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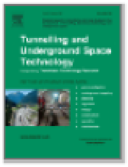

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Abstract

Although Eurocode 7 is not meant to be applied to the design of tunnels, the lack of standard codes explicitly focusing on tunneling leads to the situation that it is increasingly used in practice, at least for shallow tunnels in soil. However, there is no general agreement on which of the approaches defined in EC7 is the most suitable for the design of shallow tunnels, in particular when it comes to applying them in combination with numerical analyses. In this paper, the results of numerical calculations aimed at assessing ultimate limit state conditions of a shallow tunnel in soil are presented. The focus of the present work is firstly on the verification of the applicability of the Eurocodes in combination with numerical analyses and the comparison of different design approaches. Secondly, the consequences of the choice of a nonlinear material model for the shotcrete primary lining are discussed.

Keywords: tunnel design, EC7, numerical methods, shotcrete.

1. Introduction

Currently, there is no accepted standard code which defines and regulates the design of underground openings. Therefore geotechnical engineers have to refer to other codes, defined for conventional geotechnical structures (Eurocodes, BS, DIN, AASHTO, etc.) and engineering judgement still plays an important role.

Concerning tunneling design, two different problems can be generally distinguished: the stability of the tunnel face and the stability of the lined excavation.

The stability of the tunnel face is a typical geotechnical problem, usually tackled by means of limit equilibrium methods (LEM) applied to predefined failure mechanisms (Horn, 1961). On the contrary, the stability of the supported excavation is primarily a problem of soil-structure interaction. This aspect implies that strength and stiffness of both soil and structural support affect the stress-strain distribution and, consequently, the safety of the tunnel.

Moreover, in order to carry out this kind of analysis without neglecting the interaction effects between soil

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and support, numerical codes are required (i.e., Finite Element or Finite Difference Methods).
To guarantee a safe design, engineers often address to Eurocode standards. Eurocode 7 (CEN 2004 - EN 1997), devoted to geotechnical design, is based on the general framework of Limit State Design (LSD). Limit State Design consists in the verification that no limit state is exceeded under relevant design situations. Regarding tunnels, leaving aside serviceability limit states, two possible ultimate limit states should be checked:

- exceeding limit strength or excess deformation in the soil (GEO);
- exceeding limit strength or excess deformation in the structural elements (STR).

The GEO-type limit state is appropriate for the verification of the face stability. On the contrary both GEO and STR-type limit states are relevant for the support design.
When the verification of the different limit states is performed through calculations, according with Eurocode 7, it is possible to use three different design approaches, synthesis of different design methods traditionally used in European countries.
In this paper, the results obtained from the application of different design approaches and modelling strategies for the primary lining design of a typical shallow tunnel in soil are compared and the limits of each of them are pointed out. The interaction with Eurocode 2, as well as the possibility to take into account the non-linear and time-dependent behavior of shotcrete, is also considered.
Deep tunnels and rock tunnels are explicitly excluded at this stage because other design criteria are more relevant and the role of the observational method, in common practice, plays a much more pronounced role than in shallow tunneling (Schubert, 2010).

2. Overview of Eurocode 7

The current version of Eurocode 7 (EN 1997-1) provides the possibility to use three different design approaches (Table 1).

Table 1 - Summary of Eurocodes design approaches.

Design approach	**DA1**		**DA2**	**DA3**
Combination	1	2		
Partial factors applied to	Actions	Variable actions and Material properties (x)	Actions and resistance	Structural actions and material properties
Partial factor sets	A1+M1+R1	A2+M2+R1/R4	A1+M1+R2	A1/A2(*)+M2+R3

Sets A1-A2= factors on actions/effects of actions
Sets M1-M2=factors on material properties
Sets R1-R3=factors on resistances
(x) for axially loaded piles and anchors sets M1 (unless specific situations) and R4
(*) A1 for structural load; A2 for loads derived from the ground

Design approach 1 requires the verification of two different combinations: in DA1-1 partial factors are applied to unfavorable actions whereas in DA1-2 they are applied mainly to ground strength parameters. Design approach 2 requires the application of partial factors on actions (or effects of actions) and ground resistances and design approach 3 requires partial factors on structural actions and ground strength parameters. The possibility to introduce partial factors on the sum of the effect of actions rather than on the action itself is usually referred to as DA2* approach.
In the current version of Eurocode, partial coefficients for the actions (γ_F) and for the effects of actions (γ_E) are the same. This implies the assumption of a linear relationship between action and effect of action, which is not always the case. In order to tackle this problem, according to a general principle of Eurocode 1990 – Basis of structural design (CEN 2002 - EN1990), the following procedure may be adopted: whenever the effect of

an action increases faster than the action, the partial factor must be applied to the action; vice versa, if the effect of an action increases less than the action, the partial factor must be applied to the effect of the action. It is not always straightforward, however, to determine which of these two cases applies for a particular design situation.

Partial factors should be applied to the so-called characteristic values. Regarding material parameters, whereas in structural codes they are defined as a fractile of tests results, characteristic geotechnical values are defined as a cautious estimate of the value affecting the occurrence of a specific limit state. Therefore, the designer expertise and understanding of the ground play a major role (Simpson & Powrie, 2001).

From a general point of view the three approaches can be grouped into two main families (Bauduin et al. 2005): Load and Resistance Factoring Approach (LRFA, also called Load Resistance Factor Design – LRFD according to Shuppener et al. 2009) and Material Factoring Approach (MFA). LRFA, represented by DA2, requires the application of partial factors to both actions and resistances. Instead, in the MFA, represented by DA3, partial factors are mainly applied to ground strength parameters. DA1, which includes two combinations, can be considered as LRFA for piles and anchors. However, for every other geotechnical work, DA1.1, appears to be a particular case of LRFA, in which values of partial factors on resistances are equal to 1, whereas DA1.2 is a MFA.

When defining the partial factors to be applied to the actions, it is important to keep in mind the so-called single-source principle (EC7-1, Clause 2.4.2), which eliminates the need to distinguish between favorable and unfavorable permanent actions. In fact, it states that "*Unfavourable (or destabilising) and favourable (or stabilising) permanent actions may in some situation be considered as coming from a single source. If they are considered so, a single partial factor may be applied to the sum of these actions (or to the sum of their effects)"*. In the case of underground excavations, as well as for retaining structures, actions and resistances derive from the same source, namely the soil.

Unlike for foundations and retaining walls, there is no guidance provided for tunnels regarding resistance factors. Moreover, the soil surrounding the tunnel can be action or resistance and it would be extremely difficult to apply resistance factors in numerical computations. If partial factors on resistances are taken equal to 1, DA1-1 and DA2 are identical. DA3 and DA1-2 can also be considered identical when no structural actions (i.e., actions deriving from another structure applied directly to the retaining structure) are involved in the problem.

3. Current application of EC7 to tunnel design

Even if the geotechnical design of tunnel excavations is not explicitly covered by Eurocode 7, the general procedure can still be applied and used as a reference. In general, two issues have to be addressed, namely tunnel face stability and the stability of the lined excavation. Concerning the first problem, most of the considerations regarding slope stability can still be held valid. In this case, being the ground the main element providing resistance (GEO-type limit state), an approach requiring partial factors on material strength is recommended, although not explicitly stated in EN 1997-1.

The general tendency of geotechnical designers all over Europe is to prefer DA3 for this class of problems. With regard to the design of the supported excavation, the approach DA2* is the most frequently used in Germany and Austria, even if the latter (in contrary to Germany) also allows the use of DA3, when numerical methods are employed. In Italy, DA1 is prescribed for flexible retaining walls but tunnels are not explicitly addressed; the current tendency of the practitioners for the design of the supported excavation is to use DA2*. The recent revision of the National Italian Code (not yet issued) requires the use of DA1. The recommendations of the French Tunnelling and Underground Space Association (AFTES) suggest, consistently with the French National Annex, the application of DA2 approach, in the form of DA2* (Colombet et al. 2007). Nevertheless, the National Annex leaves the possibility to use also DA3.

It should be noted that, if geotechnical failure governs the design as a decisive ultimate limit state, an approach prescribing partial factors on forces may not be the most appropriate one to guarantee a sufficient margin of safety. Instead, considering the major role played by soil properties, it would be more logical to use safety factors on soil strength parameters. In other cases, the opposite situation can take place: factoring the strength of the ground leads to a less safe design when compared to the approach of factoring the effects of actions. This is particularly true when the rock or soil mass is competent enough to guarantee the formation of a good arching effect and, therefore, reducing soil resistance leads to a negligible increase of structural forces. For this reason, it could be suggested to check both the combinations (DA2* and DA3) in order to avoid, on the one hand, the underestimation of the structural forces and detect, on the other, the possible failure mechanisms in the soil mass.
Another design strategy, which would allow applying partial factors on actions instead of the effect of actions, is to increase the soil unit weight. However, in the opinion of the authors, it is not suitable because it would have the double effect of increasing both actions and resistances. In fact, the effective stresses would raise and consequently the shear resistance would be higher. When using advanced constitutive laws with stress-dependent stiffness, the soil stiffness would increase too. Moreover, this design strategy is not in accordance with Eurocodes in their current form.

4. Application of EC7 approaches to numerical calculation of underground excavations

Focusing on the stability of the supported excavation, the application of Eurocode approaches should consider that:

- it is a soil-structure interaction problem;
- it is usually analyzed by means of numerical methods;
- shotcrete, largely used in NATM excavations, is a material characterized by a strongly non-linear and time-dependent behavior.

Referring to the soil-structure interaction, it is largely debated all over Europe which is the best approach for this kind of problems. Both in Italy and in France a material factoring approach (MFA) in combination with numerical codes raises some concerns about the unrealistic distribution of stresses and strains in the soil, due to the formation of plasticity zones much wider than the ones which can be expected in situ. Authors do not agree with this consideration, against which two objections can be expressed. First of all the goal of ULS checks, as defined in the Eurocodes, is not to reproduce a realistic stress-strain distribution in the soil mass, but to make sure that the soil-structure system is far enough from failure or collapse. Secondly, the application of partial factors only to actions or effects of actions does not allow the check of ground failure.
It is true that reducing soil parameters might lead to a different and less realistic distribution of structural forces as well as to a deviation from the real mass behaviour, but a material factoring approach would ensure a higher margin of safety against geotechnical failure, with respect to a load factoring approach. In fact, reducing material strength parameters seems to be a good strategy for checking how far the current problem is from geotechnical failure (e.g., punching of the lining footing into the ground). The current situation should be described through characteristic values.
The hostility of some practitioners towards partial factors on material strength derives from the fact that the material parameters commonly used for tunnel design are already quite conservative. This means that, in normal practice, engineers do not use “best estimates” of material parameters (Simpson & Powrie, 2001). Therefore, a further reduction seems not appropriate because it would lead to uneconomic design. In any case, reducing soil parameters, as highlighted before, should not be judged as a too conservative way to consider soil strength properties. In fact, other uncertainties are implicitly included in these partial factors.
Being soil-structure interaction problems, such as tunneling, governed to a large extent by stiffness and relative stiffness, it could be considered to introduce partial factors on soil stiffness, which in turn would have an influence on calculated internal forces of the lining. However, it has to be considered that this strategy can

have different effects depending on the particular soil model used in the calculation and therefore this approach would be difficult to control. Moreover, checking the ultimate limit states (normally associated with failure conditions) by just acting on parameters that control the soil deformation behavior does not seem to be a consistent approach. In addition, it would not be in line with Eurocode7 in its present status.

For a general overview about the applicability of Eurocodes to a numerically based design, one can refer to (Bauduin et al. 2000, Schweiger 2005, Schweiger 2009, Schweiger 2014, Simpson 2000, Simpson 2007, Simpson and Junaideen 2013, Potts and Zdravkovic 2012). Even if some studies have been already carried out to evaluate and compare different design approaches (Schweiger et al. 2010, Schweiger 2010, Jones 2007, Walter 2007, Walter 2010, Hofmann et al. 2010), there is no general agreement, especially when it comes to apply these methods in combination with numerical analysis. It is clear that, when dealing with numerical calculations, neither actions nor resistances can be clearly distinguished and factorized. The soil surrounding the underground opening represents, in fact, a source of action and resistance at the same time. For this reason DA2*, which overcomes this issue by applying partial factors on effect of actions (namely structural forces on the tunnel support) and allowing for a single check, has gained popularity among practitioners.

An important step towards the investigation of the applicability of Eurocodes to tunnel excavation is represented by the workshop organized by the Austrian Society for Geomechanics, whose proceedings are included in Geomechanics and Tunnelling 3.1, 2010.

Another important contribution is provided by the French tunneling association (Colombet et al., 2007), whose recommendations deal with the compatibility of both EC7 and EC2 to tunnel construction and the applicability of EC2 to shotcrete and reinforced-shotcrete linings.

The main outcomes deriving from the previous works, regarding the application of Eurocode approaches in combination with numerical methods, show, as expected, that different results can be obtained from different combinations of partial safety factors depending on the soil constitutive model adopted and on the specific benchmark considered (soil properties and excavation sequence). However, differences in results due to the implementation of different design approaches seem to be smaller when advanced constitutive models are used (Schweiger 2010). Therefore advanced soil models seem to have not only the merit to give more realistic SLS predictions but also advantages for ULS calculations.

As third aspect, it should be also considered that traditionally excavated (NATM) shallow tunnels are usually supported by shotcrete. This construction material is characterized by a strongly non-linear and time-dependent behavior. However, how to deal with non-linear materials, in terms of the design approach using numerical analysis, is not clearly defined anywhere. Therefore this aspect is also addressed in this paper by applying a non-linear constitutive model for shotcrete and comparing the results with those obtained by using an elastic material. This aspect is of particular interest, since assuming the shotcrete linear elastic could lead to very uneconomic design. On the contrary, introducing non-linearity can lead to a reduction of structural forces due to the stress redistribution capacity of the support. Some calculations and preliminary results deriving from the application of Eurocodes design approaches in combination with advanced constitutive laws for the tunnel lining can be found in Walter 2007 and 2010. It seems clear that the advantage of using such models is to exploit as much as possible the stress redistribution both in the soil and in the structural support, avoiding an excessive increase in lining forces, which could lead to an unreasonably conservative design.

5. Calculation example

This contribution originates from an initiative supported by the Austrian Society for Geomechanics (OeGG), aimed at investigating the compatibility between Eurocodes approaches and numerical analyses applied to the primary lining of shallow tunnels. The current version of the Austrian guideline RVS 09.01.42 (Tunnel structures in soft soil under built-up areas) specify that for standard cases, like 2-D finite element analyses of tunnel excavations, carried out through non-linear constitutive models for the soil and linear for the support,

DA2* approach should be applied. If the tunnel support is modelled with a non-linear constitutive model, these guidelines are not directly applicable. Therefore, the goal of the calculation example presented here is not only to compare different design approaches but also to evaluate different possibilities for their application in combination with non-linear models for the tunnel support. This example also provided the opportunity to assess the impact of these models on the design, in terms of safety and cost-effectiveness.
The benchmark problem here considered is a shallow NATM tunnel characterized by an equivalent diameter of 10 m and overburden of about one diameter. Geometry and material parameters have been taken based on an actual project but have been slightly modified for the purpose of this study. The model, shown in Figure 1 and the prescribed excavation sequence, schematically represented in Figure 2, have been analyzed through the finite element code Plaxis 2D (Brinkgreve, 2014). 15-noded triangular elements were used.
The excavation area is divided in right side drift (excavated first) and left side drift. Each side drift has been excavated in two phases: top heading and bench. In order to simulate the three-dimensional effects of the face advancement, a pre-relaxation factor of 0.4 has been adopted for each section. The surface load of 15 kN/m^2, applied after the completion of the excavation, has been considered as a permanent load.
The analyses aim at assessing the applicability of different design approaches and the impact of different modelling strategies given a certain set of characteristic parameters. The constitutive model assigned to each soil layer and the corresponding soil characteristic parameters are provided in Table 2.

Figure 1 - FEM model of the calculation example.

Table 2 - Characteristic soil parameters.

		I	II	III	IV
		Backfill	**Loess**	**Sandy gravel**	**Clayey silt**
Parameter		**MC***	**HSS#**	**HSS**	**HSS**
γ_k	kN/m³	19.00	19.00	21.00	20.00
$K_{0,NC}$	-	-	0.5	0.5	0.5
φ	°	20.00	25.00	35.00	23.00
c	kN/m²	0,00	50.00	0.00	50.00
E	kN/m²	5,000	-	-	-
ν	-	0.2	-	-	-
$E_{50,ref}$	kN/m²	-	25,000	100,000	50,000
σ_{ref}	kN/m²	-	100	100	350
E_{ur}	kN/m²	-	75,000	250,000	150,000
ν_{ur}	-	-	0.2	0.2	0.2
m	-	-	0.6	0.5	0.6
$E_{0,ref}$	kN/m²	-	200,000	800,000	450,000
G_{0ref}	kN/m²	-	83,333	333,333	187,500
$\gamma_{0,7}$	-	-	0.0002	0.0002	0.0002
$E_{oed,ref}$	kN/m²	-	25,000	100,000	50,000

*Mohr-Coulomb Model
#Hardening Soil Model with Small Strain Stiffness

Figure 2 - Schematic representation of the excavation sequence.

The shotcrete primary lining has a thickness of 35 cm at the tunnel circumference and of 25 cm at the central support. In the FE analysis, it has been modelled using volume elements. The calculation of the structural

forces derives from the stress integration along the different cross sections of the lining, automatically performed by a special tool implemented in the software.
With the use of continuum elements for modelling the shotcrete lining, it has been possible to assign both a linear elastic constitutive model and an advanced constitutive model taking into account the non-linear behavior of the shotcrete, which plays a very important role in tunnel constructions since, unlike normal concrete structures, the shotcrete is loaded at very early age. Moreover, this model does not consider only the time-dependency of stiffness and strength but also creep and shrinkage effects as well as plastic deformations before and after reaching the maximum strength. More details about the model formulation can be found in Schaedlich & Schweiger (2014) and Schaedlich et al. (2014). Since the model is able to limit the stress state to the maximum value bearable by the material, it allows, by including all partial factors in the analyses, to carry out an implicit design.
The material parameters adopted for the non-linear shotcrete model are given in Table 3. In the following analyses, only the time-dependency of stiffness and strength and the pre-peak plastic deformation behavior of the material have been taken into account. Therefore no creep and shrinkage effects, as well as no softening behavior, have been considered. The increase of Young's Modulus follows the recommendation of CEB-FIP model code (1990) whereas, for the strength evolution, the J2 range defined in EN 14487-1 (2006) has been chosen (Figure 3).

Figure 3 - J curves, EN 14487-1 (2006) on the left and Young's Modulus time-dependency according to CEB-FIP model code (1990) on the right (after Schaedlich & Schweiger, 2014).

The model, as implemented in Plaxis, takes automatically the mean values of the ranges defined in the standard. The steel reinforcements are considered in the model as an equivalent tensile strength of the shotcrete material. In the calculations where the shotcrete has been modelled as linear elastic, a stepwise increase of the Young's Modulus has been adopted: 5 GPa and 15 GPa for young and old shotcrete respectively.
No interface has been modelled between soil and structure. Therefore they are tied together, and no relative displacements can occur. This approach is not be a conservative when the goal is to estimate soil displacements. However, when the structural support have to be designed, it is a cautious assumption. Moreover, it is not easy to define the properties of the interface. Even though assuming soil and structure fully tight might lead to an overestimation of the structural forces, the comparison between design approaches or between different modelling strategies is still valid.
A detailed list of the calculation phases performed can be found in Table 4. The calculations have been carried out according to both DA2* approach (which is identical to DA1-1 when resistance factors are neglected) and DA3 approach (which corresponds to DA1-2).
All the combinations taken into account in the present example are listed in Table 5.

Table 3 - Material parameters adopted for the non linear shotcrete model.

Parameter	Description	value	
$\mathbf{E_{28}}$	Young's modulus of cured shotcrete at t_{hydr}	kN/m²	2.50E+07
$\boldsymbol{\nu}$	Poisson's ratio		0.2
$\mathbf{f_{c,28}}$	Uniaxial compressive strength of cured shotcrete at t_{hydr}	kN/m²	2.00E+04
$\mathbf{f_{t,28}}$	Uniaxial tensile strength of cured shotcrete at t_{hydr}	kN/m²	2,000
$\boldsymbol{\psi}$	Dilatancy angle	°	0
$\mathbf{E_1/E_{28}}$	Time dependency of elastic stiffness	-	0.648
$\mathbf{f_{c,1}/f_{c,28}}$	Time dependency of strength	-	-2
$\mathbf{f_{c0,n}}$	Normalized initially mobilised strength	-	0.15
$\boldsymbol{\varepsilon_{cp}^{p}}$ **at 1h**	Uniaxial plastic failure strain at 1h	-	-0,03
$\boldsymbol{\varepsilon_{cp}^{p}}$ **at 8h**	Uniaxial plastic failure strain at 8h	-	-2.00E-03
$\boldsymbol{\varepsilon_{cp}^{p}}$ **at 24h**	Uniaxial plastic failure strain at 24h	-	-1.00E-03
$\boldsymbol{\varphi_{max}}$	Maximum friction angle	°	35
$\mathbf{t_{hydr}}$	Time for full hydration	day	28

Table 4 - Calculation phases performed.

PHASE	description	Δ days (time necessary to develop each phase)	Total number of days (Σ Δ days of each phase)
0	Initial stress	0	0
1	pre relaxation (area 1)	1.5	1.5
2	total relaxation (area 1) and shotcrete activation	1.5	3
3	pre relaxation (area 2)	1.5	4.5
4	total relaxation (area 2) and shotcrete activation	1.5	6
5	pause	7	13
6	pre relaxation (area 3)	1.5	14.5
7	total relaxation (area 3) and shotcrete activation	1.5	16
8	pre relaxation (area 4)	1.5	17.5
9	total relaxation (area 4) and shotcrete activation	1.5	19
10	Removal of the central support	1	20
11	Activation of the surface load	0	20

The verification of the tunnel support is usually carried out by means of the interaction diagrams between normal force and bending moment (M-N check). In order to be fulfilled, this check requires the structural forces to fall inside the diagram.

When DA3 has been adopted in combination with a shotcrete lining modelled as non-linear material, the analysis has been carried out both using the characteristic shotcrete strength (this requires an M-N check at the end) and reducing it by the 1.5 factor, prescribed in EN 1992-1-1. The advanced shotcrete model implemented in Plaxis offers the possibility to directly introduce, in the model settings, partial safety factors on both tension and compression strength. These factors automatically scale the strength envelope during its evolution with time. The reduction of shotcrete strength, performed directly during the analysis, as previously described, instead of performing the final M-N check, could be considered an alternative way to apply the DA3 approach in combination with non-linear models for the structural support.

Table 5 – Design calculations performed.

Combination		Actions		Soil parameters		Structural resistances	
		Partial factor on effect of actions	**Partial factor on surface load[(+)]**	**Partial factor on cohesion**	**Partial factor on friction angle (tanϕ')**	**Shotcrete model**	**Partial factor on shotcrete resistance[(#)]**
1	**DA2***	1.35	-	-	-	**Linear elastic**	-
2	**DA2***	1.35	-	-	-	**Non-linear**	-
3	**DA3**	-	-	1.25	1.25	**Linear elastic**	-
4	**DA3**	-	-	1.25	1.25	**Non-linear**	-
5	**DA3**	-	-	1.25	1.25	**Non-linear**	1.5

[(+)]considered as permanent geotechnical action

[(#)]directly given as input in the calculation

6. Results and discussion

In order to compare the results obtained from the different calculations, not only the final stage of construction but also an intermediate stage, namely the end of the excavation of the right drift (phase 4), is considered. In fact, it represents a critical stage, showing particularly severe stress conditions, especially at the upper junction between the circumferential lining and the central support. In the figures below, the different calculations are identified by the corresponding numbers of Table 5. The shotcrete modelled as linear elastic material is referred to as LES, whereas the new shotcrete model is referred to as NSM. The structural forces depicted in the graphs are design forces, which means that they include the factor on the effect of action (namely 1.35 for DA2*). In the legend, a further distinction is made between calculations with characteristic or design shotcrete strength (respectively 20 MPa and 13.3 MPa at 28 days of curing).
From Figure 4, showing bending moments obtained from phase 4, it can be seen that modelling the lining as a stepwise elastic material leads to much higher bending moments when compared to the non-linear shotcrete model. In the case of elastic shotcrete, DA3 results are more conservative. This means that, in this specific phase, the plasticity of the soil induced by a reduction of soil strength properties plays a major role, leading to higher bending moments when compared to DA2* approach, where the factor 1.35 is applied to the calculated forces. The opposite situation holds for the case of non-linear shotcrete, where DA2* results in higher bending moments. The differences between design approaches are less pronounced when the lining is modelled with the advanced constitutive model. Concerning normal forces (Figure 5), differences in calculated values are not significant independently from the shotcrete model and the use of characteristic or design strength parameters for the soil. Therefore DA2* results in higher normal forces due to the application of the partial factor 1.35 on structural forces.

Figure 4 - Bending moments from phase 4, combinations (outer lining).

Figure 5 - Normal forces from phase 4 (outer lining).

In the final phase, the differences in the calculated forces due to the use of a specific lining model or to soil characteristic parameters instead of design parameters are less remarkable. Therefore, also in this case, the main difference in the curves plotted in Figure 6 and in Figure 7, stems from the 1.35 factor applied on the computed values of the lining forces.

Figure 6 - Bending moments from the final phase (outer lining).

Figure 7 - Normal forces from the final phase (outer lining).

The maximum and minimum values of the lining forces obtained from each combination for both the intermediate and the final phase are listed in Table 6.

Table 6 - Maximum and minimum values of the lining forces from phase 4 and from the final phase (outer lining).

Combination	**Intermediate phase (n°4)**				**Final phase**			
	$\mathbf{M_{max}}$	$\mathbf{M_{min}}$	$\mathbf{N_{max}}$	$\mathbf{N_{min}}$	$\mathbf{M_{max}}$	$\mathbf{M_{min}}$	$\mathbf{N_{max}}$	$\mathbf{N_{min}}$
1	26	-168	-386	-669	136	-72	-484	-1278
2	40	-115	-402	-690	151	-72	-544	-1304
3	20	-214	-273	-562	89	-81	-415	-1040
4	34	-66	-267	-468	98	-66	-430	-1008
5	30	-78	-272	-494	92	-59	-419	-1015

For the central support, whose critical conditions are also reached at the end of the right drift excavation (i.e. phase 4), similar results to those emerged for the outer lining have been obtained (Figure 8, Figure 9). This is reasonable considering that the central support is rigidly connected to the circular lining. The maximum and minimum values of the lining forces deriving from each combination are listed in Table 7.

Figure 8 - Bending moments from phase 4 (central support).

Figure 9 - Normal forces from phase 4 (central support).

Table 7 - Maximum and minimum values of the lining forces from phase 4 (central support)

Combination	**Intermediate phase**			
	$\mathbf{M_{max}}$	$\mathbf{M_{min}}$	$\mathbf{N_{max}}$	$\mathbf{N_{min}}$
1	16	-165	-356	-717
2	36	-105	-374	-687
3	15	-214	-276	-619
4	32	-68	-287	-558
5	34	-57	-277	-544

For all the analyzed combinations, the M-N interaction diagrams have been plotted to evaluate the structural margin of safety. The different calculations, whose results are shown in Figure 10, Figure 11 and Figure 12, are identified with the corresponding number indicated in Table 5. The reference sections are the most critical ones and coincide with the upper corner for phase 4 and with the lowest section of the lining for the final phase. The black curve represents the characteristic domain, whereas the grey one is the design domain, reduced by the partial safety factor on material strength, namely 1.5. For the intermediate phase (i.e. phase 4), a different domain has been considered because, after the excavation of the right drift, the shotcrete curing process is not completed and its resistance is around 90% of the final one.
The combination DA3 with partial factors applied on shotcrete resistance (combination 5) was also included in the graphs, even though it does not require an M-N check, being the design criteria implicitly satisfied. All the other combinations are supposed to fall in the grey domain in order to fulfill the structural ULS requirements.
As previously underlined, phase 4 represents one of the most critical conditions the tunnel support is subjected to. From Figure 10 and Figure 11, it can be noticed that both calculations with purely elastic shotcrete are outside the design domain in both the outer lining and the central support. In the latter case, also

combination 2, including a non-linear structural support, does not fulfill the structural requirements. This means that the initially designed and analyzed section has not enough strength to sustain the calculated forces increased by 1.35 and therefore the structural design should be reviewed.

Figure 10 - M-N check for the outer lining (phase 4).

Figure 11 - M-N check for the central support (phase 4).

Considering the final phase, the first two combinations (i.e. all those with characteristic soil parameters and partial factors applied on the computed lining forces) fall slightly outside the design domain, as it can be seen from Figure 12.

Figure 12 - M-N check for the outer lining (final phase).

It seems clear, from these results, that no general rule can be established regarding which approach governs the structural design. In general, when adopting the advanced shotcrete model, the differences between DA2* and DA3 in terms of distance with respect to the M-N domain seem to be smaller.
In some cases, the adoption of an elastic model for the structural support with stepwise increase of stiffness can lead to an extremely conservative design.
Considering the 1.35 factor to be applied to the calculated internal forces, DA2* approach in combination with non-linear shotcrete would theoretically allow for a lower exploitation of the redistribution capacity of the tunnel support, with respect to DA3. However, this issue is attenuated due to the application of the 1.35 factor to both bending moments and normal forces that, in most cases, increase the bending capacity of the section.

7. Conclusions

In the present paper, after an overview on the applicability of Eurocodes to a numerically-based design for underground excavations, the results of a calculation example regarding an NATM shallow tunnel are discussed.
It has been shown that a nonlinear shotcrete model can be applied in combination with design approach DA2*, the approach intuitively favored by many geotechnical engineers in the common practice. On the other hand, at least to the opinion of the authors, DA3 approach in combination with a partial factor on the shotcrete strength directly applied in the numerical calculation would also be in accordance with the material factoring approach as defined in EC7 for soils. Moreover, it would allow performing a completely implicit design, meaning that no further M-N check is required.
The benefits, in terms of cost-effectiveness, of a design carried out by using a constitutive model that accounts for the non-linear and time dependent behavior of shotcrete can be considerable, especially when high stress concentrations cause an unrealistic increase of structural forces. Furthermore, provided that it is common practice to use non-linear material models for soils in numerical calculations, considering non-linearity and plasticity also in the structural support seems to be consistent. This is particularly true when dealing with highly non-linear and time dependent lining materials such as shotcrete, for which stress redistributions capabilities have to be considered in order to avoid uneconomic designs.
After analyzing the diagrams of structural forces resulting from the calculations performed in this study, the following conclusion can be drawn:

- in some cases, the adoption of a linear elastic material for modelling the tunnel support leads to considerably higher bending moments when compared to the advanced shotcrete model;
- DA2*, except for bending moments of phase 4, provided higher internal forces.
- DA3 approach is feasible, in particular when considering the nonlinear material behavior of soil and shotcrete, resulting in a consistent (implicit) design.

In any case, when structural safety has to be assessed, an M-N check is required, unless all the partial factors are introduced in the calculation, and the design is therefore totally implicit (as for combination 5).
Considering the M-N interaction domains the structural verifications still show that DA2* is the critical combination, i.e. the one governing the support design. The only exception is represented by the results obtained from the linear elastic shotcrete model. In fact, in this case, the material factoring approach DA3, at the intermediate stage (phase 4), exceeds the design domain much more than DA2*.
Concerning the question which is most suitable and conservative design approach to be used for shallow tunnel projects, it can be concluded that it depends not only on the specific example but also on the excavation stage considered. Using only one of the two analysed approaches might lead to a less safe design either from a geotechnical or a structural point of view. In this sense, a combination of both DA2* and DA3 (which is equivalent to DA1 approach) would achieve the purpose of ensuring a safe design from both the geotechnical and structural sides. This goal would be accomplished by concentrating the uncertainties once on the effect of actions and once on the material properties. This design strategy seems to be the most suitable according to the present status of Eurocode 7.

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