

LARGE SETTLEMENTS DUE TO TUNNELING IN POROUS CLAY

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ABSTRACT

The Brasilia underground transportation system involves 6.5 km of shallow tunneling in a soft red soil known as the porous clay that overlies harder residual soils. These materials are inter-layered saprolites from sandstones and silstones or residual soil from slate.

A site investigation programme including laboratory and in situ testing was carried out to obtain design parameters.

Settlements observations indicated that surface settlements were two to three-fold the initially predicted value, although no indication of excavation instability was observed. Another striking feature was settlement amplification between the top of the excavation and the surface by a factor of two or three, which has never been observed to the writers' knowledge. This occurred due to the collapsible nature of the porous clay that presents a considerable amount of volume change as the tunnel face passes.

INTRODUCTION

The Brasilia underground transportation system is a US \$ 650 M project, 42 km long, linking the south wing of the city to the suburbs or the satellite towns (figure 1). It encompasses 6.5 km of tunneling employing the NATM (New Austrian Tunneling Method) and over 4 km of cut-and-cover slurry-walls false tunnels.

For the south wing of Brasilia a 9.6 m diameter tunnel was designed. It was based on the bulk of Brazilian tunneling experience gathered in more than 30 km of NATM tunneling in soft

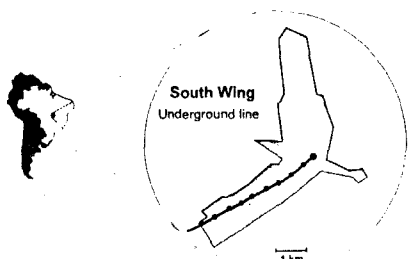


Fig. 1 - Situation de l'ouvrage.

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porous clays of Sao Paulo (Negro et al, 1992). Therefore, design was carried out using parameters derived from the Sao Paulo porous clay, which was supposed to be very similar to the Brasilia soil. Initial studies forecasted maximum settlements in the 60 to 80 mm range. These large values were not expected to cause problems, because only a few nearby structures exist, and most of them are founded on deep foundation.

At the beginning of the construction it was observed that the measured displacements were two to threefold the predictions, although the excavation face did not present signals of instability. A striking feature was displacement amplification that took place between the tunnel crown and the surface. At the tunnel crown 50 to 60 mm vertical displacements were observed, but at the surface the values were in the 150 to 200 mm range. It was soon verified that the collapsible characteristic of the Brasilia clay was unique and very close field surveillance was necessary.

A field investigation programme was carried out to reassess design parameters and employed, Marchetti dilatometer (DMT), Ménard pressuremeter (PMT), piezocone (CPTU) and horizontal plate loading tests (PLH). Block samples were obtained in test pits excavated above the water level and a series of triaxial and oedometer tests were carried out.

This paper summarizes the results of the site investigation and monitoring of displacements that reached 400 mm at the surface, the largest value ever observed in Brazil.

DES TASSEMENTS IMPORTANTS DUS AU PERCEMENT D'UN TUNNEL DANS L'ARGILE POREUSE

Le réseau des transports souterrains de Brasilia comprend 6,5 km de tunnel, peu profond, percé dans un sol rouge tendre dit "argile poreuse" recouvrant des sols résiduels plus durs. Ces matériaux sont des saprolites interstratifiés venant de grès ou de limons, ou bien des sols résiduels de schistes.

Un programme de reconnaissance du site comprenant des essais en laboratoire et in situ a été réalisé pour obtenir des paramètres d'étude. Des observations de tassement ont montré que les tassements en surface étaient de deux à trois fois plus importants que prévu initialement, bien que aucune indication d'instabilité du creusement n'a été constatée. Un autre aspect remarquable était l'augmentation des tassements entre le haut du creusement et la surface par un facteur de deux à trois, jamais observée auparavant à la connaissance de l'auteur. Ceci s'est produit à cause du caractère affaissable de l'argile poreuse qui présente un changement considérable en volume au passage du front

GEOLOGY AND SITE CONDITIONS

Regional geology and the geomorphology of Brasilia have been described by Berberian [1], Carvalho et al [2] and in detail by de Mello et al [3]. The region is flat, as a characteristic of the central plateau highlands. It is covered by a layer of red latosols and lateritic soils designated here by **porous clay**, overlying residual soils from slate or a sequence of interlayered metasilstones and quartzites, that geologists prefer to call **metarhytmites**. All encompassing the **Paranao** formation of the upper pre-Cambrian age.

Climates alternates from a 6 month rainy season to a very dry winter, leading to laterization process of leaching soluble salts at the top of the porous

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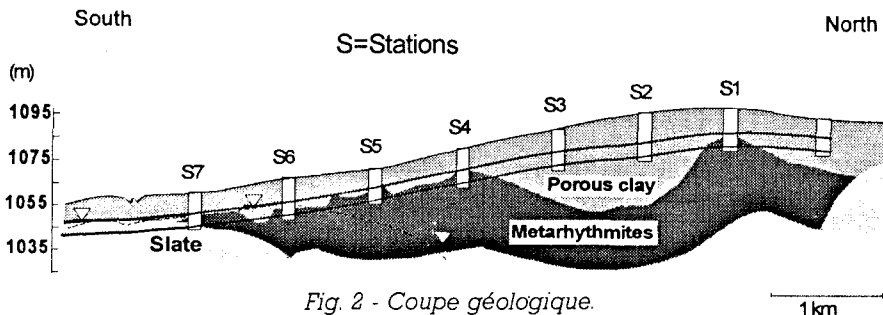


Fig. 2 - Coupe géologique.

clay and deposition below. This is responsible for the large amount of pores at the top of the clay layer resulting in high void ratios, low unit weights, and high permeability.

The soil profile was initially investigated by 65 mm diameter 20 to 30 m spaced boreholes in which SPT's were carried out at every metre along depth. The porous red clay is 8 to 20 m thick, the SPT index is low, varying from 2 to 3. The very narrow range of SPT blowcount in this clay precludes attempts to correlate deformation moduli with this parameter, signaling to the use more elaborate in situ

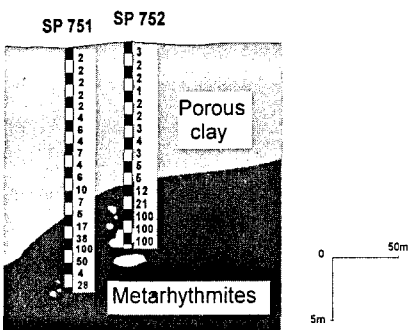


Fig. 3 - Changements des valeurs de SPT à la fin de la couche d'argile.

tests, such as the electric cone or the dilatometer.

The water level is generally very deep, except at tip of the south wing (figure 2), where it can be found at 8 to 10 m depth only. High seasonal variation of the water level due to the high permeability of the porous clay is another characteristic of the soil conditions.

The bottom of the porous clay is clearly indicated by a sudden rise in the SPT values (figure 3), as the boreholes strike the residual soil from slate or the metasiltstone or quartzite layers. These residual soils were investigated during excavations and show an inherent anisotropy as a dominant feature. Bedrock characteristics such as bedding and shear planes, present in the residual soil, control its behaviour. Strength and deformation depend on the direction in relation of these planes. Therefore, it is unlikely that in situ tests such as those used for the investigation of the porous clay could be useful in these residual soils.

CHARACTERIZATION OF THE POROUS CLAY

A summary of the laboratory test results on the porous clay is presented in figure 4. They were carried out on undisturbed block samples from test pits.

Atterberg limits are : liquid limit $LL = 50-80\%$, plasticity limit $LP = 35-50\%$ and water content $w = 35-55\%$. The clay fraction, i.e., the percentage of soil particles less than $2\ \mu\text{m}$ lies between 70 and 55%. The percentage of fines, i.e., less than $60\ \mu\text{m}$ in diameter, varies from 70 and 80%.

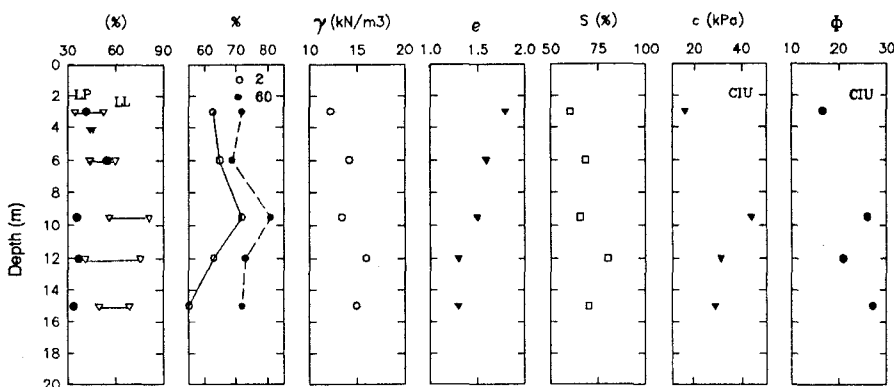


Fig. 4 - Résultats des essais en laboratoire sur l'argile poreuse.

The average unit weight of the porous clay is $\gamma = 15\ \text{kN/m}^3$, except at the top where severe leaching took place even lower values can be found. The void ratio is close to $e = 1.7$ and the degree of saturation is low above water level, about 60-80%.

Undrained triaxial tests on unsaturated and saturated samples were carried out. The strength of the unsaturated clay can be represented by the following Mohr-Coulomb's parameters : cohesion c' from 20 to 40 kPa and friction angle ϕ' lying in the 25-28° range. These parameters can be regarded as effective strength parameters because measured porepressures were very low in CIU tests on unsaturated samples.

Local experience with construction on this clay since the foundation of Brasilia in 1960 has proved that it is collapsible.

Low rise buildings on shallow foundations tend to crack one two years after construction. Good practice is to adopt deep foundations consisting of small diameter bored piles, even for just one story building.

IN SITU TESTS

An in situ test investigation programme was carried out encompassing horizontal plate loading tests (PLH), Marchetti dilatometer (DMT), Ménard presssuremeter (PMT) and piezocone (CPTU). A detailed description and analysis of the results has been given by Ortigao (1993) and a brief summary is presented in figure 5 where the stiffness of the porous clay was evaluated by several in situ tests. Young's moduli E from different tests are plotted and compared. E values are very low at the clay top and reach 30 MPa at 20 m depth. The smaller values yielded by the PMT can be explained as due to excessive soil disturbance.

Due to non-linearity in soil behaviour, secant modulus E depends of the

strain or stress level. This should be considered when comparing moduli from different tests and a reference strain level should be obtained for each test. The Young's moduli plotted in figure 5 refer to **macrodeformation** strain level (Ortigao, 1983).

GROUND CONDITIONS AT THE TIP OF THE SOUTH WING

The observed behaviour of the tunnel will be described for a particular area at the tip of the south wing of Brasilia, between progressives 650 m to 550 m, where very poor ground conditions are encountered. This area presents a high ground water level reaching the midsection of the tunnel (figure 6) and

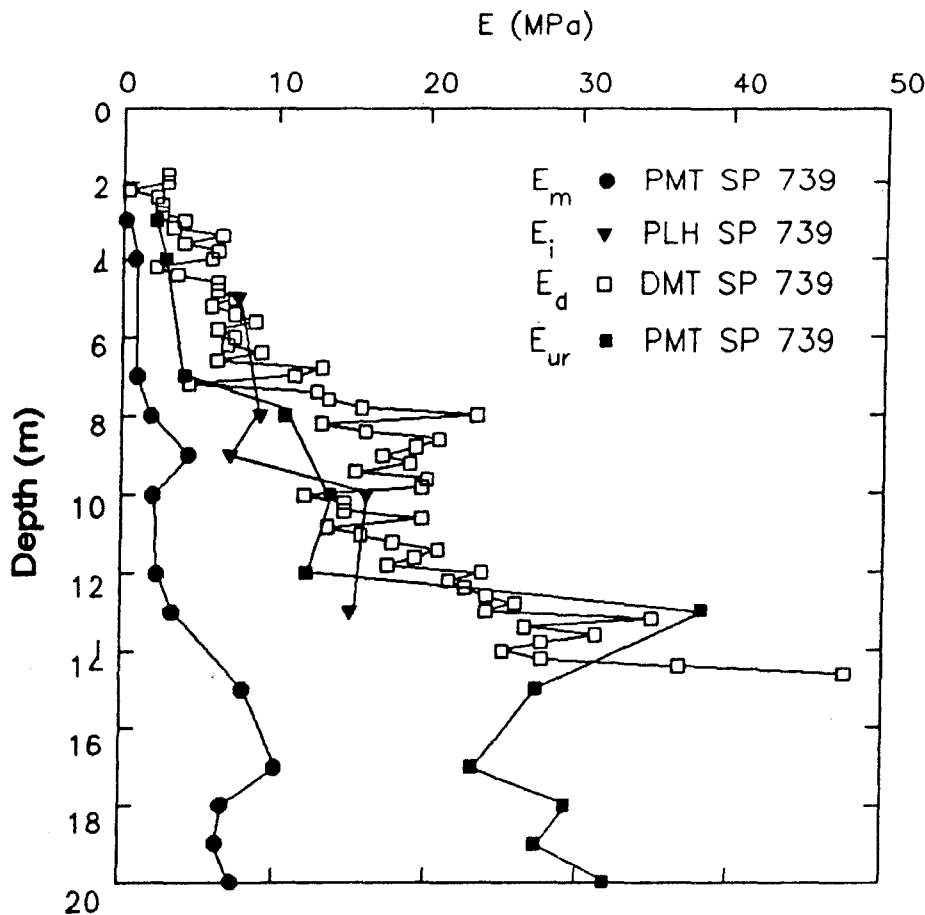


Fig. 5 - Comparaison des valeurs de module obtenues à partir de différents essais.

the presence of a silty clay interlayered between the porous clay and the residual silt from slate. This figure also shows the position of the tunnel excavated section.

The silty clay is very similar to the porous clay both in terms of strength and deformation, but it has not been severely leached leading to much

lower permeability, which prevents dewatering to be effective. This fact is responsible for **undrained** behaviour of this clay during tunneling, opposite to drained conditions that prevail at the porous clay. This has been particularly important when this layer occupied most of the tunnel section being excavated, as occurred close to tunnel length 550 m.

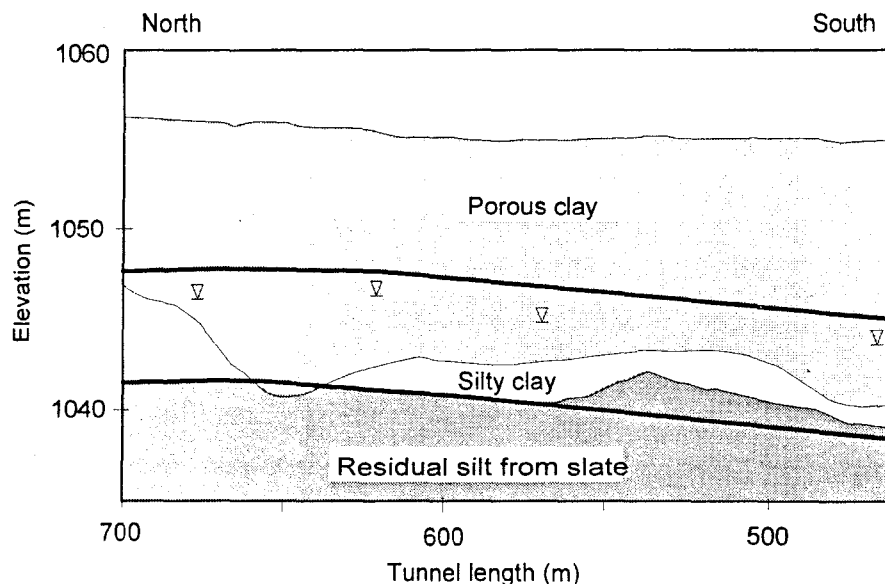


Fig. 6 - Détail de la coupe géologique entre les PM 500 et 700.

TUNNEL DESIGN AND CONSTRUCTION

Since its introduction in Brazil in 1970 the NATM became the preferred tunneling method for its flexibility to adapt to different soil conditions, smaller settlements (Negro & Eisenstein, 1981) and for the type and characteristics of the standard construction equipment used for tunneling. The construction sequence is presented in figure 7. It includes the following stages: excavation, placement of lattice girders, shotcreting the primary layer of the lining, closing the invert, and finally shotcreting the final or secondary layer of the lining.

Figure 8 presents a cross-section of the tunnel in which the equivalent diameter is around 9.6 m.

The design was carried out employing similar experience in Sao Paulo where some 15 km of NATM tunneling has already been constructed with reported success (Negro et al, 1992). It encompassed four construction methods shown in figure 8.

- Method A: full face excavation, shotcreting and closing the invert at a minimum distance of 4.8 m to a maximum of 7.2 m behind the face;
- Method B: excavation of the heading and the use of a temporary invert, closed at a minimum distance of 3 m to a maximum of 5.40 m behind the face;
- Method C: excavation employing a side wall and an invert closed at distances between 3 to 5.4 m behind the face;
- Method D: side-drift excavation at the heading, the invert closed at distances from 3 to 5.40 m behind the face.

Method A is the least expensive and employed in favorable conditions of face stability and negligible damage to nearby structures. On the other hand, if excavation conditions deteriorate, such as, decreasing thickness of soil cover, poor soils are encountered, sensitive nearby structures exist, or deformation observation indicates that stability may be decreasing, one of the additional methods B to D may be selected.

INSTRUMENTATION

Displacements and groundwater observations play an important role in NATM construction and the design considered it accordingly.

A typical fully instrumented section is presented in figure 9. It was laid in 50 m intervals along the tunnel. It comprises the following external instruments: surface marks, vertical single

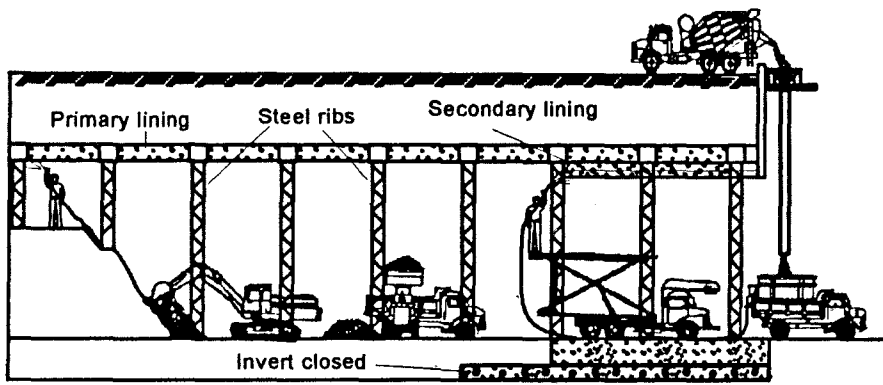


Fig. 7 - Schéma de principe de la nouvelle méthode autrichienne.

Tunneling methods

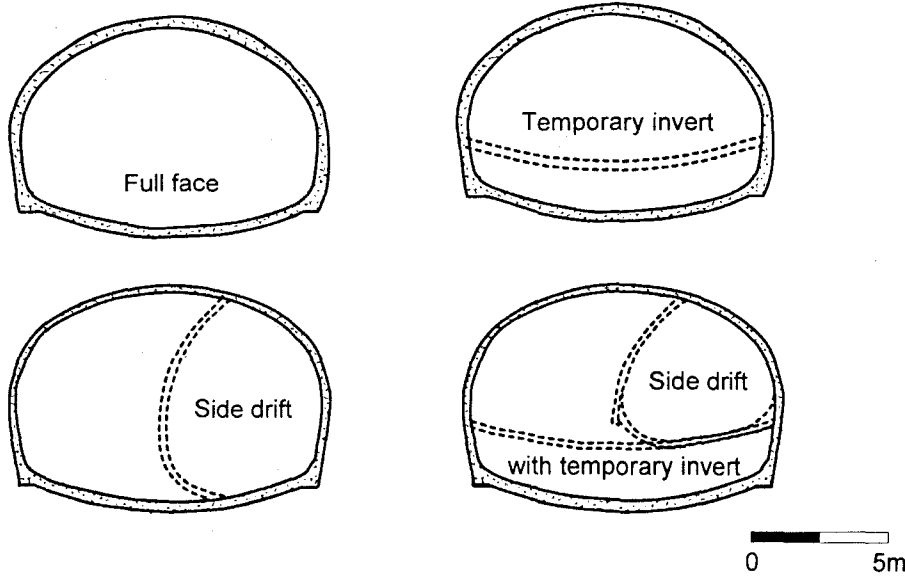


Fig. 8 - Différents phasages possibles.

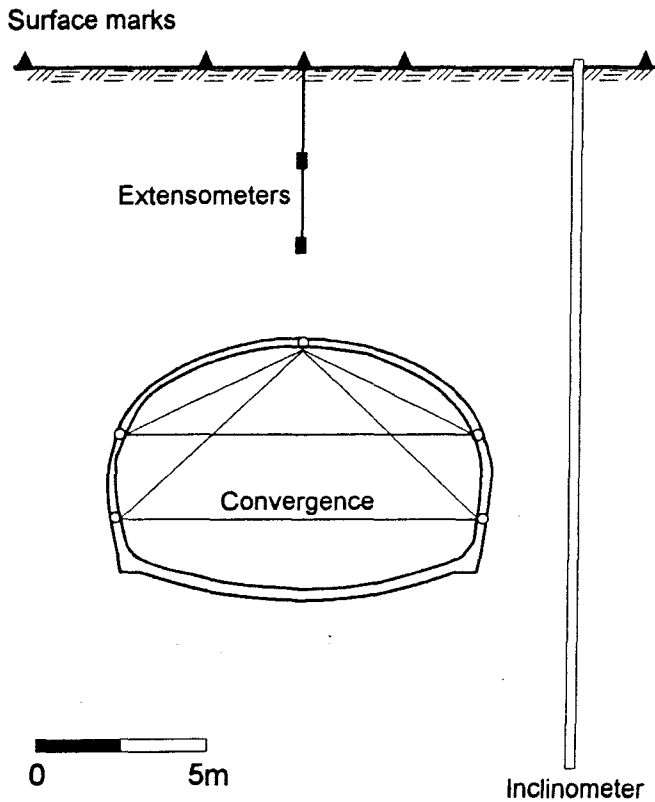


Fig. 9 - Section de mesure complète.

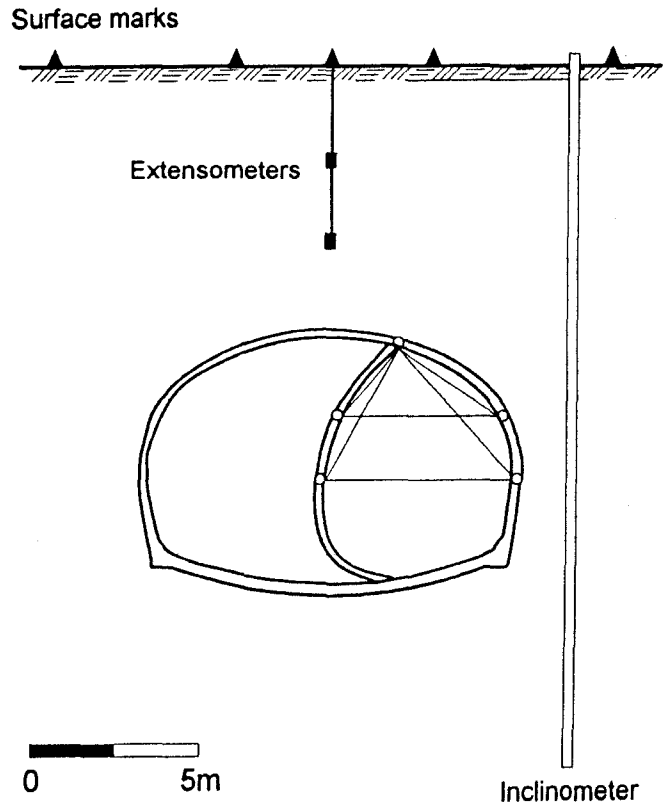


Fig. 10 - Section de mesure en section divisée.

point extensometers and an inclinometer. The internal instrumentation included convergence measurements with tape extensometers and settlement observations of pins embedded in the shotcrete. Figure 10 presents the lay-out of the instrumentation for the side-drift construction methods C and D.

Groundwater observations took place at the southern tip of the south wing of Brasilia where the water level is observed at shallow depths interfering with tunneling. In this case, dewatering is necessary,

As a rule of thumb in tunneling instrumentation, the radius of influence of displacements caused by excavation is $1.5D$ ahead of the tunnel face and $2D$ behind, where D is the tunnel diameter. In this case $D \approx 10$ m. Therefore, secondary instrumented sections with surface marks only were laid in 10 m intervals between fully instrumented sections. This scheme enabled displacements to be monitored in three instrumented sections at any time, being one ahead two behind the tunnel face.

OBSERVED BEHAVIOUR

• CONSTRUCTION METHOD

The construction started from a vertical shaft located at progressive 686 m and proceeded southwards in the direction of progressive 550 m employing full face excavation (method A) and dewatering by deep wells alongside the tunnel.

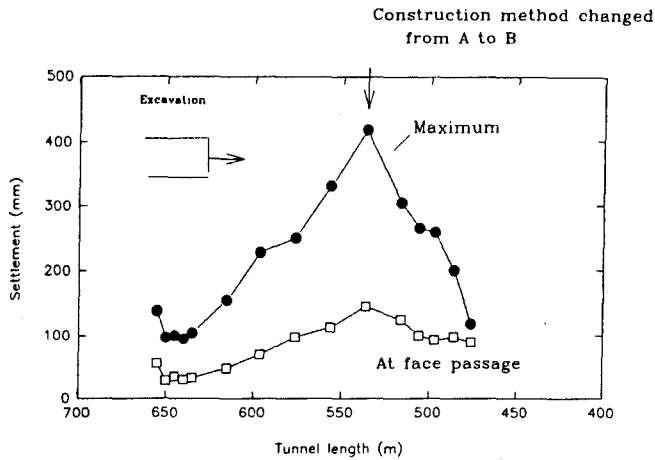


Fig. 11 - Valeurs des tassements en surface.

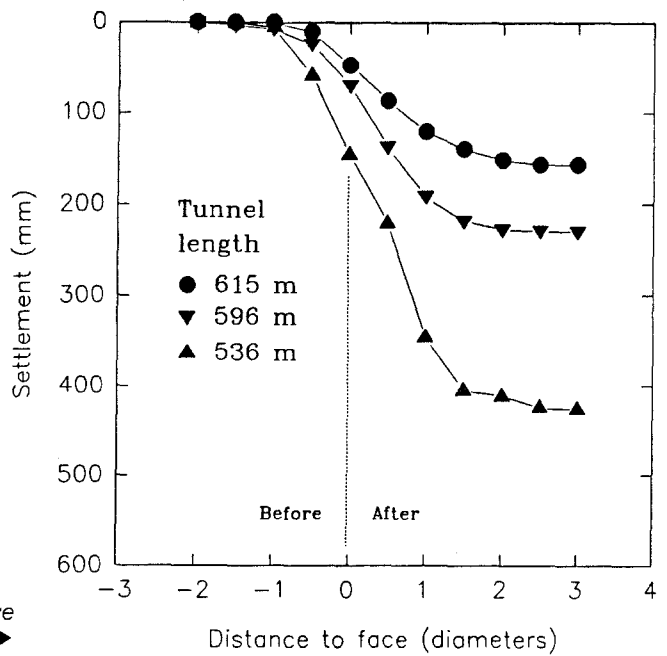


Fig. 12 - Tassements en surface en fonction de la distance au front ▶

● SURFACE LONGITUDINAL SETTLEMENT PROFILE

A longitudinal profile of settlements measured at the surface are presented in figure 11. At the beginning of the construction settlements were in the range of 100-150 mm. Stability analyses carried by the designer and the fact that there are no sensitive structure nearby, led to the conclusions that this large range of settlements was acceptable.

As tunneling proceeded, settlements increased as shown in figure 11. At tunnel progressive 550 m settlements reached values of 400 mm. Safety considerations led to the decision of changing the construction method from A (full face) to B (heading excavation and temporary invert). The results were immediately observed and by progressive 520 m settlements had returned to the 100 mm level.

● SETTLEMENTS AGAINST DISTANCE TO FACE

Figure 12 presents surface against distance to the tunnel face for three different instrumented sections. Ground settlements start as the excavation approaches of a distance of 1.5 tunnel diameters and ceases after 2 diameters after the passage of the face.

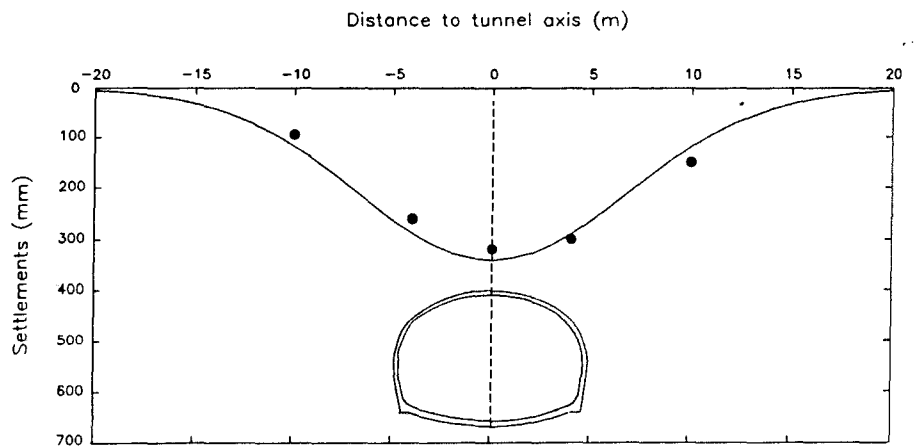


Fig. 13 - Cuvette de tassement.

● OBSERVATIONS AT INSTRUMENTED SECTIONS NEAR PROGRESSIVE 536 M

The fully instrumented section at progressive 536 m was selected for a detailed description of its behaviour. Figures 14 to 16 summarize the results.

A cross-section of settlements is presented in figure 13. An inverted Gauss function was fitted in the measurements and presented the following properties, maximum settlement close to 350 mm. At the tunnel lining (figure

14) measured settlements were much lower in the range of 120 to 140 mm.

A vertical profile of settlements versus depth including from surface marks, extensometers and pins embedded in the lining is presented in figure 15. The striking feature of this plot is that **settlement amplification** from tunnel lining to surface is observed, as a result of the collapsible nature of the ground.

Observed convergence and divergence measurements are shown in figure 16. Maximum divergence was close to 110 mm and convergence was very small, close to 10 mm.

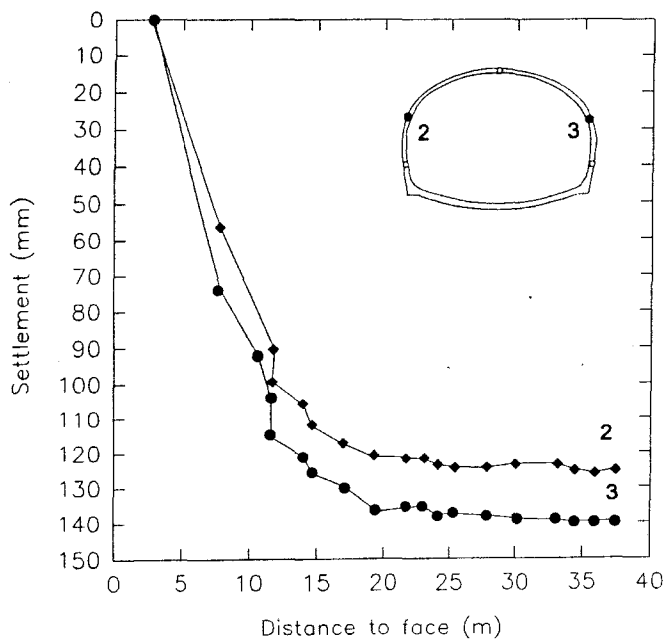


Fig. 14 - Tassements aux points 2 et 3.

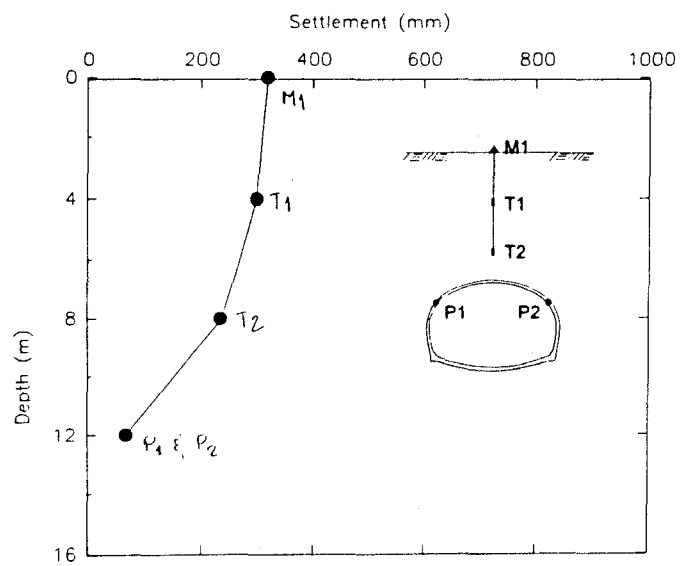


Fig. 15 - Variation du tassement en fonction de la profondeur.

CONCLUSIONS

The use of SPT blowcount to correlate with deformation moduli, a standard practice in Brazil (Negro et al, 1992) is useless in the Brasilia porous clay. The very narrow range of SPT blowcount in this soil precludes any correlation of this type.

Surface settlements due to tunneling in Brasilia reached levels of 400 mm,

which was never observed in Brazil. This is a consequence of the collapsible nature of the porous clay that amplifies by a factor of three the settlements from the tunnel lining to the surface.

A laboratory testing programme is currently under way at the University of Brasilia to evaluate the effect of the tunneling induced stress path in the volumetric strains in the porous clay.

ACKNOWLEDGMENTS

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The site investigation programme was carried out by Bureau Ltd, Sao Paulo, (PLH), Geomecânica SA, Rio de Janeiro (CPTU and DMT), Geotécnica SA, Rio de Janeiro, (PMT) and the laboratory tests by the University of Brasilia. The authors are indebted to Dr R Kochen for his support and for the helpful comments on the text. Support from Edson Grossi, Wadir Brandao, Gildson Alves, Guilherme Pinto from the Metro Company, and from Lucia Alves and Alejandro Far during the preparation of this article is acknowledged.

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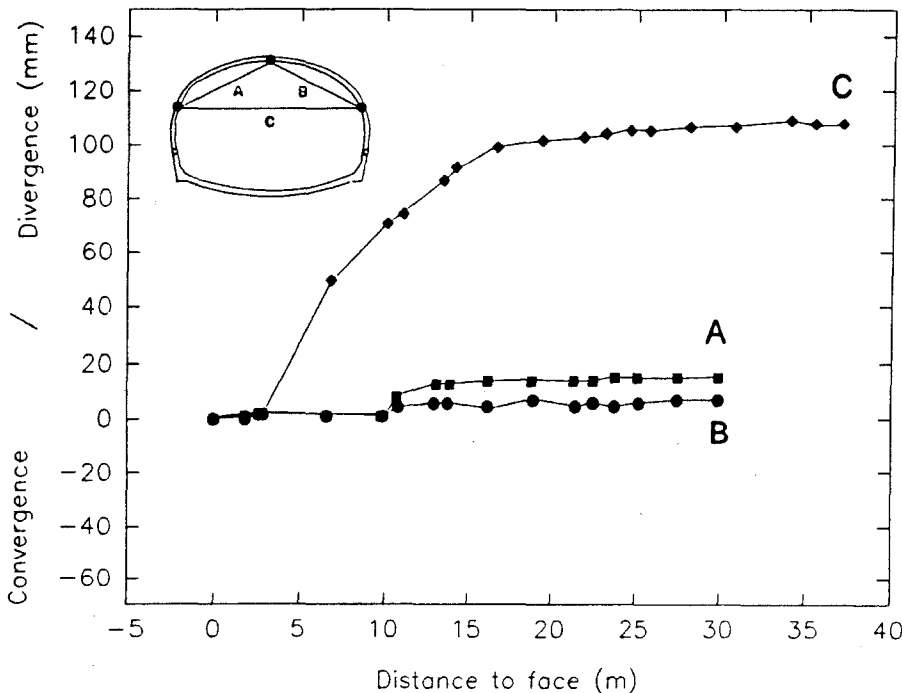


Fig. 16 - Convergences en fonction de la distance au front.

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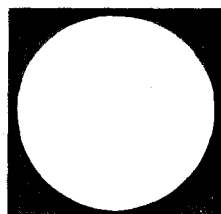
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SOMMAIRE

<i>Le mot du Président</i>	233
<i>AFTES Infos</i>	235
<i>Conseil d'Administration : Compte rendu de la réunion du 10 juin 1993</i>	239
<i>Interview de M. François TRUCY, Sénateur-Maire de Toulon</i>	243
<i>Large settlements due to tunnelling in Porous clay par J. ORTIGAO et P. MACEDO</i>	245
<i>Tunnels urbains au tunnelier par J. HERVET</i>	253
<i>La sécurité et l'intervention des secours dans la conception du projet de bouclage de l'autoroute A86 par M. BROTO</i>	259
<i>La conduite des tunneliers par G. PIQUEREAU</i>	263
<i>Les avancements des tunneliers par J. GUILLAUME et J. PIRAUD</i>	269
<i>Boucliers à pression de terre : avantages, inconvénients, recherche et développement par M. MOHKAM</i>	279
<i>Compte rendu sommaire de la RETC 93, Boston, 13-17 juin 1993 par P. DUFFAUT</i>	282



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