Panel discussion: Delayed lining activation and ground stress relaxation in shallow tunnels

Débat de spécialistes: Retard dans la sollicitation du revêtement et relaxation des contraintes dans le sol pour les tunnels peu profonds

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ABSTRACT: A procedure is proposed to estimate the ground stress release to be used in two dimensional drained analyses of shallow tunnels in soil, in order to account for the three dimensional stress changes induced by the heading excavation of the tunnel. This stress release is defined as a function of the delayed activation of the lining. The procedure is based on a numerically derived ground response model, that assumes a non-linear elastic constitutive behaviour, and relates radial stresses and displacements. An independent procedure for radial displacements prediction allows the stress changes to be assessed. The potential of this procedure is tested in actual tunnel cases with encouraging results.

RESUME: On propose une méthode pour la prise en compte de la relaxation des contraintes dans le sol, dans les calculs bidimensionnels de tunnels peu profonds, permettant de prendre en compte les modifications de contraintes tri-dimensionnelles liées à la progression du font d'excavation. La relaxation est définie en fonction du délai de mise en place du revêtement. La procédure, basée sur un modèle numérique de la réponse du sol, avec une hypothèse de loi de comportement élastique non-linéaire, s'applique aux contraintes et déplacements radiaux. Une procédure indépendante pour la prévision des déplacements radiaux permet d'évaluer l'influence des variations de contraintes.

1 INTRODUCTION

The relation between the delay in supporting a tunnel and the magnitude of the lining loads is well known, being supported both by many field evidences (see Negro et al, 1996, for instance) and by three dimensional numerical modelling (see, for example, Eisenstein et al, 1984). The increasing distance from tunnel face to the section where the lining is put in contact with the excavated ground causes a reduction in the ground loads onto the lining. Some contend that beyond certain value, the delayed support will experience increasing loads, related to the "ground loosening", an effect, however, not as well established, still raising considerable amount of debate (see for instance, Schwartz et al, 1980). This fact, though, is not consequential in the design of shallow tunnels in urban areas, since for them provision has to be made to ensure minimum induced displacements, therefore earlier support activation. And this is normally done well before any instability or "loosening" process takes place. Despite this, the lining installation always takes place after a certain degree of stress changes in the ground. Even in pressurized face TBMs, the lining is installed after stress changes and displacements occurred in the ground, and it will thus withstand ground loads smaller than the in situ stresses.

Two dimensional analysis of tunnels is always preferred in routine design practice. In this analysis, the effect of the delayed installation of the lining is accounted for by imposing a reduction on the original stresses at the tunnel perimeter, prior to the lining installation. This is particularly true in "stress reversal" procedures of excavation simulation, where each point of the tunnel contour has the in situ stress reduced by an amount α defined as a percentage of the original in situ stresses. Usually this amount, the "stress release factor", is a fixed quantity applied to each point of the excavation perimeter. This reduction is performed numerically after complete "removal" of the ground material within the excavated area. The stress reduction in the 2D model mimics the stress transfer or "arching" that takes place around the unsupported tunnel heading both transversal and longitudinal to the tunnel axis.

The importance in assessing such a stress release factor is enhanced by the fact detected in recent inquiries of design practice that the "full overburden" traditional assumption (Peck, 1969) is being gradually replaced by a "reduced overburden" hypothesis (see Negro & Eisenstein, 1991 and Negro & Leite, 1994) in the design of shallow tunnels in soil.

What is usually unclear is how to estimate the stress reduction. Normally this is done based on past experience (see Negro & Leite, 1994), on field observations (see Swoboda and Laabmayr, 1978), guided by experienced recommendation of some authors (for instance, the 50% stress reduction proposed by Muir Wood 1975). Except for some studies for deep seated tunnels in elastic ground (see for instance Schwartz & Einstein, 1980), no procedure has been made available yet to assess the stress relaxation around shallow tunnels in soils as a function of the delayed installation of the support. The aim of this paper is an attempt to fill this gap and to provide a simple tool allowing lining performance anticipation, which in turn may support lining load monitoring interpretation.

2 STRATEGY

Relations between the delay in lining activation (the distance to the tunnel face) and the radial displacements (convergence) at the opening seems easier to assess than relations for ground stresses (loads). In fact, the former exist for deep tunnels in elastic ground (see Niwa et al 1979, for instance). If we were able to establish such a relation for a shallow tunnel, then we could possibly assess the associated ground stress changes, provided a relation between displacements and ground stress were known. This relation, for deep tunnels in hydrostatic stress field is well known and it is termed "Ground Reaction (or Response) Curve". If this relation could be extended to shallow tunnels in a sensible form, with considerable generality (for instance, for non hydrostatic stress field, and non linear drained behaviour of ground), then one could solve the problem through two sets of solutions: (a) the Ground Response, linking radial stresses with displacements at the tunnel perimeter, and (b) the Ground Convergence, linking radial displacements with distance from tunnel face to where the lining is activated. By coupling both solutions one would get the aimed relation between ground stress and delayed lining activation.

The first solution, in a general form, would be expressed in the following terms, for each point of the tunnel contour

$$F = f(\sigma_r, u_r, \text{tunnel size, depth, deformation and strength parameters}) \quad (1)$$

where σ_r is the radial stress and u_r is the radial displacement at the tunnel perimeter.

The second solution would involve a model accounting for the three dimensional nature of the problem. In both cases a numerically derived approach is favoured in order to achieve generality. However, considerable amount of simplification will be required to make the problem tractable, as shown in following sections.

3 MODEL FOR GROUND RESPONSE

To assess the first solution described, preference was given to the hyperbolic ground model as extended by Duncan and Chang (1970). The preference was mainly due to the ability of accounting for the non linearity of the soil behaviour through a hypoelastic incremental constitutive relation, with variable and stress dependent tangent modulus. The known limitations of this model (Duncan, 1980) to predict soil response under certain stress paths, under near collapse conditions, for highly dilating soils under shear, for strain softening materials, or for far from failure situations (see Lo Presti et al, 1997) are largely overridden by its enormous potential for generalization (see for instance Resendiz, 1979).

Moreover, for maximum generalization potential and to reduce the number of variables, an incrementally reversible (unique) stress-strain behaviour was assumed. This ensures the similarity of the ground response and its independence on the scale of the problem, though it implies in more stored energy than it should (Winter, 1982). If the tangent modulus is made independent of the stress level thus making Janbu's (1963) exponent equal to zero and if the cohesive component of strength is also set equal to zero, homothetic stress-strain curves are found as shown in Figure 1.

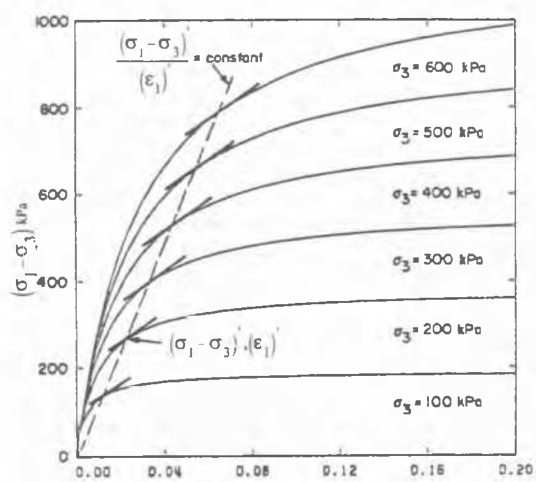

Figure 1: Homothetic hyperbolas.

The centre of homothety or of similitude of these curves is their origin, at zero strain and stress. The homothetic transformation maps a curved segment into a parallel segment with length equal to the ratio of homothety times the length of the original segment. If the coordinates of the homothetic points on these curves (the intersection of the latter with an arbitrary axis through the centre of similitude) are used as reference values, $(\sigma_1 - \sigma_3)'$ and ε_1', and if the deviatoric stress and the major principal strains of each curve are normalized to these reference values, a single normalized hyperbola is found for the entire ground mass, regardless of the value of the minor principal stress. For a constant in situ stress ratio K_0 and horizontal ground surface, the initial tangent modulus is constant with the depth:

$$E_{ti} = K \cdot p_a \cdot \left[1 - R_f \frac{1 - \sin\phi}{2 \cdot \sin\phi} \cdot \frac{1 - K_0}{K_0} \right]^2 \tag{2}$$

where the variables have the same meaning as in Duncan and Chang, op. cit. formulation.

In a 2D representation, unlined circular tunnels with different diameters will exhibit homothetic geometries if their ratios of cover to diameter (H/D) were the same. If geometrically homothetic tunnels are driven through a ground mass exhibiting homothetic hyperbolic stress-strain curves, the ground mass response will be also homothetic. In fact, unique normalized stress and strain responses are found at corresponding (homothetic) points around those tunnels perimeters, in pre-failure stages (see Figure 2).

The above findings simplified the parametric finite element analyses needed for generalization: two dimensional plane strain modelling of circular, unlined tunnel, under gravitational stress field, with constant K_0 and E_{ti}, with stress reversal excavation technique performed incrementally, with stress reduction factor applied uniformly, in each step, to all points of the opening.

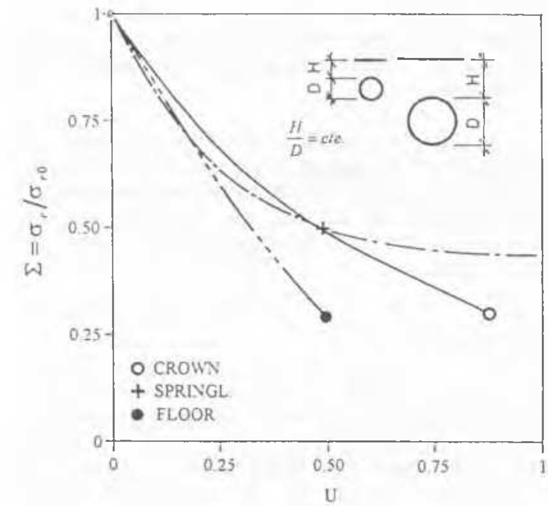

Figure 2: Normalized ground response of homothetic tunnels in ground mass with homothetic constitutive law.

Dimensional analysis indicated convenient normalized radial stress and normalized radial displacement parameters:

$$\Sigma = \frac{\sigma_r}{\sigma_{r0}} \tag{3}$$

$$U = \frac{u_r \cdot E_{ti}}{D \cdot \sigma_{r0}} \tag{4}$$

where σ_r and σ_{r0} are current and initial radial stresses, u_r is the radial displacement and D is the opening diameter. The simplifications introduced allow equation (1) to be expressed in a more compact form:

$$F = f(\theta, \Sigma, U, H/D, K_0, \phi) \tag{5}$$

where angle θ indicates the position of opening points. Note that the stress reduction factor is

$$\alpha = 1 - \Sigma \tag{6}$$

and that, without much consequence the Poisson's ratio can be kept constant (equal to 0.4, for instance).

Moreover, note that by making the cohesion equal to zero, it is always possible to find an equivalent friction angle ϕ_e, associated with a failure ratio (R_{fe}) equal to unity, which gives the same tangent modulus. From equation (2), setting $R_{fe}=1$, one gets:

$$\phi_e = \arcsin(1 - R_f + R_f \csc\phi)^{-1} \tag{7}$$

Therefore, there is no need to vary the failure ratio in the generalization analyses, being enough to replace ϕ by ϕ_e in equation (5).

4 GENERALIZATION OF GROUND RESPONSE

The dimensionless equation (5) is useful for the definition of the ranges of variables for the parametric numerical analyses. These ranges were selected to cover the most common situations found in practice. The dimensionless ground cover H/D was made to vary between 1.5 to 6, the friction angle from 20° to 40°, and the in situ stress ratio from 0.6 to 1.0. These ranges cover about 80% of cases found in recent practice reviewed.

The parametric analyses performed with this ground model revealed another singularity. Figure 3 presents normalized response curves for tunnel springline, for distinct friction angles. These curves were found to be nearly homothetic, with the centre of similitude at the point O where $\alpha = U = 0$ ($\Sigma = 1$).

Draw through point O an arbitrary axis OP that will intersect curves for ϕ=20° e 30° at points M and N. The coordinates of these points are α' and U': use these coordinates as reference values and normalize once more each of these curves, to each respective reference value. The numerical results replotted in terms of the new abscissa U/U' and of the new ordinate

$$\lambda = 1 - \alpha / \alpha' \tag{8}$$

and are shown in Figure 4. The normalized results furnishes a single response curve for each point of the tunnel contour, which is independent of the ground friction angle. This finding provides more generality to the aimed solution. One should note, however, that the points shown in Figure 4 are related to the pre-failure conditions and that the curve fitted through them is valid for this condition only.

Non linear least-squares regression techniques were used to best fit the twice normalized numerical data. The best function fitting all data, with minimum residual mean square (varying from 10^{-3} to 10^{-5}), was found to be:

$$\lambda = 1 - (x / (P_1 + P_2 \cdot x + (1 / (P_3 + P_4 \cdot x + P_5 \cdot x^2)))) \tag{9}$$

where $x = U/U'$ and P_i are function parameters. This function seems suitable in the sense it has a definable limit for large displacements, though being clear that it should be used for interpolation within pre-failure conditions only. The parameters for crown, springline and floor of the tunnel are shown in Table I. Figures 5, 6 and 7 present these functions in graphic form, for three K_0 values, for the three points of the tunnel contour and for three normalized ground covers. They furnish the relation being sought between the stress reduction factor and the convergence displacement of a shallow tunnel.

Figure 8 furnishes the ratio α'/U' which is the slope of the arbitrary homothety axis, as a function of the tunnel ground cover. This ratio was made independent of ϕ and α' was selected, for the reduction process, in such a way as to make it independent of the ground cover.

Figure 9 shows α' as a function of the frictional resistance expressed as:

$$m - 1 = \frac{2 \cdot \sin\phi}{1 - \sin\phi} \tag{10}$$

Table I: Parameters of the twice normalized response curves.

			P1	P2	P3	P4	P5
K_0=0.6	H/D=6.0	Crown	0.7863	0.1643	5.8485	-0.9192	15.2721
		Springl.	0.3270	0.5378	2.2039	1.3714	4.4509
		Floor	0.7550	0.0791	4.4239	-2.7928	4.6785
	H/D=3.0	Crown	0.7592	0.1959	5.5995	-1.8853	18.2108
		Springl.	0.3262	0.5428	2.3007	1.1162	4.9612
		Floor	0.8201	0.0558	6.3141	-6.6624	8.7052
	H/D=1.5	Crown	0.5830	0.3175	2.9017	1.5671	5.6173
		Springl.	0.3173	0.5517	2.2334	1.0538	5.0694
		Floor	0.8312	0.0476	6.4333	-5.5703	7.8069
K_0=0.8	H/D=6.0	Crown	0.5480	0.3849	3.0527	2.9276	9.9043
		Springl.	0.4223	0.5317	3.4086	3.2167	12.6102
		Floor	0.7230	0.2316	3.5999	-2.8499	14.5303
	H/D=3.0	Crown	0.5055	0.4191	2.8752	3.6904	6.8780
		Springl.	0.4197	0.5380	3.4399	2.6640	14.5739
		Floor	0.7328	0.2234	3.8938	-3.9446	15.8681
	H/D=1.5	Crown	0.5052	0.4630	3.6081	5.5775	17.3331
		Springl.	0.4096	0.5500	3.4225	2.1201	15.7848
		Floor	0.7171	0.2256	3.6040	-2.8964	11.6415
K_0=1.0	H/D=6.0	Crown	0.4274	0.5208	4.2706	4.9524	9.1674
		Springl.	0.4416	0.4977	4.0454	5.6946	6.1024
		Floor	0.4323	0.5090	4.0270	5.0692	7.6430
	H/D=3.0	Crown	0.4097	0.5446	4.4999	5.3435	10.7461
		Springl.	0.4484	0.5015	4.6854	3.7948	10.6789
		Floor	0.4574	0.4914	4.3206	4.2243	10.4066
	H/D=1.5	Crown	0.3575	0.6118	5.1904	5.6765	17.2042
		Springl.	0.4207	0.5227	4.0515	4.5578	8.2557
		Floor	0.4258	0.4880	3.2546	4.6773	3.4674

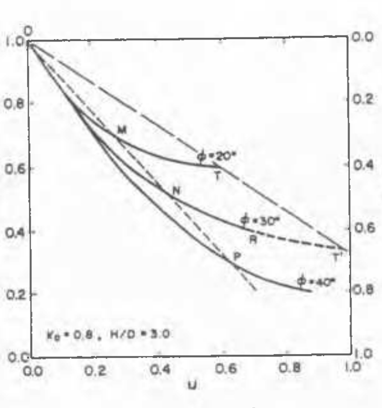

Figure 3: Normalized ground responses at tunnel springline for different friction angles.

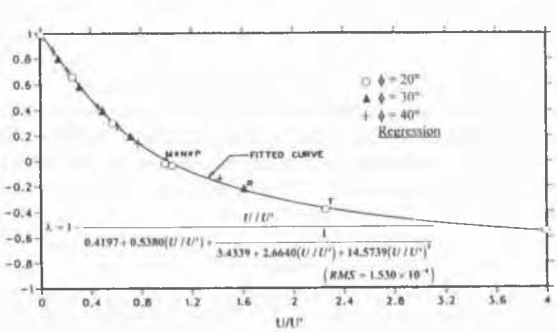

Figure 4: Twice normalized ground response for springline and for any friction angle (K_0=0,8).

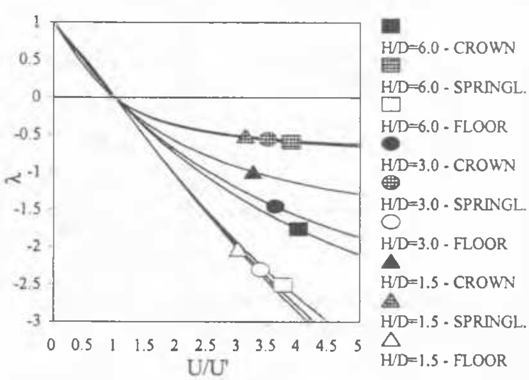

Figure 5: Twice normalized ground response for K_0 = 0.6.

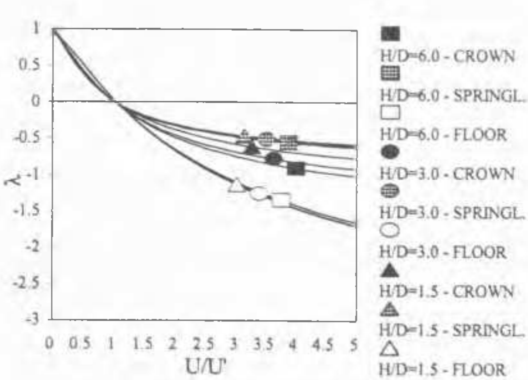

Figure 6: Twice normalized ground response for K_0 = 0.8.

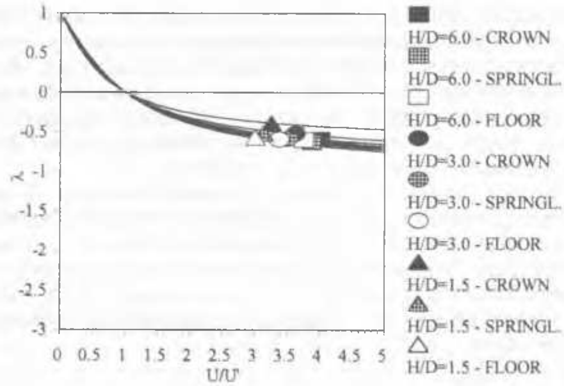

Figure 7: Twice normalized ground response for K_0 = 1.0.

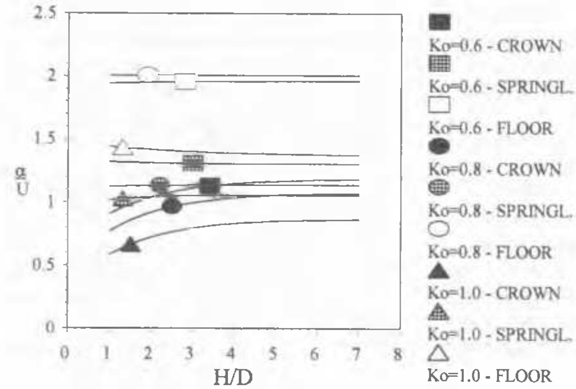

Figure 8: Slope of the arbitrary homothety axis and normalized tunnel cover.

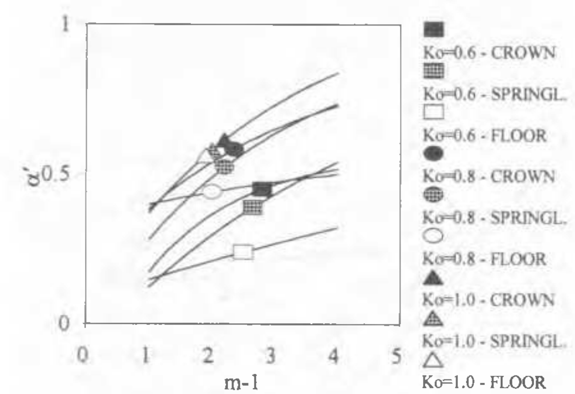

Figure 9: Relationships between the reference ground stress reduction factor and ground frictional resistance.

The set of functions shown in Figures 5 to 9 represents pre-failure non linear ground response curves, for three points at the perimeter of a shallow circular tunnel, in cohesionless ground. In other section it will be shown how to account for moderate cohesive component of strength and how to account for increasing ground stiffness with depth. A similar set of functions can be defined for undrained (frictionless) conditions.

5 GROUND CONVERGENCE AT LINING ACTIVATION

The estimate of ground stress reduction in the two dimensional model presented in the previous section requires an independent evaluation of the ground displacements around the tunnel perimeter, prior to the instant when the lining is put in contact with the soil and starts interacting with it.

Solutions available for tunnel convergence estimates were developed for deep tunnels, were numerically derived from either 3D or axisymmetric finite elements or boundary integral, assumed the ground mass to behave linear-elastically and assumed that the opening is unlined. Some solutions account for in situ stress other than unity (e.g. Niwa et al, 1979). Negro et al (1986), on the other hand, used a similar approach but for a shallow circular tunnel condition, taking into account the effect of the stress free ground surface and of the gravitational stress gradient. They used 3D finite element parametric simulations to obtain a numerically derived solution for prediction of radial displacements at the face of shallow tunnels, for K_0 values between 0.6 and 1.0 and ground covers between 1.5 and 3 diameters. This unlined tunnel solution for elastic ground was later extended to furnish radial displacement of the tunnel perimeter at points behind the tunnel face.

The extended solution furnishes the dimensionless radial displacement at tunnel crown and floor as:

$$U = a - b \cdot K_o \qquad (11)$$

and at tunnel springline as:

$$U = a - b / K_o \qquad (12)$$

where a and b are the coefficients shown in Figure 10, as a function of the relative distance to the tunnel face.

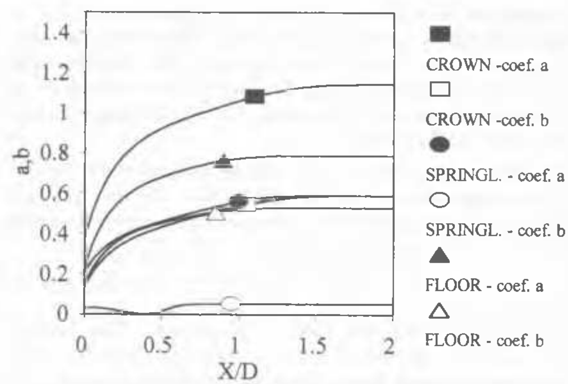

Figure 10: Coefficients of dimensionless radial displacements at points of the tunnel contour.

It is known that the presence of the lining does not affect much the displacement at the tunnel face. However, for points behind the face the influence of the lining becomes more pronounced, the extended solution tending to overpredict the elastic displacements. This is a limitation of the solution which is balanced by the assumption of linear elasticity that underpredicts the actual non linear displacements of the ground mass.

Notwithstanding these shortcomings, the extended solution was found to be consistent with Niwa at al (1979) solution. Additionally, it yielded estimates of tunnel convergence at crown, springline and floor in more than 50 actual case histories of shallow tunnels, that compared favourably with the measured displacements prior to the lining activation (Negro, 1988). Departures between predictions and measurements were observed whenever poor ground control conditions prevailed, with localized instabilities and larger losses of ground, regardless the construction method or the ground type.

A key issue in applying this simplified solution to actual problems is the definition of the point of lining activation. For NATM built tunnels lining activation occurs at the section where the lining ring is closed by shotcreteing the invert arch at tunnel floor. For TBM or shielded tunnels, lining activation occurs at the section the liner is put in full contact with the soil, where grout fills the void behind it or where the lining is expanded. Good ground control conditions normally implies in lining activation between 0.5 to 1.5 times the tunnel diameter. Larger delays in lining activation can imply in poorer control conditions, where this simplified method ceases to furnish good estimates of tunnel wall closure.

6 APPROXIMATIONS

One restrictive assumption made in the development of the ground response model refers to the zero cohesion. The influence of moderate cohesive component of strength, however, can be taken into account approximately. This can be done by adjusting the friction angle of the soil with $c = 0$ in such a way that the in situ strength defined in terms of the principal stress difference, at the tunnel axis elevation, is made equal to the strength of the actual soil with non-zero cohesion. This adjustment has been tested by numerical modelling that revealed that the best and safer approximations were obtained with the assumption that the stress difference at failure is reached by keeping the minor in situ principal stress constant and by increasing the major principal stress. For this stress path, the adjusted friction angle becomes:

$$\phi_a = arcsin\left[\frac{1+(\sigma_3/c)\cdot tan\phi}{1+(\sigma_3/c)\cdot sec\phi}\right] \tag{13}$$

Another restrictive assumption made refers to the Janbu's exponent n, which was also set equal to zero. This is the parameter controlling the rate of increase of the in situ tangent modulus of a soil deposit with depth. If n is equal to unity, the in situ tangent modulus increases linearly with depth. If n is equal to zero, a constant modulus profile with depth is found, what usually does not correspond to reality. To take into consideration the modulus change with depth, a number of numerical analyses was performed. It was found that if the ground responses at the crown and at the floor were normalized to the in situ stiffness of the ground at points located half diameter above and below the tunnel respectively, the resulting normalized ground response curves would be practically independent of the rate of increase of the modulus with the depth, thus independent of n. This artifice, therefore, allows the effect of the modulus increasing with depth to be approximately accounted for, from the results of analyses where a constant in situ stiffness is assumed, as is the case.

7 APPLICATIONS

To assess the stress release factor α for a particular tunnel where a 2D numerical analysis is to be performed, one should firstly identify the distance X from the face where the lining is activated. Equations (11) and (12) and Figure 10 furnishes the dimensionless radial displacements U at three points of the tunnel contour, at a distance X, for a given in situ stress ratio K_0. Using equations (7) and (13) the adjusted frictional resistance of the soil $(m-1)$ is found through equation (10). Figure 9 furnishes the reference stress release factors α' and Figure 8 yields the slope of the arbitrary homothety axis. From the latter, the reference dimensionless displacements U' for the three points of the tunnel perimeter are found. With the ratio U/U' for each of these points, one obtains three values of λ through Figures 5, 6 or 7. If required, interpolation of λ is made for odd K_0 values. From equation (8) one obtains, for each point of the tunnel contour:

$$\alpha = (1-\lambda)\cdot\alpha' \tag{14}$$

The stress release factor to be applied to the 2D numerical analysis of a particular tunnel will correspond to the average α value for those points of the tunnel contour. Note that the springline α value is counted twice in the averaged factor:

$$\overline{\alpha} = (\alpha_{CROWN} + 2\cdot\alpha_{SPRINGLINE} + \alpha_{FLOOR})/4 \tag{15}$$

The above procedure was applied to a dozen of tunnel cases and samples of results obtained are shown in Figures 11 and 12, that refer to the ABV NATM tunnel in Sao Paulo (Negro and Eisenstein, 1981) and to the Edmonton LRT Shielded Tunnel in Canada (Branco, 1981).

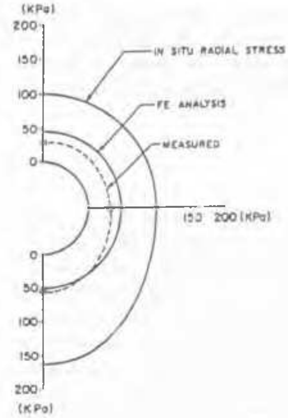

Figure 11: Calculated and measured radial stresses on the ABV tunnel's shotcrete lining, Sao Paulo (α = 53%).

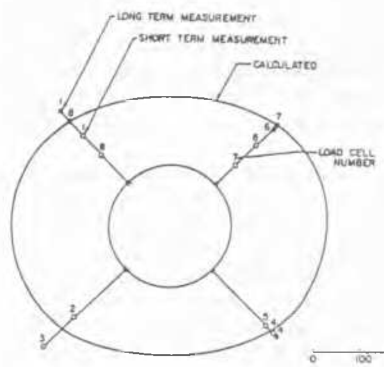

Figure 12: Calculated and measured thrusts on the Edmonton LRT Shielded tunnel primary lining (α = 62%).

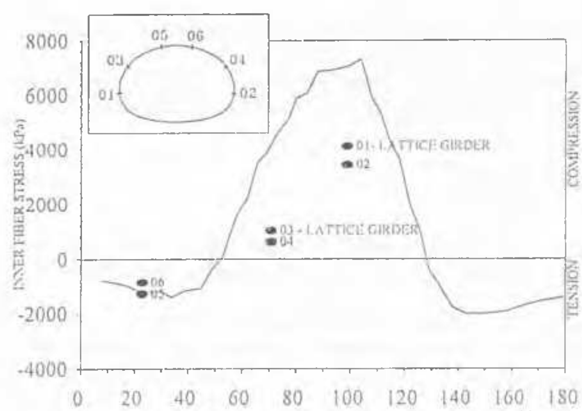

Figure 13: Calculated and measured stresses on Brasilia Metro tunnel shotcrete lining (α = 57%).

In the applications of this method to actual tunnel cases, no attempt was made to best fit the observed performances.

These tests were not back-analyses, since it was assumed that all parameters governing the tunnel response were known. These tests covered a wide range of conditions, of soil types, geometries, of construction methods. From these applications it was noted that the calculation procedure either matched or overestimated the lining loads. Similar agreement was obtained with respect to ground displacements.

After theses tests involving Lambe's type C predictions, this procedure has been used in more than a hundred of tunnel cases, as type A prediction. As an example, Figure 13 presents a comparison of measured and predicted (type A prediction) lining loads in the primary shotcrete lining of Brasilia Metro (Negro and Kochen, 1996).

The amount of ground stress release at lining activation in these applications varied from 20% to 70%, the most frequent value being around 50%, consistent with the arbitrary stress reduction proposed by Muir Wood (1975) for lining design.

8 FINAL COMMENTS

Though validated by a large number of applications, the present procedure has a number of limitations. The procedure is strictly valid for the conditions it was developed, clearly stated in the preceding sections. Its use is restricted within the limits of the variables it considers. Of course some degree of extrapolation can be exerted but is unwarranted. In any case, the method is applicable for good ground control conditions, in which any form of instability is preclude. It is, therefore, a basic requirement for this procedure application, to assess the limiting stress relaxation that brings about the collapse of the unlined tunnel. This assessment can be performed, for instance, through the Bound Theorems of Plasticity. Stress release factors greater then the limiting factor causing collapse are not acceptable.

Some construction methods, for instance the slurry pressure balanced shield, may pose difficulties in applying the present procedure. This may require an alternative method for prediction of the tunnel convergence, on account of the slurry pressure.

Finally, the procedure can be used and is being used, not only to tunnelling performance prediction, but to tunnel support monitoring analysis. In fact, tunnel convergence from extensometers, deep settlement points, inclinometers can be treated as an input into the method, that can then furnish the corresponding ground stresses. If the latter are known from lining loads measurements, then back analysis can be performed rendering ground variables sometimes difficult to asses (K_0 for instance).

9 ACKNOWLEDGEMENTS

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