Three-dimensional numerical simulation of a tunnel excavated using NATM – Brasilia underground case

Simulation numérique tridimensionelle d'un tunel excavé par la méthode NATM – Metro de Brasilia

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During the first half of the 90's a subway tunnel was dug in porous clay using NATM in Brasilia, the Brazilian capital city. The main observed features were excessive settlement and an unexpected trend, with displacements increasing from the tunnel crown towards the surface. Many unsuccessful attempts to simulate this behaviour using two-dimensional numerical analyses have been reported. The first two authors, using some numerical artifices to simulate the structural collapse of Brasilia's porous clay, obtained partial success. This paper presents a three-dimensional numerical analysis using the FEM, in which the construction sequence of Brasilia's underground was carefully matched. The soil behaviour was simulated using elastoplastic (Drucker-Prager) and critical state (Cam clay) models. The Drucker-Prager model was unable to reproduce the observed pattern, even when a non-associated flow rule was adopted. Compatible results were obtained with a three-dimensional analysis using the Cam clay model. Two-dimensional analyses, even with the Cam clay model, are unable to reproduce the registered results. These later analyses cannot account for the load transfer along the excavation direction. This arching effect, together with the appropriate constitutive model, seem to be the key to reproduce the observed field behaviour.

Pendant la premièrc moitié des années 90, ont a fait construire à Brasilia, capital du Brésil, un tunnel de metro en argile poreuse par la méthode NATM. Les principales caractéristiques observées ont éte les grandes deformations et une tendence d'augmenter les deplacements en direction de la surface. Plusiers tentatives de reproduir ces resultats ont éte publiés en utilisant des analyses numériques bidimensionlles, même sans obtenir succês pour les resultats. Les premieurs auteurs de cet article ont réussi parciellement dans cet but en utilisant des artifices pour simuler la proprieté collapsible de l'argile poureuse de BrasiliaDans cet article, on present une analyse numérique tridimensionelle qui represente d'une façon trés precise les etapés d'excavation adoptées dans la construction du tunnel de Brasilia. Ont a utilisé des modèles elastoplastique (Drucker-Prager) et des états critiques (Cam clay) pour simuler le comportment du sol. Le modèle de Drucker-Prager est incapable de simuler le comportment observé, malgré la consideration d'une loi de flux non-associée. Comportment compatible par rapport au observé a éte obtenu en utilisant le modèle Cam clay et une analyse tridimensionelle sans tennir en compte la simulation du colapse structural. Analyses bidimensionelles, même en utilisant le modèle Cam clay, ne sont pas capables de reproduir le comportment observé due à leurs incapacités de simuler le transfer de charge vers la direction de la excavation. Cette constatation a montré être un facteur determinant, ensemble avec le modèle acopté, pour la simulation du vrait comportment au chantier.

1 BRASILIA UNDERGROUND

The high cost of urban space has significantly increased the demand for tunnels in big urban centres. In many countries, tunnels in soils and rocks are constructed using the New Austrian Tunnelling Method (NATM). More than 300 km of tunnels in hard rock and over 40 km in soft soil have been constructed with flexible shotcrete lining in Brazil since 1970.

An underground transportation system was planned in Brasilia to link the administrative centre of the Brazilian capital to a few suburban cities. It totals 42 km and is divided in several sections. The main section, in the so-called South Wing of the Pilot Plan (PP) area, was excavated in clay using NATM and is 7.2 km long.

The regional geology and geomorphology has been described in detail by Macedo et al. (1994). It is covered by a layer of lateritic soils locally known as porous clay. Under the porous clay, layers of residual soils from slate, mostly silt and sand, may be found at the tip of the South Wing. A suspended water table is also found in this region. Elsewhere the porous clay overlays a sequence of interlayered metasiltstones and quartzites, that geologists call metarhythmites. Due to the high permeability of this fractured metarhythmites, no water table is found in this region.

Most of the tunnel in the South Wing of Brasilia was excavated in the domain of the porous clay. The equivalent diameter of the tunnel is 9.8 m and the overburden is about 10 m in average. The tunnel design was based on tunnel experience in soft porous clay in Sao Paulo (Negro et al. 1996). Initial studies predicted maximum settlement in the 60-80 mm range. Although high, these were not expected to cause any problem, since the tunnel is excavated in a green field, dividing two large avenues and there are no major structures, except a few viaducts and petrol stations founded in deep piles.

However, observed displacements at the start of construction were two to three times higher than those predicted. In some sections, settlements in the order of 450 mm were registered, amazingly without any signs of instability. Another striking feature was a settlement amplification towards the surface. Settlements above the tunnel crown were smaller than those registered at the surface. A complete analysis of these displacement patterns and field tests performed during construction may be found in Ortigão et al. (1996).

The porous clay shows a very peculiar and problematic behaviour, which became the focus of several researches at the University of Brasilia (Camapum de Carvalho et al. 1998). Its main characteristics are high void ratio (1.0-3.0), low unit weight (11-15 kN/m^3) and well-defined micro and macro structures. Such structures consist of aggregations of clay and/or sand particles bonded by clay bridges with iron oxide as a stabilizing component. That is the result of intense weathering and leaching processes to which tropical lateritic soils are submitted.

From the engineering point of view, the so-called porous clay presents very low bearing capacity with number of blows counts (N-SPT) in the standard penetration test in the range of 1 to 4. Most importantly it exhibits a highly contractile behaviour. It may experience high volumetric decrease (up to 10%) when either saturated or disturbed. This behaviour has been identified as a kind of progressive structural collapse.

2 NUMERICAL ANALYSIS

Several attempts to simulate numerically the observed displacement pattern of Brasilia underground have been reported (Kochen & Negro Jr. 1996). So far these analyses were all two-dimensional under the hypothesis of plane strain conditions. Despite some success in reproducing the surface displacements, most have failed to reproduce qualitatively the displacement profile with depth.

Some success has been reported by the authors when structural collapse was introduced into the 2D analyses (Farias et al. 1998). This was done by means of a scheme similar to the one used for collapse due to saturation (Farias 1993). However this method required the intervention of the user to trigger the collapse mechanism or the introduction of an automatic collapse criterion, which is difficult to define.

A displacement pattern similar to that observed in Brasilia was reproduced in laboratory tests with 1g models using very low density kaolin samples (Nakai et al. 1997). The results were successfully simulated using 3D FEM analyses with a sophisticated constitutive model.

2.1 *Simulation of the Excavation Process*

In the present study, a full three-dimensional analysis was performed and no collapse mechanism was explicitly introduced. The constitutive model was expected to account for the contractile behaviour of the soil during the excavation process. Figure 1 shows the finite element mesh used. It has 2828 brick elements of 8 nodes with 2x2 Gaussian integration and 3435 nodes, totaling 10305 equations. The mesh is divided in 14 longitudinal segments of variable size. At the central zone their length corresponds to ¼ of the tunnel diameter (*D*). This is a relatively coarse mesh due to memory and time limitations to run in a personal computer.

Numerical simulation of the excavation tries to match as closely as possible the real excavation sequence. This is depicted in Figure 2, where the upper part shows 4 typical cross sections, and the lower part shows the excavation advance along the longitudinal direction. The arrow shows the excavation direction and the double letters indicates the typical cross sections in the regions representing a segment or block of finite elements. In a first stage, the tunnel crown and sides are excavated (Figure 2b). In the next stage, the excavation front progresses 4.8 m into a second block of elements, while the bench and invert of the first block are excavated at the same time that lining in the roof and sides are activated (Figure 2c). Then the excavation of the roof and sides advances another 4.8 m into a third block, while bench and invert excavation plus tunnel lining are performed in the second block, and the invert lining is closed in the first block. The process advances in this way in 14 stages up to the final section at the end of the mesh.

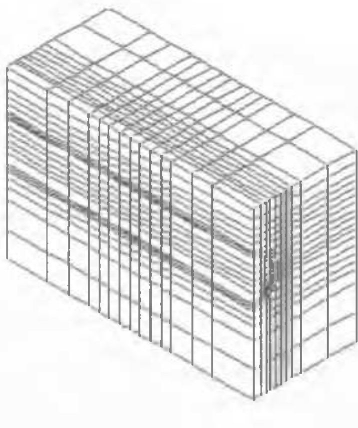

Figure 1. Three-dimensional finite element mesh.

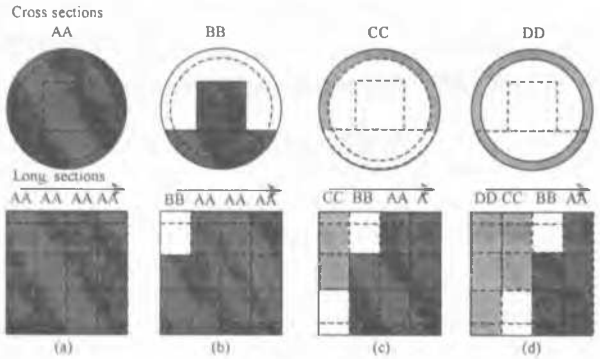

Figure 2. Illustration of excavation sequence.

The simulation used the finite element code ALLFINE (Farias 1993). Elements in the excavated areas are deactivated and the mesh is internally renumbered and optimised. Lining construction is simulated by reactivating the appropriate elements with new material properties (concrete). Stresses in deactivated elements are converted into forces along the excavation boundaries. The total force vector is divided into smaller portions and applied incrementally. It was decided to fix the number and sizes of load increments and to use a purely tangential solution scheme in order to control the total analysis time. The norm of the residual forces with respect to the total applied forces was monitored and never exceed 1% in the relevant stages.

2.2 *Models and Properties*

The influence of the constitutive relation for the soil was investigated for the following models: (a) linear elastic; (b) elastic perfectly-plastic Drucker-Prager and (c) modified Cam clay. The concrete lining was considered linear elastic with Young Modulus of 10 GPa and Poisson ratio equal to 0.3.

Definition of soil properties was based on several in situ tests, laboratory tests and previous back-analyses of instrument data. Initial stresses were generated from geostatic conditions, using the unit weight of the porous clay equal to 15 kN/m^3 and a coefficient of earth pressure at rest equal to 0.5.

For the implemented version of the Cam clay model the input parameters and values were: Poisson ratio compatible with initial stress; compression index, $\lambda/(1+e_o)$=0,175; recompression index, $\kappa/(1+e_o)$=0,019; friction angle at critical state, ϕ_{cs}=32°. The adopted value for the compression index must be interpreted as an average value since it is extremely variable with stress level due to the collapsible nature of the porous clay.

To simulate a cohesion of 6 kPa, an equivalent shift in the mean stress axis (p) is introduced by adding to the initial stresses a hydrostatic field with $\sigma = c.\mathrm{cotg}\phi$. That is counterbalanced by an equal value of suction in the initial pore pressures to keep the equilibrium condition of the initial stresses.

The Elastic Modulus varies with mean stresses in the Cam clay model according to the expression:

$$E = \frac{3(1-2\nu)(1+e_O)}{\kappa} p \qquad (1)$$

In order to ensure the same initial properties for all models, the value of Young modulus for the linear elastic and Drucker-Prager models were computed from Equation 1 with $p=p_o$ for the initial stress. In this way, this parameter was made to vary linearly with depth.

For the Drucker-Prager model the strength parameters were compatible with a cohesion of 6 kPa and friction angle of ϕ=32°. It was used a non-associated flow rule with dilatancy angle, $\psi=0$. That was necessary to avoid undue dilatancy during failure, since it was known that the behaviour of Brasilia porous clay is contractile.

3 NUMERICAL RESULTS AND INTERPRETATION

The surface settlement trough at the end of the excavation process is shown in Figure 3 for different constitutive models. Note that displacement forecasts by the linear elastic and Drucker-Prager models did not differ much. This indicates that few zones actually reached ultimate strength. In fact most of the observed failure zones were in the bench area, what does not reflect much in the surface. The displacement forecast with the Cam clay model has the same order of magnitude as observed displacements. The shape of the trough is a bit odd due to boundary conditions and model as well. In retrospect it could be said that the lateral extension of the mesh of *2.5D* is too close to the centreline. Also the vertical displacements at the lateral boundaries were fixed, producing that V shaped trough. However, the authors believe that the Cam clay model itself must respond in part for that, expanding too much the displacements laterally, what was not observed with other models.

Displacements of a surface point in the centreline against tunnel face advance are shown in Figure 4. The face distance (Z_1) is normalized by the tunnel diameter (*D*). Movements start when the tunnel face reaches a distance of about *3D* before the control section and stabilize about *3D* after passage of the excavation front. That is in good agreement with empirical data and numerical results reported by several authors (Negro et al. 1996).

Figure 5 shows the final vertical displacement profile with depth Y (normalized by the soil overburden, H) in the centre of the mesh. It is interesting to note that the displacement pattern for the elastic linear and Drucker-Prager models is such that displacements decrease from the tunnel crown to the surface. Meanwhile, for the Cam clay model, there is an amplification of displacements towards the surface similar to the observed displacement pattern in Brasilia underground excavation.

The behaviour produced by the Cam clay model is related to mechanisms of load transfer along the longitudinal direction and to the fact that stiffness is related to the mean stresses in this model, specially the elastic portion according to Equation 1.

Load transfer in the longitudinal direction is caused by soil arching in the unsupported zone in the tunnel roof between the excavation front and the concrete lining 4.8 m behind. This is shown by monitoring both the mean (*p*) and deviatoric (*q*) stresses for a line of nodes just above the tunnel lining, such as in Figure 6. The upper bar in this figure shows the free span gap during excavation of block 8, when the excavation face is at section 9. The lines show the mean stress values at the end of excavation of blocks 7, 8 an 9 and also the geostatic mean stresses. When the excavation front reaches a given block of elements, the mean stresses a little ahead of the tunnel face increase above the geostatic level, while decreasing in the unsupported zone. Besides, mean stresses also increase immediately behind the supported area due to the higher stiffness of the concrete lining.

Similar figure may be plotted for the deviatoric stresses (*q*). It was also observed an increase in *q* values ahead of the tunnel face, while it decreases strongly in the unsupported zone. However, there is not much change of the deviatoric stress behind the tunnel lining, since distortion strains are more restricted in this zone. Thus, for points above the tunnel lining, stresses in the *p-q* plane describe a Z-like stress path during the three-dimensional excavation process. This stress disturbance is obviously less significant in the surface zone away from the tunnel excavation. Therefore there is a stronger relative increase in stiffness in the zone immediately above the tunnel. That causes the displacement amplification towards the surface not only for stabilized settlements, but also in the pre-convergence stage. This is nicely shown in Figure 7, which depicts the evolution of the displacement profile with depth in section 9 for excavation stages behind this section (block 7), reaching the section (block 8), passing the section (block 9) and finally stabilized near the end of excavation (block 13).

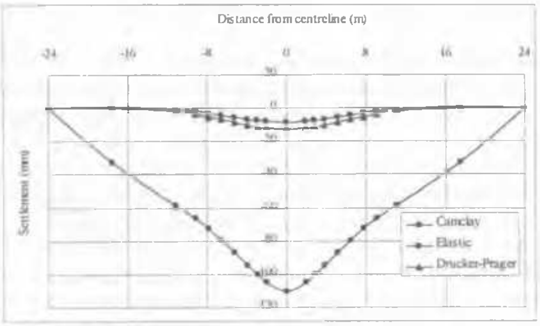

Figure 3. Stabilized settlement trough.

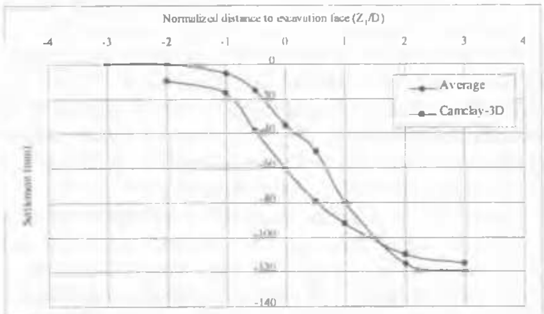

Figure 4. Stabilized longitudinal settlement profile.

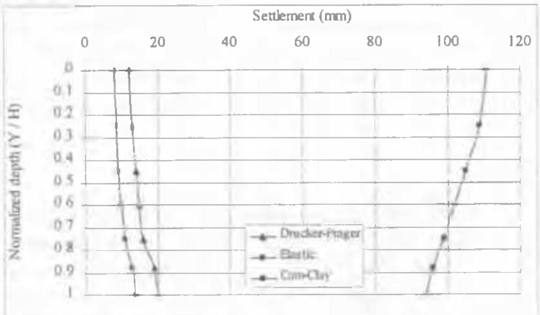

Figure 5. Displacement profile with depth.

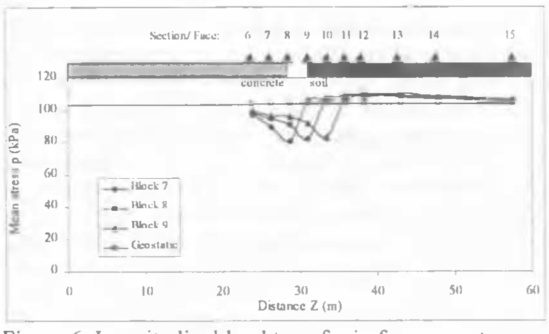

Figure 6. Longitudinal load transfer in for mean stresses (*p*).

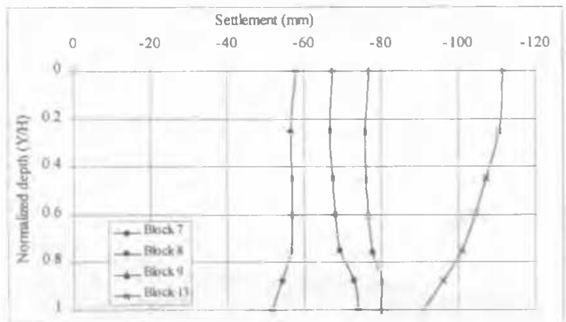

Figure 7. Evolution of displacement profile with depth.

4 NUMERICAL AND OBSERVED DISPLACEMENTS

During the excavation of Brasilia underground, displacements were monitored in several sections along 628 m between stations PP2 and PP3. Surface displacements were monitored topographically with surface marks and settlements with depth by means of extensometers and inclinometers. Internal tunnel convergence was also monitored. Some of these recorded measurements are compared with predictions of the Cam clay model.

Figure 8 shows the Cam clay numerical results and some experimental surface settlement troughs. Numerical results are in good agreement with the average trough. Displacements in two sections (S4104 and S4294) for which complete instrumentation was available are also shown. Maximum displacements in these sections were 150 and 170 mm, what is higher than observed average and computed numerical values. That is because the unsupported span between the excavation face and the tunnel lining in these sections were 7.05 and 8.10 m, respectively.

Evolution of maximum vertical displacement at surface with excavation advance is shown in Figure 9. There is a good overall agreement between numerical and observed results, although observed pre-convergence was less than forecasted.

Finally the vertical displacement profile with depth is shown in Figure 10 for the numerical results and for sections S4104 and S4294, where these readings were available. Note the qualitative agreement for the computed and observed settlement amplification. Moraes Jr. (1999) showed that the quantitative values are proportional to the length of the unsupported zone. Although this proportion is not linear, if the observed values were affected by factors 4.80/7.05 and 4.80/8.10, respectively, all curves would almost match.

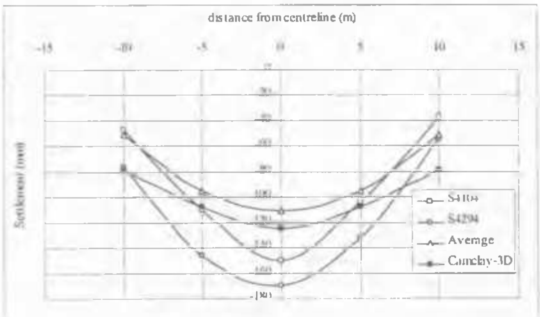

Figure 8. Observed and numerical surface settlements troughs.

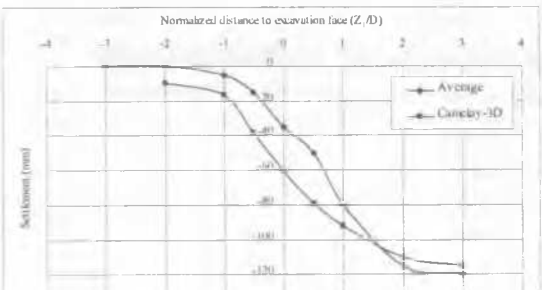

Figure 9. Surface displacements versus tunnel advance.

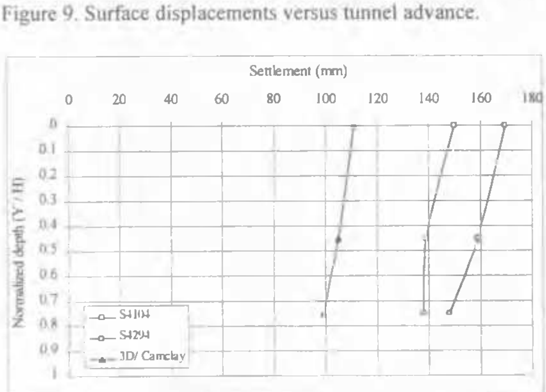

Figure 10. Displacement profile with depth.

5 CONCLUSIONS

A 3D FEM back-analysis of the tunnel excavation of Brasilia underground using different constitutive models was presented. The main observed features include high surface displacements without face instability and, most important, a settlement amplification from the tunnel crown towards the surface.

Good overall qualitative and quantitative agreement with observed displacements was accomplished with the Cam clay model. However, this model showed some trend to extend the zone of influence of the excavation further than that observed both laterally and longitudinally. Besides a constant value of the compression index (λ) was used in this analysis, what can not account for the progressive collapse of Brasilia's porous clay. Nevertheless, it is still amazing the good performance obtained with this model.

Of paramount importance to the success of the simulation was the ability to produce a longitudinal load transfer and to incorporate that into the model response.

Load transfer due to soil arching in the unsupported zone, only can be appropriately reproduced with full three-dimensional analyses. That explains why most previous analyses, including those which used the Cam clay models, failed to reproduce the observed displacement pattern.

Besides a proper simulation of the three-dimensional process, the constitutive model is fundamental in the analyses. The good prediction using the Cam clay model was credited to the fact that it incorporates the stress changes due to load transfer by changing the soil stiffness accordingly.

ACKNOWLEDGEMENTS

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