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Face stabilization of deep tunnels using longitudinal fibreglass dowels

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ABSTRACT

When the stability of an excavation face in a deep tunnel is not ensured by the strength characteristics of the rock mass, it is current practice to proceed with reinforcement of the face using longitudinal fibreglass dowels. An analysis of the effect of these reinforcements and a correct dimensioning can be obtained through three-dimensional numerical modelling, which is very burdensome from a computational point of view. The simplified methods that are at present available do not allow the maximum force in the dowels to be obtained and cannot therefore be used for their dimensioning.

A finite difference procedure for deep tunnels is presented in this paper. This procedure is able to analyse the presence of dowels in the rock mass in a distinct manner, and it adopts the conceptual schematisation of a spherical geometry void for the approximation of the tunnel face zone. A calculation comparison of the simplified procedure and the three-dimensional numerical modelling has offered satisfactory results. The procedure has also been applied to a real case and the results agree with in situ measurements.

1. Introduction

Full-face excavation of a tunnel is currently preferred, even in poor rock masses and in difficult conditions. In this way, it is possible to take full advantage of the potentiality of the machines and large size plants. The problem of face stability, however, is fundamental for the stability of the tunnel in these cases [1,2]. When the stability of the face is not guaranteed by the strength characteristics of the rock mass, fibreglass reinforcements are usually installed; they offer the advantage of producing a confinement of the cortical zone of the face through the application of concentrated forces, which are balanced by other forces that develop deeper down in the anchorage zone. The fibreglass dowels are connected along all their length to the surrounding ground through cement grouting and there is no contrast plate in correspondence to the excavation face. They can be easily cut by the excavation machines, because of the brittleness of fibreglass material; this typology of reinforcement technique therefore does not reduce the excavation efficiency.

While radial bolting around tunnel walls have been studied in the past [3–7] no adequate calculation instruments have been developed that are able to proceed quickly with an analysis of the behaviour of fibreglass dowels and therefore with their dimensioning. In particular, it is at present problematic to define the number and type of reinforcements that should be used for deep tunnels: numerical methods, in fact, being obliged to use three-dimensional models to study the reinforcement system, require very long calculation times.

By deep tunnels, we generally mean those tunnels in which there is a variation in the lithostatic stress of less than 10% between the edges (crown or bottom) and centre of the tunnel. They are therefore those tunnels that have an overburden of more than 4–5 times the tunnel diameter and for which analytical and numerical studies, for the sake of simplicity, consider a homogeneous initial stress field equal to the stress field that exists in the centre of the tunnel (the stress variation due to gravity is not considered). Deep tunnels are different from shallow tunnels as shallow tunnels suffer from different forms of face collapse that involve the ground surface [1,2]. It is not possible to neglect the lithostatic stress variations with depth in studies on shallow tunnels.

In recent years, various authors have studied the problem of excavation face stability in deep tunnels in the presence of fibreglass reinforcements using three-dimensional numerical methods. Among the most recent, mention can be made to the following studies [8–10]. All these works have made it possible to study the role and action mechanisms of fibreglass reinforcements on the stability of the excavation face.

Other authors have attempted to develop simplified analytical methods that are able to estimate face extrusion, in the presence of fibreglass reinforcements, considering the homogenisation criterion. The works [11–15] are among the most interesting of these studies. However, as these methods cannot evaluate the tensile force induced in the dowels, it is not possible to define their dimensioning in a correct way.

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A new simplified calculation procedure is presented in this paper for the analysis of a reinforced excavation face in deep tunnels, and for the dimensioning of fibreglass dowels. This procedure is based on a finite difference analysis of the geometric problem of the spherical symmetry. The fibreglass dowels are considered in a distinct way, through the shear stresses that they apply to the dowel–rock interface.

A comparison between the calculation results of an extensive parametric analysis, obtained using the proposed procedure, and three-dimensional numerical modelling has led to good results. The procedure has also been applied to a real case with satisfactory results.

2. Analysis of face behaviour using spherical geometry approximation

It is possible to study the behaviour of an excavation face of a deep tunnel, with a certain approximation, using the spherical symmetry hypothesis (Fig. 1), which considers a hemi-spherical void, with a radius R_i in a homogeneous and isotropic medium initially subjected to a hydrostatic pressure p_0 (for the sake of simplicity, the variability of the stress state with depth is neglected, and deep tunnel conditions are hypothesised). The stresses and strains are determined inside the medium, and in particular on the surface of the hemi-sphere, when the radial pressure acting on the surface reduces to zero [16,17]. The value of the radial displacements obtained on the surface of the sphere is an estimation of the mean extrusion displacement of the excavation face. For the sake of simplicity, the stress state produced by the presence of tunnel supports, close to the excavation face, is neglected.

Using the theory of elasticity in spherical symmetry conditions, it is possible to obtain the radial stresses and displacements in an elastic field for $r > R$, from the equilibrium of the forces in the radial direction and from the congruency of the strains, where R is the generic radius for which the radial stress $\sigma_{r,R}$ is known

$$\sigma_\vartheta = p_0 + \frac{1}{2}\left(p_0 - \sigma_{r,R}\right)\frac{R^3}{r^3} \quad (1)$$

$$\sigma_r = p_0 - \left(p_0 - \sigma_{r,R}\right)\frac{R^3}{r^3} \quad (2)$$

$$u = \frac{1+\nu}{2E}\left(p_0 - \sigma_{r,R}\right)\frac{R^3}{r^2} \quad (3)$$

where ε_r and ε_ϑ are the radial and circumferential strains, respectively (positive compression strains); E and ν are the elastic modulus and the Poisson ratio of the rock mass. R can be represented by the tunnel radius R_i, when a plastic area is not formed ahead of the face or by the plastic radius R_{pl} (see below), when a plastic area is formed close to the face and the elastic behaviour zone is outside the plastic area.

In the plastic zone close to the face (the brittle elastic–plastic behaviour is shown in Fig. 2a), the connection between σ_ϑ (σ_1) and σ_r (σ_3) is given by the residual strength criterion. Considering the Mohr–Coulomb criterion (Fig. 2b), we obtain [16–19]

$$\sigma_\vartheta = f_{res} + N_{res}\sigma_r \quad (4)$$

where

$$f_{res} = \frac{2c_{res}\cos\varphi_{res}}{1-\sin\varphi_{res}}, \quad N_{res} = \frac{1+\sin\varphi_{res}}{1-\sin\varphi_{res}}$$

and c_{res} and φ_{res} are the residual cohesion and friction angle of the rock mass, respectively.

The radial stress calculated with the residual strength criterion on the plastic radius R_{pl} (distance of the border between the plastic and elastic zones) should be equal to the radial stress calculated with Eq. (2). It is then possible to determine the value of the plastic radius

$$\frac{R_{pl}}{R_i} = \left[\frac{\left(\frac{1.5p_0 - f_{peak}}{N_{peak}+0.5} + \frac{f_{res}}{N_{res}-1}\right)}{\left(p + \frac{f_{res}}{N_{res}-1}\right)}\right]^{\frac{1}{2(N_{res}-1)}} \quad (5)$$

where p is the the radial pressure applied to the surface of the sphere (on the excavation face); usually $p=0$, and

$$f_{peak} = \frac{2c_{peak}\cos\varphi_{peak}}{1-\sin\varphi_{peak}}, \quad N_{peak} = \frac{1+\sin\varphi_{peak}}{1-\sin\varphi_{peak}}$$

where c_{peak} and φ_{peak} are the peak cohesion and friction angle of the rock mass, respectively. Therefore, the rock mass shows plastic behaviour for $r \le R_{pl}$ and elastic behaviour for $r > R_{pl}$.

The strains in the plastic field are made up of an elastic component and a plastic component. By making the radial and circumferential strains in the plastic field explicit, it is possible to obtain the following differential equations of the radial displacement u with a variation in the distance r

$$\frac{du}{dr} + k\frac{u}{r} + lr^m = n \quad (6)$$

where

$$n = \frac{f_{res}}{E}\left[(k - k\nu - 2\nu) - \frac{1 - k\nu + N_{res}(k - k\nu - 2\nu)}{N_{res}-1}\right] + \frac{p_0}{E}(2k\nu - k + 2\nu - 1)$$

$$l = -\frac{p + \left(\frac{f_{res}}{N_{res}-1}\right)}{E}\frac{1}{R_i^m}\left[1 - k\nu + N_{res}(k - k\nu - 2\nu)\right]$$

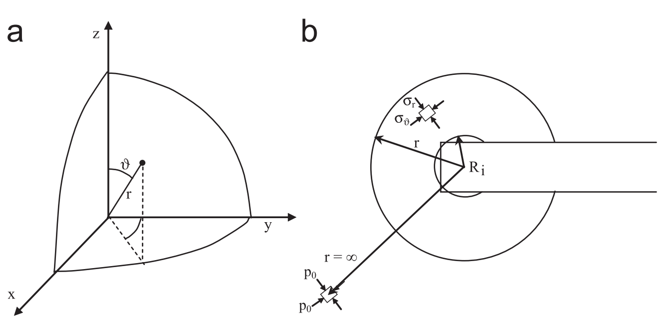

Fig. 1. (a) Geometry of the problem of a spherical void for the study of an excavation face in a deep tunnel; (b) longitudinal section of the tunnel. key: r: generic distance of a point from the centre of the sphere; x, y and z: Cartesian axes; σ_ϑ: circumferential stress (positive compression stresses); σ_r: radial stress; R_i: internal radius of the sphere; p_0: lithostatic stress present for $r = \infty$.

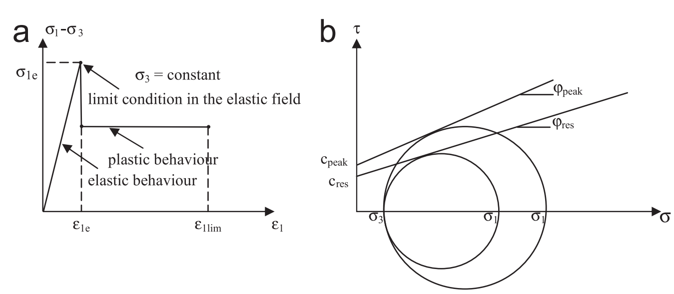

Fig. 2. Material with brittle elastic–plastic behaviour. (a) Stress–strain laws σ_3=constant); (b) peak and residual strength criterion (the peak parameters define the stress state of the limit conditions in the elastic field, while the residual parameters define the stress state in the plastic field). Key: σ_1 and σ_3: principle maximum and minimum stress; σ_{1e}: principle maximum stress in elastic behaviour limit conditions (in correspondence to the plastic radius); ε_1: maximum principle deformation; ε_{1e}: maximum principle deformation in the elastic behaviour limit conditions; ε_{1lim}: limit deformation on rock breaking; τ and σ: shear and normal stress; c_{peak} and φ_{peak}: peak cohesion and peak friction angle; c_{res} and φ_{res}: residual cohesion and residual friction angle.