Geotechnical Aspects of Underground Construction in Soft Ground – Yoo, Park, Kim & Ban (Eds)

Tunnel modelling: Stress release and constitutive aspects

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ABSTRACT: Tunnel construction in soft ground has evolved significantly over the last 20 years, especially on the matter of settlement control. This was achieved by guiding the TBM operation to control the main factors that induced soil displacements, like the face pressure and the soil-lining void closure. However, the design methods and numerical modeling procedures where not adapted to these new conditions, sometimes applying boundary conditions, constitutive parameters or state variables with no physical meaning to match field measurements. This paper presents an analysis of the basic principles of plane strain numerical modeling of tunnel construction. The literature review is followed by an analysis of the stress release factor and the effects of different constitutive models to represent the soil. The tunneling convergence and settlement trough as well as the stress paths on soil elements at the crown and at springline will be presented.

1 INTRODUCTION

Tunnel construction in soft ground has evolved significantly over the last 20 years, especially on the matter of settlement control. The routine volume loss of mechanized tunnels decreased from 6% to less than 1%. This was achieved by guiding the TBM operation to control the main factors that induced soil displacements, like the face pressure and the soil-lining void closure. This approach was effective but tended to analyze these factors individually with no account for their possible influence on the global mechanism of tunnel construction.

The same pattern emerged from tunnel design methods that are particularly focused on the surface settlements. Again, significant contributions came from this; the settlement trough has been thoroughly analyzed since its effect on buildings was addressed by Burland & Wroth (1974). Its description and quantification was of utmost importance for the development of underground construction in urban centers without jeopardizing the stability of surface structures.

Alongside of these trends, the use of the finite element method for the analysis of geotechnical problems has increased notably. However, the vastness of choices in modelling procedures, which include the constitutive model, mesh, parameters and boundary conditions, tend to produce results that are user-dependent and often limited in the aspects they can reproduce from real cases or physical models. Ergo a great deal of tunnel modelling aimed solely on predicting the settlement trough, and did so without accounting for the effect of the new construction techniques. To achieve a better prediction, boundary conditions, constitutive parameters or state variables were sometimes adapted without a clear physical meaning. This normally hindered the model's reliability and capacity to cope with the different tunneling or soil conditions.

Considering these aspects, this paper presents an analysis of the basic principles of plane strain numerical modeling of tunnel construction. The literature review presents how the basic stress release concept evolved to account for TBM tunnel construction. That is followed by two groups of analyzes: a simple linear elastic model to remark on the effects of the soil compressibility and initial stress state and a comparative analysis on the effects of different constitutive models to represent the soil. The tunneling convergence and settlement trough as well as the stress paths on soil elements at the crown and at springline will be presented.

2 LITERATURE REVIEW

The underground opening of a tunnel can be evaluated mechanically as a simple process. For an unlined tunnel the stress state on the imaginary boundary surface of the excavation is taken from the initial in-situ stress to a condition of zero normal and shear stress. This path can be done in increments that are normally called stress release factors (λ) and represent a percentage of the full path. For the case of a lined tunnel it is normally assumed that part of the stress release will develop with a free boundary (λ), as in the unlined tunnels. After the lining is installed the remaining stress (1-λ)

will be released and reach equilibrium with the lining. The soil-lining interaction and the lining rigidity will dictate the equilibrium state of the tunnel.

A series of analyzes on modeling procedures for TBM tunnels was initiated by Rowe et al. (1983), that also remarks the importance of modeling the soil's cross-anisotropic deformability to achieve better displacement predictions. Knowing the gap that exists between the excavated boundary and the lining, the partial stress release factor will be the one that induces a tunnel converge that closes that gap. From then on soil-lining interaction develops. On subsequent studies this gap parameter was revised to account for the quality of workmanship, face protrusion and other tunneling aspects. Bottom line is that a displacement criterion controlled the partial stress release factor.

The three-dimensional aspect of a TBM tunnel was attempted to be simulated by combining two plane-strain models, for the transversal and longitudinal sections of a tunnel (Finno & Clough 1985). Cohesive soils were represented by the modified cam clay model and cohesionless soils by a nonlinear pseudo-elastic law. The tunnel face in the longitudinal model was displaced until the horizontal forces were equal the measured jacking forces of the TBM. The ratio between the vertical and the out-of-plane horizontal displacement was applied in the transversal model until the horizontal displacements matched the inclinometer measurements by the side of the tunnel. From this state the first stress release was applied until the tunnel convergence closed the gap to the lining. The subsequent steps were soil-lining interaction and consolidation of the excess pore water pressure (PWP).

Abu-Farsakh & Voyiadjis (1999) tried to model the same TBM tunnel, with the same soil models, but relying less on measured parameters. The longitudinal model advanced until the specified face pressure was achieved. The outward displacement of the transversal model had an elliptical profile with a major/minor axis ratio of five and was applied until the increment of PWP on the tunnel springline was the same as the one calculated on the longitudinal section. After the correct displacement was determined a new tunnel section was modelled with a smaller diameter so that after the displacements are imposed the boundary geometry matches the excavated diameter. The stress release factors were applied along the tunnel boundary by an elliptical profile, this time with a major/minor axis ratio of 1.50. Again the final phases were the soil-lining interaction and consolidation of excess PWP.

Other studies only analyzed the tunnel transversal section, but accounted for the pressure increase due to grout injection on the tail-void. Bernat et al. (1999) modelled the TBM excavations of the Lyons-Vaise metro by calibrating the partial stress release factors of an unlined tunnel with the measured tunnel crown displacements. The soil was represented by the CJS model that account for kinematic strain-hardening. A single partial stress release was compared to a cycle of stress reduction, increase ($\lambda<0$) and reduction to account for the grout injection at the tail void.

The tunnel excavation can also be modeled directly by imposing displacements on the excavation boundary. However, there might be a case where the resultant boundary stresses are not representative of an excavation and the procedure only mimics the measured displacements. Dias et al. (2000) present a 2D model with a fixed tunnel invert and a ring plate element on the excavation boundary. The soil was modeled as a drained Mohr-Coulomb material. As the excavation advances the plate rigidity changed from TBM to grout properties. The passage of the cutter-head was simulated by reducing the soil-plate interface strength to induce sliding and by reducing the diameter of the plate ring. The TBM passage, grout injection and grout consolidation were simulated by a sequence of decrease, increase and decrease in the ring diameter.

Ding et al. (2004) analyzed the different TBM phases by combining stress release factors with special interface elements and a beam-joint discontinuous lining. A simple model and empirical relations were presented to calibrate the normal and tangential interface stiffness by the properties of the fresh and consolidated grout. Two distributions of grout pressure are also tested against measurements of an Osaka subway line. Just recently, Konda et al. (2013) modeled the different phases of the TBM by a set of normal forces on the tunnel boundary together with a full stress release (λ=1). That allowed the internal tunnel pressures to be determined with no relation to the in-situ stress. The soil was assigned the t_{ij} constitutive model on an undrained analysis.

Among these stress-based approaches for plane-strain modeling of TBM tunnels, a progression can be traced from basic stress release factors to representations of boundary pressure gradients related to the different TBM elements. As mentioned, when using stress release factors, the boundary stresses of the excavation are always connected to the gradients of the initial stress state. The gradient adjustment can be done through an asymmetric distribution of λ, as in Abu-Farsakh's model, by a combination of partial stress release factors and internal pressures as in Ding's or by a total stress release and internal pressures as in Konda's model. How to assess the real pressure gradient is still not a straightforward procedure, but it has been analyzed at the excavation face (Bezuijen et al. 2006a), along the TBM (Bezuijen 2009) and on the lining (Bezuijen et al. 2006). For the practice of tunnel design this is an important understanding in order to create numerical models with consistent physical significance.

Regarding the constitutive model for the soil, there is also no agreement in geotechnical engineering in general. Compromise is always necessary over the aspects to be modeled and the availability of soil tests to assess the required parameter. Even so, Shirlaw (2000) presents how when tunneling through the same geological strata, using the same tunneling method and crew, the settlements can change over 2.5 times.

Table 1. Constitutive parameters.

	LE	MC	MC.D	HS	HS.s
E (MPa)	45	45	45	–	–
ν	0.2	0.2	0.2	0.2	0.2
φ (°)	–	37	37	37	37
ψ (°)	–	–	7	7	7
E_{50}^{ref} (MPa)	–	–	–	45	45
E_{oed}^{ref} (MPa)	–	–	–	45	45
E_{ur}^{ref} (MPa)	–	–	–	135	135
m	–	–	–	0.466	0.466
G_0^{ref} (MPa)	–	–	–	–	111
$\gamma_{0.7}$	–	–	–	–	$1.25\ 10^{-4}$

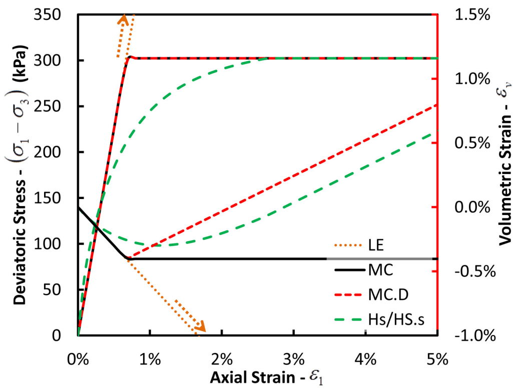

Figure 1. Triaxial test simulations with various constitutive models (see also text).

For TBM tunnels several aspects cannot be determined accurately before the construction. Nevertheless the soil model is a necessary input, and how their characteristics will affect the results of a tunnel analysis is something that will be analyzed hereafter.

3 METHODOLOGY

A set of drained plane-strain finite element calculations were performed to analyze the patterns of boundary stress and the effects of the constitutive models on the response on an unlined tunnel modeled by increments of stress release. The stress path, tunnel convergence and surface settlements were assessed on the models. The calculations were performed on Plaxis 2D v2012.01 software.

The first analysis considered a circular tunnel (8 m in diameter; 30 m deep at springline) and a dry linear elastic soil (Young's modulus E=50 MPa; Poisson's ratio $\nu = 0.30$; Volumetric weight $\gamma = 20\,kN/m^3$). The effects of variations of the coefficient of earth pressure at rest (k_0) and ν were assessed. A second group of models considered the same layout and $k_0 = 0.50$. The soil was analyzed with four constitutive models: LE – Linear Elastic; MC – Linear elastic perfectly-plastic with a Mohr-Coulomb failure criterion and a non-associated flow rule; HS – Hardening Soil and HS.s – Hardening Soil with small-strain stiffness. The parameters were obtained from the empirical correlations from Brinkgreve et al. (2010) for sand at a relative density of 0.75, but with the same volumetric weight of the first analysis (Tab. 1). For the formulations of the models, the reader is referred to Brinkgreve et al. (2013).

There is a recurrent discussion on whether soil dilatancy should be considered when employing the Mohr-Coulomb model. The formulation implied that on drained analyzes, shear strains can develop indefinitely without reaching the critical state. To evaluate this aspect for tunnel modeling the Mohr-Coulomb model was employed considering dilatancy (MC.D) and with zero dilatancy (MC). A triaxial test was simulated for each constitutive model by the Soil Test software in the Plaxis Program and the results are on Figure 1.

The hyperbolic stress-strain relation of the Hardening Soil models (Hs and Hs.s) can be seen on the upper curves. All models, with the clear exception of the LE, converge to the Mohr-Coulomb failure criterion, corresponding to a deviatoric stress of 300 kPa. The dilatant response of the MC.D model can be seen in contrast with the MC. Both the Hardening soil models together with the MC.D present unrestrained dilatancy as no critical state is reached and no dilatancy cut-off was assigned.

4 RESULTS AND DISCUSSION

The results will be divided by the linear elastic analysis and the comparison of different constitutive models. As discussed, with the stress release factors the boundary stresses remain related to the original stress state, and not the constitutive model. That also implies that when that state is not isotropic ($k_0 \neq 1$), both normal and shear stresses will be acting on the boundary (Fig. 2).

4.1 *Linear elastic analyses*

Considering k_0 equals 1, 0.5 and 2, the increments of isotropic (p) and deviatoric (q) stress invariants on the tunnel crown and springline were evaluated (Fig. 3). For the isotropic state ($k_0 = 1$) the increment is predominantly of deviatoric stress. For the other states the response is distinguishable on whether the normal stress is the initial major (σ_1) or minor (σ_3) principal stress. The initial normal stress is the minor principal stress on the tunnel crown for $k_0 = 2$ and on the tunnel springline for $k_0 = 0.50$. In those cases the tunnel excavation, a reduction in the normal stress, represents a decrease in σ_3 and an increase in the hoop stress, which is σ_1. Therefore a path of increase of deviatoric and mean stress is the result. On the opposite conditions, the decrease of σ_1 and the increase in σ_3 result in negative increment of isotropic stress.

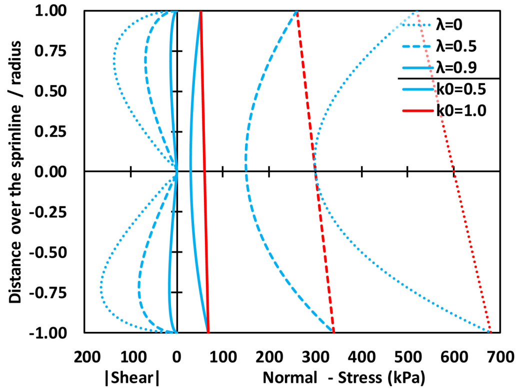

Figure 2. Boundary stresses. The type of line represents different stress release factors and the color represents the different coefficients of earth pressure at rest.

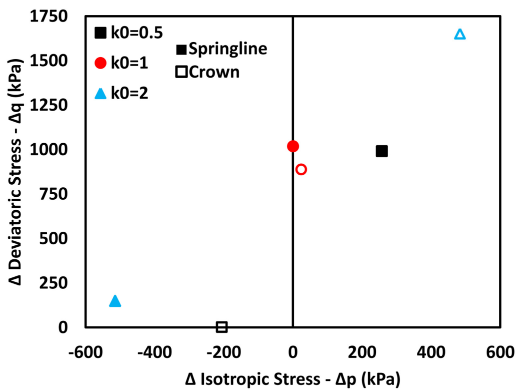

Figure 3. Increments of p and q for different k_0 values.

The difference on the initial stress state also affects the volume loss measured by the tunnel convergence and by the settlement trough (Fig. 4). The increase in the horizontal stress increases the tunnel convergence without significantly affecting the settlement trough. It is important to notice that the notion of equivalency between the tunnel convergence and the settlement trough can only be applied in undrained conditions, when the soil is practically incompressible ($\nu = 0.5$). However it is common in practice to assume this equivalency even for compressible soils.

For a linear elastic analysis, being all the other parameters fixed, a relation can be traced between the ratio of volume losses measured on the surface and by the tunnel convergence and the Poisson's ratio as it can be seen on Figure 5. Leca & New (2007) reported that the so called deformation dampening can also be a consequence of the presence of a stiffer or dilating soil over the tunnel.

4.2 *Constitutive models analyses*

With the conditions described in the methodology, each analysis was conducted in stress release

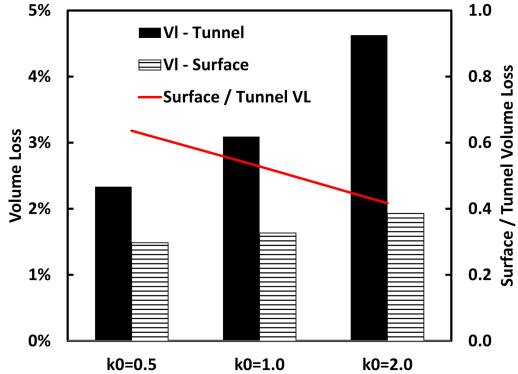

Figure 4. Volume losses for different k_0 values. Bars correspond to the left Y-axis, the red line with the right.

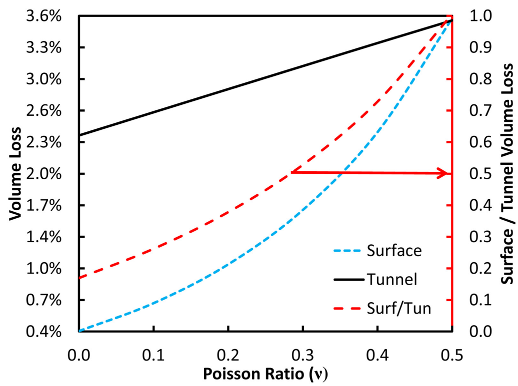

Figure 5. Volume losses as a function of the Poisson's ratio.

increments of 10% the original state until the solution did not reach convergence. The maximum λ for the different models was: MC 0.7; MC.D 0.8; HS 0.9; HS.s 0.8. The linear elastic analysis does not account for a failure criterion. For the sake of comparison the surface settlements are presented for $\lambda = 0.7$ (Fig. 6) while the volume losses (Fig. 7) and the stress paths (Fig. 8) are presented from $\lambda = 0.1$ to $\lambda = 0.7$.

The typical problem with the settlement trough of 2D models is evident in the LE results: the trough is shallow and narrow. It can be observed that when plastic deformations are taken into account the trough becomes deeper. On the other hand, when hardening is considered the trough becomes wider. The consideration of small strain stiffness reduces the maximum settlements without affecting the trough extent. The HS settlement troughs were reported to be significantly deeper than the MC trough for higher overconsolidation ratio (OCR) values (Vermeer et al. 2003). The parameters of the typical Gaussian model for the settlement trough are on Figure 6.

As discussed in the previous sections, drained analyzes do not hold the equivalency of volume loss measured on the surface and on the tunnel boundary. There is a direct correlation between stress release and

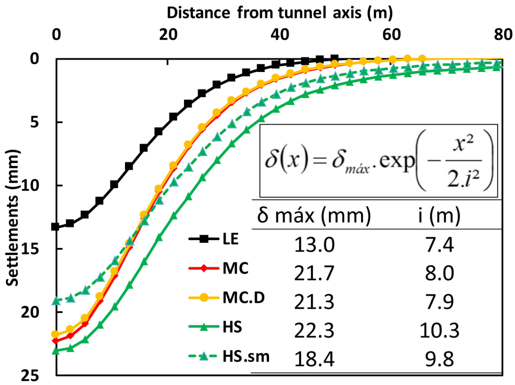

Figure 6. Surface settlements for different constitutive models.

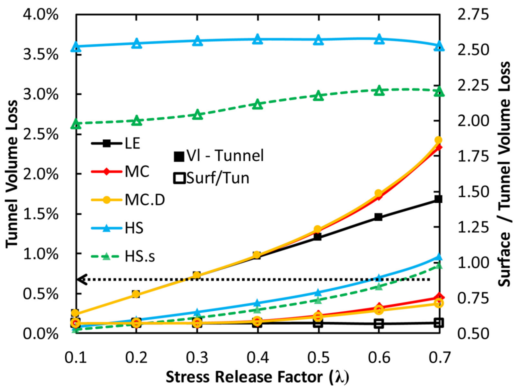

Figure 7. Tunnel convergence and surface/tunnel volume loss ratio for different constitutive models. All the lines that are intercepted by the dotted arrow should be read on the left axis.

the tunnel volume loss. For the MC and MC.D models the relation deviates from the linear elastic from $\lambda = 0.5$ on, as the model is actually linear elastic until plasticity is reached. On the other hand, the Hardening Soil models that present a hyperbolic stress-strain relation deviate from the linear elastic model from early stages and present always smaller volume losses at the tunnel level.

The ratios between surface and tunnel volume losses are also on Figure 7. Again there is a clear distinction between the LE, MC and MC.D models and the Hardening Soil models (HS and HS.s). As presented in Figure 5, for the LE model the ratio is constant and smaller than 1 for all values of λ. The MC models present a ratio increase from $\lambda = 0.5$ on, when both volume losses increase, but on a higher pace on the surface. In contrast to that, both HS models present ratios above 2 for all values of λ, as the volume loss on the surface was higher than on the tunnel.

From the previous section it is understood that the final stress boundary condition results in zero vertical stress on the tunnel crown and zero horizontal stress on the tunnel springline, inducing deviatoric and positive

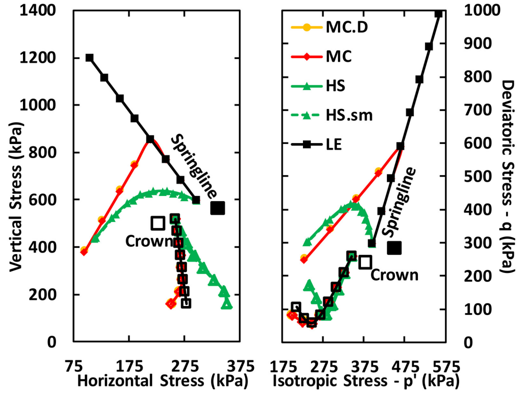

Figure 8. Stress paths for vertical x horizontal stress and isotropic x deviatoric stress for different constitutive models.

isotropic stress increment on the springline and a negative isotropic stress increment on the tunnel crown as $k_0 = 0.5$. The stress path on the springline is of typical loading, leading to failure. For the MC and MC.D materials the path is the same until the Mohr-Coulomb failure line is reached. From there on there is a decrease in the vertical stress that reduced the deviatoric stress along the failure line. For the Hardening Soil models the decrease in the horizontal stress is not followed by the increase in the vertical stress, therefore there is a negative increment on isotropic stress that leads more directly to the failure line.

5 CONCLUSION

From the literature review it was possible to demonstrate how the concept of plane-strain modeling of TBM tunnel is evolving. The gradient of boundary pressure due to a partial stress release factor holds a relation to the initial stress state and not to the TBM elements acting on that section. However, the basic linear elastic analysis presents how the initial stress state is important for the stress paths that are imposed in the soil elements due to the excavation. The effects of the soil's compressibility on the relation between the volume losses measured on the surface and on the tunnel boundary were also assessed by the linear elastic analysis.

When different constitutive models were considered, and a maximum strength was assigned to the soil, the tunnels were not stable over the whole path of stress release. The surface settlements and their relation to the volume losses measured on the tunnel boundary were significantly different among different models, especially between the models with linear and nonlinear stress-strain relations. The recurrently discussed consideration of the soil dilatancy when applying the Mohr-Coulomb model did not results in significant differences for the results.

This clear understanding of the modelling conditions and implications is very important for the practice of tunnel design, so that the analyses can reach reliable predictions with consistent physical significance.

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