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Design Approaches of Eurocode 7 and their Effect on the Safety of Shallow Foundations

K. Lesny
Institute of Soil Mechanics and Foundation Engineering, University of Duisburg-Essen, Germany

ABSTRACT: With issue of Eurocode 7 in 2005 the concept of Limit State Design has been consistently implemented in geotechnical design in Europe. However, the member states themselves define the design approach as well as the partial safety factors according to national design and safety requirements. The design approaches differ in the way the partial safety factors are applied. In case of shallow foundations under complex loading the different approaches may have consequences on the overall safety as the resistance is entirely load depended. In the present paper the correlations between loads and resistances of a shallow foundation within the design philosophy of Eurocode 7 are analysed by use of interaction diagrams. The effects of the design approaches on the overall safety are discussed. An alternative design method based on an unique failure criterion for the Ultimate Limit State is introduced and the implementation of the partial safety concept is discussed.

1 INTRODUCTION

The aim of Eurocode 7 (2005) was to harmonize geotechnical design in Europe. Hence, limit state design together with the partial safety factor concept was consistently introduced in all member states.

However, as a result of extensive discussions the final version of Eurocode 7 (EC7) provides several design approaches for the ultimate limit state (ULS), which differ in the way the partial safety factors are applied. Within these approaches either actions or their effects and either material parameters or resistances are factorized. The design approach and the corresponding partial factors are defined by each member state according to their national safety requirements.

The design approaches are especially relevant in case of shallow foundations where failure of the ground usually controls the footing dimensions within the ULS. For a foundation under complex loading failure of the ground is described by different failure modes (bearing resistance failure, sliding, uplift, overturning). Their main feature is that they are entirely load dependent. Not only the magnitude but also the combination of load components and their ratios determine the resistance of the shallow foundation, e. g. load inclination and load eccentricity both reduce the bearing capacity. Hence, the choice of the design approach may affect the performance of the foundation, especially its overall safety. However, the differences among the design approaches may be more or less pronounced depending on the complexity of loading and on the limit states relevant for design. As a consequence the most critical design situation is difficult to define and the actual safety level can only be approximated.

In the present paper the correlations between loads and resistances of a shallow foundation under complex loading within the design philosophy of EC7 are analysed using the well-known interaction diagrams. The most critical load combinations in the different design approaches of EC7 are pointed out and the effects of these approaches on the overall safety are discussed. Finally, an alternative design method for the ULS of shallow foundations is introduced. In this method the artificial distinction of different limit states is avoided by an unique failure criterion which thoroughly describes the ultimate limit state. This concept allows the visualization of the effect of different actions on foundation stability and a clearer determination of the safety. The implementation of the partial safety concept is discussed as well.

2 DESIGN PHILOSPHY OF EUROCODE 7

2.1 *Ultimate Limit States for Shallow Foundations*

Table 1 summarizes the ULS defined in EC7. For the design of shallow foundations the most relevant ULS are UPL (loss of equilibrium due to uplift by water pressure) and GEO (failure of the ground).

Within limit state UPL vertical stabilizing (stb) and destabilizing (dst) permanent (G) and variable (Q) actions are compared:

$$V_{dst,d} \le G_{stb,d} \tag{1a}$$

$$\text{with } V_{dst,d} = G_{dst,d} + Q_{dst,d} \tag{1b}$$

Table 1. Relevant ultimate limit states according to EC 7

ULS	Failure Mode
EQU	Loss of equilibrium of structure or ground ⇒ resistances not significant
STR	Internal failure of strucure / structural element ⇒ strength of structural material significant
GEO	Failure of the ground ⇒ strength of soil or rock significant
UPL	Loss of equilibrium due to uplift (buoyancy) ⇒ resistances not significant
HYD	Hydraulic heave, internal erosion, piping

Within GEO actions or their effects E and resistances R are compared. Hence, the limit state equation with the design values E_d and R_d is defined according to (2).

$$E_d \le R_d \tag{2}$$

If the partial factors are applied directly to actions, the design value E_d of the effect of actions is defined according to (3). If the effect of actions is factorized the design value is defined according to (4).

$$E_d = \{\gamma_F \cdot F_{rep} \quad X_k/\gamma_M \quad a_d\} \tag{3}$$

$$E_d = \gamma_E \cdot E\{F_{rep} \quad X_k/\gamma_M \quad a_d\} \tag{4}$$

In (3) and (4) the effect of actions E is a function of representative actions $F_{rep} = \psi \cdot F_k$, ground strength parameters X_k (for geotechnical actions) and geometrical data a_d multiplied with a partial safety factor for actions γ_F or for the effect of actions γ_E. The combination factors ψ for the characteristic actions F_k are required for multiple variable actions. Additionally, a material factor γ_M may be required for geotechnical actions.

Partial factors on resistances are applied either to ground properties X_k according to (5) using the material factor γ_M or to the resultant resistance R_k according to (6) using the resistance factor γ_R. If the effects of actions are factorized, $\gamma_F = 1$ in (5) and (6).

$$R_d = R\{\gamma_F \cdot F_{rep} \quad X_k/\gamma_M \quad a_d\} \tag{5}$$

$$R_d = R\{\gamma_F \cdot F_{rep} \quad X_k \quad a_d\}/\gamma_R \tag{6}$$

In case of shallow foundations limit state GEO includes the failure modes bearing resistance failure and sliding, as these resistances are significantly influenced by the strength of the ground.

Overturning may only occur for structures on hard ground and is therefore not considered within this study. An explicit limitation of the eccentricity of the resultant vertical loading is not provided by EC 7. However, the German DIN 1054 (2005) limits the eccentricity resulting from characteristic loading to 1/3 of the foundation width in the ULS.

2.2 *Design Approaches and Partial Safety Factors*

In GEO three design approaches are distinguished, which differ in the way the partial safety factors are applied.

In design approach 1 (DA1) two combinations have to be checked, that means two calculations are required. In combination 1 (DA1-K1) the partial factors are only applied to actions according to (3). The resistance is determined according to (5) with $\gamma_M = \gamma_R = 1$ in (3) and (5). In combination 2 (DA1-K2) ground strength parameters are factorized according to (3) and (5), hence $\gamma_R = 1$. Partial factors γ_F are applied to unfavourable variable actions only.

In design approach 2 (DA2) actions according to (3) or effects of actions according to (4) are factorized. At the same time, partial factors are applied to the ground resistance according to (6). In all equations $\gamma_M = 1$ is assumed and only one calculation is required. In DA2 the resistance is calculated using the design values of actions or their effects. However, in Orr (2005) an alternative concept DA2* is discussed. In this approach the effect of actions and the resistance are calculated with characteristic values of actions and ground strength parameters. The partial factors are applied at the final stage of design when the limit state equation according to (2) is checked.

Design approach 3 (DA3) also requires only one calculation. Regarding the partial factor γ_F actions resulting from the structure and geotechnical actions are distinguished. Actions or their effects are factorized according to (3) or (4) and the resistance is factorized according to (5), hence $\gamma_R = 1$ and $\gamma_M \neq 1$.

Table 2 summarizes the partial safety factors recommended by EC7 for UPL and GEO.

Table 2. Partial safety factors for UPL and GEO (EC7, 2005)

UPL		
	$\gamma_{G,dst} = 1.0$, $\gamma_{G,stb} = 0.9$, $\gamma_{Q,dst} = 1.5$	
GEO		
	$\gamma_G = 1.35$, $\gamma_{G,inf}{}^1 = 1.0$, $\gamma_Q = 1.5$	DA1-K1
Factors on actions		DA2/DA2*
or effects of actions		DA3^2
(γ_F or γ_E)	$\gamma_G = \gamma_{G,inf}{}^1 = 1.0$, $\gamma_Q = 1.3$	DA1-K2
		DA3^3
Material factors (γ_M)	$\gamma_{\varphi'} = \gamma_{c'} = 1.25$, $\gamma_{cu} = 1.4$	DA1-K2 DA3
Resistance factors (γ_R)	$\gamma_{R,v} = 1.4$ (bearing capacity) $\gamma_{R,h} = 1.1$ (sliding resistance)	DA2/DA2*

^{1}: $\gamma_{G,inf}$ for favourable permanent actions only, ^{2}: only on actions from the structure, ^{3}: only on geotechnical actions

Each member state finally defines the design approach and the corresponding partial factors to be used. These decisions are regularized in the national annexes to EC7.

3 SAFETY ANALYSIS OF A SHALLOW FOUNDATION

3.1 *Example of a Vertical Breakwater*

The analysis of the overall safety of shallow foundations is based on the example of a vertical breakwater which already has been introduced in Lesny & Kisse (2004) and Lesny (2006). The system is illustrated in Figure 1.

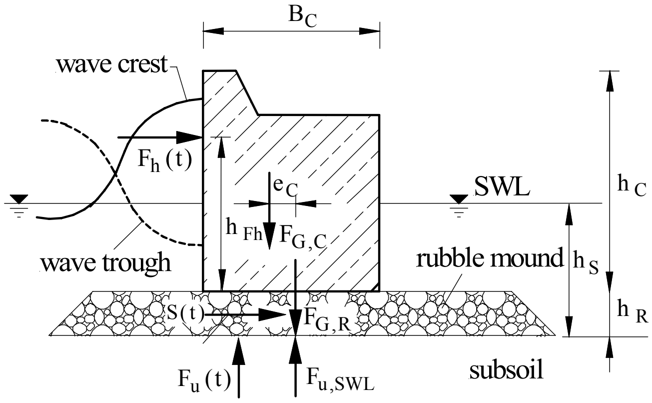

Figure 1. Geometry and loading of a vertical breakwater

The vertical breakwater is a strip footing under combined vertical (F_1), horizontal (F_2) and moment (M_3) loading placed on a thin rubble layer on sandy subsoil. The width B_C and the height h_C of the breakwater are the design parameters. The buoyant unit weight of the sand is assumed to $\gamma' = 10$ kN/m³, the angle of internal friction to $\varphi' = 38°$. The rubble layer of height $h_R = 0.5$ m consists of coarse material with a buoyant unit weight of $\gamma' = 11$ kN/m³ and an angle of internal friction of $\varphi' = 44°$. The roughness of the footing base is characterized by a roughness factor $\mu = 0.8$. The loading of the breakwater has been calculated according to de Groot et al. (1996) for a still water level (SWL) of $h_S = 15.5$ m, a wave height of $H_{max} = 10$ m with a related wave period of 12 s.

Permanent actions result from the dead weights of breakwater ($F_{G,C}$) and rubble layer ($F_{G,R}$) and from uplift due to SWL ($F_{u,SWL}$). Variable actions result from wave impact, hence a horizontal wave load (F_h), a seepage force (S) in the rubble layer and an additional uplift force (F_u) have to be considered. The variable actions are correlated and entirely stochastic. However, within design the system is usually idealized and three load combinations (LC) are investigated: SWL (LC1), wave crest (LC2) and wave trough (LC3).

With that, the characteristic effects of actions for LC1 (SWL) are:

$$F_{1,k} = F_{G,C} + F_{G,R} - F_{u,SWL} \tag{7a}$$

$$F_{2,k} = 0 \tag{7b}$$

$$M_{3,k} = F_{G,C} \cdot e_C \tag{7c}$$

For load cases LC2 (wave crest) and LC3 (wave trough) the characteristic effects of actions are:

$$F_{1,k} = F_{G,C} + F_{G,R} - F_{u,SWL} - F_u \tag{8a}$$

$$F_{2,k} = F_h + S \tag{8b}$$

$$\begin{aligned} M_{3,k} = {} & F_{G,C} \cdot e_C - F_h \cdot \left(h_{Fh} + h_R\right) - S \cdot 0.5 \cdot h_R \\ & - F_u \cdot {}^{B_C}\!/_{6} \end{aligned} \tag{8c}$$

The loading at wave trough (LC3) differs in magnitude and direction from the loading at wave crest (LC2) resulting in different magnitudes and signs of the load components in (8).

The design values for the three load combinations and the different design approaches are determined according to the procedure described in the previous chapter. This step requires an assessment of the effect of permanent actions on foundation stability within DA1-K1, DA2 (DA2*) and DA3. However, it is not completely clear, if the permanent actions act favourable or unfavourable. Hence, the following cases have been investigated:

1 all permanent actions are unfavourable
2 dead weights of caisson and rubble layer are favourable, uplift due to SWL is unfavourable
3 dead weights of caisson and rubble layer are favourable, uplift due to SWL is favourable

The latter case is only relevant for the vertical load F_1 as it does not have any consequence for the moment M_3.

In order to get a first insight in the design of the vertical breakwater the maximum vertical bearing resistance F_{10}, i. e. the capacity under centric vertical loading, has been calculated according to EC7 Annex D (2005) for a caisson width of $B_C = 24.0$ m and a height $h_C = 27$ m. Figure 2 shows the characteristic and the design values of F_{10} following the different design approaches.

For DA1-K1 the resistance coincides with the characteristic value as partial factors are only applied to actions. The dramatic decrease of the resistance in DA1-K2 and DA3 results from the material factor approach in which a design value of the shear strength of $\varphi'_d = \varphi'_k/\gamma_{\varphi'} = 38°/1.25 = 30{,}4°$ is used. As the bearing capacity factor N_γ is very sensitive to changes of the shear strength even a moderate reduction leads to a disproportionately large decrease of N_γ and, with that, of the resultant bearing resistance.

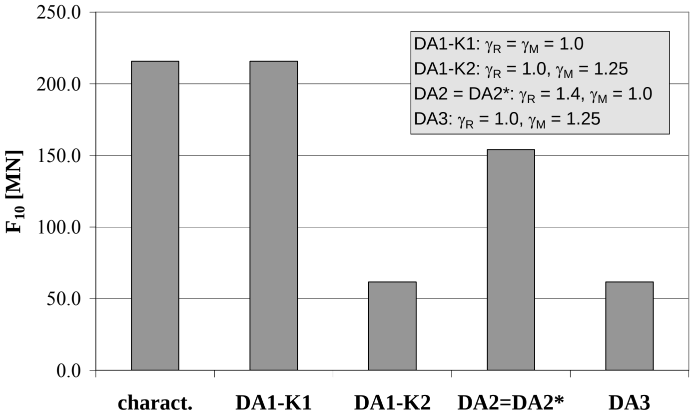

Figure 2. Bearing resistance F_{10} under vertical centric loading (partial factors acc. to Table 2)

3.2 *Safety of the Vertical Breakwater*

The safety of the vertical breakwater for the chosen dimensions has been analysed by the help of interaction diagrams which directly display the interaction of the load components within the respective limit states.

Figure 3 shows the interaction diagram of the F_1-F_2 interaction for LC3 (wave trough). The closed curves represent the characteristic bearing resistance and the design resistances according to the different design approaches. The difference between DA2 and DA2* appears only in the proof of the bearing resistance, hence both design resistances are identical here. The straight lines in Figure 3 are the characteristic and design sliding resistances. Normally, one interaction diagram has to be produced for each eccentricity which varies depending on the type of loading (characteristic or design loading). For simplification an average value of $e/B_C = 0.063$ has been chosen in Figure 3. Additionally, in Figure 4 the F_1-M_3/B_c interaction for LC2 (wave crest) is presented for an average load inclination of $F_2/F_1 = 0.22$. Beside the characteristic and the design bearing resistance the limitation of the eccentricity to $B_C/3$ is indicated as well.

On the first sight there is no big difference in the characteristic and the design values of the resultant loading in Figures 3 and 4. On the other hand, both figures strongly emphasize the decrease in N_γ for design approaches DA1-K2 and DA3, which leads to a striking reduction of the bearing capacity under inclined and eccentric loading.

For the design load according to DA1-K1, DA2 and DA3, assuming that the dead weights of caisson and rubble layer are favourable, the design procedure for bearing resistance and sliding resistance is shown in Figure 3 as well. Within the bearing resistance calculation the effect of vertical actions F_1 is compared to the bearing resistance R_v. Hence, the bearing resistance is also defined as a vertical load resulting from the given load inclination as indicated by the radial load path in Figure 3. The sliding resistance R_h is a horizontal load defined as a function of the resultant vertical load F_1. Hence, it follows a steplike load path. It is compared to the effect of horizontal actions F_2 only as shown in Figure 3 (see also Lesny 2006).

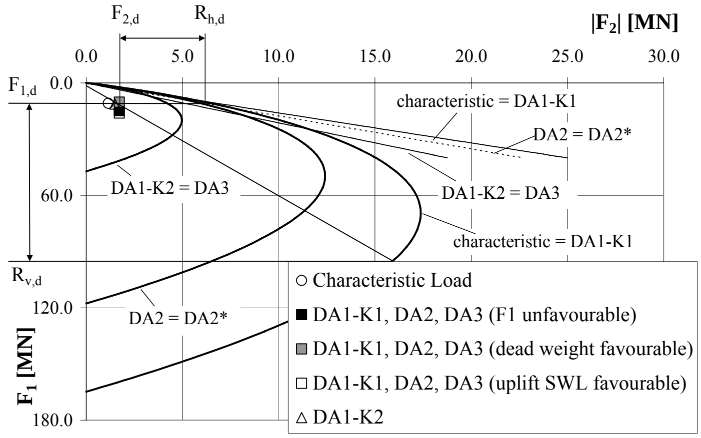

Figure 3. Interaction Diagram for F_1-F_2 – LC3

The distance between the effect of loading and the resistance indicates the degree of mobilization which may be calculated according to equation (9).

$$\mu_v = \frac{F_1}{R_v},\ \mu_h = \frac{F_2}{R_h},\ \mu_M = \frac{e/B_C}{\text{adm } e/B_C} \qquad (9)$$

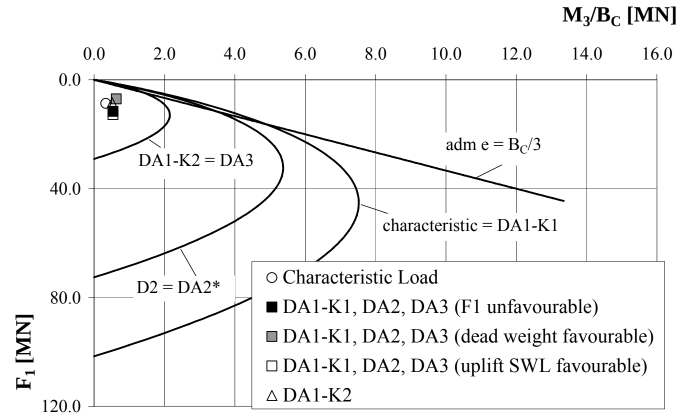

Figure 4. Interaction Diagram for F_1-M_3/B_C – LC2

The degree of mobilization according to (9) has been determined for all limit states following the different design approaches. Figure 5 shows the degree of mobilization of the bearing resistance for all load combinations. It should be noted that in Figure 5 only one value is presented for the characteristic and the design load in DA1-K2, because the influence of the permanent load components does not play a role.

According to Figure 5 the highest degree of mobilization is reached in LC2 as the wave impact at wave crest is higher than at wave trough. In case of SWL, where only permanent loads occur, the degree of mobilization is relatively small.

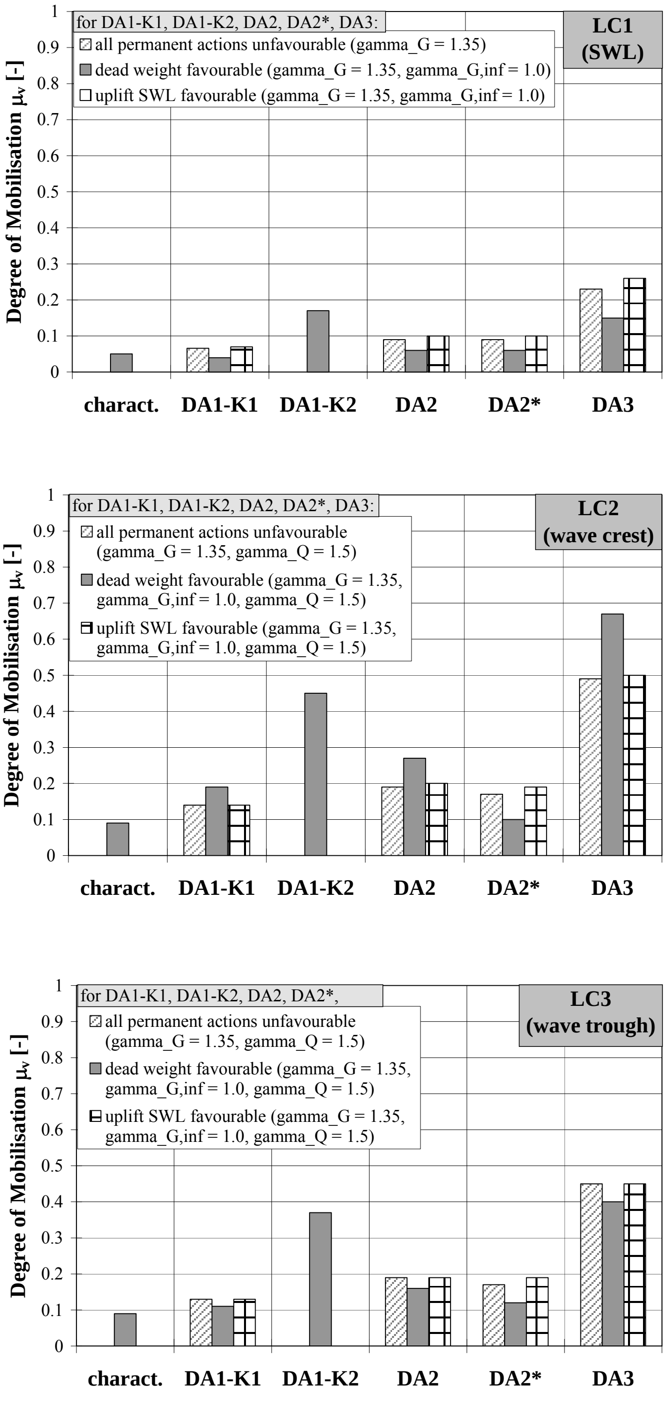

Figure 5. Degree of mobilization of bearing resistance (partial resistance and material factors as in Figure 2)

The material factor approach in DA3 together with partial factors on actions ≠ 1 results in the highest degree of mobilization for all load combinations. Similar though smaller results are gained with DA1-K2 where partial factors ≠ 1 are only applied to variable actions. However, the difference of these design approaches compared to the results of the other approaches are significant and may lead to an uneconomic design.

The smallest footing dimensions may be achieved if DA1-K1 is used. Only for LC2 the degree of mobilization is significantly greater than the one for the characteristic values of load and resistance.

The degree of mobilization in DA2* is generally smaller than in DA2. However, the differences are especially pronounced only for LC2 where load inclination and eccentricity reach their greatest values and are further increased by the application of the partial factors.

On the other hand, the degree of mobilization of the bearing resistance is not very high at all. This has already been indicated by the radial load path in Figure 3. In contrast to this, the sliding resistance is more critical especially in LC2 where the horizontal load reaches it's maximum. This can also be observed in Figure 3, where the steplike load path describes a much shorter distance to the ULS than the radial load path. These findings are confirmed by the results presented in Figure 6.

The degree of mobilization regarding the limitation of the eccentricity varies between $\mu_M = 0.09$ and $\mu_M = 0.27$. However, it does not depend on the design approach, since all partial factors are equal to one.

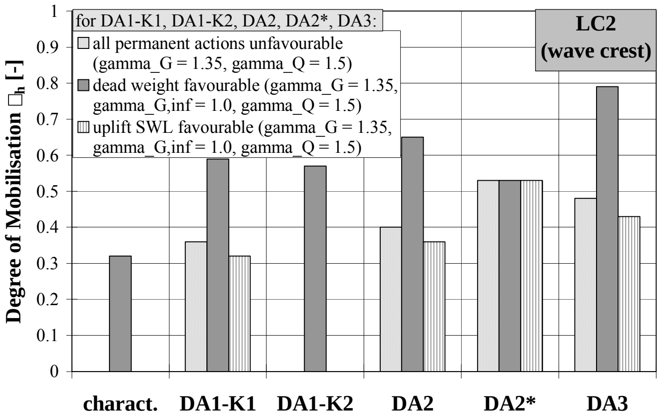

Figure 6. Degree of mobilization of sliding resistance for LC2 (application of partial resistance and material factors as in Figure 2 with $\gamma_R = 1.1$ and $\gamma_M = 1.25$)

Regarding the influence of permanent actions on foundation stability neither Figure 5 nor Figure 6 show a big difference between the assumption that all permanent actions are unfavourable and the assumption that only the dead weights of caisson and rubble layer are unfavourable. The reason for this is that the contribution of the uplift force at SWL is relatively small.

On the other hand, the assumption that the dead weights of caisson and rubble layer are favourable leads to results which seem to be unlogic at the first sight. For LC2 in Figure 5 and 6 this assumption leads to a higher degree of mobilization compared to the other two cases. This means that the influence of a load, which initially has been assumed to be favourable, finally would be more negative as if the load has been assumed to be unfavourable.

However, due to a partial factor of $\gamma_{G,inf} = 1$ the resultant vertical load in this case is about 40 to 50% smaller than in the other two cases. Maximum values of resultant load inclination and eccentricity are the consequence leading to a drastic reduction of the bearing resistance. Further on, the smaller vertical load directly reduces the sliding resistance. Design approach DA2*, in which the bearing resistance is calculated with characteristic values of load inclination and eccentricity, shows exactly the opposite result. This seems to be logic as it corresponds to the initial assessment of the load effect.

In case of the sliding resistance the use of the characteristic vertical load leads to identical results in all cases.

However, this example points out the difficulties in correctly assessing the influence of permanent actions on foundation stability at an early stage of the design. It seems to be more straightforward to evaluate the influence of the effect of vertical actions, i. e. of the resultant vertical load instead of the actions themselves (Lesny 2006). Such an assessment may be based on the interaction diagrams which visualize the interrelations of the load components and their influence on the foundation stability.

In the example of the vertical breakwater the resultant loading is located in the upper portion of the interaction diagrams (see Figure 3 and 4). Hence, an increase of the resultant vertical load improves the safety of the foundation as a greater vertical load reduces load inclination and eccentricity. Therefore it acts favourably, whereas a decrease of the vertical load deteriorates foundation design.

As has already been evaluated in Lesny & Kisse (2004) and Lesny (2006), all design results presented here depend on the specific load paths indicated in Figure 3. If the actual load path is different from these anticipated load paths, the safety of the foundation may be underestimated. For example, an additional load which follows the closest distance from the design load to the resultant resistance is most hazardous for the foundation. Thus, it describes the most critical load condition. Such a load condition is not unrealistic as a higher wave load than the one assumed here would lead to an increase of the resultant horizontal load and moment but a decrease of the vertical load due to higher uplift forces.

For this situation the degree of mobilization can be determined analogous to Butterfield (1993) by

$$\mu = Q/(Q + \Delta Q) \tag{10}$$

In (10) Q is the resultant design load and ΔQ is the maximum additional load following the most critical load path:

$$Q = \sqrt{F_1^2 + (M_3/B_C)^2} \tag{11a}$$

$$\Delta Q = \sqrt{\Delta F_1^2 + (\Delta M_3/B_C)^2} \tag{11b}$$

Exemplary for the design load according to DA1-K1 (dead weights favourable), equation (10) leads to a degree of mobilization of $\mu = 0.76$, whereas the traditional design leads to values of $\mu_v = 0.19$ and $\mu_h = 0.59$. Hence, the actual safety is much more critical than calculated by the traditional design.

After all, it can be summarized that the design procedure of EC7 may lead to a misinterpretation of the safety of a shallow foundation under combined loading. A reliable and finally economic design cannot be guaranteed. For this reason an alternative design method for the ULS of shallow foundations is introduced in the following chapter which allows a better understanding of the interrelations of the different load components and may help to improve the design.

4 AN ALTERNATIVE DESIGN METHOD

4.1 *Concept of Design*

The alternative design method includes an unique failure condition which describes the ULS without distinguishing the different failure modes mentioned above. It has been developed for foundations on non-cohesive soils with and without embedment. The main aspects are described in the following, for more details see Lesny (2001), Lesny & Richwien (2002), Lesny & Kisse (2004).

In the general case a single footing is loaded by a vertical load F_1, horizontal load components F_2 and F_3, a torsional moment M_1 and bending moment components M_2 and M_3. The load components may be summarized in a load vector $\vec{Q}$:

$$\vec{Q}^T = [F_1\, F_2\, F_3\, M_1\, M_2\, M_3] \tag{12}$$

For a footing on non-cohesive soil without embedment the geometry of the footing, described by the side ratio $\bar{b} = b_2/b_3$, weight γ, shear strength $\tan\varphi'$ of the soil and the roughness of the footing base μ_s have to be considered as well. With these input parameters the failure condition of the general form

$$F(\vec{Q}, \bar{b}, \gamma, \tan\varphi', \mu_S) = 0 \tag{13}$$

is defined by the following expression:

$$\sqrt{\frac{F_2^2 + F_3^2}{(a_1 \cdot F_{10})^2} + \frac{M_1^2}{(a_2 \cdot (b_3) \cdot F_{10})^2} + \frac{M_2^2 + M_3^2}{(a_3 \cdot b_2 \cdot F_{10})^2}} - \frac{F_1}{F_{10}} \cdot \left(1 - \frac{F_1}{F_{10}}\right)^{\alpha} = 0 \tag{14}$$

In (14) all load components are referred to F_{10}, the bearing resistance under centric vertical loading.

This quantity is calculated using the traditional bearing capacity formulae (e. g. EC7 Annex D). The advantage of this formulation is that the complex mutual interaction of the load components is described directly without using reduction factors or the concept of the effective area. Other influences on the bearing capacity are included in F_{10}.

In an interaction diagram like the one in Figure 3 or 4 the failure condition spans a failure surface, which is the outer boundary of the admissible loading. The parameters a_1, a_2 and a_3 govern the inclination of this failure surface for small vertical loading where the limit states sliding and, if required, limitation of the eccentricity have previously been relevant. These limit states are integrated by defining the parameters a_1, a_2, a_3 and α acc. to (15).

$$a_1 = \pi/2 \cdot \mu_S \cdot \tan\varphi' \cdot e^{-\pi/3 \cdot \tan\varphi'},\quad a_2 = 0.098 \cdot \bar{b},$$
$$a_3 = 0.42,\quad \alpha = 1.3 \qquad (15)$$

The limit state uplift is already included in (14), because only positive vertical loads are admissible. The parameters have been derived from an analysis of numerous small scale model tests conducted at the Institute of Soil Mechanics and Foundation Engineering in Essen and verified against other tests reported in the literature (Lesny 2001). Figure 7 shows the failure condition for combined F_1-F_2-M_3/b_2 loading compared to various model test results.

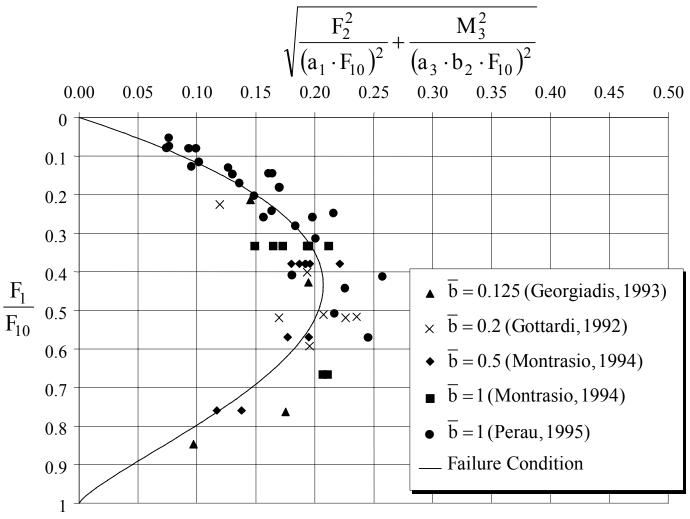

Figure 7. Failure condition for inclined and eccentric loading vs. failure loads from small scale model tests

4.2 *Implementation of the Partial Safety Concept*

To implement a safety concept for the ULS following the Limit State Design first the characteristic bearing capacity shall be considered. The characteristic bearing capacity F_k of the vertical breakwater defined by the failure condition (13) is shown in Figure 8. Additionally, the characteristic and the design loading in load cases LC2 (wave crest) and LC3 (wave trough) are displayed. The effect of the load components on the stability of the foundation has been assessed on the basis of the load interaction presented in Figure 8. Hence, the vertical load acts favourably as an increase would stabilize the system. However, to maintain a certain safety level in case of favourable loads as well, F_1 consequently has to be reduced by a partial factor. Here, a factor of $\gamma_{F1,inf} = 0.75 \approx 1/1.35$ has been chosen. Horizontal and moment loading are unfavourable, so they are factorized by a factor of $\gamma_{F2} = \gamma_{M3} = 1.5$. This procedure means that the design loads move closer to the failure surface defined by the failure condition providing the required safety level. LC1 does not need to be checked here as load cases LC2 and LC3 are more unfavourable.

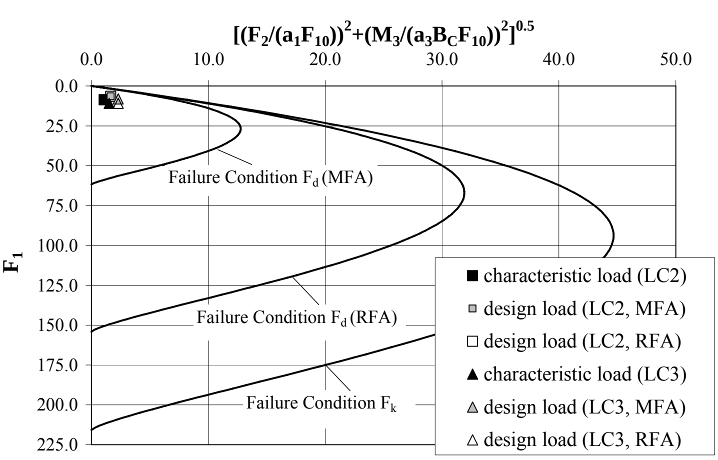

Figure 8. Characteristic and design failure condition and loading

The design failure condition F_d has been determined according to the Resistance Factor Approach (RFA) as well as to the Material Factor Approach (MFA). In the RFA the parameters a_1, a_2 and a_3 in equation (14) are divided by the required resistance factor $\gamma_{R,i}$ which has been set here to a constant value of 1.4. In the MFA the failure condition has been calculated using the design value of the angle of internal friction φ'_d with a material factor $\gamma_M = 1.25$. The maximum bearing capacity $F_{10,k}$ and it's design values already have been determined in chapter 3.1.

This procedure practically means that the failure surface shrinks in the way as indicated in Figure 8. Finally, the foundation stability is verified, if it can be shown that :

$$\underset{\vec{L}_d \in L_d}{\forall} F\left(\ldots, \gamma_{R,i} \text{ or } \gamma_M, \vec{L}_d\right) < 0 \qquad (16)$$

where $\vec{L}_d$ is one design load combination of the set of all design load combinations L_d which need to be checked.

If the inequality (16) is fulfilled all design load combinations are located inside the design failure surface. Here, the MFA leads to $F_d = -0.064$ in LC2 and

F_d = -0.072 in LC3. The RFA results in F_d = -0.029 in LC2 and F_d = -0.049 in LC3. Again LC2 is the critical load combination as it produces the smallest vertical load. Hence, the design load moves closer to the failure surface, although the moment for LC3 is greater than in LC2.

The procedure described before also reveals that within the alternative design method the partial safety factors are no longer distinguished according to different limit states but according to possible load interactions. Hence, at least separate resistance factors for centric vertical loading, inclined loading, eccentric loading and torsional loading must be defined. However, as the interrelations between the load components are coupled the partial factors may be coupled as well in case of more complex loading, e. g. inclined, eccentric loading as in the example presented here. Further on, the resistance factors should also cover uncertainties in the limit state equation (failure condition), so they have to be calibrated on the basis of this method. However, more research work is required for definition and calibration of the partial factors.

5 CONCLUSIONS

The safety of a shallow foundation under combined loading following the design procedure of Eurocode 7 (2005) has been analysed using the example of a vertical breakwater. Striking differences between the design approaches of EC7 have been found out. Obviously, DA3 leads to the most uneconomic design whereas DA1-K1 results in the smallest footing dimensions. However, the most straightforward design is provided by DA2* in which partial factors are only applied on effect of actions and resistance at the final state of design. This procedure follows a clear Load and Resistance Factor Approach.

However, two main problems still remain in the traditional design, which have been identified as well. First of all, the assessment of the influence permanent vertical actions have on foundation stability is obviously difficult. Design results may indicate that the influence of a vertical load assumed to be favourable finally is unfavourable.

Second, due to the load dependency of the safety calculated by the procedure of EC7 the actual safety may be underestimated. This may lead to an unsafe design especially in case of high variable loading. An alternative design method has been introduced which allows a clear foundation design as it directly considers the interrelations of the load components and is not load path dependent. The implementation of a partial safety concept has been discussed as well.

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