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Research Paper

Numerical analyses of stability and deformation behavior of reinforced and unreinforced tunnel faces

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ABSTRACT

In traditional tunneling, an analysis of the face stability is required to avoid failure mechanisms or excessive face extrusion. Face reinforcement can improve face stability and reduce deformations. In the present work a numerical study of both unreinforced and reinforced tunnel excavation faces by means of 3D FEM analyses is presented. The results are compared with those of the traditional limit equilibrium method and with an analytical solution based on previous numerical studies. It could be shown that the LEM may lead to non-conservative results. Finally, the deformation response is assessed and the benefits of face reinforcements are investigated.

1. Introduction

A realistic assessment of face stability and deformation behavior of shallow tunnels is of significant importance not only for guaranteeing the workmanship safety but also because it is directly related to surface deformations, at least for shallow tunnels. The occurrence of excessive face extrusion or the developing of a failure mechanism could cause significant subsidence phenomena, damaging pre-existing buildings and infrastructures.

Several experimental studies have been carried out in the past to assess the stability of an unreinforced tunnel face and different analytical or semi-empirical solutions have been presented in the literature. A detailed overview of these studies for both drained and undrained conditions is provided in Ruse (2004) [1]. Moreover, advantages and shortcomings of the most common methods employed for geotechnical stability analysis (namely limit equilibrium method, upper/lower bound limit analysis and displacement finite-element analysis) are summarized in Sloan (2013) [2].

The solutions formulated by different authors for tunnel face stability are usually expressed in the form of stability numbers (N_C, N_Q and N_γ) which can be introduced in Eq. (1) to calculate the minimum support pressure required to obtain the desired factor of safety for face stability. These factors are related to soil cohesion (c'), surface surcharge (q) and soil unit weight (γ) and were mainly derived from upper/lower bound calculations or limit equilibrium methods. The stability numbers are dimensionless factors and similarly to those derived for footings depend on friction angle.

$$p_f = -c'N_C + qN_Q + \gamma DN_\gamma \tag{1}$$

The choice between drained and undrained analysis should depend on the ratio between the ground permeability and the advance rate (Anagnostou, 1993) [3]. When long excavation standstills have to be considered, even a low permeability material should be analyzed in drained conditions.

If necessary, the stability of the face, in traditionally excavated tunnels, can be improved either by a partial excavation of the cross section or by reinforcing the core ahead of the face. The first method allows for a reduction of the deconfinement inevitably induced by an underground excavation. The second provides an improvement of the mechanical characteristics of the ground to be excavated. For cohesive or semi-cohesive soils, bars made of Glass Fiber Reinforced Polymer (GFRP) are often adopted. They are basically pipes inserted in longitudinal holes drilled into the core and immediately grouted using a shrink controlled or an expansive mixture. During the excavation, a certain length of these elements has to be maintained in order to have beneficial effects. For this reason, when a new series of fiberglass bars is installed, a prescribed overlap length should be kept. The success of this material is due to the considerable strength and stiffness properties combined with high fragility. Therefore, it is easy to excavate the core with the same tools regularly used for an unreinforced

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material but at the same time excessive de-confinement is avoided by improving ground stiffness and strength. The reliability of this method has been proved by many successful projects completed over the past 30 years, especially in Italy and France. For a complete overview of the method and its applications in different projects, one can refer to Lunardi & Bindi (2004) [4]. The design of reinforcing elements is commonly carried out through limit equilibrium methods (LEM) which allow, given a prescribed failure mechanism, an evaluation of the safety factor. The advantage of the LEM is to be a very simple and fast method to assess the stability conditions of an excavation. However, being based on several simplifying hypotheses, it cannot always guarantee realistic estimations of the face stability. Performing a 3D numerical analysis, on the other hand, implies a significant computational effort, given the level of mesh fineness necessary to achieve a satisfactory accuracy of the results and the high number of structural elements required to simulate the reinforcements. Considering that an optimization of their design features requires more trials, this effort increases considerably.

The design of reinforcements implies the definition of their number, length and arrangement. One criterion to define these characteristics could be a target safety level to be reached. Nevertheless, when the limitation of the surface settlements is a major concern, it makes sense to adopt a criterion based on limiting the volume loss which consequently would limit excessive surface subsidence. Either way, the evaluation of the deformation behavior of the reinforced tunnel requires, at least for shallow tunnels, a full 3D numerical analysis including direct modelling of the face reinforcements. Alternative ways to consider the presence of these elements for the modelling of the reinforcement in a 3D model could be the application of a face pressure or the definition of equivalent material properties for the soil clusters belonging to the tunnel core. Besides the detailed 3D model the second option is also considered in the present study. With respect to the previous literature, the present work includes a more extensive study covering a wide range of material properties and reinforcement configurations and providing safety factors for both unreinforced and reinforced face. The so-called strength reduction technique is adopted for analyzing face stability. An important outcome of this study is that results from limit equilibrium methods may be non-conservative for this type of problem and the authors believe that this is not common knowledge, at least not in practice. A further series of numerical calculations is aimed at investigating the effectiveness of face reinforcement in reducing both extrusion and subsidence phenomena. In this way, both the behavior under working conditions and the ultimate limit state of the excavation face are taken into account.

2. Application of the limit equilibrium method for tunnel face stability

The principal hypothesis characterizing the limit equilibrium method regards the definition of a failure mechanism, which is represented by rigid blocks sliding along surfaces where the maximum shear resistance is assumed to be mobilized. A mechanism commonly assumed to represent a potential tunnel face failure is the Horn mechanism (Horn 1961) [5], represented in Fig. 1.

The sides of the square front of the sliding wedge can be equal to the tunnel diameter, as in Fig. 1. Alternatively, the square dimensions can be calculated based on the criterion of maintaining the same area of the tunnel section. On the lateral surfaces of the sliding wedges, the shear forces depend on the geostatic stress distribution, which can be evaluated according to the Silo's theory (Janssen 1895 [6]), as shown in Fig. 2. In particular, the vertical stress at a certain depth (z) is computed as:

$$\sigma(z) = \frac{\gamma r}{2k_0\mu}\left(1 - e^{-\frac{2k_0\mu z}{r}}\right) \tag{2}$$

where γ is the soil unit weight, k_0 the at-rest earth pressure coefficient, r the silo's radius and μ the friction coefficient. Given a silo's geometry and prescribed values of γ and k_0, the maximum value which can be reached from the vertical stress is $\gamma r/(2k_0\mu)$.

The safety factor can be calculated as the ratio between forces and resistances along the sliding wedge, whose angle (θ) has to be defined by minimizing this ratio. With reference to Fig. 1, the safety factor can be calculated as follow:

$$FoS = \frac{R_s + R_{l2} + H\cos\theta}{[(W - R_{l1}) + V]\sin\theta} \tag{3}$$

Horn's model has been widely used in the past for both conventional and mechanized tunneling (Oreste 2011 [7], Anagnostou & Kovári 1994 [8], 1996 [9], Anagnostou & Serafeimidis 2007 [10], Segato et al. [11]). One of the main limits of the LEM is that the failure mechanism is apriori defined whereas, as shown in previous works carried out by means of numerical calculations (Vermeer at al. 2002 [12], Kavvadas & Prountzopoulos 2009 [13]), the potential failure mechanism depends on the specific case, in particular on the material strength properties and on the overburden.

In the LEM, the wedge mechanism can be properly modified in order to consider an unsupported span, whereas the presence of reinforcements can be taken into account by considering an equivalent force S supporting the excavation face. The unsupported span can be easily taken into account by increasing the horizontal dimension of the upper block. The value of the supporting force, which is necessary to guarantee a prescribed safety factor, can be assessed by considering the force itself in the equilibrium of the sliding wedge. Once the supporting force has been determined, one can define the minimum number of reinforcements necessary to reach the desired safety level. The procedure usually adopted takes into account only the axial force developed in the bars, while the flexural stiffness is assumed to be negligible (Oreste 2011 [7]). Each bar can develop a maximum force equal to the lower value among the axial force the structural element itself can take (calculated with the admissible axial stress, Eq. (4)) and the two main forces developed around the borehole. These forces are the frictional force which can be mobilized between the ground and grout in the segment inside the sliding block (Eq. (5)) and the one which can be mobilized in the segment falling in the stable part of the core (Eq. (6)).

$$S \leqslant n \cdot \sigma_{adm} \cdot A_{bar} \tag{4}$$

$$S \leqslant n \cdot \tau_{adm} \cdot (\pi \cdot \phi_{hole} \cdot L_a) \tag{5}$$

$$S \leqslant n \cdot \tau_{adm} \cdot (\pi \cdot \phi_{hole} \cdot L_p) \tag{6}$$

In these equations, σ_{adm} is the admissible tensile stress of the element, τ_{adm} is the maximum shear stress at the contact ground-grout, ϕ_{hole} is the hole diameter, n is the number of bars and L_a and L_p are the bar lengths falling inside and outside the sliding prism (Fig. 2).

Another failure mechanism which could theoretically occur is the pull-out of the bar along the contact surface between the structural element and the injection material. However, given the very high values of the pull-out resistances, especially for corrugated profiles, derived from laboratory tests (Zenti et al. 2012 [14]), this mechanism does not govern the reinforcement design. Moreover, considering the structural properties of these elements, the overlap lengths commonly adopted (0.5D-D) and the average ultimate bond strength between grout and ground for mainly clayey and silty soils (τ_{adm} < 150 kPa), the structural failure is not likely to occur either, being the structural strength higher than the pull-