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Safety approach for tunnel lining calculations in 3D-continuum models

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ABSTRACT: The state-of-the-art method for calculations on bored tunnel lining is considered to be a 3D-continuum model using Finite Element Software, in which both the soil and concrete lining and their interaction are modelled. According to Eurocode 0 it is preferred to perform a ULS verification using partial factors on individual loads and resistances as described in Eurocode 1, 2 and 7. Eurocode 7 provides 3 design approaches which may be applied. A common approach is to perform SLS calculations and apply an overall safety factor on the resulting forces in order to obtain ULS results. This is similar to Design Approach 2 in Eurocode 7 (EC7-DA2). However, a more economic design may be reached by applying partial factors on the loads and soil parameters rather than the calculation results. In this paper an approach is presented for ULS tunnel lining calculations in a finite element environment, following Design Approach 3 as described in EN-1997 and applying partial factors within the finite element model to achieve the desired safety level (EC7-DA3). Furthermore, a case study is presented, illustrating the difference between the results for EC7-DA2 and EC7-DA3.

1 INTRODUCTION

Traditionally, the prefab concrete lining for shield driven tunnels may be calculated using framework models in which the segmented lining is represented by beams. The support of the lining by the surrounding soil can be modelled as pressure only springs, who's stiffness is generally determined using formulae as presented by Schulze & Duddeck (1964). The loads acting on the tunnel lining are modelled explicitly. This method is commonly referred to as the "Duddeck method", and the Design Approaches described in the Eurocodes can be applied to perform ULS calculations.

However, the structure soil interaction and its effect on the loads cannot be modelled accurately with this approach. Continuum models in which both tunnel lining and soil can be modelled have become more widely used as these allow for a more accurate assessment of the soil-lining interaction. These models are usually made in Finite Element Software. As geotechnical actions and support are not modelled explicitly, applying partial factors is not as straightforward compared to framework models (Boxheimer et al 2008).

Since the introduction of Eurocode 7, tunnels are classified as a geotechnical construction (European Committee for Standardization 2004). Therefore, ULS verifications of the structure should be made using one of the design approaches as prescribed by EC7. This paper describes the application of the Design Approaches from Eurocode 7 for the design of a shield driven tunnel lining in a continuum model. Section 2 describes the available safety approaches from Eurocode 7 with their application of partial factors, and how they may be applied to a continuum model. Section 3 provides the proposed approach of ULS modelling for continuum models. Finally, section 4 provides a case study based on the design calculations for the RijnlandRoute bored tunnel and compares the results for the various design approaches.

2 DESIGN APPROACHES

Eurocode 7 specicfies three possible Design approaches for ULS calculations. The national annex to the Eurocode may specify which Design Approach is to be used. Otherwise, the designer may choose one based on preference or practicability. All three approaches are expected to result in a sufficiently safe design.

The design approaches prescribe one or more combinations of partial factors to be applied either on the loads and soil parameters directly, or on the forces resulting from the calculations without partial factors.

2.1 *Partial factors*

In the Eurocode, partial factors are divided into four categories:

- A1, for structural actions
- A2, for geotechnical actions
- M1/M2, for soil parameters
- R1/R2/R3/R4, for foundations and (slope) stability

For a shield driven tunnel lining, factors for structural and geotechnical actions and soil parameters are relevant. Depending on the chosen Design Approach, a selection of these may be applied. For all soil parameters an analysis should be performed whether decreasing the value is favourable or unfavourable. When a decrease proves favourable, a partial factor of 1,00 should be applied instead. The following tables present the partial factors for reliability class 3.

2.2 *Design Approaches*

The Design Approaches prescribe combinations of partial factors for which a “limit state of rupture or excessive deformation” should be verified. As factor set R1 to R4 are not relevant for shield driven tunnels, this leaves the following combinations for the design approaches:

Design Approach 1: Combination 1: A1“+” M1
Combination 2: A2 “+” M2

Table 1. Partial factors for loads in A1 and A2.

Action	Factor	
	A1	A2
Permanent unfavourable	1,50	1,00
Permanent favourable	0,90	1,00
Variable unfavourable	1,65	1,45
Variable favourable	0,00	0,00

Table 2. Partial factors for soil parameters.

Soil parameter	Symbol	Factor	
		M1	M2
Friction angle	$\gamma_{\varphi'}$	1,00	1,25
Effective cohesion	$\gamma_{c'}$	1,00	1,25
Volume weight	γ_{γ}	1,00	1,00
Stiffness*	γ_{e}	1,00	1,30

* partial factor on stiffness from Dutch National Annex

Design Approach 2: Combination 1: A1“+” M1

Design Approach 3: Combination 1: (A1 or A2) “+” M2

In Design Approach 1 (EC7-DA1), both Combination 1 and Combination 2 should be checked, in which the partial factors are applied on the actions and ground strength parameters. However, as the loads from for example the soil are not modelled explicitly in a continuum model, it is not possible to apply a partial factor 1,5 to the soil load. Therefore, EC7-DA1 cannot be applied properly when using a finite element model.

In EC7-DA2, partial factors may be applied to either actions or the effects of actions and to ground resistances. Again, it is not possible to apply the partial factors from set A1 onto the soil loads. Therefore, the only viable approach for EC7-DA2 is to apply partial factors on the effects of actions, and ground resistances. For structural linear calculations, it is possible to calculate each load case separately, and combine them afterwards with their corresponding partial factors. For nonlinear calculations however, this is not necessarily possible. Therefore, a more practical approach is to apply overall factors to the calculated forces.

For the main reinforcement of a segmented tunnel lining, normal forces are generally favourable while bending moment is unfavourable. Consequently, a factor 0,9 should be applied on normal force, while bending moment is factored by a value somewhere between 1,50 and 1,65 depending on the relative effect of the permanent and variable unfavourable loads. It is not straightforward to accurately assess this relation. A sensible choice is to apply a factor of 1,65.

EC7-DA3 finally, partial factors may again be applied to either actions or the effects of actions and to ground resistances. However, EC7-DA3 makes a distinction between structural actions (A1) and geotechnical actions (A2). Unfavourable geotechnical actions and structural actions are modelled explicitly, and the partial factors from set A1 or A2 can be applied within the model. All implicit actions, such as soil loads, are generally permanent geotechnical actions, which receive a partial factor of 1,00 from set A2. As a result, it is possible to include all partial factors within the model, rather than applying them to the calculation results.

Concluding this chapter, EC7-DA1 cannot be applied in a continuum model as partial factors on soil loads other than 1,00 cannot be modelled properly. EC7-DA2 can be used by applying partial factors to the effects of actions. In EC7-DA3 all partial factors can be included within the model, as it prescribes a partial factor of 1,00 on permanent geotechnical actions.

3 CASE: ULS DESIGN OF A TUNNEL

3.1 *Introduction*

This case presents a calculation of the Structural Limit State and Ground Limit State for the segmental lining of a shield driven tunnel. The conditions are based on the RijnlandRoute tunnel in the Netherlands. For sake of clarity, not all load combinations are presented. Also, only one tunnel tube is modelled, whereas in reality two tubes are constructed.

3.2 *Tunnel geometry*

The tunnel is built up with 2 m long rings, each divided into 7 roughly equal segments. The rings are constructed in a staggered configuration. As the keystone is similar in dimensions to the other segments, there will be no X-joints and the location of the keystone is not relevant for the calculations. Rings are connected using dowels, of which 28 are applied per ring. Guiding rods are applied in the longitudinal joints.

3.3 *Soil conditions*

The soil profile consists of three layers: a relatively strong Pleistocene sand layer below a more clayey sand layer, covered by soft clay. The tunnel axis is located at a depth of 26 meters below surface, and lies completely in the Pleistocene sand.

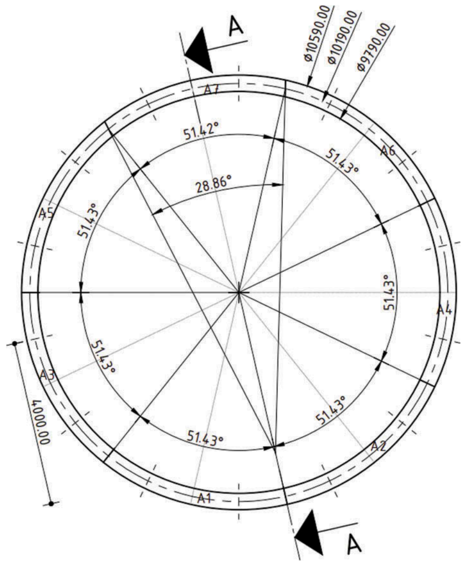

Figure 1. Ring geometry.

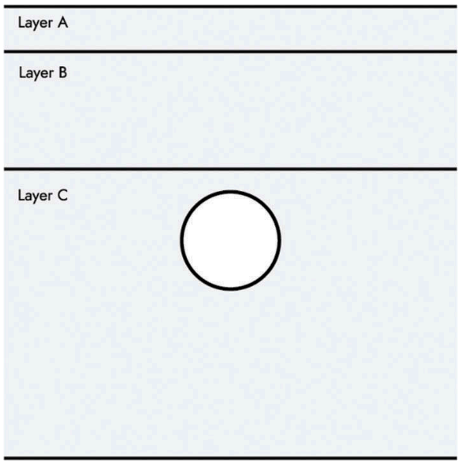

Figure 2. Soil stratigraphy.

Soil parameters were determined by a statistical analysis of the results of laboratory tests performed on actual samples from the field. The statistical analysis results in average, high and low characteristic values for every parameter. These are presented in table 3.

According to Eurocode 7, the most unfavourable combination of low and high values for parameters should be used for independent parameters. Laboratoy tests have shown that in this project, some soil parameters are co-dependent. For the Pleistocene sand, a higher volume weight is correlated with a higher stiffness and friction angle. Therefore it is allowed to use either high, medium or low characteristic values for all parameters, whichever is unfavourable. Doing this will result in a more economic design, however additional analyses are needed in order to determine which values need to be used, as this greatly depends on the range between the upper and lower boundary for the individual parameters. Whether the high average or low values should be used can change between projects or even soil types.

Table 3. Soil parameters.

	volume weight		strength params.		modulus of elasticity		
characteristic value	γ_{bulk} [kN/m^3]	γ_{sat} [kN/m^3]	φ' [°]	c' [kN/m^2]	$E_{50;ref}$ [MPa]	$E_{ur;ref}$ [MPa]	$E_{oed;ref}$ [MPa]
$\mathbf{X_{gem;k;low}}$	18,4	19,4	35,5	1,7	26	102	26
$\mathbf{X_{gem;k}}$	18,9	19,6	37,5	2,0	32	128	32
$\mathbf{X_{gem;k;high}}$	19,3	19,9	40,0	2,4	38	154	38

Table 4. Results sensitivity analysis soil parameters Layer C.

Parameter values	M_{min} [kNm]	corr. N [kN]	M_{max} [kNm]	corr. N [kN]
low	-135	-1637	150	-1998
average	-130	-1621	140	-1988
high	-122	-1597	129	-1975

Whether the set of low, average and high characteristic values are unfavourable is determined using simplified 2D finite element analyses, in which the tunnel is modelled as a continuous ring in a homogenous soil body. Table 4 shows the extreme bending moments and their accompanying normal forces.

For Layer C, a relatively low variation in volume weight is found. Bending moments are affected mostly by soil stiffness and horizontal soil coefficient (which is dependent on angle of internal friction). The set of low characteristic values is found to be unfavourable. For layers A and B, above the tunnel, only the unit weight affects the tunnel. The high characteristic value is used. The complete set of parameters is presented in table 5.

3.4 *Hydraulic head*

While groundwater may always be present, there can be fluctuations in the hydraulic head. As a result, part of the water pressure can be regarded as a permanent load, and part can be regarded as a variable load. Initial pore pressures follow a hydrostatic gradient. A higher water pressure increases normal forces in the tunnel, which generally is a favourable effect. Drained conditions are assumed. Construction of the tunnel and external loads do not cause excess pore pressures that may negatively affect the effective soil stresses.

When sufficient data from measurements are available, a statistical analysis can be performed to determine the extreme values for the hydraulic head with a chance of occurrence

Table 5. Soil parameters.

Parameter		Layer A	Layer B	Layer C	Unit
Soil model		Hard. Soil	Hard. Soil	Hard. Soil	-
Unit weight	$\gamma_{unsat}/\gamma_{sat}$	12,7/12,7	17,6/18,4	18,4/19,4	kN/m^3
Triaxial stiffness	$E_{50;char}$	1100	11000	26000	kN/m^2
Oedometer stiffness	$E_{oed;char}$	600	11000	26000	kN/m^2 kN/m^2
Unloading stiffness	$E_{ur;char}$	4800	45000	102000	kN/m^2
Reference stress	p_{ref}	100	100	100	kN/m^2
Power	M	0,90	0,60	0,55	-
Poisson ratio	v	0,17	0,20	0,17	-
Cohesion	c'	6,0	0	0	kN/m^2
Friction angle	φ'	22,6	34,0	35,5	°
Dilatancy angle	d_f	0	0	0	°
Initial stress ratio	K_0	0,62	0,44	0,42	-

that fits the required safety level. The lower bound value determines the permanent part of the load, while the variance is the variable load.

3.5 *Modelling*

The tunnel and soil continuum were modelled using the Finite Element software DIANA FEA. Two half widths of tunnel rings (1,0 m) have been modelled as plate elements. The connecting dowels are simulated by linear elastic zero thickness interface elements. The longitudinal joints are modelled with a zero thickness interface using the Janssen material model included in the DIANA software (Manie & Kikstra 2018). This material simulates the deformation in the longitudinal joints based on the formulae presented by Janssen (1983). The soil body is modelled as 3D volume elements using the Modified Mohr-Coulomb material model . This soil model uses Hardening Soil design parameters (Manie & Kikstra 2018). For the soil lining interaction, a zero thickness interface is modelled with Mohr-Coulomb properties. As the friction between lining and soil is uncertain, the friction angle is reduced to near zero. Figures 2 and 3 illustrate the Finite Element mesh.

A three dimensional model is required in order to correctly assess the effect of the staggered configuration of the lining rings. As the contact area in the longitudinal joint has a height of only 220 mm, compared to the 400 mm thickness of the segments, the longitudinal joints do not behave as stiff as the concrete segments. Due to the staggered configuration, forces will be transferred to adjacent rings depending on the locations of the joints.

While soil deformations may be approximated in a two dimensional model by reducing the stiffness of a continuous ring, the redistribution of load and the resulting forces can only be assessed properly by actually modelling multiple rings and joints.

3.6 *Construction sequence*

Due to the conical shape of the TBM shield, the surrounding soil will relax to a certain degree. Subsequent application of grout under high pressure in the tail void gap will then put additional stress on the soil. The combined effect of ‘constructing’ the tunnel is sometimes accounted for by the convergence confinement method which applies to 2D calculations (Eisenstein & Branco 1991).

In three dimensional models, a sequenced calculation may be made, in which tunnel rings are excavated and installed one by one. A face pressure, relaxation of soil, and grout pressure may be modelled explicitly. The effect of the sequenced construction, including soil relaxation and grout pressures is found to have a favourable effect when structural design of

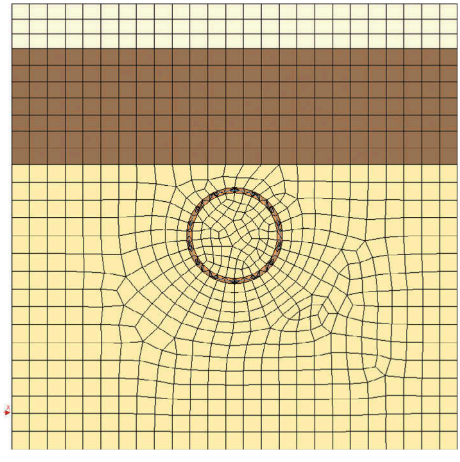

Figure 3. Finite Element Mesh.

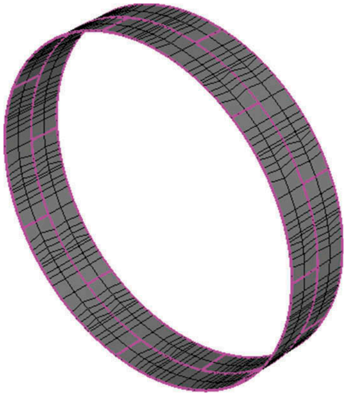

Figure 4. Tunnel Segments.

the lining is concerned. However, these effects are greatly dependent on the execution of the construction works, where especially grout pressures are difficult to control during execution. A certain amount of safety is incorporated in the model by disregarding these favourable but highly uncertain effects. Instead, the construction is modelled as if the excavation of soil and installation of the lining for the complete tunnel occurs instantaneously. The temporary situation in which a tunnel ring is floating inside the grout should be assessed in a separate calculation model.

3.7 *Model phasing*

The calculation starts by determining the initial soil stresses in the virgin soil. Then, the soil is excavated and the lining activated. After activation of the tunnel lining, additional loads are applied in separate phases. First, a load from the tunnel inlay is activated, representing the cable duct, backfill and asphalt structure. Then, a surface surcharge of 20 kN/m^2 is activated. Other loads, such as passage of the TBM backup train, temperature variations, and traffic inside the tunnel, are excluded from this example calculation.

The calculation scheme is derived from Brinkgreve & Post (2015). The initial stresses in the virgin soil are calculated in Phase 0. The tunnel tube is then excavated and activated at once in Phase 1. In phase 2, the inlay is modelled as a distributed load on the tunnel invert. A surface surcharge load of 20 kN/m^2 is then applied on the full extent of the model in phase 3.

The calculation scheme for the case study is shown below.

Phases 1 to 3 are SLS calculations for different load combinations, in which the characteristic values for soil parameters and loads are applied. Phases 4 to 6 are similar to phases 1 to 3 respectively, however partial safety factors are applied to the relevant parameters and loads in order to obtain ULS values. For EC7-DA3, all partial factors are incorporated in the model. For EC-DA2, partial factors on effects of actions are applied as an overall factor on the calculation results.

Table 6. Calculation scheme.

Phase	State	Start from phase
0. Initial phase	SLS	-
1. Activate tunnel tube 1	SLS	0
2. Activate tunnel inlay	SLS	1
3. Variable surcharge 20 kN/m^2	SLS	2
4. ULS of phase 1	ULS	0
5. ULS of phase 2	ULS	1
6. ULS of phase 3	ULS	2

Table 7. Calculation results SLS.

Phase	M_{max} [kNm/m]	N_{Mmax} [kN/m]	M_{min} [kNm/m]	N_{Mmin} [kN/m]
0. Initial phase	-	-	-	-
1. Activate tunnel tube 1	150	2012	-135	1637
2. Activate tunnel inlay	143	1979	-131	1654
3. Variable surcharge 20 kN/m^2	261	2454	-266	1822

Table 8. Calculation results ULS for EC7-DA3.

Phase	M_{max} [kNm/m]	N_{Mmax} [kN/m]	M_{min} [kNm/m]	N_{Mmin} [kN/m]
4. ULS of phase 1	183	2025	-163	1635
5. ULS of phase 2	168	1980	-152	1657
6. ULS of phase 3	392	2739	-379	1934

Table 9. Calculation results ULS for EC7-DA2.

Phase	M_{max} [kNm/m]	N_{Mmax} [kN/m]	M_{min} [kNm/m]	N_{Mmin} [kN/m]
4. ULS of phase 1	248	1811	-223	1473
5. ULS of phase 2	236	1781	-216	1489
6. ULS of phase 3	431	2209	-439	1640

3.8 *Results*

The SLS results for extreme bending moments and the normal forces corresponding to these bending moments are shown in table 7. Table 8 presents the ULS results for EC7-DA3. In table 8, the SLS results from the Finite Element calculations are multiplied by a partial factor of 1,65 for bending moment (unfavourable) and 0,9 for normal forces (favourable) to obtain ULS results for EC7-DA2.

Figure 5 shows the calculation results plotted against the capacity of a 1 m concrete tunnel section with a 400 mm thickness and 10–100 mm main reinforcement. For EC7-DA3, the results fall within the capacity of the section. The normative results for EC7-DA2 however, fall outside the section capacity. Additional reinforcement would be required when using EC7-DA2.

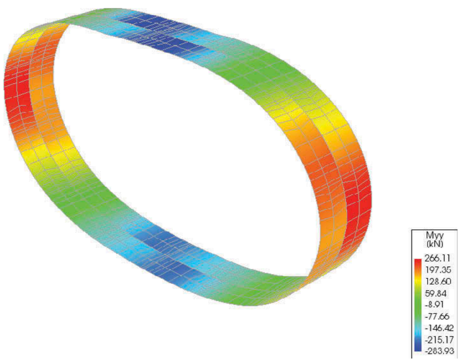

Figure 5. Distributed bending moments.

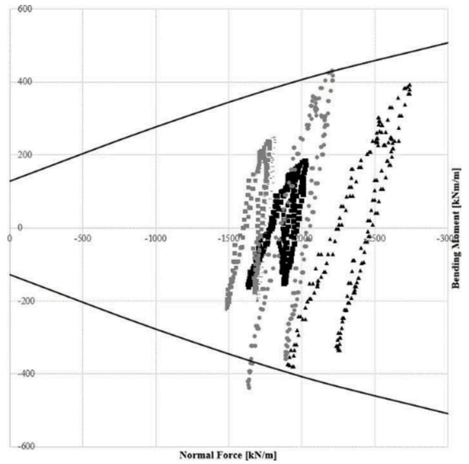

Figure 6. Bending moments versus capacity EC7-DA2 (grey), EC7-DA3 (black).

4 DISCUSSION OF THE RESULTS

From the SLS calculations we can make a number of observations. First is the effect of the tunnel inlay, which has a mild, but favourable effect on the normative forces in the tunnel. This is explained by how the soil load presses on the tunnel lining. As the vertical soil stresses are significantly greater than the horizontal soil stresses, the tunnel will initially deform into an flat horizontal oval shape. The bending moments at the top and invert act inwards, while the bending moments at the sides act outwards. Any action inside the tunnel on the tunnel invert then acts counter to the initial load from the soil. This can be extrapolated to a degree for all structural actions on the tunnel invert, as long as these are significantly smaller than the soil pressure on the tunnel. For very shallow tunnels, the deformation of the tunnel may actually resemble more of a standing oval due to uplift. Actions on the tunnel invert are then expected to be unfavourable. As a whole, the effect of the inlay is relatively small.

Phase 3 shows the effect of a surface load after construction of the tunnel. Bending moments more than double, while normal forces increase as well, but to a much smaller extent. This shows how surface loads can significantly influence the reinforcement design of the tunnel, and why it is of paramount importance not to underestimate these future loads during design, as well as not to exceed the loads that are used for the design by future surface works. Conversely, being very conservative in the estimation of surcharge loads can lead to an uneconomic design.

From the results of the EC7-DA3 calculations it can be observed that between the SLS and ULS calculations of phase 1, only a 20% increase is found in the bending moments, while normal forces remain practically unchanged. This increase can be almost entirely attributed to the partial factor of 1,3 on the stiffness of the soil, which is added by the Dutch National Annex. The partial factor on permanent geotechnical actions is 1,00, there are no substantial structural actions, and a partial factor of the internal friction angle would actually be favourable and is therefore set to 1,00. Without this partial factor on stiffness, no significant differences would be found.

Phase 2 shows the effect of structural actions on the tunnel invert. In comparison to the results for phase 1, the effect is limited.

Between the SLS and ULS calculation of phase 3, a 40% increase is found in the bending moments compared to a 10% increase in normal forces. This is explained by the partial factor of 1,35 on the surcharge load.

From the results it is clear that the ULS bending moments directly obtained from the Finite Element model by EC7-DA3 are more favourable compared to the EC7-DA2 results. This is due to the distinction made by EC7-DA3, where the partial factor for permanent geotechnical actions is 1,00, while in EC7-DA2 it is 1,50. Also, the normal forces are reduced with a partial factor 0,9 in EC7-DA2, where in EC7-DA3 they are similar between SLS and ULS, or increase when the surface load is also increased by its partial factor.

Figure 5 confirms our findings, in that more reinforcement would be needed when EC7-DA2 is used compared to EC7-DA3.

In general we can state that EC7-DA3 provides a more economic design than EC7-DA2. We have also observed that most of the safety for the ULS calculations comes from the partial factor on stiffness parameters, which is specific to the Dutch national annex only. Without it, only a small difference may be found between SLS and ULS calculations, and one may argue whether this provides enough safety in the design.

5 CONCLUSIONS AND REMARKS

This paper presents a comparison between the different Design Approaches for geotechnical structures as described in EN-1997-1 within a finite element environment, with regards to the structural design of the lining for a shield driven tunnel. A case study is performed to illustrate the integration of the Design Approaches in the finite element environment, and show the difference in the results.

Both Design Approach 2 and 3 may be applied to perform Ultimate Limit State (ULS) calculations in finite element models. As all actions that are transferred through the soil are modelled implicitly, partial factors for EC7-DA2 should be applied on the effects of actions. In EC7-DA3, partial factors for permanent geotechnical actions are 1,00, which allows all partial factors to be modelled in the finite element model.

Ultimately, EC7-DA3 can lead to a more economic design, at the expense of additional time and analyses during the design phase.

When applying EC7-DA3 as described, the engineer should be confident that he or she provides a sufficient level of safety, regardless of what the Eurocodes allow for. This may be achieved partly by incorporating a margin of safety in the construction model, for which we propose to disregard favourable effects of the tunnelling process which are difficult to predict and quantify, partly by incorporating a partial factor on the soil stiffness as prescribed in the Dutch national annex, and partly by taking a somewhat conservative attitude towards the estimation of future surface loads.

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