Finite Element Method (FEM) for verifications in geotechnical design

La méthode des éléments finis pour les vérifications géotechniques

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ABSTRACT
Verifications in geotechnical design according to national and European standards are mostly based on limit state models like slip circle or sliding wedges – and the safety factors are related to these models. To use numerical models like the FEM it has to be checked, to what extent the results coincide. Numerical methods such as the FEM enable failure patterns to be forecast from deformations occurring close to the point of failure instead of assuming a priori a limit state geometry for a particular verification. Also boundary conditions can be considered much more in detail. In addition, one single FE calculation is able to provide not only the effects of the actions for ultimate limit state design (ULS) but also those needed for serviceability limit state design (SLS). Obviously there are possibilities and limitations of the FEM for verifications in geotechnical design according to standards, here concentrating on the new European standard EN 1997-1 (EC 7).

RÉSUMÉ
Les vérifications géotechniques selon les normes nationales et européennes s'appuient sur des modèles de certaines états limites comme des cercles ou des prismes de glissement. Les coefficients de sécurité sont établis pour ces modèles. Pour appliquer la méthode des éléments finis (FEM) il faut examiner, dans quelle mesure les résultats sont comparable. Des méthodes numériques comme FEM permettent de déterminer l'état limite sur la base des déformations près de la rupture au lieu de supposer à priori une géométrie de rupture. Pareillement, les conditions aux limites sont mieux pris en considération. En outre, une seule analyse donne les résultats pour l'état limite ultime et pour l'état limite de service. De toute évidence la méthode des éléments finis offre des chances mais a également des restrictions concernant les vérifications géotechniques comme l'indique le norme européenne EN 1997-1.

Keywords : Verifications, finite element method, safety factors, European Standards

1 INTRODUCTION

Every design is based on a concept of a model. When approaches are used that are well established since a long time, this might be forgotten sometimes. Modelling includes drawing up a priori rules which may have a considerable influence on the results. Examples of this are the magnitude of the wall friction angle or the shape of the failure surface. Furthermore there is a strong relation of the model and the safety factors. Any safety factor defined in a standard is related exclusively to the calculation procedure described in that standard, since only for that combination exists the long-term experience that allowed to establish a reliable safety factor. The new generation of standards therefore discusses models concerning loads, materials and calculation methods.

Numerical models like the Finite Element Method (FEM) are well established to predicting the material behaviour of the ground and the interaction of ground, water and structure far better than with any conventional calculation method. But an entirely different type of modelling is adapted when methods such as the FEM are used. Therefore the obvious idea to perform verifications required by the standards by making use of FEM has to be checked very carefully.

2 PRINCIPLES OF DESIGN IN STANDARDS

National geotechnical safety standards are currently based on the European Standard EN 1997-1. Unfortunately, it was not possible to reach agreement on geotechnical design in Europe, with the result that three different design approaches are given in the standard. Germany mostly follows Design Approach 2. The European Standard does not stipulate when the partial safety factors have to be introduced into the calculation. One way of proceeding, which is also used in structural engineering, is to apply the partial safety factors to the actions and resistances at the outset and to perform the calculation with design values (known as "factoring the input parameters"). However, another way is to perform the calculation with the characteristic loads – as in the past – and to determine the effects of actions required for each verification. The partial safety factors are then applied to the effects of actions obtained in the calculation (e.g. the internal structural forces) ("factoring the effects of actions") before performing the verification. The second method, which is adopted for most German standards, is referred to as "Design Approach 2*" (Frank et al. 2004) and lends itself to verifications performed with numerical methods, as will be shown later.

3 MODELS

The awareness that the models on which verifications are based are only more or less appropriate is reflected either in the numerical values of the safety factors or in the requirement for two verifications (e.g. DIN 1054 (2005), subclause 10.6.7).

Modelling means selecting representative parameters which will result in safe and economic construction. There are many points at which it is required, as listed below.

- The appropriate possible loads and combinations of loads must be selected.
- The existing geometry must be simplified but still provide safe results.
- The characteristic values of the materials must be representative.

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- The most important stages of the construction process must be specified and the appropriate verifications performed.
- Dispersive results must lead to a single set of (or only a few) instructions for dimensioning.
- Contradictory results obtained by different calculation methods must be reconciled.

3.1 *Material models*

Selecting the appropriate characteristic values of the materials is always a critical, yet essential part of the construction project as a whole. As the cost and effort involved in conducting ground investigations is often disproportionately high compared with the overall volume of the construction project, there was no prospect of introducing a safety concept based on a probabilistic method in geotechnical engineering. There is a tendency to produce very conservative calculations instead of conducting an appropriate and specific programme of investigations or performing loading tests – but when are permeability values or consistencies, for example, conservative and when are they not? So it cannot be emphasised strongly enough that it does not pay to save on ground investigations.

On the other hand, the design engineer is also in a dilemma if sufficient characteristic values of the ground are available. As it is not practical to alter construction methods and the use of materials within short spaces of time during a construction project a representative design profile must be drawn up. After the load models have been drawn up, the next important step is therefore to establish models for the soil and for the geometry of the strata and the ground prior to performing the first calculations.

The material laws of numerical methods are specified on the basis of concepts of models which are able to provide a reasonably good picture of the real behaviour as a function of the loading history. They allow also to model laboratory tests which can be used for calibration purpose.

3.2 *Calculation models*

There are essentially two ways of assessing failure of the ground (Arslan 1980). One involves determining the deformations of the ground and structure, often on the basis of the assumption that the deformation behaviour under the given loads is at least partially linear-elastic, while the other involves considering failures on the basis of rigid plasticity. The models generally used for the latter are the rigid body failure mechanisms which dominate many geotechnical designs. The process, which is actually a continuous one, starting with an initial condition and progressing through deformations with elastic and plastic components to yielding and failure, is therefore dealt with as two separate problems for which there are different concepts of models.

It must never be forgotten that there will always be discrepancies between models and reality. The usual equation for earth pressure provides an earth pressure distribution which increases linearly from the top to the toe of the wall. In reality, however, the earth pressure is redistributed. Supporting points which are displaced only slightly bear greater loads than sections that can avoid loading. The final distribution always depends on the movement and deformation of the wall. Furthermore, a disproportionate degree of importance is attached to the selection of the wall friction angle in models for traditional calculation methods. The level of wall friction must first be estimated, although this also compensates for shortcomings in the calculation model.

The classical calculation methods often do not take account of deformations until the end of the design procedure and then do so with very simple models. However, the deformation behaviour must be considered at the very beginning as loads and deformations are mutually dependent, especially where structure-soil interaction is concerned. The redistribution of the active earth pressure is a well-known example.

3.3 *Numerical models*

An entirely different type of modelling, which may also have a considerable influence on the results, is required when numerical methods such as the FEM are used. The soil is initially considered as a continuum (which is inconsistent with its granular structure) and then split into elements to enable a numerical solution to be found. In a continuum, the development of failure wedges, slip circles or other slip surfaces as used in conventional ultimate limit calculations is inherently impossible, even though there is research effort to overcome this limitation.

Since the mesh of elements can only cover a limited area, the boundary conditions will have a considerable effect on the results, in flow calculations even more than in deformation calculations. So care has to be taken that the boundaries are chosen far enough from the area considered.

4 SERVICEABILITY LIMIT STATE (SLS)

There is already general agreement that numerical models such as the FEM are far more suited to predicting deformations than any conventional method when calculations for the serviceability limit state (SLS) are performed. These calculations do not include safety factors and the limiting values (e.g. maximum differential settlements) are specified empirically. The FEM is far superior to models based on springs or an elastic half-space. Of course, a tested material law which reliably reflects the actual behaviour of the soil is required. If FE calculations only cover the elastic behaviour of the soil the results cannot be expected to be better than those obtained by other methods. The quality of the results depends on that of the material law. The forecasts can be considerably improved by calibrating the models against measurements taken during different construction stages, for example (Schwab 2002).

5 ULTIMATE LIMIT STATE DESIGN (ULS)

The decision to introduce in German standards Design Approach 2* of EN 1997-1, in which the effects of actions are first determined with characteristic values and then factored as appropriate for the respective verification, is an advantage for design with the FEM. Design Approach 2* enables the ground behaviour to be simulated realistically in the FE analysis. By contrast, calculating safety levels by factoring the shear parameters (Design Approach 3) will result in inaccurate material behaviour.

A limit state must first be defined before a verification that takes into account the ground resistance can be performed according to the standard. In classical rigid body analyses the limit state is defined by assuming that the shear strength of the soil is achieved in each of the (preselected) failure surfaces (Fig.1). In the numerical method, however, the soil is treated as a deformable material. Failure surfaces are not generally predetermined - by contact elements, for instance – but nor can they occur in a mesh of continuum elements. In order to achieve good estimates in spite of this, the deformation pattern is used to identify potential failure surfaces in the simplest cases. The concentration of high values of strain, shear strain or comparative strain may also need to be included. Zones exhibiting high strain values can reasonably be interpreted as potential failure surfaces (Fig.2). However, the sliding wedge and the stationary soil are not decoupled. In addition, the shear strength is not fully utilised everywhere at the same time – mirroring what happens in situ. In each individual case it has to be discussed how to approach the limit state. Well known procedures are the stepwise reduction of the shear strength, the application of increasing surcharge or the application of displacements. One might consider increase or decrease of the soil unit weight, the introduction of excess pore water pressure or else. All procedures suffer from the lack of experience to allow for a reliable safety assessment.

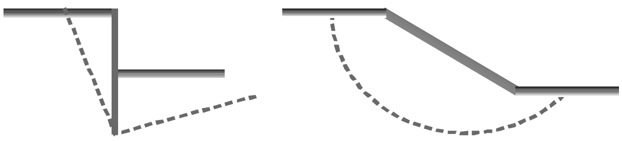

Figure 1. Preselected failure surfaces for classical ULS design.

In FE calculations, zones in which the shear strength of the soil has been achieved cannot resist any further loads. Additional stresses must be redistributed to other zones. The limit state is therefore frequently defined as a state in which no further redistribution of stresses can be calculated, in other words in which a numerical (!) limit state has been reached. However, a numerical limit state may occur whenever a numerical equilibrium is not maintained at one particular point while in fact a limit state would not yet actually have been reached in the soil. Therefore, it must not be assumed that a limit state will occur in the soil whenever a numerical limit state has been achieved.

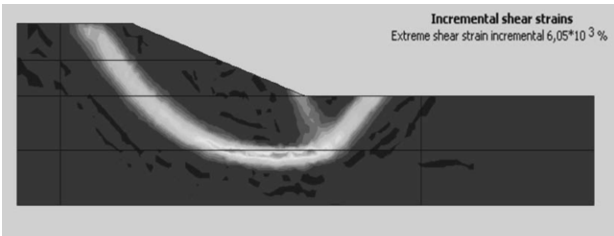

Figure 2. Failure surface indicated by band of high incremental shear strains.

6 VERIFICATION OF SLOPE FAILURE

The verification that a slope has sufficient stability is performed in classical earthworks design by determining the equilibrium of forces at one (or several) selected rigid sliding wedge(s). The geometry of the sliding wedge(s) must be progressively altered until the most unfavourable case has been found. However, slope failure calculations performed with the FEM cannot consider rigid bodies but are based on the interaction of deformable, interconnected elements instead. Experience to date has fortunately shown that the verification of slope failure can be performed very well with the FEM, as demonstrated by Schanz (2006) in several examples. Design Approach 3, i.e. safety factors are imposed on the shear strength parameters of the ground, is used both for classical design and for FEM analysis.

7 STABILITY OF ANCHORED SHEET PILE WALLS

When designing anchored sheet pile walls several verifications must be performed for the ultimate limit states (the serviceability limit state is not dealt with further here).

7.1 *Wall thickness and section modulus*

To enable the wall thickness or the section modulus to be selected, the partial safety factors must be applied to the action, i.e. the relevant design moment for example, after it has been broken down into its permanent and variable components (the partial factors being γ_G and γ_Q respectively). Splitting the action into the components referred to here is one disadvantage of the new standard as it makes the calculation procedure far more complex. Owing to the fact that the linear superposition commonly used in structural calculations is not possible in numerical non-linear calculations, more time and effort is required to perform the calculations than was formerly needed for global verifications. To apply the safety to the resistance, the section modulus must be reduced by the appropriate partial safety factor

In classical design procedures, any differences in the design moment are solely due to the selected redistributed earth pressure. It is largely up to the engineer performing the calculation to decide how, and to what extent, the earth pressure is redistributed so that the results will always exhibit a certain degree of spread.

If numerical methods are used, it is not necessary to assume the redistribution of the earth pressure as it is obtained in the calculation. Several other parameters are also taken into consideration "automatically", these being the stiffnesses of the soil, the wall and the support as well as the magnitude and distribution of the earth pressure which are a function of the stiffness.

7.2 *Earth support (horizontal)*

The required embedment depth is generally determined with the aid of the required passive earth pressure. For verifications using FEM the force of the earth support must be determined first and it is then checked that the ground is able to mobilise sufficient resistance in order to withstand that force. This force is obtained from the integral of the normal stresses on the wall over the embedment depth.

There are several different ways of determining the passive earth pressure in classical earthworks design. Consequently, the results may differ considerably, depending on whether the method according to Coulomb, Caquot-Kerisel or Gudehus or another method is used. However, DIN 1054 only specifies one partial safety factor to cover all of these methods.

A limiting value of the passive earth pressure can also be determined by means of a FE calculation, e.g. by moving the wall until a constant passive earth pressure is achieved, if increasing deformations, but no increase in the load, are obtained in the calculation. But sufficient experience with passive earth pressures obtained by numerical methods is not yet available.

7.3 *Vertical equilibrium*

In classical earthworks design it must be verified that the absolute value of the assumed passive wall friction angle is not too great (as the passive earth pressure increases with that value). Being a reaction force, it can never exceed the action. The requirement for equilibrium and compatible relative displacements is automatically satisfied in calculations performed with the FEM so that its application facilitates the design process in this case.

7.4 *Vertical loadbearing capacity*

The question of the appropriate limiting value arises in cases in which a base resistance has to withstand part of the vertical load. Vertical loading tests are not usually conducted on walls so that the designer has to rely on empirical values. Yet there are no clear-cut guidelines for such values which results in rather high partial safety factors being applied. Calculations of the base resistance with numerical methods have unfortunately not yet been sufficiently verified by measurements. The definition of the base resistance in a FE analysis is problematic particularly because the thickness of the wall is frequently taken to be zero in the calculation.

7.5 *Stability in the lower failure plane*

Classical verifications of the stability in the lower failure plane aimed at determining the required anchor length assume one or two rigid sliding wedges and the forces that occur (actions and resistances) are compared.

When calculating the limit state at the lower failure plane by means of finite elements the anchors, which are usually installed a certain distance apart from each other, and the contact between the anchors and the ground must be simulated realistically. In two-dimensional simulations the anchorage element forms a "slab" in the soil. The failure plane cannot intersect it and is always diverted to the end of the anchor even if contact elements are present. Calculations to determine the stability in the lower failure plane thus require a three-dimensional simulation (with the exception of anchorages in the form of a continuous anchor wall) and contact elements placed between the anchorage element and the ground.

Verifications in the lower failure plane performed by FE calculations appear to be problematic at present, not only as regards the way in which the limit state is achieved (specified wall displacement, additional external loads on the anchor, reduction in the shear parameters or increase in the weight density) but also as regards the interpretation of the results.

The reflections on this verification indicate the general problem of how the resistances could also be determined appropriately by means of FE calculations. Perau (2007) clearly demonstrates that different partial safety factors/safety levels are obtained depending on the cause of failure (reduction in the characteristic values of the ground or application of an addition action) and that the stiffness of the system as a whole has a significant part to play.

7.6 *Hydraulic heave*

The safety against hydraulic heave is determined by comparing the forces exercised by the weight of the soil block (at buoyancy) and those exercised by the upwards-directed seepage forces. The comparison of these forces simplifies a very complex process. The limit state occurs when high porewater pressures, which reduce the intergrain stresses, and seepage forces, which cause the grains to move, coincide. In spite of the high degree of simplification, this approach to determining safety, which was established by Terzaghi, continues to be used successfully today and a better approach has not yet been found. The awareness that the model behind this verification is not a particularly appropriate one has thus so far always resulted in relatively high partial safety factors being selected

In the finite element method the grain structure is modelled as a continuum although the processes taking place in the ground when hydraulic heave occurs are macroscopic ones in granular soil. Hydraulic heave cannot therefore be modelled with certainty by the FE method. One of the main advantages of numerical calculations is, however, that they enable the pore water pressure distribution to be taken into consideration to a greater extent so that sufficiently reliable initial values are obtained for the actual verification.

8 SUMMARY

The Eurocode EN 1997-1 permits three different design approaches, each of which mostly leads to quite different results. The design approach adopted in the German standard DIN 1054 is the one that best enables geotechnical engineers to benefit from the experience gathered with the method commonly used hitherto in Germany, in which global safety factors are applied. The procedure generally specified in the new edition of the standard (Design Approach 2*) is suitable for FE calculations; it involves first performing the entire calculation with characteristic values and only then considering which safety factors should be applied. Performing the calculation with design values would result in a distorted picture of the actual material performance and lead to unrealistic results.

All verifications are based on model assumptions. This fact is sometimes forgotten, being accustomed to calculation methods that are being established for a long time, but has to be considered when comparing design approaches. Numerical methods such as the FE method enable the boundary conditions to be taken into account to a far greater extent than in classical earthworks design. They also enable failure patterns to be forecast from deformations occurring close to the point of failure instead of having to assume them a priori for a particular verification. Experience has shown that actions and the effects of actions can be determined reliably by the FE method. The FE method certainly delivers more satisfactory results than other models, particularly in the case of difficult geometries and complex construction processes. In addition, a single FE calculation is able to provide not only the effects of the actions for ultimate limit state design (ULS) but also those needed for serviceability limit state design (SLS).

Another advantage of calculations performed with the FE method is that it is possible to check the results for each stage of construction by means of measurements as only characteristic values are used. The FE method has thus been shown to be the best possible tool for the observational method. There are only a few exceptions, such as hydraulic heave, where the FE method is not able to simulate deformation processes.

However, a great deal of uncertainty is associated with attempts to determine the resistances in the ground by means of the FE method. To enable the resistances to be computed it would have to be ensured that it is possible to calculate the deformations reliably up to a point just before failure occurs and that a false appearance of failure is not caused by numerical instabilities. In addition, a consensus of opinion on how to bring about failure would have to be reached. This can either be done by reducing the shear parameters incrementally or by applying an additional action, although different numerical limit states are then obtained.

Therefore, it can be recommended at the present time that the actions and effects of actions obtained in FE calculations should be included in verifications and the resistances computed in accordance with classical earthworks design. The partial safety factors used to compare the effects of actions and resistances should be the same as those specified in standards for classical earthworks design as more extensive experience is not yet available.

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