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IMPLICIT ULS DESIGN USING ADVANCED CONSTITUTIVE LAWS WITHIN THE FRAMEWORK OF EUROCODE 7

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Ultimate limit state design in terms of the Eurocodes requires the application of partial safety factors on actions and resistances, respectively. During the last few years the applicability and implementation of this concept in geotechnics has been widely discussed; Eurocode 7 is about to become a mandatory standard in Europe.

Nonlinear constitutive laws for retaining structures in general and tunnel linings in particular allow for an "implicit design"; i. e. the numerical model itself confines the stress state to values bearable by the material at each instant of time, in analogy to nonlinear soil models. In connection with such an implicit design, a strategy has to be developed in order to provide sufficient safety margins for both, retaining structure and soil, at all construction stages by proper implementation and choice of partial safety factors. In order to illustrate possible strategies two examples are examined:

In the first case, fire safety of a reinforced concrete cut-and-cover tunnel had to be investigated: Due to the development of the temperature with time, strength and elastic properties of concrete and steel decrease. Stress redistribution within the structure is essential to achieve an economic design.

In the second case, a viscoplastic constitutive law for shotcrete permits considering the rapid development of strength and stiffness of young shotcrete in combination with high ductility at young age and its interaction with the soil. Stress concentrations at the intersection would result in an enormous amount of reinforcement without consideration of the load-redistribution of both, shotcrete and soil, in the vicinity of the intersection which is calculated by a time-dependent 3-D-FE-simulation.

The applicability of each Design Approach of Eurocode 7 in connection with such an implicit design is discussed. The actions taken in order to provide safety margins comparable to conventional models (which are focussed on linear elastic material models for structures) and recommendations for the choice of Design Approach are given.

1 INTRODUCTION

Ultimate limit state (ULS) design in terms of the Eurocodes requires the application of partial safety factors on actions and resistances, respectively. During the last few years the applicability and implementation of this concept in geotechnics has been widely discussed; Eurocode 7 (EN 1997-1 [1]) is about becoming a mandatory standard in Europe. Consensus has been achieved about possible strategies of performing ultimate limit state design using numerical tools with non-linear constitutive models for soil and linear elastic behaviour of retaining structures [2].

Nonlinear constitutive laws for retaining structures in general and tunnel linings in particular allow for an “implicit design”; i. e. the numerical model itself confines the stress state to values bearable by the material at each instant of time, in analogy to nonlinear soil models.

In order to cope with a multitude of solution techniques for various kinds of geotechnical problems three so-called Design Approaches– differing in the method of applying partial safety factors on actions, material parameters and/or resistances – have been introduced in EN 1997-1. Performing an implicit design puts some restrictions on the choice of Design Approaches (and safety concepts in general) which will be discussed in this contribution.

Two examples have been chosen in order to illustrate possible choices of Design Approaches:

The first example deals with the fire safety of the reinforced concrete lining of a cut-and-cover tunnel near Innsbruck, Austria. The second example is a tunnel intersection of a metro station in Istanbul. In the first example, constitutive laws considering temperature dependent properties of concrete and reinforcing steel were employed, in the second example time dependent properties of shotcrete were considered in the material formulation. In both cases, performing an implicit design with help of these material laws resulted in an economic design of the structure.

2 ULTIMATE LIMIT STATE DESIGN - CONCEPT

2.1 Ultimate limit state design – leaving reality

For a long time, research efforts in numerical analyses of engineering structures have focussed on developing constitutive laws which match given experimental data, either on laboratory scale or on full scale, and which are able to predict the behaviour of structures under various loading conditions correctly.

Conventional analysis tools, on the other hand, were developed with a focus on design: It has always been more important to provide a solution on the safe side than matching all aspects of structural behaviour.

Purposefully reality is left behind. Safety concepts have been introduced and optimized with the goal of providing tools for the design of reliable and durable engineering structures. In terms of the Eurocodes: Ultimate limit state (ULS) analyses have to be performed.

Quality and ease of use of numerical methods like the finite element method (FEM) have developed to a point where they are competing with traditional approaches in everyday practice of the (ULS-) design of engineering structures.

As a result, additional applications of numerical methods have come into focus: Not only real structures with given material properties and given loading conditions have to be investigated. Instead, structures with nominal dimensions, made of fictitious materials and loaded by actions which are expected not to be exceeded with a certain probability, have to be analysed.

If an adequate statistical base is available, and if excellent computer hardware is at hand, failure probability can be calculated and checked against accepted risk levels [3]. In engineering practice, however, the opportunities to perform complete statistical analyses are scarce. Other design concepts have to be resorted to, and have been established in conventional design procedures.

2.2 Conventional design concepts

In conventional design concepts a sufficient safety margin against failure or inadmissible deformations of a structure frequently has been achieved by applying a single global safety factor on actions. In the course of time, values of the global safety factor have been optimized in order to match safety requirements with economic constraints. For different types of structural materials, different types and durations of actions, and different types of structures a data base of global safety factors has been developed, adapted to changing requirements and fixed in national standards.

In some cases, global safety factors on actions appeared as not being appropriate. Then other types of safety factors have been used: Slope stability and sliding resistance, for instance, have traditionally been checked by applying a safety factor on soil strength parameters.

2.3 Design concept of the Eurocodes and its connection with conventional safety concepts

The design concept of the Eurocodes for ultimate limit state design is based on a so-called semi-probabilistic approach (which has already a tradition in some European countries): "*When considering a state of rupture or excessive deformation of a structural element or section of the ground, it shall be verified that the design value of the effects of all the actions is smaller or equal to the design value of the corresponding resistance*" [1]. Actions, or effects of actions, are increased to "design values" by multiplying them by partial safety factors, γ_F or γ_E. Effects of actions in conventional models are e. g. cross-sectional forces and resulting forces or stresses at sections through a system. In terms of numerical models, stresses at integration points can be considered as effects of actions. (If the model connecting actions and effects of actions is non-linear one gets different results from analyses where partial safety factors are applied directly on actions (γ_F), or on effects of action (γ_E), respectively.)

Resistances are decreased to design values by dividing them by a different set of partial safety factors, γ_R. The values of these partial safety factors are either derived from a statistical basis, or from previous experience. Previous experience with this concept is only available in a few European countries. In most countries previous experience has been gathered by applying the safety concept using a single global safety factor on actions described above.

Making use of all the valuable experience with the conventional concept using global safety factors in order to define values for the new partial safety concept proved to be difficult. It was comparatively easy to agree on partial safety factors on actions: Analysis of the available data base resulted in the recommendation of the values 1.35 for permanent actions and 1.5 for variable actions.

Most of the values of partial safety factors on resistances were derived from conventional global safety factors bearing in mind predominant mixes of permanent and variable actions and considering the amount of variability of strength parameters of common structural materials.

2.4 Application of the semi-probabilistic safety concept of the Eurocodes in geotechnics

In geotechnical engineering, application of the semi-probabilistic safety concept of the Eurocodes is complicated by some issues:

- Soil can function as action – like the action on the back of a retaining wall – or as resistance – like the resistance of a foundation, and can be both at the same time, as in slope stability analysis or in tunnelling in soft soil.
- Traditionally, different analysis tools have been in use for different tasks. And, even with the conventional safety concept, different safety factors have been applied for different tasks. For some of the tasks, different approaches have evolved in different countries.
- Several special types of resistances have to be considered, most of them resulting from an interaction between soil and certain types of structure, like soil nails, anchors, piles or geotextiles. These resistances – like the pull-out resistance of an anchor or the skin friction of a pile – are dependent both on the quality and shape of the resisting structural material and on the properties of the soil.

In order to cope with the peculiarities of soil-structure-interaction, and in an effort to deal with different solution strategies in different countries, EN 1997-1 was developed. It allows choosing between three so-called Design Approaches.

They differ in the size of partial safety factors on actions (or effects of actions), on material properties (strength parameters) of the soil and on resistances (structural properties). The recommended values of the most important partial safety factors for retaining structures and overall stability analysis are summarized in Table 1 for all three Design Approaches.

<table>
<tr><th rowspan="2">Design Approach</th><th colspan="2">unfavourable permanent action</th><th colspan="2">unfavourable variable action</th><th>soil strength parameters c, tan φ</th><th>earth resistance bearing capacity</th><th>sliding resistance</th></tr>
<tr><th colspan="2">γ_F, γ_E</th><th colspan="2">γ_F, γ_E</th><th>γ_M</th><th>γ_R</th><th>γ_R</th></tr>
<tr><td>DA 1-1</td><td colspan="2">1.35</td><td colspan="2">1.5</td><td>1.0</td><td>1.0</td><td>1.0</td></tr>
<tr><td>DA 1-2</td><td colspan="2">1.0</td><td colspan="2">1.3</td><td>1.25</td><td>1.0</td><td>1.0</td></tr>
<tr><td>DA 2</td><td colspan="2">1.35</td><td colspan="2">1.5</td><td>1.0</td><td>1.4</td><td>1.1</td></tr>
<tr><td>DA3</td><td>1.35
1.0</td><td colspan="2">structural
geotechnical</td><td>1.5
1.3</td><td>1.25</td><td>1.0</td><td>1.0</td></tr>
</table>

Table 1: Partial safety factors (ULS) for retaining structures and overall stability analysis [1]

In Design Approach 2 (DA 2) characteristic values of soil strength parameters are used. Either the actions or the effects of actions have to be augmented by a partial safety factor. Additionally a partial safety factor on ground resistances like bearing capacity, sliding or earth resistance (which do not explicitly appear in a continuum mechanics approach) is used. Frequently a variant of DA 2 is used in conventional design where characteristic values of actions are used during the analysis, and partial safety factors on effects of actions are introduced immediately before checking effects of actions against design resistances of the structural members involved. Also soil resistances (earth resistance, bearing resistance and sliding resistance) are checked at this late state against soil stresses by multiplying them by partial safety factors on actions immediately before the check.

In Design Approach 3 (DA 3) the partial safety factor for permanent geotechnical actions is 1.0, whereas the soil strength parameters tan φ' and c', with φ' being the friction angle and c' being the cohesion, are reduced. The partial safety factor on ground resistances is 1.0. This Design Approach will be used for overall stability analysis in most European countries.

Design Approach 1 consists of two combinations: DA 1-1 is similar to DA 2, whereas DA 1-2 is similar to DA 3. The more unfavourable of the two combinations governs the design.

Design strength properties of structural materials and design resistances of structural elements shall be calculated in accordance with other Eurocodes (EN 1992 to EN 1996 and EN 1999) for all three Design Approaches. (EN 1992-1-1 for concrete structures [4] requires a partial safety factor of 1.5 for the compressive strength of concrete, and a partial safety factor of 1.15 for the yield strength of reinforcing steel.)

Whether the choice of Design Approach is left to the engineer, or whether a certain Design Approach is mandatory for a certain type of analysis, and the values of partial safety factors are specified in the National Annex of Eurocode 7, which is provided by the Standard Institutes of the European countries.

2.5 Implicit Design

Up-to-date constitutive laws in combination with nonlinear finite element methods and similar techniques permit an economic design of engineering structures: The constitutive law limits the stresses to a level which is bearable –according to the model – by the material in each (integration-) point of a structure. Yielding or rupture of the material causes additional strains, but not inadmissible stresses. If there is still load-carrying capacity left in the system, equilibrium can be achieved by stress-redistribution to regions of the system with a lower stress level. As soon as the whole potential for load-redistribution is exhausted, the structure fails. This is indicated by progressive increase of deformations and displacements.

Exploiting the whole load redistribution potential of a system is cumbersome to achieve with conventional approaches. This becomes especially obvious in the case of time or temperature dependent nonlinearities of the material: These nonlinearities may result in different stiffness and strength at each point of the structure which is difficult to handle with conventional analysis tools.

The constitutive laws employed should be based on standard material parameters, like uniaxial compressive strength, and behave in a way that scaling of the standard parameters by partial safety factors results in appropriate scaling of the (e. g. time or temperature dependent) strength envelope.

3 DESIGN APPROACHES AND NUMERICAL METHODS

The following discussion of the Design Approaches is focussed on retaining structures (according to the very general definition in EN 1997-1: "*The provisions of this Section shall apply to structures, which retain ground comprising soil, rock or backfill and water. Material is retained if it is kept at a slope steeper than it would eventually adopt if no structure were present. Retaining structures include all types of wall and support systems in which structural elements have forces imposed by the retained material*.") analysed by help of numerical methods using non-linear constitutive laws. (There are other categories of problems in geotechnics for which conventional approaches are established where – also as a result of previous experience and local traditions – recommendations for the choice of the appropriate Design Approach can be different [5].)

3.1 Geotechnical Aspects

- Safety factors on soil resistances, like earth resistance or sliding resistance, are based on the method of sections. They require either a clear understanding where soil functions as action and where it is resistance, or some automatism which allows to find the section through the structure which separates regions of action from regions of resistance (as it is done in overall stability analysis).
  It is frequently possible to perform numerical analyses, e. g. for a construction pit, and to check calculated design stresses at certain sections against design resistances afterwards. However, unless a specialized computer program is available, this check can be a tedious task.
- Augmenting actions does not automatically create a higher level of safety. Dead load of soil can be action as well as resistance. Increasing of dead weight for the whole soil region under investigation therefore does not consider this distinction and may yield misleading results, as in overall stability analysis. (If, as another example, Mohr-Coulomb-friction is assumed, and sliding resistance has to be checked, an increase of load does not effect safety against sliding.)
- Putting safety factors on soil strength parameters is analogous to the treatment of other types of resistances in the safety concept of the Eurocodes. In case that soil functions as an action, or if the distinction between the function of action or resistance is not known in advance, the actions of the soil are increased automatically. The resulting safety factor on soil as action in terms of a factor on stresses or forces is not a constant number, but depends on soil strength parameters (besides the chosen partial safety factor).

3.2 Implicit Design

- Design Approaches in which effects of actions are augmented are inappropriate in connection with constitutive laws for structural materials with built-in strength envelope: Most constitutive laws define stresses (and stiffness) as functions of strains. Multiplying stresses by partial safety factors γ_E before entering the constitutive law is impossible.
  As a remedy – instead of increasing stresses – strength parameters could be decreased by an appropriate factor. However, this is not provided for in the design concept of EN 1997-1.

4 EXAMPLE 1 – FIRE SAFETY OF A CUT-AND-COVER TUNNEL

4.1 Description

The first example deals with a 2-track railway tunnel which is part of section H7 of the Unterinntal railway line near Innsbruck, Austria. Fig. 1 shows a characteristic cross section of the reinforced concrete structure which runs parallel to an existing railway line. Fire safety of the new line is a major concern, aggravated by the bundling of important infrastructure. The resulting project specifications for the ULS-design in case of fire required a thorough numerical investigation: Utilizing stress redistribution capacities within the structure is essential to achieve an economic design. Soil-structure-interaction is not much of an issue: Earth pressure acts on the walls and the ceiling of the tunnel and can be considered as an action; the base slab is bedded in the soil which in this case is clearly a resistance.

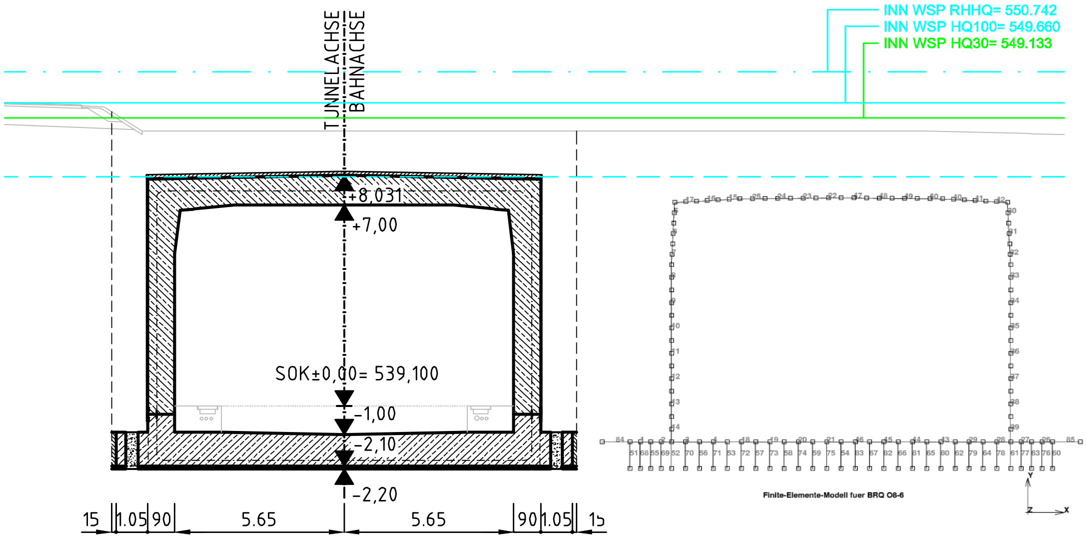

Figure 1: Analysed cross section and FEA-model of example 1

4.2 Constitutive Laws

During a tunnel fire temperatures rise from around room temperature to values of 1000 °C and above. In this temperature range the properties of both, concrete and steel, change considerably. Strength in compression and tension, stiffness and thermal elongation are all affected.

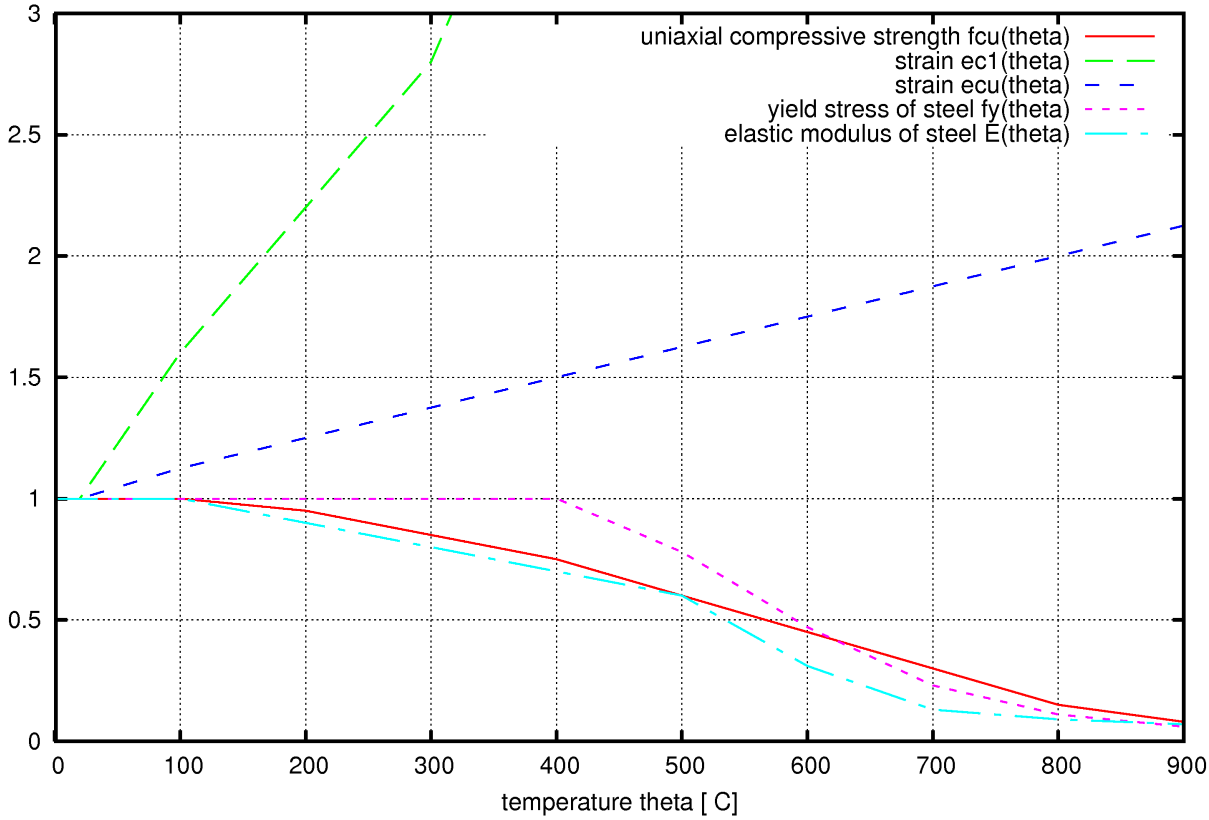

Figure 2: Temperature dependent material properties (ratios compared to cold state)

Fig. 2 shows some examples of temperature dependent properties according to EN 1992-1-2 [6]. In order to accomplish an implicit design, a model in accordance with this standard has to include a non-linear stress-strain-relation for concrete, including cracking in tension. Yielding of the reinforcing steel at high stress levels has to be part of the model as well. All temperature dependencies of stiffness and strength parameters according to EN 1992-1-2 should be part of a decent constitutive law in agreement with this standard.

The analyses for the current example have been performed using the nonlinear FE program MSC.Marc. Each member of the structure is computationally divided into a number of concrete layers and two steel layers. A combination of user-subroutines and tabulated data has been used to define temperature dependent material properties. A tensile strength of concrete of 5 % of the compressive strength and some tension softening have been assumed. In the constitutive laws applied, yield functions and strength envelopes are defined in terms of uniaxial yield stress of steel and uniaxial compressive strength of concrete. The whole strength envelope is scaled appropriately. Scaling of these envelopes inevitably effects (temperature dependent) stiffness – an independent definition of temperature-dependent stiffness, as suggested in the Eurocodes, is not possible.

During the development of a tunnel fire, temperatures increase first at the fire-exposed surface and propagate into the interior of the concrete structure with time. If excessive spalling is prevented, e. g. by adding poly-propylene fibres to the concrete mix, the temperature development with time is largely independent of changes of concrete properties. The resulting temperature distribution is a function of the temperature increase at the surface and the distance from the fire-exposed surface. Fig. 3 shows the temperature distribution – part of the project specifications – as function of time and of distance to the fire-exposed surface [7].

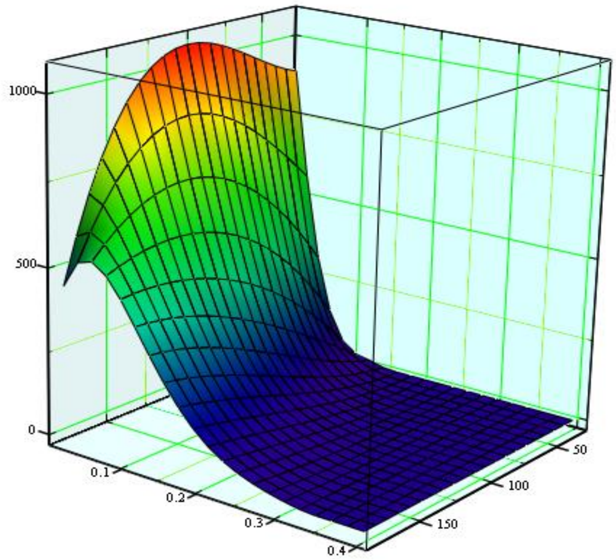

Figure 3: Temperature distribution as function of time and distance to fire-exposed surface [7]

4.3 Choice of Design Approach

In this example, the structure can be analysed without even thinking of EN 1997-1 and Design Approaches: Soil and water pressure acting on roof and walls of the structure can be treated as ordinary permanent external loads (the at-rest earth pressure has been used to calculate soil pressure on the walls). These loads, as well as other actions like dead weight, traffic loads or building loads, can be amplified by appropriate partial safety factors on actions, γ_F, according to EN 1992-1-1 and EN 1992-1-2. There is no partial safety factor on temperature loads – the chosen time-temperature-curves are worst-case scenarios anyway. Resistances of concrete and steel are decreased by dividing concrete compressive strength and yield stress of steel by partial safety factors according to EN 1992-1-1 and EN 1992-1-2, respectively.

Partial safety factors on soil resistance only effect bedding stiffness, and, in consequence, stress distribution of the base slab. Having in mind the inaccuracies of assumptions about bedding, effects of the size of partial safety factors on earth resistance can be neglected in this example.

It has to be pointed out that the time-dependent stress-strain-law for concrete applied here requires application of partial safety factors directly on actions, and not on effects of actions.

Applying these safety factors immediately before comparison with the design resistance of concrete and reinforcing steel, respectively, would not work.

Just as thought experiment, let us view the cut-and-cover tunnel as a geotechnical structure, and think in terms of Design Approaches as well. For this experiment, we choose a state at room temperature where the partial safety factors of table 1 are valid:

There are different types of actions:

- Structural actions, like dead weight and actions acting directly on the structure, like traffic loads.
- Geotechnical actions, possibly including contributions from traffic or structures on the surface
  - Vertical earth pressure
  - Horizontal earth pressure

The procedure described above matches DA 1-1 or DA 2, respectively: Characteristic values of soil strength parameters are used, actions are augmented by familiar partial safety factors, and soil resistances are of minor importance.

In DA 1-2 and in DA 3 partial safety factors on permanent geotechnical actions are 1.0. Instead, there is a safety factor γ_M on soil strength parameters. For the deposits of the Quaternary in this example, characteristic values for the friction angle of $\varphi' = 37°$ and cohesion $c' = 0$ were derived. Looking at the horizontal earth pressure, the horizontal load on the structure would be based in DA 1-1 or DA 2 on the at-rest earth pressure coefficient, $k_0 = 1 - \sin \varphi' = 0.398$, multiplied by $\gamma_F = 1,35$ yielding a factor of $k_{0d} = 0.537$. In case of DA 1-2 or DA 3 the corresponding factor is calculated with a design friction angle $\varphi'_d = \arctan(\tan \varphi' / \gamma_M) = 31.1°$ as $k_{0d} = 1 - \sin \varphi'_d = 0.484$. Thus, DA 3 would result in smaller loads acting on the walls.

Whereas in Combination 1 of DA1 dead weight and traffic loads acting directly on the structure would not be factorised, in DA3 these types of loads would be increased by a partial safety factor γ_F. A closer look at the vertical earth pressure in this example, with a thin soil layer above the structure (see Fig. 1), illustrates a shortcoming of DA3: Whereas a traffic or building load acting directly on the structure would be increased by the familiar partial safety factor for actions, there would be a much lower partial safety factor in case of a soil layer between traffic or building load and structure.

In this example it is obvious that treating vertical earth pressure as a geotechnical action and applying DA 3 is not appropriate. In case of a mined tunnel with low overburden, where soil is explicitly modelled as part of the "structure" applying DA 3 may be the best choice. In this case, the treatment of vertical loads deserves special consideration.

4.4 Results

This example is also part of a second contribution to this conference [8]. The results are shown there in some detail and need not be repeated here. Summarizing, it can be stated that utilization of the load redistribution capacity of the structure by help of implicit design resulted in a very economic design which could not be achieved by using conventional techniques.

5 EXAMPLE 2 – METRO STATION

5.1 Description

This example deals with a 3-D-model of a part of a metro station which is under construction in Istanbul. The station consists of two parallel station tunnels with top of rail about 34 m below street level, five connection tunnels (of two different types) and two escalator tunnels. All of them will be excavated by an NATM excavation technique in relatively weak rock (Trakya formation). A historical building above the station required a thorough investigation. Fig. 4 shows a geological cross section in the vicinity of the building. The model includes the region around this building and has been set up in a way to contain all types of connections.

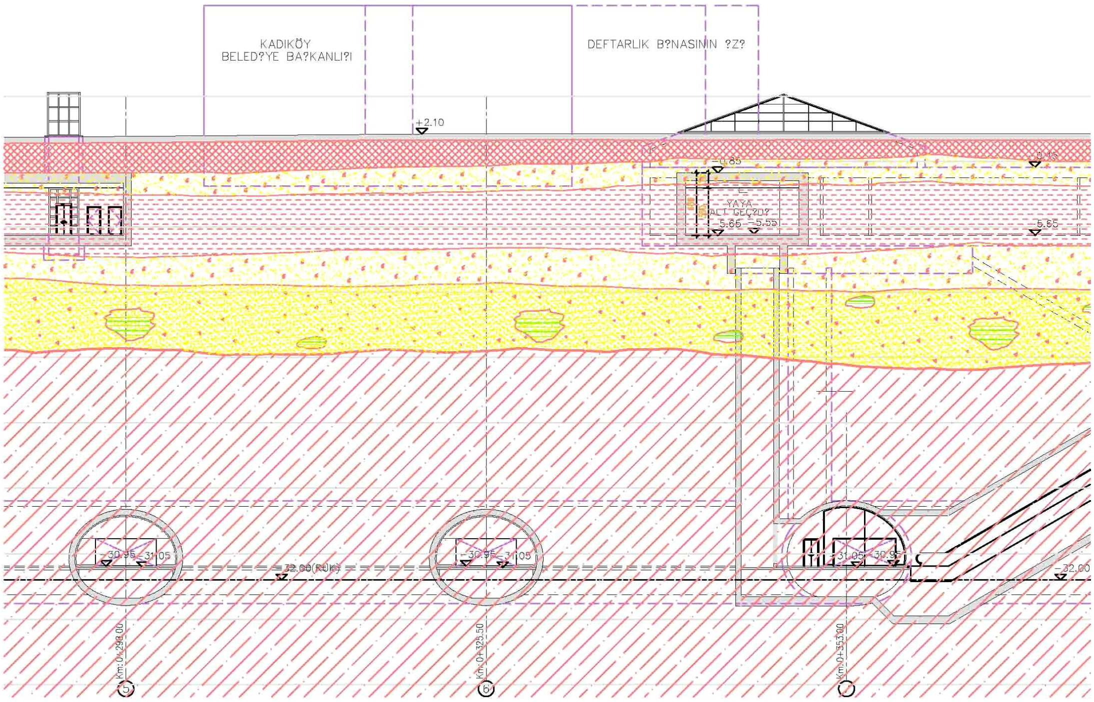

Figure 4 – Longitudinal geological cross section through the metro station

Primary support of the tunnels is a shotcrete lining with a thickness of 30 cm. The linings will be reinforced with two layers of wire mesh. Rock bolts and other support measures have been neglected in the analyses. Each tunnel is subdivided into top heading, bench and invert.

Stress concentrations in the lining at intersections would result in massive reinforcement and increased lining thickness without consideration of the load-redistribution of both, shotcrete and soil. As a result, extremely costly and time consuming provisions during the excavation process would be required. An economic design was accomplished by help of implicit design applying models for shotcrete and soil described below.

5.2 Constitutive Laws

5.2.1 Shotcrete with time dependent properties

Shotcrete, or sprayed concrete, is a material with a rapid increase of stiffness and strength at very young age. A few hours after application it is already able to carry considerable loads while its viscosity is still high enough to be able to suffer considerable straining without being destroyed. Besides the pronounced time dependent behaviour and its loading at early age, shotcrete behaves very similarly to ordinary concrete, with a non-linear stress-strain relation in compression, tensile cracking at low stress levels and a tendency to creep at high stress levels.

The shotcrete lining consists of layered shell elements. A viscoplastic constitutive model developed by Meschke and Mang [9], and extended by additional creep and shrinkage terms, has been applied. Two independent hardening and softening mechanisms control the constitutive behaviour of shotcrete subjected to compressive and tensile stresses, respectively. The increase of elastic stiffness during hydration of shotcrete as well as the time-dependent increase of compressive strength, tensile strength, and yield surface are all considered. The original Meschke-Mang-model describes creep effects with one single parameter, the viscosity. At stress levels within the current yield surface, no creep or relaxation occurs. Therefore, additional creep terms based on the rate of flow method and in agreement with experimental observation were added in the course of a research project [10].

This model captures most of the essential properties of shotcrete and guarantees – by proper choice of model parameters – time dependent properties in agreement with specifications of standards and guidelines [11], see Fig. 5. In this example, a shotcrete of type SpC 20/25/J2 has been anticipated.

In the progress of cyclic excavation, the shotcrete applied in each round has a different age. In the numerical model, the excavation sequence is simulated step by step in the time domain by removing soil elements and stress-free activation of shotcrete elements. Details of this procedure can be found in [12].

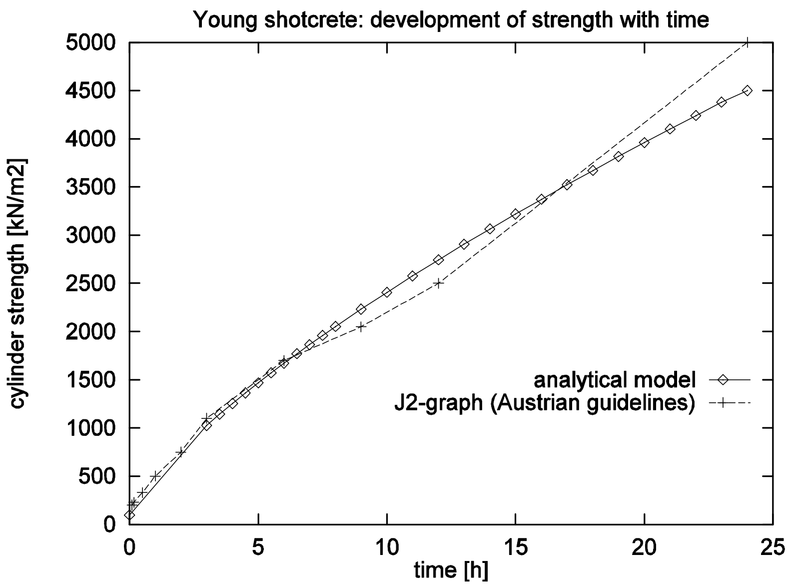

Figure 5. Development of the strength of young shotcrete with time.

5.2.2 Soil

Application of constitutive laws containing a Mohr-Coulomb yield surface in numerical analyses produces practically identical sliding surfaces and failure loads as conventional approaches for some classical geotechnical problems, like overall stability analysis. Using this model type in numerical analysis is therefore in good agreement with the requirements of EN 1997-1. For the analyses of this example, a simple linear elastic - ideal plastic material model with Mohr-Coulomb yield surface has been applied. (Weaknesses of simple models like this are not topic of this study.)

5.3 Choice of Design Approach

For mined tunnels, an a priori distinction of regions where soil functions as action from those where soil functions as resistance is not possible. DA1-2 and DA3 with $\gamma_M > 1.0$ are therefore a first choice. However, in case of tunnels with very low overburden the calculated soil pressure acting on the tunnel lining might not contain enough safety margin, see the thought experiment for example 1 above. An additional analysis applying partial safety factors on permanent actions should be performed as well.

Increasing earth pressure on the structure by increasing the specific weight of soil seems inappropriate: There is little scatter in the specific weight of the soil, and shear resistance is hardly influenced by higher specific weight. On the other hand, increasing effects of actions is impossible in connection with implicit design. As a remedy, resort has been taken to a variant of DA1-1 and DA2, respectively, where the analysis is done applying characteristic actions, specific weight and soil strength parameters, but strength of the structural materials is reduced twice: First by the partial safety factor for material resistances, and then by γ_E in order to introduce the partial safety factor on effects of actions indirectly. (Performing solely this second type of analysis is problematic as well: There is no safety margin, for instance, on face stability.)

Table 2 summarizes the chosen partial safety factors used in the two analyses performed for the ULS design of this example.

partial safety factor on	analysis 1 (Design Approach 3)	analysis 2 (variant of Design Approach 2)
permanent actions	1.0	1.0
soil strength parameters	1.25	1.0
shotcrete strength parameters	1.5	$1.5*1.35 \approx 2.0$
reinforcing steel yield stress	1.15	$1.15*1.35 \approx 1.55$

Table 2: Partial safety factors for ULS analyses of example 2

5.4 Results

Figures 6 and 7 show comparisons of equivalent plastic strains in the soil for the two analyses, whereas Figures 8 and 9 depict comparisons of normal stresses in the middle layer of the shotcrete shell in circumferential direction. All figures show analysis steps 83 to 85, with the start of the excavation of the top heading of a connection tunnel. In Figures 6 and 7 a vertical cut through the axis of the connection tunnel allows investigating the development of plastic strains around the excavation in progress. As had to be expected, decreased soil strength parameters result in increased deformations of the face, larger plastified areas and larger plastic strains. In Figures 8 and 9 the observer looks in direction of the new excavation, soil is invisible. Comparison of the stresses in the middle layer of the shotcrete shell shows that the automatic stress confinement works, but with some limitations: In analysis 2 the stresses level is lower than in analysis 1. This is due to the combination of larger deformations caused by weaker soil behaviour and higher admissible stresses in concrete. However, viscoplasticity allows – for short periods of time – stress levels outside the current strength envelope. This can be observed at the locations with maximum stress of the intersection – stresses are momentarily considerably higher than the specified uniaxial strength, but decrease in the course of time due to creep. More details about limitations of the constitutive laws applied and measures taken to guarantee a sufficient level of safety can be found in [13].

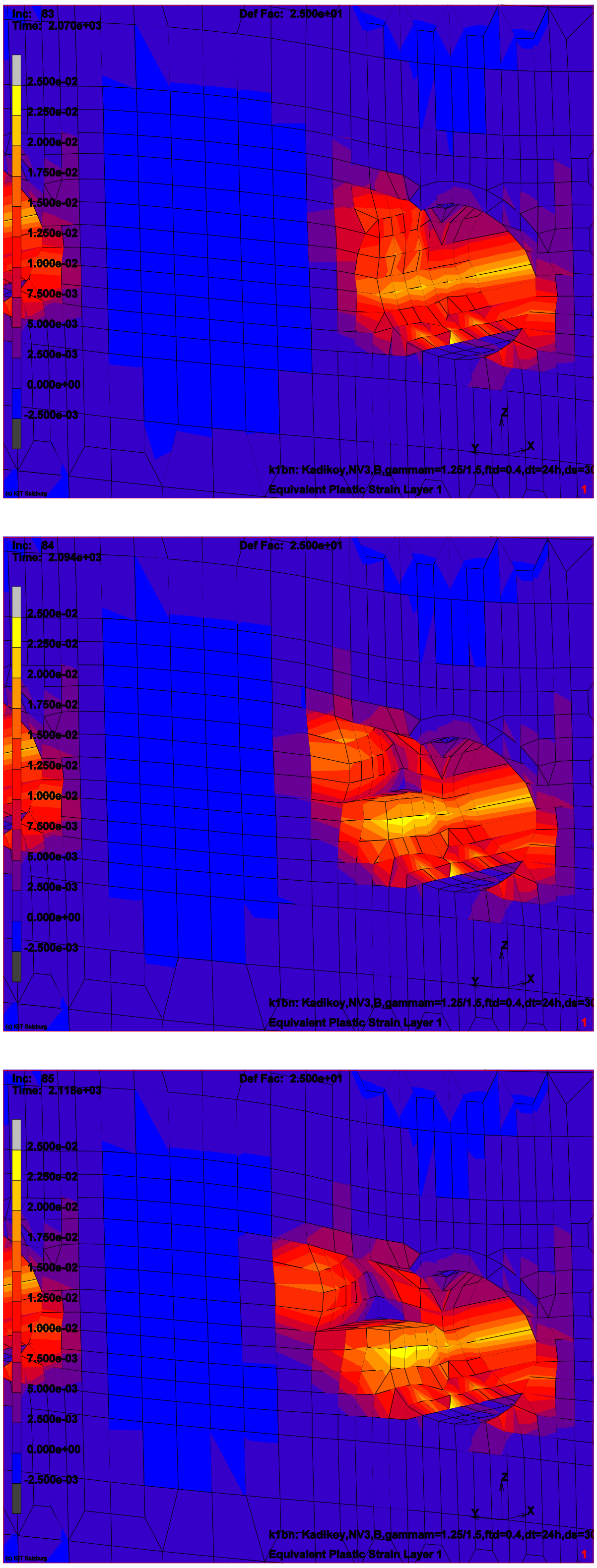

Figure 6: Equivalent plastic strains for analysis 1 (DA 3)

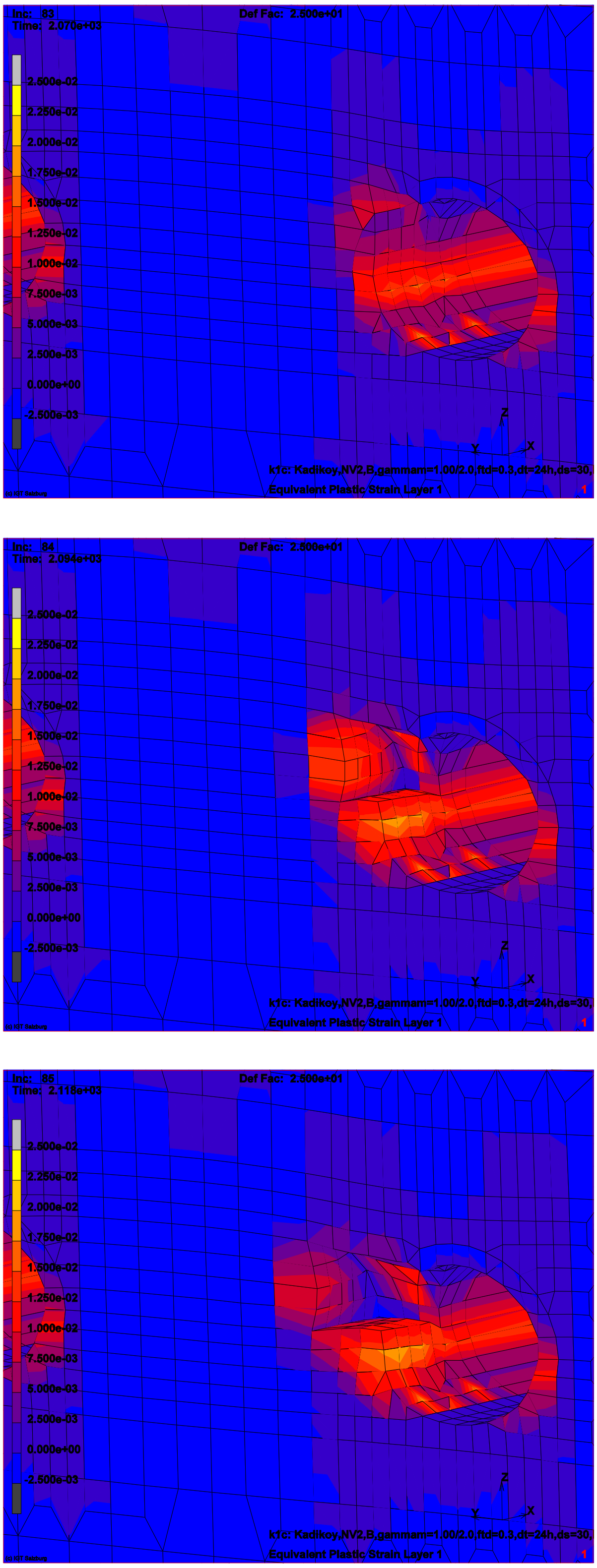

Figure 7: Equivalent plastic strains for analysis 2 (Variant of DA 2)

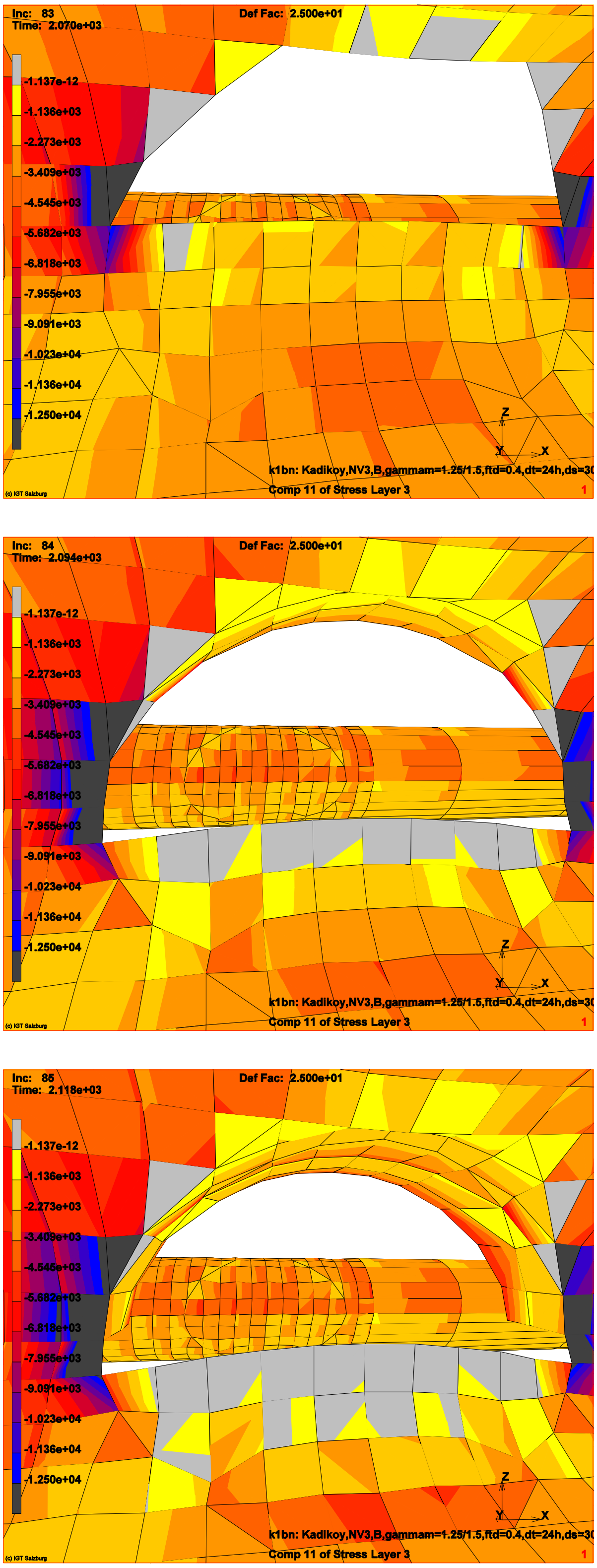

Figure 8: Normal stresses in circumferential direction for analysis 1 (DA 3)

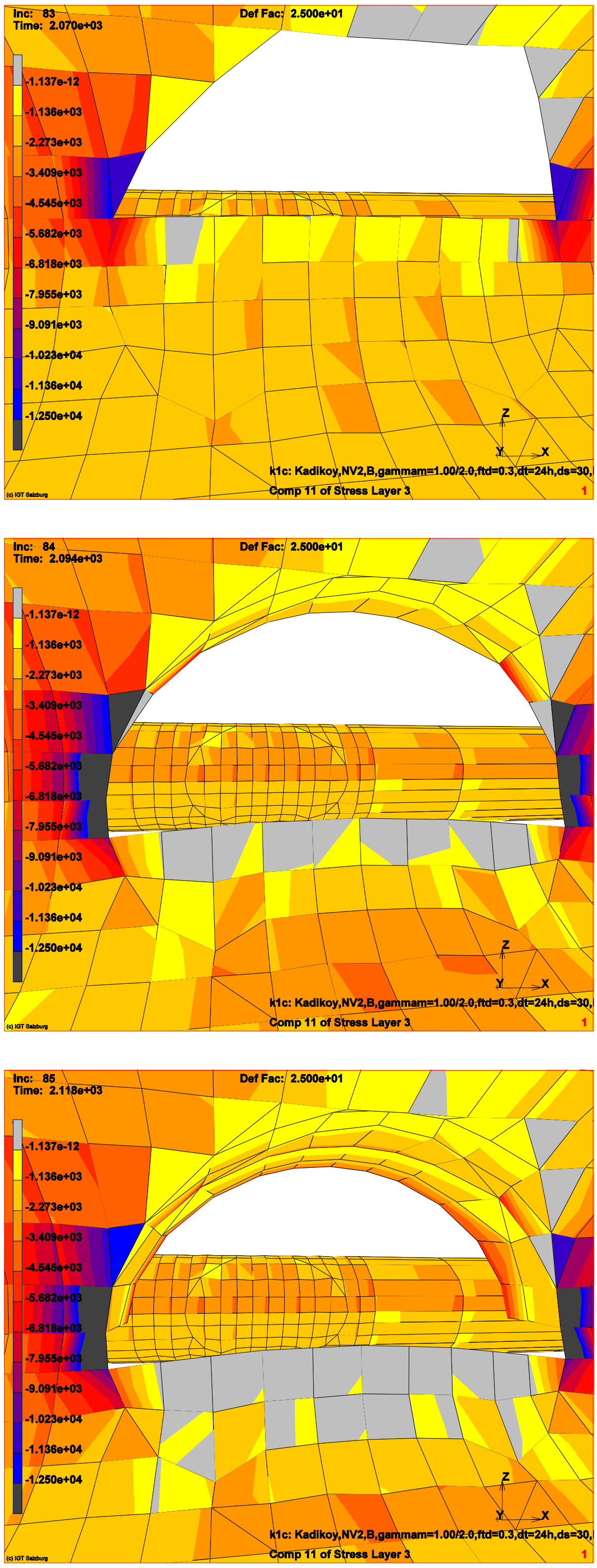

Figure 9: Normal stresses in circumferential direction for analysis 2 (Variant of DA 2)

6 CONCLUSIONS

The applicability of each of the Design Approaches of Eurocode 7 has been investigated with a focus on retaining structures (in a very general sense) where time dependent or temperature dependent nonlinear properties of structural materials are important. Numerical methods using advanced constitutive laws enable a so-called implicit design – requirements of standards like the Eurocodes are part of the material laws applied. Two examples were chosen which illustrate economic advantages of such an implicit design. Limitations of the Design Approaches of Eurocode 7 (EN 1997-1) (and of the familiar global safety concept) in connection with such an implicit design are as follows:

- Increase of effects of actions does not work in combination with constitutive laws where stress and stiffness are defined as function of strains.
- Design Approach 1 is suited as long as actions are applied directly. Whereas Combination 1 does not guarantee the automatic distinction between soil as action and as resistance but directly covers uncertainties of external actions, Combination 2 automatically separates the behaviour of soil as action or resistance, respectively.
- Application of Design Approach 2 is restricted in connection with implicit design. Augmenting effects of actions does not work if nonlinear constitutive laws are used for the structures involved. Safety factors on soil resistances require elaborate post-processing. In order to circumvent this restriction, variants of DA 2 and DA 1-1 are suggested in connection with implicit design with safety factors on structural resistances only. DA 2 and its variants do not contain safety margins for failure of soil, as for face stability of a mined tunnel.
- Design Approach 3 is favourable if regions where soil functions as action or as resistance cannot be determined beforehand. However, vertical soil pressure is not augmented in this approach which might yield lower stresses, e. g. in tunnel linings, than other Design Approaches. The relation between partial safety factors which are applied on actions directly, and partial safety factors on actions caused indirectly by reduction of soil strength parameters is non-linear and soil-strength-dependent. Thus, it is difficult to establish the same level of safety (probability of failure) as with conventional design techniques. Previous experience cannot be transferred to the new concept in a straight-forward way.

Structural elements with strong soil-structure interaction, like anchors, piles or geotextiles, have not been treated in this study.

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