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The Role of Favourable and Unfavourable Actions in the Design of Shallow Foundations according to Eurocode 7

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Abstract

For the design of shallow foundations under combined loading Eurocode 7 provides different partial safety factors for unfavourable and favourable actions and for different ultimate limit states. Several load combinations need to be checked to find the most critical combination of loading and limit state which governs design. Hence, the resulting safety is often not apparent. This paper illustrates the role of actions within the design of shallow foundations and points out the effects on the safety of the system.

Introduction

The design of geotechnical structures according to Eurocode 7 (prEN 1997-1, 2004) is based on limit state design (LSD). Ultimate limit states (ULS) and serviceability limit states (SLS) are defined and the safety is calculated with the help of the partial safety factor method. To analyze the ULS of shallow foundations several failure modes, e. g. bearing resistance failure, sliding or uplift, need to be checked. Consequently, the partial safety factor method provides partial safety factors for each failure mode. The effect of an action, whether it is favourable or unfavourable, is considered by different partial safety factors as well. This procedure has apparent disadvantages, because actions directly influence the resistance of the foundation which is a function of the loading. Particularly in the design of foundations under complex loading classifying actions as favourable or unfavourable within a certain limit state is often problematic. In the following the role of favourable and unfavourable actions in the design of shallow foundations is pointed out and the consequences on the resultant safety are discussed.

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Design procedure for the ULS of shallow foundations according to Eurocode 7

The main aspects of the design procedure according to Eurocode 7 (EC 7) for the ULS of shallow foundations under combined loading are briefly summarized here. Table 1 shows the most relevant ULS which determine the footing dimensions together with the respective limit state equation. For the limit state UPL (loss of equilibrium due to uplift) vertical stabilizing (stb) and destabilizing (dst) permanent (G) and variable (Q) actions are compared. Within GEO (failure of ground) three design approaches are distinguished, which differ in the way the partial safety factors are applied. In design approach 1 the partial safety factors are applied either on actions or on ground strength parameters, which requires always two calculations. In design approaches 2 and 3 actions or effects of actions are factorized. At the same time, in design approach 2 partial safety factors are applied on ground resistances, whereas in design approach 3 ground strength parameters are factorized. Hence, design approaches 2 and 3 both require only one calculation.

Table 1. Relevant ultimate limit states for shallow foundations according to EC 7

ULS	Failure mode	Limit state equation
UPL	Loss of equilibrium due to uplift $\Rightarrow$ resistances not significant	$V_{dst,d} \le G_{stb,d}$ with $V_{dst,d} = G_{dst,d} + Q_{dst,d}$
GEO	Failure of the ground $\Rightarrow$ strength of soil or rock significant	in general: $E_d \le R_d$ bearing resistance: $V_d \le R_{v,d}$ sliding resistance: $H_d \le R_{h,d}$

If we focus on design approach 2 for GEO and factorize the effect of actions, its design load E_d in Table 1 is defined according to annex B of EC 7 as follows:

$$E_d = \gamma_E \cdot E\{F_{rep} \quad X_k \quad a_d\} \tag{1}$$

with the effect of actions E as 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 the effect of actions γ_E. The combination factors ψ for the characteristic actions F_k are required for multiple variable actions. Design resistance R_d in Table 1 is defined correspondingly (annex B of EC 7):

$$R_d = R\{F_{rep} \quad X_k \quad a_d\}/\gamma_R \tag{2}$$

with F_{rep}, X_k and a_d as defined above and the partial safety factor for the resistance γ_R. Table 2 shows the partial safety factors for GEO provided by EC 7. Within the bearing resistance calculation the effect of actions results only from vertical actions. The resultant vertical load is compared to the bearing resistance usually calculated with the traditional bearing resistance formula (see Annex D of EC 7). Hence, the bearing resistance is defined as a vertical load. It is a function of the load inclination H/V and the eccentricity e = M/V and therefore load-dependent. The sliding resistance is a horizontal load component derived from the shear resistance in the footing base as a function of the resultant vertical load V. It is compared to the effect of actions resulting from horizontal actions only.

Table 2. Partial safety factors for GEO according to EC 7 (design approach 2)

Effect of actions	
Permanent actions	
- favourable	$\gamma_{E,G} = 1.0$
- unfavourable	$\gamma_{E,G} = 1.35$
Variable actions	
- favourable	$\gamma_{E,Q} = 0$
- unfavourable	$\gamma_{E,Q} = 1.5$

Resistance	
Bearing resistance	$\gamma_{R,v} = 1.4$
Sliding resistance	$\gamma_{R,h} = 1.1$

Effect of favourable and unfavourable actions on foundation stability

The procedure described above is the most straightforward way to introduce the partial safety concept in geotechnical design. On the other hand it does not represent the basic idea of applying partial safety factors directly on the sources of uncertainties, actions and ground strength parameters. However, as geotechnical resistances are load dependent, calculating the foundation resistance with design loads implies a change of the real design situation. This can be demonstrated by the example of a strip footing on sand without embedment depicted in Figure 1. The footing is loaded by a permanent vertical load including weight of the footing $V_{G,k}$ and a variable horizontal load $H_{Q,k}$. The present load inclination $\tan\delta_k = H_{Q,k}/V_{G,k}$ reduces the characteristic bearing resistance $R_{V,k}(H_{Q,k} = 0)$ by the factor $i_\gamma = (1 - \tan\delta_k)^3$. However, if the resistance is determined from the design loads $H_{Q,d}$ and $V_{Q,d}$ and assuming that $V_{G,k}$ acts unfavourably the design load inclination is

$$\tan\delta_d = \frac{H_{Q,d}}{V_{G,d}} = \frac{1{,}5 \cdot H_{Q,k}}{1{,}35 \cdot V_{G,k}} = 1{,}1 \cdot \tan\delta_k \qquad (3)$$

which is greater than the actual load inclination. If additionally the horizontal load acts on top of the footing as indicated in Figure 1, it produces a bending moment, which reduces the effective width of the foundation $B' = B - 2 \cdot e_d$ by the eccentricity $e_d = (H_{Q,d} \cdot d)/V_{G,d}$. This eccentricity is also 1.1 times larger than the eccentricity from characteristic loads resulting in a greater reduction of the effective width.

However, if the vertical load acts favourably and we use $\gamma_{G,inf} = 1.0$, load inclination and eccentricity are even 1.5 times larger than for characteristic loads as shown in Figure 1b. Together with the partial safety factor for the resistance γ_R this procedure leads to an excessive reduction of the bearing resistance and a much more conservative design as reported by Schuppener and Vogt (2005).

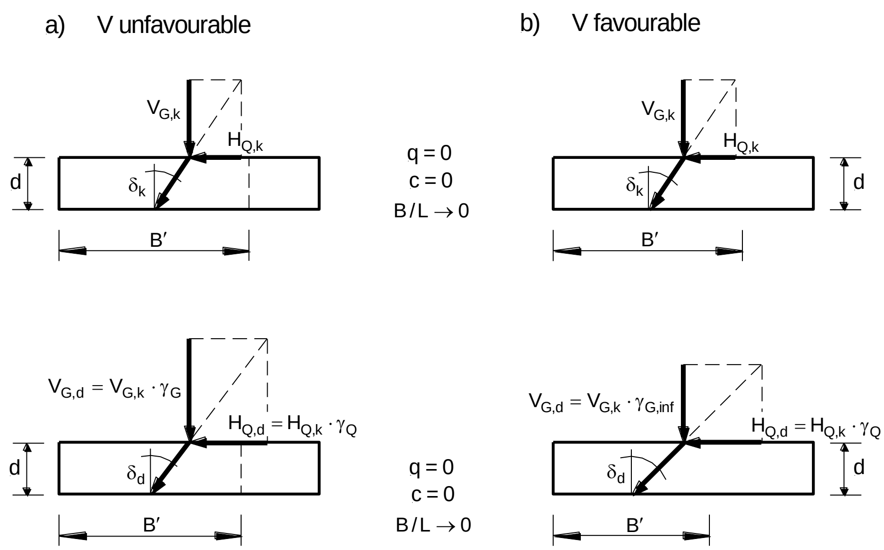

Figure 1. Influence of partial safety factors on load inclination and effective width

This simple example indicates another problem especially for foundations under combined loading arising from the classification of actions as favourable or unfavourable. In Figure 2a bearing resistance and sliding resistance of a shallow foundation under inclined loading are qualitatively depicted as functions of the load components. In this interaction diagram the sliding resistance is illustrated as a simple straight line, whereas the bearing resistance is a closed curve.

Referring to the design procedure described in the previous chapter Figure 2a shows that the bearing resistance calculation implies a radial load path, which is the same for loading and resistance. The sliding resistance calculation is based on the assumption of a steplike load path. The distances between design loads and design resistances represent the actual degree of mobilization. In Figure 2b bearing resistance and sliding resistance are referred to the maximum vertical resistance V_{max}.

Hence, the diagram shows the pure interaction of the load components. In this illustration a maximum horizontal load can be applied for $V/V_{max} \approx 0.42$. When starting from the maximum resistance V_{max} it is apparent that the application of an arbitrary horizontal load is always unfavourable as it decreases the vertical resistance. For a given horizontal load, on the other hand, the application of a vertical load always acts favourable as a minimum vertical load (min V) is required to carry the horizontal load. This means, the load inclination is limited and the limit is provided by the sliding resistance for a certain roughness of the footing base $\tan \delta_s$. With increasing vertical load the resultant load inclination decreases and the resistance of the system increases. However, because of the convex shape of the bearing resistance the degree of mobilization increases if $V/V_{max} > 0.42$, so the magnitude of the applicable vertical load is limited as well (max V).

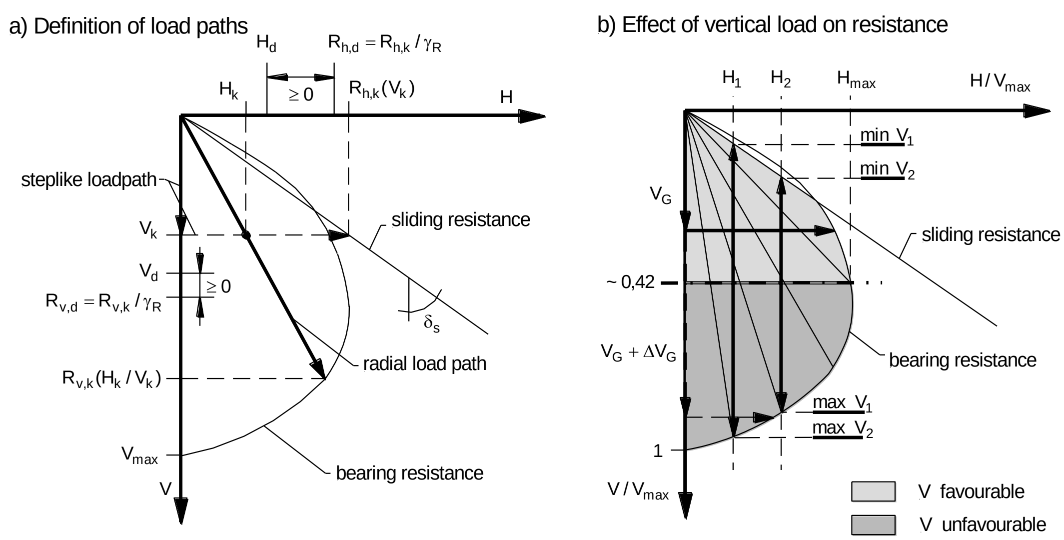

Figure 2. Influence of load components on bearing resistance and sliding resistance

If we now consider a given vertical load, e. g. the weight of the footing V_G as a permanent load, only one maximum horizontal load exists. A larger horizontal load can only be applied if the vertical load, the weight, is increased simultaneously. So, the vertical load acts favourably, which affirms the previous statement. However, this is only valid for $V/V_{max} < 0.42$. Larger vertical loads ($V_G + \Delta V_G$) act unfavourably as they reduce the maximum allowable horizontal load. An arbitrary increase of the dead weight of the footing would be counterproductive as it does not help to improve the performance of the system. These complex interrelations confirm that especially the role of the vertical load component is apparently not clear and it is difficult to classify it as favourable or unfavourable. The effect of favourable and unfavourable actions also has consequences for the actual safety of the system as may be explained by the following example of a vertical breakwater. The breakwater is a strip footing of width B on sand under combined $[V; H; M]$ loading. Details are described in Lesny and Kisse (2004). In Figure 3 the bearing resistance and the sliding resistance of the breakwater are depicted for a fixed eccentricity of $e_k/B = 0.12$ in the V-H-plane.

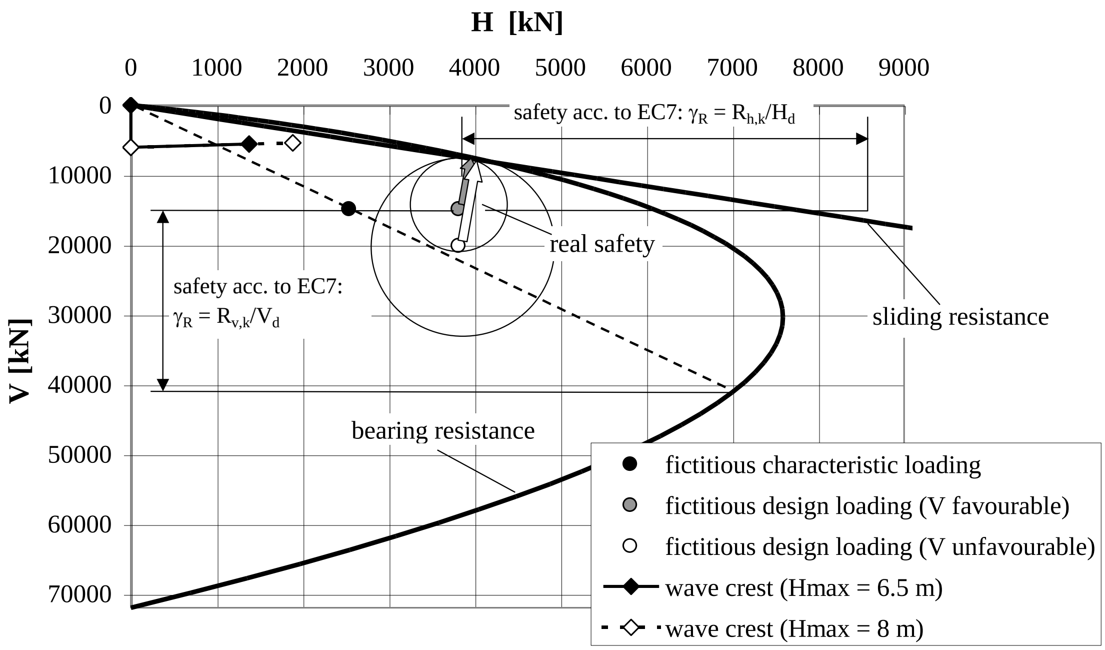

Figure 3. Stability analysis of a vertical breakwater (from Lesny and Kisse, 2004)

We assume a fictitious characteristic loading of $H_{Q,k} = 2.55$ MN and $V_{G,k} = 15$ MN. The design loads with the partial safety factors of Table 2 are $H_{Q,d} = 3.82$ MN and $V_{G,d} = 15$ MN (V favourable) or $V_{G,d} = 20.3$ MN (V unfavourable), respectively. The safety against sliding according to EC 7 is $\gamma_{R,h} = R_{h,k}/H_{Q,d} = 2.2 > 1.1$. For the bearing resistance the safety is $\gamma_{R,v} = R_{v,k}/V_{G,d} = 2.7 > 1.4$ if V is favourable, but only $\gamma_{R,v} = 2.7$ if V is unfavourable. On the other hand, the safety of the system is sufficient in all cases. However, these results do not represent the actual safety of the foundation. Within the interaction diagram of Figure 3 the actual safety is described by the closest distance of the loading to the resistance of the foundation as indicated by the arrows. Additional load components acting along this path are most hazardous. If arbitrary load paths are possible, only additional load components acting within the circles sketched in Figure 3 are admissible. The actual safety can be determined with the help of the load vector $\vec{Q} = [V_{G,d}; H_{Q,d}]$ and the additional load vector $\Delta\vec{Q}$ in the V-H-plane, which coincides with the radius of the circles in Figure 3 (Butterfield, 1993).

For the design load components given above the maximum additional loading is limited by the sliding resistance and amounts to $\Delta Q = 2.42$ MN (V favourable) or $\Delta Q = 5.0$ MN (V unfavourable), respectively. Thus, the actual safety of the system is:

$$\gamma_R = (Q + \Delta Q)/Q = \begin{cases} 1.16 & \text{V favourable} \\ 1.24 & \text{V unfavourable} \end{cases} \tag{4}$$

Hence, the actual safety in both cases is less than the required safety against bearing resistance failure, but slightly greater than the required safety against sliding. However, the safety for V assumed to be unfavourable is greater than if V is favourable as already indicated by the longer arrow in Figure 3. Not only that this is contradictory to the result of the safety calculation of EC 7, but it is also inconsistent with the classification of V as unfavourable as this load finally improves the safety of the system. The reason for these inconsistencies can be found in the convex shape of the resultant resistance. As a consequence the safety of the system depends on the load path. This may be critical for design situations with large variable actions especially if the vertical load is small. In case of the vertical breakwater the permanent load results only from the eccentric dead weight of the structure, whereas the variable loads result from the wave loading. In Figure 3 the load path of the design load for a wave height of $H_{max} = 6.5$ m is shown. The actual safety according to (4) is $\gamma_R = 1.15$, so the sliding resistance is proved but not the bearing resistance. If the wave height is actually $H_{max} = 8.0$ m only the variable loading changes resulting in an increase of the horizontal load, but a decrease of the vertical load. Hence, the safety of the system is reduced significantly to $\gamma_R = 1.06$ according to (4) and is not sufficient at all. In opposite to these results the safety prediction according to EC 7 suggests a sufficient safety in all cases with e. g. $\gamma_{R,h} = 1.2$ and $\gamma_{R,v} = 6.0$ for $H_{max} = 8.0$ m. This example shows clearly that the assumption of certain load paths within traditional design may lead to a misinterpretation of the actual level of safety. This is problematic especially if the stability of a structure is influenced by parameters with a high uncertainty like the wave parameters in the example presented here. This problem can only be solved if a safety concept is established which takes into account the load path dependency described above. As a basis for a consistent definition of the safety concept, the ULS of a shallow foundation must be defined by a unique limit state equation without distinguishing different failure modes. Such a model already has been presented by Lesny and Richwien (2002).

Conclusion

The design procedure of the limit state design according to EC 7 and the role of unfavourable and favourable actions on the stability of a shallow foundation was illustrated. It has been shown that the classification of actions for foundations under combined loading is difficult.

Especially the role of the vertical load within the determination of the foundation resistance is not clear. This influences also the actual safety of a shallow foundation, which may be underestimated by the traditional limit state design. The actual safety depends on the load path which is not reflected by the current design procedure. An example has been shown where this may lead to a miscalculation of the available safety. Three main conclusions may finally be derived:

- Geotechnical resistances must be calculated based on characteristic load components to reflect the real design situation.
- The classification of an action should be based on its effect on the actual safety of the system.
- As geotechnical resistances are load-dependent a safety concept is needed which also takes into account this load path dependency. Such a safety concept requires a unique limit state equation like the one suggested by Lesny and Richwien (2002).

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References

Butterfield, R. (1993). "A New Approach to Safety Factors for Shallow Foundations: Load Combination Factors as a Basis for Risk Assessment." *Risk and Reliability in Ground Engineering*, Thomas Telford, London, 112-125.

Lesny, K; Richwien, W. (2002). „A Consistent Failure Model for Single Footings embedded in Sand.*" Proc. Int. Workshop on Foundation Design Codes and Soil Investigation in View of International Harmonization and Performance Based Design*, Kamakura, Tokyo, Japan, 159-165.

Lesny, K.; Kisse, A. (2004). "Safety of Shallow Foundations in Limit State Design." Proc. Int. Workshop on Risk Assessment in Site Characterization and Geotechnical Design, Bangalore, India, 97-109

pREN 1997-1 (2004). *Geotechnical Design – Part 1: General Rules*, European Committee for Standardization, Brussels.

Schuppener, B.; Vogt, N. (2005). „Favourable and Unfavourable Actions in the Verification of Bearing Capacity of Footings." Proc. ERTC 10/GeoTechNet WP2 Workshop, Evaluation of Eurocode 7, Dublin, Ireland.