Evaluation of long-term ground load on conventional tunnel linings

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ABSTRACT

Conventional tunnel linings are generally composed of two different shells. The outer layer, also called temporary or primary lining, is installed immediately after excavation, and is conceived to guarantee the required ground stability, allowing an appreciable stress release in the rock mass. The inner layer, or secondary lining, on the other hand, is usually applied only at a later stage, and is conceived to carry long term ground load and, eventually, water pressure.

Current design practice is still based on semi-empirical and safe-side approaches, assuming that the whole load carried by the temporary lining is transferred, at the end of its life, directly to the secondary lining. Some authors showed that in many cases, also after 30 to 40 years, primary linings are still efficient and practically no ground loads act on the final lining. At the same time many doubts exist regarding the effective durability of the primary lining, making difficult to assume that this component can be able to guarantee the stability of the system over its whole design life.

Assuming this challenging set of boundary conditions, the present study aims to investigate load sharing mechanisms between primary and secondary lining, considering different degree of deterioration for the shotcrete layer. Plaxis 2D has been used to implement a representative set of non-linear finite element analysis, focused on the definition of a rational design approach, conceived to overcome this impass and to allow for an optimal, reliable and more sustainable design of the concrete double shell.

The calculated results show how the proposed design approach can be used in everyday practice to perform an effective design, which can lead to an appreciable reduction in terms of concrete lining thickness and, as a logical consequence, of excavation diameter. The corresponding savings represent not only an economical advantage, but also a step-forward for the sustainability of the infrastructure, lowering the associated carbon footprint.

Key Words: Lining, Shotcrete, Durability, Optimisation

1. INTRODUCTION

Concrete linings for tunnels are usually designed using two different philosophies: Single Shell Linings (SSL) and Double Shell Linings (DSL).

SSL approach, based on the implementation of a single shotcrete layer, represents an opportunity for cost optimisation, even if it has some important criticalities in terms of water tightness and durability. This is the main reason why it has been used mainly for permanent access tunnels or temporary excavations.

DSL approach, on the other hand, is based on the principle that the main function of shotcrete primary lining is to guarantee excavation stability allowing, thanks to its intrinsic flexibility, a pressure redistribution in the surrounding rock (in accordance to Rabcewicz “load-bearing ring” assumption). The secondary lining is installed only after this initial stress release, taking advantage of the associated load reduction. Thanks to its characteristics, DSL guarantee excellent performance in terms of water tightness, reliability, and durability, being the most adopted solution for permanent tunnels built all over the world.

In spite of this, field experience showed that in many cases the secondary lining remained unloaded for years after its installation [1]. For this reason many interesting papers have been published trying to investigate if the load bearing capacity of the primary lining can be taken into account for an economical design of tunnel lining system in general (see [2], [3] and [4]).

On the other hand, progressive shotcrete degradation can represent an important issue in terms of reliability, mainly because a rigorous estimation of the reduced load bearing capacity of the primary lining is hard to predict over the required service life of the tunnel.

Moving from these assumptions, the objective of this paper will be the definition of a rational design approach, thereafter called DPL (Degraded Primary Lining) method, which could be reasonably implemented in everyday practice for the design of optimal concrete double shell linings.

2. DEGRADATION OF UNDERGROUND CONCRETE STRUCTURES

Generally speaking, in everyday design practice the most attractive solution should be chosen bearing in mind infrastructure requirements in terms of resistance and durability. Even if in some cases the convenience of considering shotcrete layer load bearing capacity has been discussed, in practice it should be remembered that long term durability is a decisive factor to preserve functional efficiency throughout the whole service life of the tunnel.

With this objective in mind, many researchers spent their time to identify the most probable causes of degradation, classifying internal and external factors in three categories: chemical, physical and mechanical attacks.

An accurate discussion of concrete degradation falls outside the scope of this paper. This notwithstanding it can be useful to remember that for the specific case of underground structures, the presence of water represents a key factor which can promote material damage. Focusing our attention on the primary lining, we must recognise that shotcrete can be particularly vulnerable to diffused cracking phenomena. It is quite evident that these cracks represent a preferential path for water access, promoting the penetration of dangerous substances like sulphates, chlorides and alkali. Even in absence of this aggressive chemical compounds, the presence of water can represent by itself a source of degradation, facilitating a progressive decalcification mechanism in the structural components. When water can flow through concrete cracks, in fact, it is able to dissolve various minerals that are present in the hardened cement paste, consuming the composite material until concrete will be reduced to its aggregate only.

Some promising concrete deterioration predictive models are currently available in the technical literature, mainly focused on permeation characteristics and material fracture strength. Unfortunately the main problem of these approaches is the intrinsic complexity, which makes really problematic their application in everyday design practice.

Considering this global picture of the problem the final question is: “Is it reasonable to evaluate the long-term stability of the tunnel taking advantage of the uncertain contribution due to a vulnerable structural component that we won’t be able to inspect over time?”.

3. TRADITIONAL DESIGN APPROACH AND NEW TRENDS

DSL structural systems are usually designed on the basis of the following steps:

- STEP 01: reasonable estimation of the stress field acting in the ground before tunnel excavation and of the expected stress release on the basis of the representative characteristic line;
- STEP 02: implementation of a 2D FEM model where ground and secondary lining are modelled using solid elements, while beam elements are used to simulate the behaviour of shotcrete primary lining;
- STEP 03: deactivation of the excavated cluster, implementation of the previously estimated stress release and installation of the primary lining;

- STEP 04: run the calculation required for the definition of stress resultants (mainly axial load, bending moment and shear force) acting in primary lining;
- STEP 05: activation of secondary lining;
- STEP 06: deactivation of primary lining, application of long-term rock loads (and, if required, water pressure), and calculation of stress resultants needed for the structural design of the secondary lining.

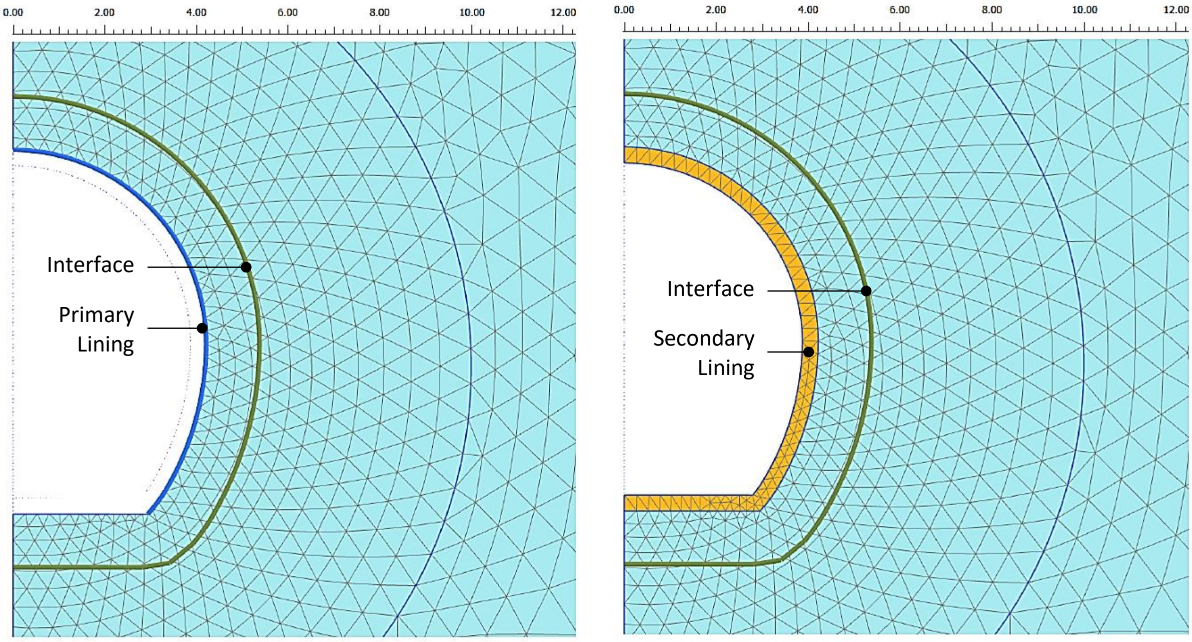


Figure 1. Plaxis 2D FEM model: STEP 04 on the left and STEP 06 on the right (traditional approach).

Just to summarise, the practical effect of this approach is to transfer 100% of the loads acting on the primary lining directly to the secondary lining.

As previously discussed, many interesting technical approaches have been introduced to overcome this simplification, considering both combined and composite behaviour of primary and secondary lining. This notwithstanding, durability issues always prevented a concrete implementation in permanent tunnels.

This seems a good reason to introduce the optimised design approach described in the following section of the paper.

4. PROPOSED DESIGN APPROACH

4.1. Introduction

The DPL design method has been conceived moving from the following assumptions:

- Nowadays powerful computational resources are not a prerogative of huge engineering players only. Non-linear analysis, in fact, represent by now a widespread implemented design tool that can be used to improve our knowledge of the problem, avoiding the introduction of safe-side simplifications or assumptions;
- In recent years there have been significant advances in why and how we design, construct and maintain our physical infrastructure. Societal demand to build more responsibly, embedding the key issues of sustainability, in now part of contemporary design, pushing towards a reduction in the primary energy spent to realise our built environment.

Taking advantage of state-of-art FEM tools, the proposed approach will show how a proper simulation strategy can be used to optimise concrete secondary lining, following its behaviour over the whole service life.

4.2. Shotcrete degradation model

The definition of a meaningful shotcrete degradation model has been a key issue for the development of this alternative design method. The main challenge related to this task has been the necessity to reproduce a progressive material transition from concrete to a loose gravel, considering its evolution in terms of stiffness and strength.

The hypothesis at the basis of the proposed approach is that the primary effect of leaked groundwater is to induce a gradual consumption of the cement paste, with a corresponding increase in its porosity. This phenomenon can be associated, under a mechanical point of view, to a progressive degradation of the elastic modulus and of material uniaxial compressive strength. The following paragraphs will summarise the main assumptions that are at the basis of DPL design method.

Stiffness: in general terms, a composite material consisting in two phases (like concrete and shotcrete) can be idealised using a composite hard material model or a composite soft material model. The first case can be representative of a continuous matrix characterised by a high modulus elastic phase and embedded particles with a lower modulus. The second option, on the other hand, is that of a material which consists of elastic particles with a high modulus of elasticity, embedded in a continuous matrix phase with a lower modulus. For the specific case discussed in this paper, the embedded aggregate has a higher modulus than the cement paste. According to [5] the following formulation can be used for a reliable estimation of the elastic modulus of the composite material:

$$E_c = \left[\frac{1-g}{E_m} + \frac{g}{E_p}\right]^{-1} \qquad (1)$$

Where:
E_c = modulus of elasticity of the composite material
E_p = modulus of elasticity of the particle phase
E_m = modulus of elasticity of the matrix phase
g = fractional volume of the particles

According to current design practice, a reasonable value of “g” for a common shotcrete mix design can be about 0.67. On the other hand, the estimation of the elastic modulus for the hardened portland cement paste is a challenging task. The experimental campaign described in [6] shows, as expected, a direct relationship between porosity and elastic modulus. Assuming that a representative value for “E_m” can be 16’000 MPa (defined using a mean capillary porosity), it seems reasonable to affirm that the degradation of the composite material can be simulated via a progressive increase in the porosity of the cement paste (corresponding to a reduction in terms of elastic modulus). “E_p”, finally, can be estimated using typical values provided in technical literature for crushed aggregates.

For the specific purpose of this paper two options have been considered: compact limestone (E_p = 65000 MPa) and granite (E_p = 40000 MPa).

Table 1 shows some values for “Ec” calculated according to different assumptions for “E_m”. Cement paste degradation has been simulated applying a percentage reduction of the original elastic modulus. It’s interesting to observe that:

- The sensitivity of the mix to aggregate quality decreases with the evolution of the degradation process;
- The initial values calculated for “E_c” (before degradation) are very close to typical elastic moduli assumed for hardened shotcrete mixes;
- The final values calculated for “Ec” (after degradation) are very close to typical elastic moduli assumed for loose gravel.

Table 1. Elastic modulus reduction due to an increase in cement paste porosity

Limestone				Granite			
E_m [MPa]	*% E_m*	E_c [MPa]	% E_c	E_m [MPa]	*% E_m*	E_c [MPa]	% E_c
16000	*100%*	33876	*100%*	16000	*100%*	27586	*100%*
14400	*90%*	31643	*93%*	14400	*90%*	26087	*95%*
12800	*80%*	29234	*86%*	12800	*80%*	24427	*89%*
11200	*70%*	26628	*79%*	11200	*70%*	22581	*82%*
9600	*60%*	23799	*70%*	9600	*60%*	20513	*74%*
8000	*50%*	20717	*61%*	8000	*50%*	18182	*66%*
6400	*40%*	17348	*51%*	6400	*40%*	15534	*56%*
4800	*30%*	13648	*40%*	4800	*30%*	12500	*45%*
3200	*20%*	9568	*28%*	3200	*20%*	8989	*33%*
1600	*10%*	5044	*15%*	1600	*10%*	4878	*18%*
800	*5%*	2592	*8%*	800	*5%*	2548	*8%*
160	*1%*	530	*2%*	160	*1%*	528	*2%*
80	*0,5%*	266	*1%*	80	*0,5%*	265	*1%*

Strength: also under the point of view of compressive strength, concrete can be treated as a composite material, where the dissipate phase (aggregate and grain of non-hydrated cement) is joined by gel with dissipate pores (which makes the matrix). Many models have been developed to describe material destruction process, which, in the broad scope of structure's development, proceeds in the matrix area. In any case it is generally recognised that concrete mechanical characteristics are given by the following factors: total porosity, pores size distribution, defect's existence and diversity of structure's level. More in detail, the porosity coefficient can be calculated using the formulation proposed in [7]:

$$x = \frac{\omega_{gel}}{\omega_{gel}+\omega_{cap}+\omega_a} \qquad (2)$$

Where: ω_{cap} = capacity of capillary pores [per unit binder mass dm^3 / kg]
ω_{gel} = capacity of molecular pores [per unit binder mass dm^3 / kg]
ω_a = capacity of air pores [per unit binder mass dm^3 / kg]

The following figure presents two meaningful graphs showing how a linear relationship can be established between concrete cubic strength and porosity coefficient:

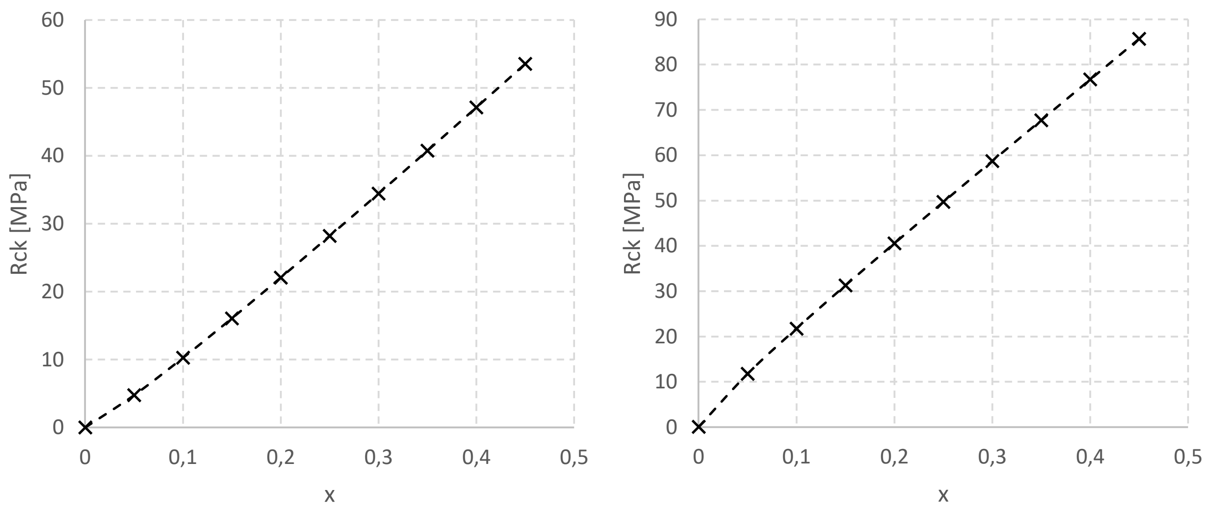


Figure 2. Porosity Coefficient vs Cubic Strength experimental graphs (representative of 4 mixes).

These curves have been empirically characterised investigating 4 different mixes.

On the basis of the proposed results, it is reasonable to conclude that the progressive degradation of the cement paste, involving a gradual increase in the value of "ω_{cap}", produces a corresponding reduction in the compressive strength of the mix, which becomes, at the end of the process, very similar to a loose gravel composed of its aggregates.

4.3. Analysed constitutive models

The selection of the best constitutive model to be used for FEM simulations has been done considering that shotcrete degradation is mainly due to a progressive decalcification of the cement paste (that means an increase in the capillary porosity and a corresponding reduction in terms of strength and stiffness). In other words, this means that the analysed material will experience a progressive loss in terms of cohesion.

Moving from these assumptions, DPL design method has been investigated assuming two representative constitutive models:

- Mohr-Coulomb;
- Plaxis Shotcrete Model (see [8] and [9])

Additional details of the adopted simulation approach will be provided in the next session of the paper, where the proposed design sequence will be summarised.

4.4. Design steps

The DPL design method can be implemented on the basis of the following steps:

- STEP 01: reasonable estimation of the stress field acting in the ground before tunnel excavation and of the expected stress release on the basis of the representative characteristic line;
- STEP 02: implementation of a 2D FEM model where ground, primary and secondary lining are modelled using solid elements;
- STEP 03: deactivation of the excavated cluster, implementation of the previously estimated stress release and installation of primary lining (both Mohr-Coulomb and Plaxis Shotcrete Model have been considered for this component);
- STEP 04: run the calculation required for the definition of stress resultants (mainly axial load, bending moment and shear force) acting in the primary lining;
- STEP 05: activation of secondary lining;
- STEP 06: progressive decalcification of the primary lining according to the previously described shotcrete degradation model. For the case of Mohr-Coulomb constitutive model the corresponding loss in terms of stress and stiffness has been reproduced via a gradual reduction of cohesion and elastic modulus. For the case of Plaxis Shotcrete Model, on the other hand, the same phenomenon has been simulated working on "E_{28}", "$f_{c,28}$" and "$f_{t,28}$";
- STEP 07: definition of the Double Shell Interaction Curve (DSIC), useful to show load history for both primary and secondary lining.

Under a practical point of view, the adoption of this modified approach allows for a more accurate simulation of the system. The progressive degradation of the primary lining, in fact, causes a "damping" effect for the secondary lining, allowing an additional stress redistribution in the surrounding ground.

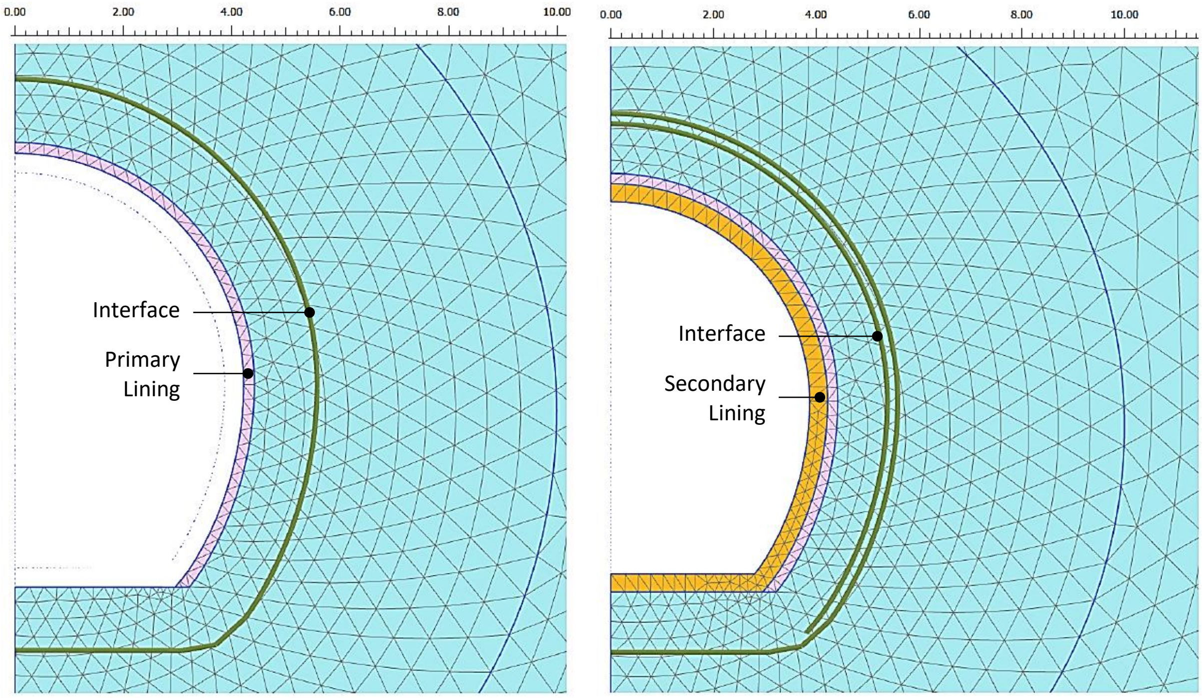


Figure 3. Plaxis 2D FEM model: STEP 04 on the left and STEP 06 on the right (DPL method).

In order to provide some quantitative data useful to demonstrate the tangible benefits associated with the adoption of the DPL method, the following sections of the paper will describe a meaningful case study and the associated results.

5. CASE STUDY

The analysed case study is characterised by an excavation area of about 60 m^2 and refers to the single track cross section shown in the following figure:

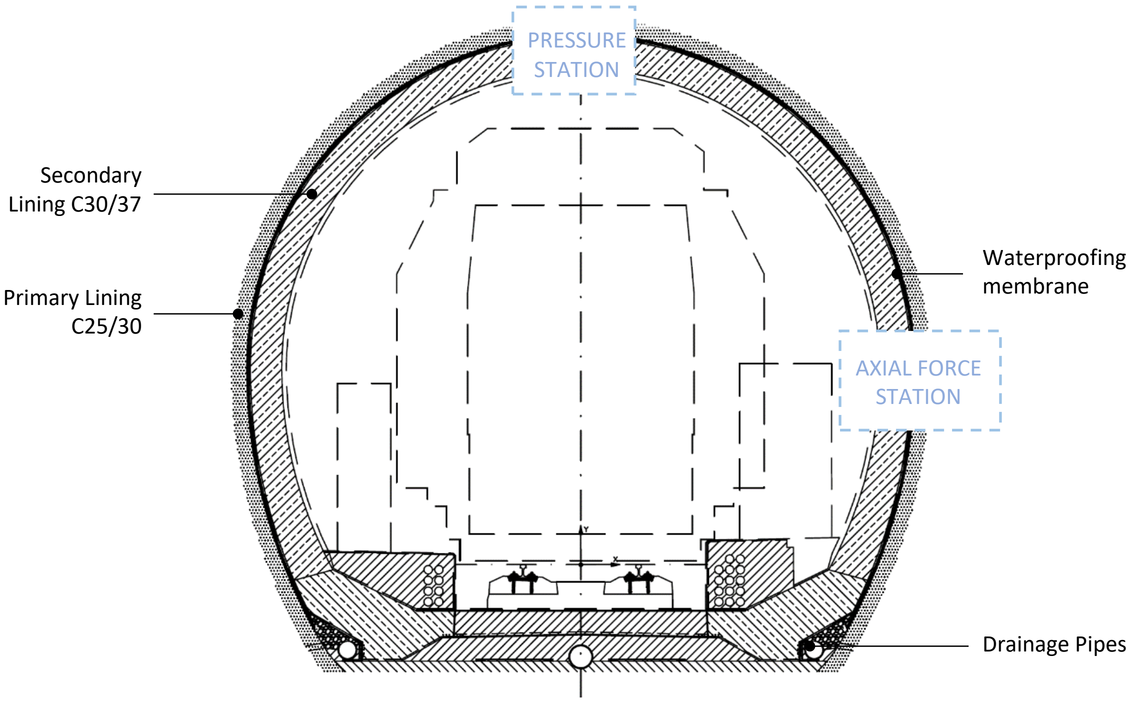


Figure 4. Single track cross section representative of the case study.

Under a practical point of view, it can be representative of a typical layout suitable for a deep base tunnel, where it is necessary to ensure water tightness, but preventing the high pressures that are commonly associated with a fully tanked solution.

More in detail the analysed system will have the following characteristics:

- Shotcrete primary lining will be built using a 200 mm thick layer with a strength class C25/30, according to EN 206-1 requirements;
- Concrete secondary lining will be built using a 350 mm thick layer with a strength class C30/37 according to EN 206-1 requirements;
- A PVC waterproofing membrane will separate primary and secondary lining, inhibiting the transmission of interface shear stresses (only normal stresses can be transferred from primary to secondary lining);
- Two draining pipes will be installed for an effective management of the collected water.

5.1. Soil parameters

The rock mass considered in the following simulations is a metamorphosed limestone.

Its specific weight has been assumed equal to 27 kN/m^3, while the associated overburden has been fixed in 700 m. The adopted K_0, finally, is 1.0. The material has been modelled using an elastoplastic behaviour with a Hoek-Brown failure criterion, based on the following parameters:

- σ_{ci} = 40 MPa
- m_i = 12
- E_i = 40 GPa
- GSI = 40

The calculation has been run according to both traditional design approach and DPL method.

A comparison between the obtained results will be presented and commented in the following paragraphs. The selected output parameters have been the axial force acting in the lining (see axial force station of Figure 4) and the normal stress acting on the top of the crown (see pressure station of Figure 4), which is usually the input value required to perform the prescribed structural checks.

5.2. FEM model

The simulation has been performed using the finite element package Plaxis 2D 2017, that is one of the most accredited tools currently available on the market for the evaluation of ground structure interaction and for the execution of stability analysis.

The following modelling assumptions have been introduced:

- 15 nodes triangular elements have been used for volume components (ground and primary or secondary lining);
- 5 nodes Mindlin beam elements have been used for linear components (mainly the primary lining in the traditional design approach);
- Considering the symmetry of the problem, only half of the system has been modelled to optimise computational time;
- Boundary conditions have been applied enlarging the size of the model to prevent border effects;
- In order to correctly reproduce actual shotcrete behaviour, the hypothetical stiffness approach has been adopted, combining technical literature suggestions with numerical results obtained

applying the advanced shotcrete model described in [8] and [9] to study different deformation paths. For short term evaluation of tunnel primary lining after installation, the so-called "young" elastic modulus has been fixed to 5000 MPa [10];

- The stress release prior to lining installation as been determined according to Sulem ground reaction curve [11] in combination with Vlachopoulos longitudinal displacement profile [12]. In order to obtain a realistic release factor, a distance of 3 times the round length from the face has been considered.

The following figure provides some useful information on the geometry of the model and the applied boundary conditions:

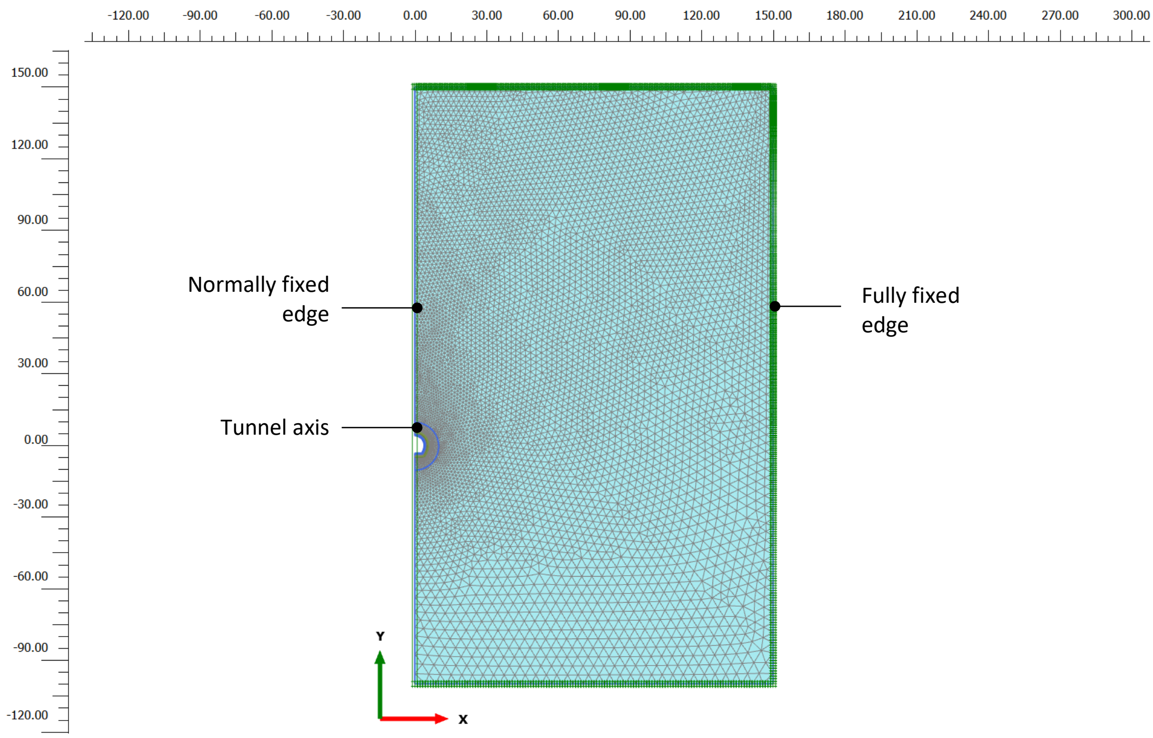


Figure 5. Plaxis 2D FEM model implemented to analyse the problem.

5.3. Traditional design approach

First of all the traditional approach has been applied to the problem. The primary lining has been modelled using a linear elastic line element that has been deactivated after final lining installation. The obtained results have been used for a meaningful comparison with the proposed design method.

5.4. DPL design approach

Successively the system has been studied adopting the DPL approach. Both primary and secondary linings have been modelled using linear elastic volume elements. Progressive long term deterioration of primary lining has been reproduced via a gradual reduction in terms of strength and stiffness of the assigned material.

For the specific case of Mohr-Coulomb constitutive law, cohesion has been directly derived from Mohr's circles equations in the case of uniaxial compression and under the assumption of a constant friction angle equal to 37°:

$$c = \frac{f_{cd}}{2} \times \frac{1-\sin\varphi}{\cos\varphi} \qquad (3)$$

The initial uniaxial compressive strength has been fixed to f_{cd} = 16.67 MPa (equal to design short term strength of shotcrete C25/30), while complete deterioration has been modelled using typical gravel strength parameters.

Table 2. Primary lining parameters for different deterioration stages.

Steps of deterioration	Primary lining parameters			
	$\%_{\text{deterioration}}$	f_{cd} [kPa]	c [kPa]	E_{cm} [MPa]
1	0	16667	4155	33000
2	10	15000	3739	31500
3	20	13333	3324	29000
4	30	11667	2908	26600
5	40	10000	2493	23800
6	50	8333	2077	20700
7	60	6667	1662	17300
8	70	5000	1246	13600
9	80	3333	831	9500
10	90	1667	415	5000
11	100	0	1	250

5.5. Results

The next figure aims at providing some information useful for a meaningful comparison between traditional and DPL design method:

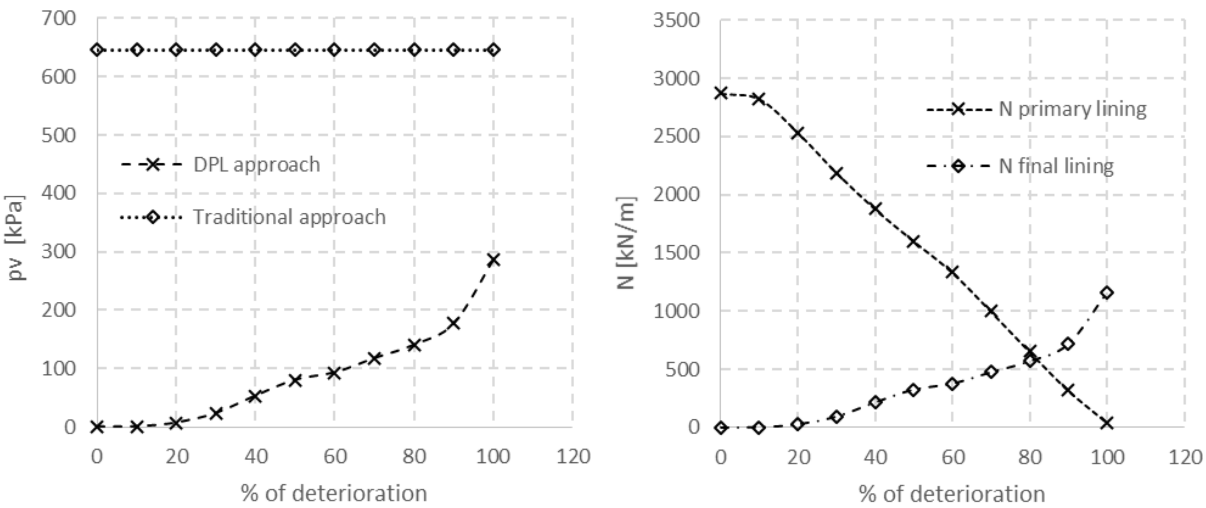


Figure 6. Time evolution of crown normal stresses (left side) and lining axial force (right side)

On the basis of the presented results, the following remarks can be done:

- The axial force acting in the secondary lining after primary lining complete deterioration is lower than the axial force acting in the primary lining at the beginning of the process, because gradual degradation allows a stress redistribution that is partially in charge of the surrounding rock;
- In general the primary lining exhibit a reserve of capacity coming from the safety factors applied at the design stage. This ensures that it is able to carry the full ground load also for non-negligible levels of deterioration;
- The final pressure acting on the lining according to DPL design method is significantly lower than that evaluated with the traditional design approach (about 50% less).

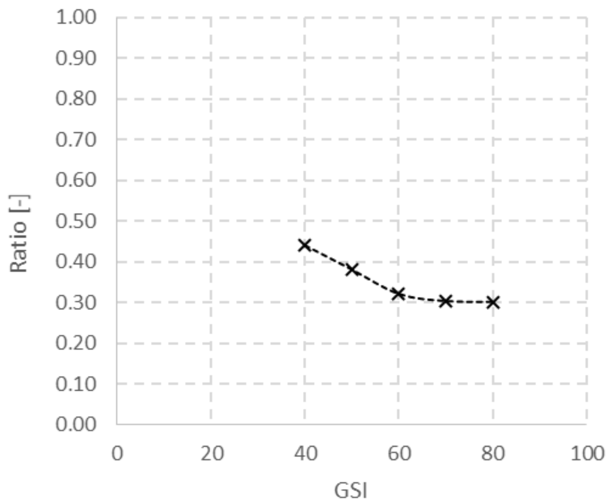
5.6. Sensitivity analysis on rock mass quality

Considering that the previously described stress redistribution is directly influenced by rock mass capacity of absorbing additional load, it is quite intuitive that the expected advantage of DPL design approach (compared with traditional design approach), depends on rock mass quality, which can be quantified on the basis of geomechanical classification, e.g. with GSI.

For this reason a sensitivity analysis has been implemented to investigate the relationship of GSI value (varying in the typical range 40-80) and the ratio between rock load calculated using both DPL and traditional design approach.

Intact rock properties are the same adopted in the previously described analysis.

Load values for DPL method refer to the case of a complete deterioration of the primary lining.



Stress on top of crown

GSI	$P_{V,DSL}$ [kPa]	$P_{V,standard}$ [kPa]	Ratio [-]
40	286	646	0.44
50	137	360	0.38
60	52	162	0.32
70	36	118	0.30
80	22	74	0.30

Figure 7. DPL design method sensitivity to rock mass quality

6. CONCLUSIONS

This paper presents a design method that can be useful in everyday design practice to optimise conventional tunnel linings.

Considering that strength and durability represent two essential requirements, the system has been studied on the basis of a new modelling approach, where the geometry of the problem and the gradual degradation of shotcrete primary lining have been analysed with a more realistic set of assumptions.

The analysed case study showed that this new design method can guarantee, especially for an high overburden and for good GSI values, an appreciable optimisation of secondary lining structural design.

Another meaningful advantage of the method is that it is not less conservative, but simply more realistic, taking advantage of state-of-art FEM capabilities without producing a massive increase in the required computational effort.

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