been calibrated for reinforced fill structures. EN 14475:2006 provides guidance on the execution of reinforced fill structures; a future European Standard will cover their design. However, there are various aspects of reinforced soil design which may be addressed by the current issue of EC7. Analysis of the external stability of reinforced soil structures is examined by modelling the reinforced fill block as a gravity retaining wall (e.g. sliding, bearing capacity and overturning). In addition, stability analysis is used to check overall stability. Furthermore, a number of National Annexes have been published which do provide requirements for reinforced soil design. One such example is Germany.

The purpose of this paper is to examine a number of issues which will arise when the current EC7 recommendations are applied to gravity retaining wall design and stability analysis, in particular, in circumstances which are likely to arise in reinforced soil design. The points raised here were previously made in a presentation during the 2011 AGM of the Geotechnical Engineering Technical Division of IEM on 11 June 2011, and have also been provided in greater detail as comments on the Draft Malaysian National Annex to MS EN 1997-1:2011. The important observation in relation to these comments is that the National Annex should be used to clarify or provide guidance to designers using the EC7 in Malaysia, especially where ambiguity or lack of experience in applying EC7 requirements exists.

CONSIDERATION OF PARTIAL LOAD FACTORS IN GRAVITY RETAINING WALL DESIGN

EC7 defines load factors as follows for permanent actions and transient actions (live loads):

ACTION		SYMBOL	SET	
			A1	A2
Permanent	Unfavourable Favourable	γ_G $\gamma_{G, fav}$	1.35 1.0	1.0 1.0
Transient	Unfavourable Favourable	γ_Q $\gamma_{Q, fav}$	1.5 0	1.3 0

For Set A1, unfavourable actions are increased, whereas favourable actions are factored by 1.0 for permanent actions and 0 for transient actions (live loads). In reality, the factors and their values appear to have slightly different functions, certainly so when applied to either gravity retaining wall analysis or stability analysis:

- For permanent actions, both the weight density and dimensions are likely to be reasonably well known, so presumably the aim of applying 1.35 to unfavourable permanent actions is to ensure a certain margin of safety, likewise favourable actions are taken at face value for the same reason.
- For transient actions (live loads), the higher partial factor for unfavourable loads would appear to reflect a greater uncertainty, which is the nature of live loads, while at the same time, providing some margin of safety. However, the value of 0 for favourable live loads is being used to model the fact that when they are favourable, the safest assumption is that they are absent.

However, it is possible that, for an unfavourable situation, although live load is present, a component of the live load is actually favourable. This gives rise to a situation where applying $\gamma_{Q,fav} = 0$ might not be logical. One obvious case is the sliding stability of a gravity retaining wall as shown in Figure 1. In terms of BS 8006-1:2010 nomenclature, this situation is examined using Load Case B where downward actions are taken as favourable, but lateral actions are unfavourable. Considering only the live loads, it is clear that the live load (LL2) must be present behind the wall to generate unfavourable lateral thrust on the wall. Therefore, $\gamma_Q = 1.5$ is applied to the horizontal component of the earth pressure action, P_{aqh}. However, there is also a vertical component (P_{aqv}) and for the worst case senario, $\gamma_{Q,fav} = 1.0$ should be used. It is not logical to apply $\gamma_{Q,fav} = 0$.

However, for the live load on top of the retaining wall (LL1), clearly the critical case is that the live load is absent, so $\gamma_{Q,fav} = 0$ is applied. Therefore, the use of $\gamma_{Q,fav} = 0$ is not so much due to uncertainty, but to establish an absolute case that the live load is not present. Therefore, there needs to be a second definition of $\gamma_{Q,fav} = 1.0$ for situations where the live load must be present, but its action is favourable.

It should be noted in the example given here that the application of the single-source principle would result in P_{aqv} being factored by $\gamma_Q = 1.5$, so that the issue would not arise. However, this approach results in increasing an action which is favourable (i.e. helping to prevent sliding), which does not seem to be logical.

Load factors applied to transient actions - Load case B

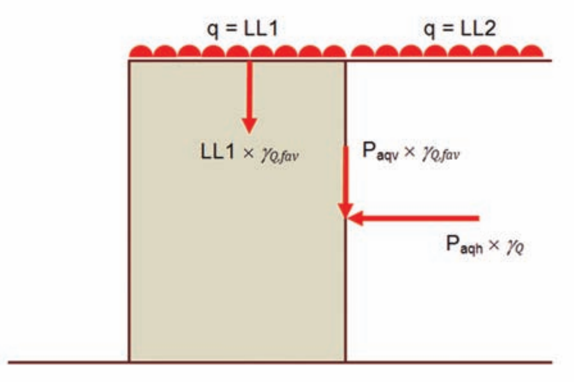

Figure 1: Forces used to analyse sliding

INVESTIGATION OF COEFFICIENT OF ACTIVE EARTH PRESSURE

EC7 provides a method of calculating earth pressure in Annex C (Informative). Section C2 includes a numerical method based on slipline fields, and according to Item (1), it includes certain approximations on the safe side, and may be used in all cases. The conventional method of calculating K_a for complex geometry is to use the Coulomb equation, in this case, giving the horizontal component K_{ah}:

$$K_{ah} = \frac{\cos^2(\phi' + \alpha)}{\cos^2\alpha \left[1 + \sqrt{\dfrac{\sin(\phi' + \delta)\sin(\phi' - \beta)}{\cos(\alpha - \delta)\cos(\alpha + \beta)}}\right]^2}$$

where

ϕ' = angle of shearing resistance of fill
δ = wall friction angle
α = angle of wall back measured against vertical (positive leaning towards the fill)
β = upper slope angle measured against horizontal (positive sloping upwards)

It should be noted that this only applies to the effect of the soil mass retained by the wall. For superimposed uniform surcharge, K_{ah} as given above should be multiplied by the following expression. It can be seen that this expression will be 1.0 unless both α and β are > 0 at the same time (i.e. there is both a sloping surface behind the wall, and the back of the wall is inclined):

$$\frac{\cos\alpha \times \cos\beta}{\cos(\alpha + \beta)}$$

The Coulomb equation is the analytical solution derived by finding the maximum lateral thrust from the backfill based on a simple wedge analysis (i.e. simple linear failure surface). The same value of K_a is found by examining a large number of wedges graphically until the wedge giving the maximum lateral thrust is found (sometimes known as the Culmann method or Coulomb sweeping wedge method).

In the case of reinforced soil design, it is common for the back of the wall to be inclined backwards, and also for the retained backfill to have an upward inclined surface as shown in Figure 2. Normally, the active earth pressure coefficient in this case would be calculated using the Coulomb approach. In order to examine the suggested EC7 slipline method versus Coulomb, a series of calculations have been carried out to compare the values of K_a given by the two methods.

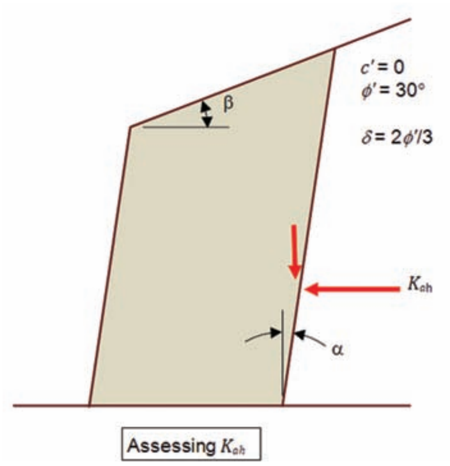

Figure 2: Definition of angles for gravity retaining wall

The results of the comparison are shown on two graphs in Figure 3, one for the lateral thrust due to the soil mass, and the other for the surcharge (UDL). The y-axis value in each graph is the ratio of K_{ah} (calculated according to Coulomb) to K_{ah} (calculated according to the EC7), and the x-axis is the wall inclination (positive is leaning backwards towards the fill as shown in Figure 2). The calculations have been carried out for three different backfill angles, and are based on ϕ' = 30° and $\delta = 2\phi'/3$. These values are fairly typical for the design of gravity retaining walls. Some observations based on this analysis:

- For Rankine conditions ($\delta = \alpha = \beta = 0$), all methods (both for soil mass and UDL) give the same result. (not shown in Figure 3)
- For a vertical wall, provided that $\delta = \beta$, all methods (both for soil mass and UDL) give the same result.
- Beyond these simple cases, the graphs in Figure 3 give some idea of the sensitivity of the calculation. In general, if the K_a ratio > 1.0 on the graph, then Coulomb gives a higher K_a, thus is more critical.
- Based on the examination of the graph for the K_a ratio for soil mass, once the wall leans backwards towards the fill, then Coulomb is more critical, except for cases with level back and steep wall angle.

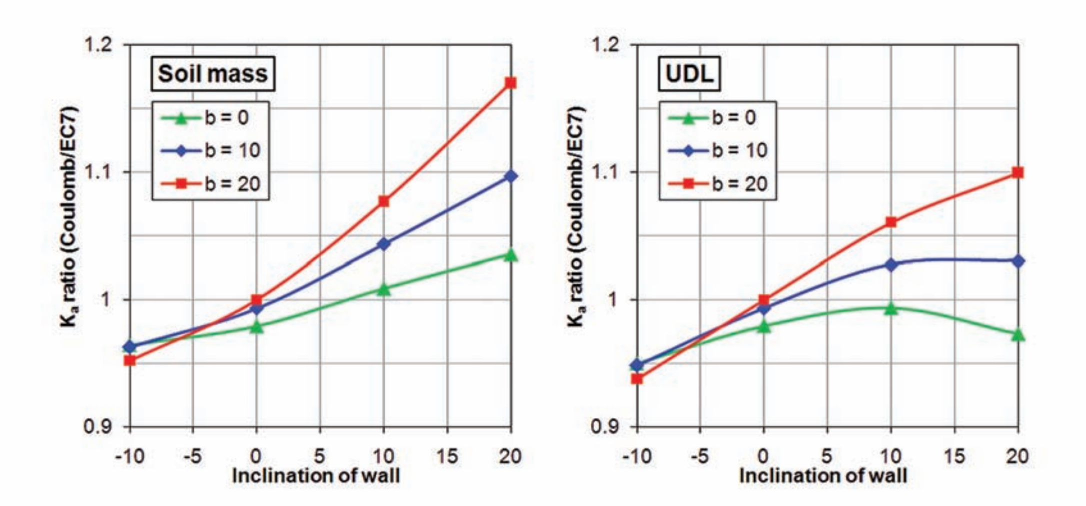

Figure 3: Comparing K_a using the method given in the EC7 with Coulomb

- In the case of UDL, for level backfill, the EC7 method is always more critical. However, for walls that lean backwards and have even gentle upward inclination of the backfill surface, then Coulomb is more critical.
- In most gravity retaining wall analyses, lateral force due to the soil mass is generally considerably higher than lateral force due to surcharge.
- The contents of items (13) and (14) of the EC7, Annex C should be noted as follows:

(13) Both for passive and active pressures, the procedure assumes the angle of convexity to be positive ($v \geq 0$).

(14) If this condition is not (even approximately) fulfilled, e.g. for a smooth wall and a sufficiently sloping soil surface when β and ϕ' have opposite signs, it may be necessary to consider using other methods. This may also be the case when irregular surface loads are considered.

INVESTIGATION OF BEARING RESISTANCE CALCULATION FOR GRAVITY RETAINING WALLS

EC7 provides a method of calculating bearing resistance in Annex D (Informative). The method is similar to the traditional Terzaghi solution, taking into account the eccentricity and inclination of the load applied to the foundation. For the case of frictional soil with c' = 0 and zero burial depth, the bearing resistance (P_v) is given by:

$$P_v = 0.5\gamma N_\gamma X_\gamma L^2_{eff}$$

where,
γ = weight density of foundation soil
N_γ = bearing capacity factor
X_γ = inclination factor
L_{eff} = effective foundation width

The effective width is defined using the Meyerhof approach. Both effective width and the inclination factor are calculated by taking into account the actions applied to the foundation. These actions consist of:

- vertical actions due to the mass of the wall and any superimposed surcharges PLUS the vertical components of earth pressure actions applied to the back of the wall,
- horizontal actions due to the retained backfill soil and any surcharges applied above the backfill.

It should be noted that the value of P_v is particularly sensitive to the value of L_{eff}, because this parameter is raised to the power of 2. In carrying out this calculation, the designer is faced with a number of choices:

- should the actions used to calculate eccentricity and inclination factor be characteristic (unfactored) or design (factored) values,
- in the case of the vertical components of the earth pressure actions on the back of the wall, should these be based on single-source principles or worst-case principles,
- using the terminology in BS 8006-1:2010, gravity retaining wall design is normally considered under two load cases for ULS: Load Case A in which all actions are considered unfavourable (i.e. both vertical and horizontal actions) which is normally the critical case for bearing resistance; and Load Case B in which downward vertical actions are considered as favourable which is normally the critical case for checking sliding on the base.

The reason for the third choice is that, in cases where horizontal actions are relatively large, Load Case B may be critical for bearing. It is, therefore, normal to carry out such bearing resistance calculations for both load cases, and take the worst case as critical.

To examine the effects of these choices on the bearing resistance calculation for a typical gravity retaining wall, two cases have been examined as shown in Figure 4 (being a common geometry for a gravity retaining wall). The main difference is the surface of the backfill: in one case, it is horizontal with a 12kPa surcharge, and in the other, it is

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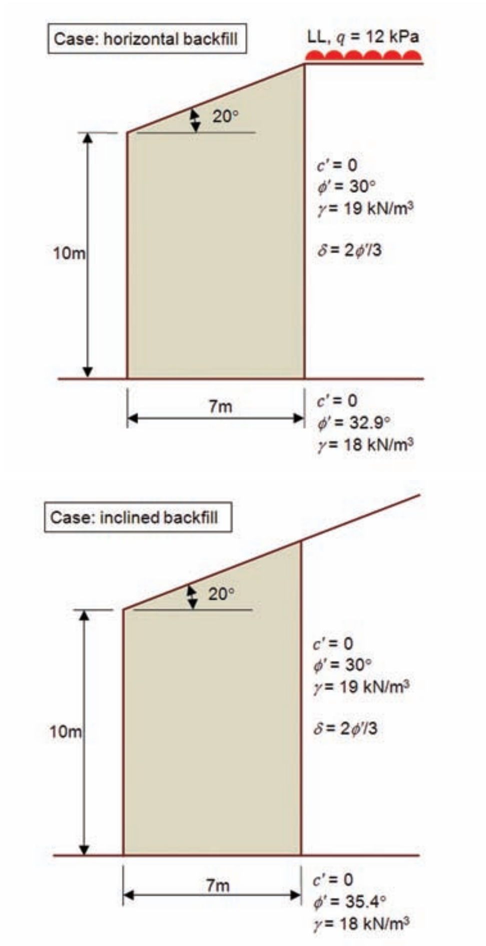

Figure 4: Design cases used to examine bearing resistance calculation according to the EC7

Load case for horizontal backfill with 12 kPa surcharge		Load Case A		Load Case B	
Actions for calculating eccentricity and inclination factor		Unfactored	Factored	Unfactored	Factored
EC7 for Germany (unfactored, worst case)		0.943		0.691	
DA1 Combination 1	SS	0.673	0.687	0.516	0.940
	WC	0.673	0.687	0.494	1.160
DA1 Combination 2	SS	1.218	2.208	1.218	2.208
	WC	1.218	2.208	1.203	2.332
DA2	SS	0.943	0.962	0.723	1.316
	WC	0.943	0.962	0.691	1.624
DA3	SS	1.218	2.208	1.218	2.208
	WC	1.218	2.208	1.203	2.332

Load case for inclined backfill without surcharge		Load Case A		Load Case B	
Actions for calculating eccentricity and inclination factor		Unfactored	Factored	Unfactored	Factored
EC7 for Germany (unfactored, worst case)		0.939		0.695	
DA1 Combination 1	SS	0.670	0.670	0.518	1.077
	WC	0.670	0.670	0.497	1.374
DA1 Combination 2 (*)	SS	1.287	3.351	1.287	3.351
	WC	1.287	3.351	1.287	3.351
DA2	SS	0.939	0.939	0.725	1.508
	WC	0.939	0.939	0.695	1.923
DA3 (*)	SS	1.287	3.351	1.287	3.351
	WC	1.287	3.351	1.287	3.351

inclined. All other parameters and dimensions are the same in both cases, except that the ϕ' value of the foundation soil has been adjusted to give FS = 2.0 on bearing according to a conventional “lumped safety factor” design method (giving 32.9° and 35.4° respectively for the two cases).

For both cases, the bearing resistance following the EC7 approach has been assessed and is reported in the following tables in terms of the "degree of utilisation" denoted by Λ_{GEO} and defined as:

$$\Lambda_{GEO} = \frac{E_d}{R_d}$$

where E_d = design actions or effects of actions
R_d = design resistance
Λ_{GEO} $\leq$ 1.0 for a satisfactory design

Λ_{GEO} is given in the tables below for DA1 (Combinations 1 and 2), DA2 and DA3. In addition, the EC7 requirements for Germany are included. These are well established, and the National Annex and related standards have defined the use of "worst-case" earth pressure and unfactored loads to calculate eccentricity and inclination factors.

From the examination of these values, it should be noted that:

- SS denotes single-source and WC denotes worst-case principles regarding actions from earth pressure applied to the back of the wall.
- For Load Case A, SS and WC give the same results, because all actions are regarded as unfavourable.
- For Load Case B, WC gives higher Λ_{GEO} for the factored cases, but lower for the unfactored cases. However, the effect is not major, and would result in only a small change in dimensions to give the same result.
- For DA1 Combination 2 and DA3, Load Case A and Load Case B give the same results because all load factors are set to 1.0, except when a live load is present.
- For the DA1 approach, where the more critical result is used for the final design, it is clear that this is provided by Combination 2 for all cases, and by a considerable amount. Combination 2 uses material factors only (plus load factor on live load), and this can be considered as a relatively new approach for gravity retaining wall design compared to load factor or lumped safety factor approaches.

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- In comparing DA1 Combination 1 with DA2 (the two "load factor" design approaches), the difference between any two equivalent cases is a factor of 1.4, because DA2 applies γ_{Rv} = 1.4 to bearing resistance, whereas DA1 Combination 1 applies γ_{Rv} = 1.0.
- By far the most important decision is whether or not to use factored or unfactored actions in calculating eccentricity and inclination factors. If factored actions are used, then Load Case B is likely to be critical.
- In order to achieve $\Lambda_{GEO} \leq 1.0$ for the unfactored cases in DA1 Combination 2 and DA3 (*), it is necessary to increase the foundation width to 7.58m (from 7m), an increase of only 8%.
- However, in order to achieve $\Lambda_{GEO} \leq 1.0$ for the factored cases in DA1 Combination 2 and DA3 (*), it is necessary to increase the foundation width to 9.65m (from 7m), an increase of 38%.

EXAMINATION OF STABILITY ANALYSIS ACCORDING TO EC7 USING BISHOP'S ROUTINE METHOD

The purpose of this section is to outline the adjustments which must be made when using stability analysis following the requirements of EC7. This is done by using Bishop's routine (aka simplified) method of slices based on a circular failure surface. In particular, part of the aim of this section is to explain some of the statements made in Section 7.3.3 of BS 6031:2009. These commentaries are extremely helpful, and give an authoritative outline of the important aspects of carrying out stability analysis to EC7 requirements. Relevant sections from Section 7.3.3 of BS 6031:2009 are repeated in italics.

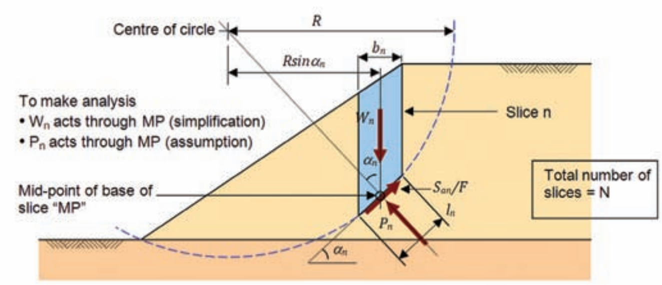

Figure 5: The method of slices based on a circular arc

The formulation of Bishop's routine method of slices is well known, and will not be repeated here. The method is based on taking moments as shown in Figure 5. However, the factor of safety is introduced by applying "F" to soil strength. This is an important point and immediately gives a point of difference in relation to EC7.

The normal situation and required assumptions/simplifications for the method of slices based on a circular arc are as shown in Figure 5. The well known equation derived by the Bishop's routine method is as follows:

$$F = \frac{R\sum\left[c'b_n + (W_n + Q_n - ub_n)\tan\phi'\right]\left[\dfrac{\sec\alpha_n}{1 + \dfrac{\tan\phi'\tan\alpha_n}{F}}\right]}{R\sum(W_n + Q_n)\sin\alpha_n}$$

Because the formulation of this equation is based on moments, the "R" term has been retained. It should be noted that:

- "F" appears on both sides of the equation, so that iteration is required to find a solution. This is an inevitable result of applying "F" to soil strength in the formulation of the equation.
- The denominator on the RHS is effectively the disturbing moment due to the weight of the slices, but it should be noted that part of this moment is actually stabilising (the slices to the left of the lowest point of the failure circle as shown).
- However, due to the way Bishop's routine method is formulated, this does not matter, and the equation will appear as shown even if the stabilising moment is initially added to the moment of the shear resistance.
- The Q_n term has been included to represent live load applied to the mid-point of the top surface of each slice.
- Pore-water pressure is included as the actual pressure (u) instead of using the pore-pressure ratio.

According to EC7, the basic equation defining the GEO limit state is:

$$\Lambda_{GEO} = \frac{E_d}{R_d} = \frac{\text{design (factored) effect of actions}}{\text{design (factored) resistance}} = \frac{1}{F} \leq 1.0$$

Bishop's routine method may be formulated following this approach, and the resulting equation will appear as follows (retaining the same structure of the equation, with disturbing moments in the denominator, so that the results is given in terms of $1/\Lambda_{GEO}$:

$$F = \frac{1}{\Lambda_{GEO}} = \frac{R\dfrac{1}{\gamma_{Re}}\sum\left[\dfrac{c'}{\gamma_{c'}}b_n + (\gamma_{G,fav}W_n + \gamma_{Q,fav}Q_n - ub_n)\dfrac{\tan\phi'}{\gamma_{\phi'}}\right]\dfrac{\sec\alpha_n}{\left[1 + \dfrac{\tan\phi'\tan\alpha_n}{\gamma_{\phi'}}\right]} - R\sum(\gamma_{G,fav}W_n + \gamma_{Q,fav}Q_n)\sin\alpha_n}{R\sum(\gamma_G W_n + \gamma_Q Q_n)\sin\alpha_n}$$

It should be noted that:

- "F" no longer appears on the RHS of the equation, because the factor on soil strength is fixed as $\gamma_{\phi'}$.
- The moments from slices which resist failure are included in the numerator, and the reason for the sign being negative is that the α value for these slices is also negative.
- All partial factors are included with favourable and unfavourable load factors being applied as appropriate for the worst-case.

The difficulty in using the equation in this form is that existing software packages would need significant rewriting. Also see comment from BS 6031:2009:

In addition the treatment of actions due to gravity loads and water is difficult since these loads might be unfavourable in *part of the sliding mass but favourable in another part. In a traditional analysis of a circular failure surface, part of the slope mass is producing a positive driving moment (i.e. it is unfavourable) and part of the slope mass is producing a negative driving moment (i.e. it is favourable) and the moments produced by the two parts depend on the position of the point about which moment equilibrium is checked. The application of different partial factors to each part of the slope introduces scope for confusion and requires a degree of complexity of analysis that is not readily available and not justified given the nature of the problem.*

In order to avoid these issues, the favourable (resisting) moment due to soil mass is considered as "negative disturbing moment", so it is added to the denominator, and to minimise complication, a single load factor definition is applied, i.e. unfavourable. This results in the equation appearing as follows, which is pretty much the same as the original equation:

$$F = \frac{1}{\Lambda_{GEO}} = \frac{R\frac{1}{\gamma_{Re}}\sum\left[\frac{c'}{\gamma_{c'}}b_n + (\gamma_{G,fav}W_n + \gamma_{Q,fav}Q_n - ub_n)\frac{\tan\phi'}{\gamma_{\phi'}}\right]\frac{\sec\alpha_n}{[1+\tan\phi'\tan\alpha_n]}}{\gamma_{Re}R\sum(\gamma_G W_n + \gamma_Q Q_n)\sin\alpha_n}$$

ADJUSTMENTS REQUIRED FOR "LOAD FACTOR" METHODS (DA1-1 AND DA2)

For these methods, all material partial factors are set to 1.0. If in addition both $\gamma_{G,fav}$ and $\gamma_{Q,fav}$ are taken as 1.0 ($\gamma_{Q,fav}$ should be taken as 1.0 if it is present but favourable - see the first section of this paper), then after some adjustment the equation reduces to:

$$F = \frac{1}{\Lambda_{GEO}} = \frac{R\sum[c'b_n + (W_n + Q_n - ub_n)\tan\phi']\frac{\sec\alpha_n}{[1+\tan\phi'\tan\alpha_n]}}{\gamma_{Re}\gamma_G R\sum(W_n + \frac{\gamma_Q}{\gamma_G}Q_n)\sin\alpha_n}$$

This method is the "approximate" method given by Frank et al, see comments from BS 6031:2009:

If the single-source principle is not applied, then a special procedure has to be followed, if using commercially available software, in order to apply different factors to stabilising and destabilising actions. Frank et al [5] describe one such procedure, but by ignoring the single-source principle, Combination 1 becomes more critical than Combination 2 in most design situations using an effective stress analysis and results in an equivalent global factor of safety of about 1.35.

In order to use this method, the procedure would be:

- Adjust the live loads by a factor γ_Q/γ_G although this will also affect the numerator, so there is a slight error.
- Carry out a "normal" stability analysis using Bishop's routine method, and find "F".
- Then $\Lambda_{GEO} = \gamma_{Re}\gamma_G / F$

The alternative is to use the single-source principle and apply unfavourable load factors to all forces. In this case, the equation becomes:

$$F = \frac{1}{\Lambda_{GEO}} = \frac{R\sum[c'b_n + (\gamma_G W_n + \gamma_G Q_n - ub_n)\tan\phi']\frac{\sec\alpha_n}{[1+\tan\phi'\tan\alpha_n]}}{\gamma_{Re}R\sum(\gamma_G W_n + \frac{\gamma_Q}{\gamma_G}Q_n)\sin\alpha_n}$$

With reference to the commentary in BS 6031:2009, the above equation follows this comment:

For this reason, a note to 2.4.2 of BS EN 1997-1:2004 states "Unfavourable (or destabilising) and favourable (or stabilising) permanent actions may in some situations be considered as coming from a single source. If they are considered so, a single partial factor may be applied to the sum of these actions or the sum of their effects." This note, commonly referred to as the "single-source principle", allows the same partial factor to be applied to the stabilising and destabilising actions. When using Combination 1, it is recommended that the partial factor for the unfavourable action of the soil is applied to the weight density of the soil.

The problem with this approach is that the margin against failure relies almost entirely on the resistance factor γ_{Re}. To see this clearly, the equation can be set for the simple case of a dry slope with c' = 0:

$$F = \frac{1}{\Lambda_{GEO}} = \frac{R\gamma_G\sum[W_n\tan\phi']\frac{\sec\alpha_n}{[1+\tan\phi'\tan\alpha_n]}}{\gamma_{Re}\gamma_G R\sum W_n\sin\alpha_n}$$

In this case, the γ_G values cancel out, so that only γ_{Re} remains. For DA1 Combination 1, γ_{Re} = 1.0, thus there is no margin against failure. For this reason, the commentary in BS 6031:2009 states:

In an effective stress analysis, the effect of the partial factor is to increase the destabilising action and to increase simultaneously the shearing resistance of the soil, which cancels the effect of the partial factor.

and

In both cases, Combination 1 tends to be less critical than Combination 2 in almost all design situations. (Exceptions might occur when extremely large variable actions apply or the soil strength is extremely low).

ADJUSTMENTS REQUIRED FOR "MATERIAL FACTOR" METHODS (DA1-2 AND DA3)

For these methods, all partial load factors and resistance factors are set to 1.0. The stability equation then becomes:

$$F = \frac{1}{\Lambda_{GEO}} = \frac{R\sum\left[\frac{c'}{\gamma_{c'}}b_n + (W_n + \gamma_Q Q_n - ub_n)\frac{\tan\phi'}{\gamma_{\phi'}}\right]\frac{\sec\alpha_n}{\left[1+\frac{\tan\phi'\tan\alpha_n}{\gamma_{\phi'}}\right]}}{R\sum(W_n + \gamma_Q Q_n)\sin\alpha_n}$$

This has the benefit of being completely unambiguous, and because actions and water pressure are all unfactored, effective stress is preserved correctly in determining soil shear resistance.

In order to use this method, the procedure would be:

- Adjust the material properties by the material factors $\gamma_{\phi'}$ and $\gamma_{c'}$.
- Carry out a "normal" stability analysis using Bishop's routine method, and find F.
- Then $\Lambda_{GEO} = \gamma_{Re}\,\gamma_G / F = 1/F$

SUMMARY OF COMMENTS

In using DA1, the requirement is that both Combination 1 and Combination 2 are checked and the most critical result is used to determine the design. Based on the comments above and the extract from BS 6031:2009, there are two main options for Combination 1:

- Use the Frank et al approximation, so that Combination 1 is likely to be critical with an equivalent traditional lumped F≈1.35.
- Use the single-source principle for Combination 1, so that Combination 2 is likely to be critical with an equivalent traditional lumped F≈1.25.

There is one final comment to make, namely, that most of the approximations and adjustments as described to permit the easy use of the Bishop's routine method of slices as formulated in existing software are acceptable as long as the target "F" = 1.0 when the analysis is performed. This is the case for DA1 Combination 2. However, for DA1 Combination 1 and DA2, if the Frank et al approach is used, then the target will be >1.0, so this also leads to some uncertainly in using the load factor methods.

The comments given in the first section of this paper concerning $\gamma_{Q,fav}$ apply equally well in stability analysis. In general, live loads are only applied to the tops of slices when $\alpha > \phi'$ for dry slopes.

CONCLUSION

The publication of the Malaysian National Annex to EC7 gives the authorities an opportunity to provide clarification and reduce ambiguity. Without such a clarification, there could be major differences in the methods used by engineers to carry out geotechnical design with subsequent differences in the resulting structures, in particular:

- Consideration should be given to establishing an additional definition of $\gamma_{Q,fav}$ = 1.0 for situations where the live load must be present, but its action is favourable
- The method of calculating active earth pressure given in Annex C.2 underestimates earth pressure in cases where a retaining wall leans backwards and the surface of the retained fill slopes upwards. This geometry is common for reinforced soil structures, and it is recommended that this point should be made and advice given in the National Annex.
- Bearing resistance for gravity retaining walls has a special problem inasmuch as the applied lateral load is of a significant proportion compared to the applied downward vertical load. The main issue that arises is: should the calculation of the foundation effective width and the inclination factor be based on factored or unfactored loads? There are supplementary issues related to the use of single-source or worst-case principles, as well as the use of Load Case A and Load Case B. In particular, there may be major issues using DA1 Combination 2 with factored loads. There is wide support for using factored loads, however, some countries in the EU are adopting a special DA2* (or presumably DA1 C1*) where unfactored loads are used. It is strongly recommended that the National Annex should give advice on this, and that extensive sensitivity calculations be carried out beforehand. If factored loads are recommended, then this is likely to result in the base width of gravity retaining walls becoming considerably wider than that provided by "conventional" design. It is strongly urged by the author that unfactored loads be used. The main logic is that resistance is resistance, so if factored loads are used to calculate a component of resistance, and load factors are used again in the final verification, then the load factors have effectively been applied twice.
- It is recommended that stability analysis requires a special section giving general advice and clarification on how to apply the EC7. The draft National Annex includes reference to BS 6031:2009. The 2009 version includes extensive reference to applying EC7 principles to stability analysis, and in particular, Section 7.3.3 is helpful. It is important that the National Annex states whether Combination 1 should follow the single-source principle (making it irrelevant in most cases) or the Frank *et. al.* approach (possibly making it critical). ■

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