Tunnelling in Brasília porous clay

J.A.R. Ortigao, R. Kochen, M.M. Farias, and A.P. Assis

Abstract: The Brasília underground transportation system comprises 6.5 km of shallow tunnel excavated in a soft red soil known as porous clay that overlies harder residual soils. The tunnel diameter is 9.6 m. Settlement observations indicated that surface settlements were two to three fold the initially predicted value, although no indication of excavation instability was observed. Settlements reached, at one section, 500 mm without failure. Another striking feature was settlement amplification between the top of the excavation and the surface by a factor that averaged 1.2 but reached up to 4. This occurred because of the collapsible nature of the porous clay, which presented a considerable reduction of volume as the tunnel face advanced. This paper describes tunnel design, construction, and instrumentation; and summarizes geology and soil properties from in situ and laboratory tests. Field measurements of settlements and horizontal displacements are described and analysed. The main cause of the large settlements was collapse of the porous clay structure.

Key words: tunnelling, porous clay, settlements, collapse.

Résumé : Le système de transport souterrain de Brasília comprend 6,5 km d'un tunnel superficiel excavé dans un sol mou rouge connu sous le nom d'argile poreuse qui recouvre des sols résiduels plus denses. Le diamètre du tunnel est de 9,6 m. Les tassements observés ont indiqué que les déplacements de surface ont été de deux à trois fois supérieurs aux valeurs prédites initialement, bien qu'aucune indication d'instabilité n'ait été observée dans l'excavation. Pour une des sections les tassements ont atteint 500 mm sans rupture. Un autre fait frappant a été l'amplification du tassement entre le haut de l'excavation et la surface par un facteur de 1,2 en moyenne mais qui a pu aller jusqu'à 4. Ceci est attribuable au potentiel d'effondrement de l'argile poreuse qui a présenté des réductions de volume considérables lors de l'avance du front du tunnel. Cet article décrit la conception du tunnel, sa construction et son instrumentation et il donne un résumé de la géologie et des propriétés du sol à partir d'essais en place et en laboratoire. Les mesures en place des tassements et des déplacements horizontaux sont présentées et analysées. La cause principale des tassements importants a été l'effondrement de la structure de l'argile poreuse.

Mots clés : percement de tunnel, argile poreuse, tassement, effondrement.

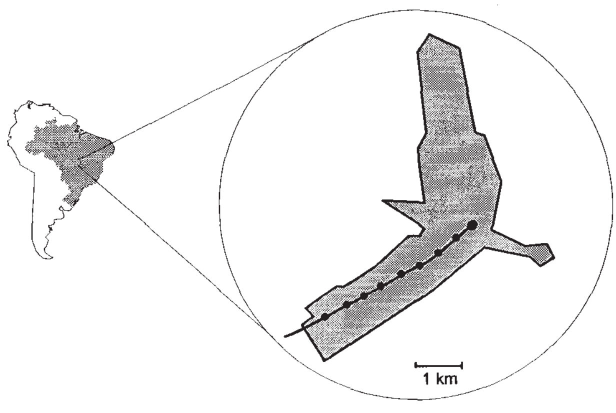
[Traduit par la rédaction]

Introduction

The Brasília underground transportation system is a $650MUS project, 42 km long, linking the south wing of the city to the suburbs or the satellite towns (Fig. 1). It encompasses 6.5 km of tunnelling and over 4 km of cut-and-cover slurry-walled false tunnels.

The tunnel design was based on the bulk of the Brazilian tunnelling experience gathered in more than 30 km of tunnelling in the soft porous clays of São Paulo (Negro et al. 1992) and more than 300 km in hard soils and rocks. Therefore, preconstruction predictions were based on soils in São Paulo porous clay, which were supposed to be very similar to the soils of Brasília. Initial studies predicted maximum settlements in the 60–80 mm range. These settlements were not expected to cause problems because only a few nearby structures existed, and most of them were founded on deep foundations.

Fig. 1. Site location.



At the beginning of the construction it was observed that the measured displacements were two to three fold greater than those predicted, although the excavation face did not show signs of instability. A striking feature was the displacement amplification that took place between the tunnel crown and the surface. At the tunnel crown vertical

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Fig. 2. Soil profile at the south wing of Brasília.

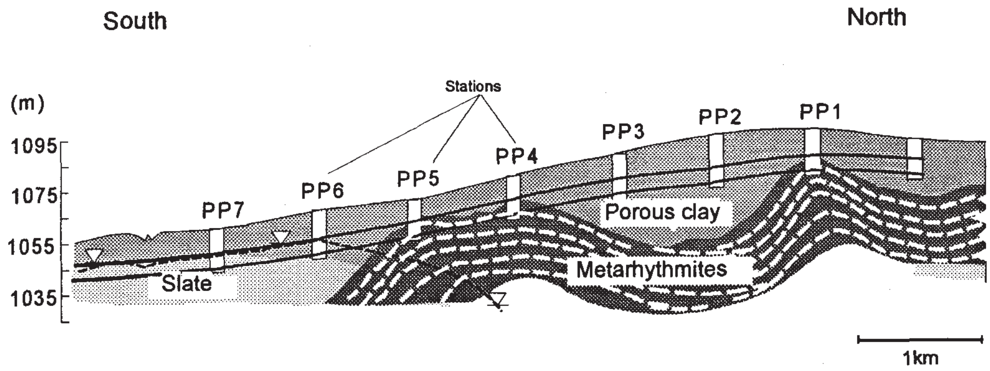
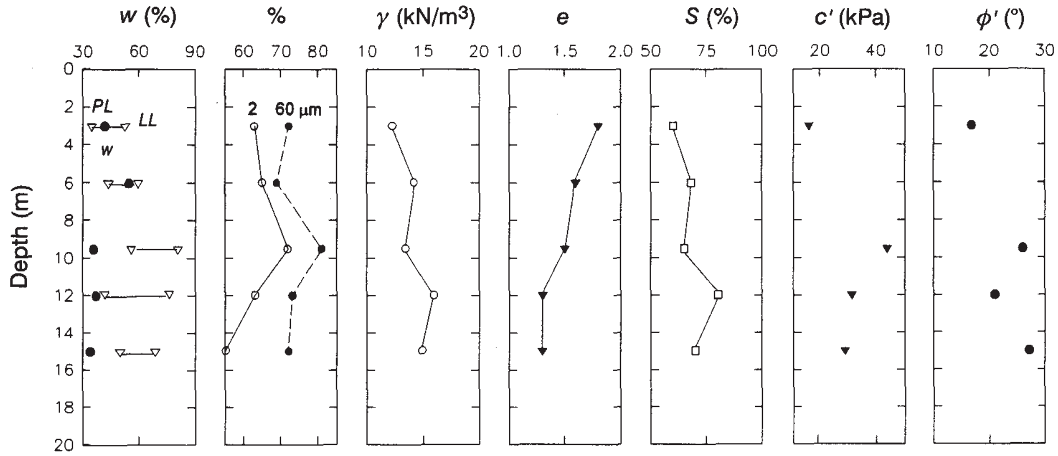


Fig. 3. Summary of laboratory test data.



displacements were 50–60 mm, but at the surface the values were much larger, in the 150–200 mm range. It was soon verified that the collapsible characteristic of the Brasília clay was unique and very close field surveillance was necessary.

A field investigation programme, described in detail by Ortigao et al. (1995), was carried out to reassess design parameters and employed Marchetti dilatometer (DMT), Ménard pressuremeter (PMT), piezocone (CPTU), and horizontal plate loading (PLH) tests. 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 observed tunnel behaviour, which led to large displacements reaching 500 mm at the surface at one single section without any sign of face instability.

Geology and site conditions

The regional geology and geomorphology have been described in detail by Macedo et al. (1994). The region is flat, as is characteristic of the central plateau highlands. It is covered by a layer of latosoils and lateritic soils named porous clay, overlying residual soils from slate or a sequence of interlayered metasiltstones and quartzites, that geologists call metarhythmites. They are named the Paranoah formation of the upper Precambrian.

The climate alternates from a 6 month rainy season to a very dry winter, leading to a laterization process of leaching soluble salts at the top of the porous clay and deposition below. This process is responsible for the large pores at the top of the clay layer resulting in high void ratio, low unit weight, and high permeability.

This study is concentrated in the south wing of Brasília where a 10 m diameter tunnel was excavated (Ortigao and Macedo 1993). The soil profile was initially investigated by boreholes at 30 m intervals along the tunnel line in which SPTs were carried out. The porous red clay is 8–30 m thick; the SPT blow count is low, varying from 2 to 3. The water level is generally very deep, except at the tip of the south wing (Fig. 2) where it can be found at 8–10 m depth only. High seasonal variations in 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 blow count, as the boreholes strike the residual soil from the slate or the siltstone or quartzite layers. These residual soils were investigated during excavations and show an inherent anisotropy as a dominant feature. Bedrock characteristics such as bedding

Table 1. Construction methods.

Method	Excavation method	Distance to close the invert behind the excavation face (m)
A	Full face	4.8–7.2
B	Heading excavation with a temporary invert; bench excavated afterwards in 3 m long steps	2.4–5.4
C	Side drift employing a side wall; enlargement to the final section, with demolition of the side wall	2.4–5.4
D	Side drift and heading; enlargement of the side drift to form the heading; advancement of the excavation, as in method B	2.4–5.4

Fig. 4. Tunnelling methods employed in Brasília.

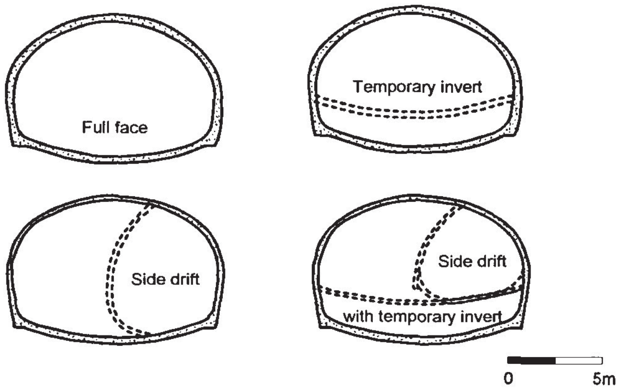
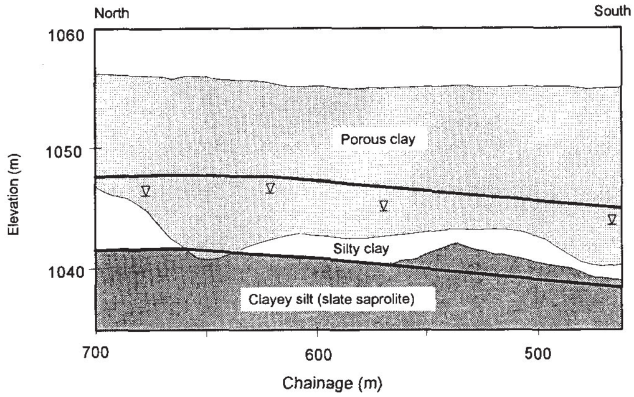


Fig. 5. Soil profile at the tip of the southern tip of Brasília (shaft-S).



and shear planes, remaining in the residual soil, control the 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.

Characteristics of the porous clay

A summary of the laboratory test results on the porous clay is presented in Fig. 3. The tests were carried out on undisturbed block samples.

Atterberg limits are as follows: liquid limit, LL = 50–80%; plastic limit, PL = 35–50%; and water content, w = 35–55%. The clay fraction, i.e., the percentage of soil particles less than 2 μm lies between 70 and 55%. The percentage of fines, i.e., less than 60 μm in diameter, varies from 70 to 80%.

The average unit weight γ of the porous clay is 15 kN/m^3, except at the top where severe leaching took place and even lower values can be found. The void ratio e is close to 1.7, and the degree of saturation is low above the water level, in the range of 60–80%. Low unit weight and high void ratio are indications of unstable ground conditions that may lead to large deformations.

Triaxial tests on saturated samples were carried out. The strength of the porous clay can be represented by the following Mohr-Coulomb drained parameters: cohesion c' from 20 to 40 kPa, and friction angle ϕ' lying in the 25–28° range. In summary, this clay exhibits a drained behaviour as a result of its high permeability, with a strength envelope that can be described by an effective cohesion and a friction angle.

Foundation experience has shown that this clay is collapsible (Ortigao 1995). Low-rise buildings on shallow foundations tend to crack 1–2 years after construction. A good practice is to adopt deep foundations consisting of small-diameter bored piles, even for a one-story building.

Tunnel design and construction

Soft ground tunnelling with flexible shotcrete lining has been used in Brazil since 1970. It has been called the New Austrian Tunnelling Method, and has become the preferred tunnelling method because of its flexibility in adapting to different soil conditions, small settlements (Negro and Eisenstein 1981) and because of the type and characteristics of the standard construction equipment used for tunnelling. Recent work by Kovari (1994*a* and 1994*b*) pointed out misconceptions and false principles on which the Austrian method is based.

The tunnelling method used in Brasília had the following stages: excavation, placement of lattice girders, shotcreting the primary layer of the lining, closing the invert, and finally placing the final or secondary layer of the lining.

Figure 4 shows a cross section of the Brasília tunnel in which the equivalent diameter is 9.6 m.

The construction was based on similar experience in São Paulo where some 15 km of tunnelling has already

Fig. 6. Layout of instruments at a fully instrumented section.

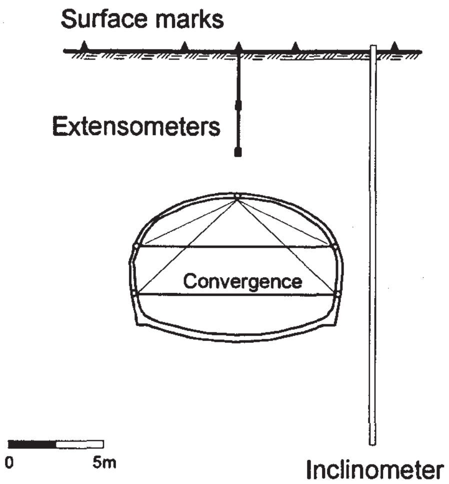
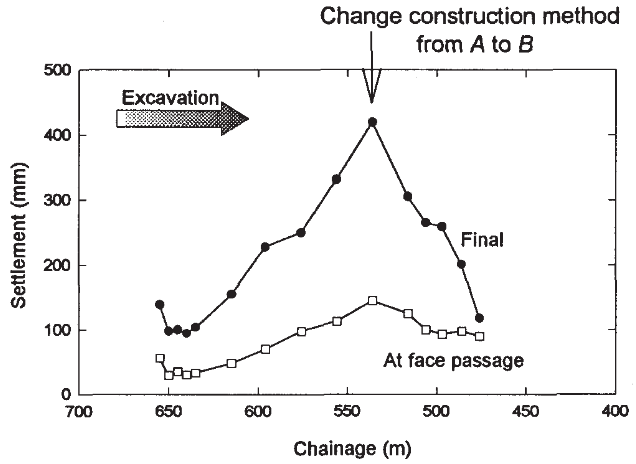


Fig. 7. Surface settlements at the face passage and maximum final values after stabilization (shaft-S drive).

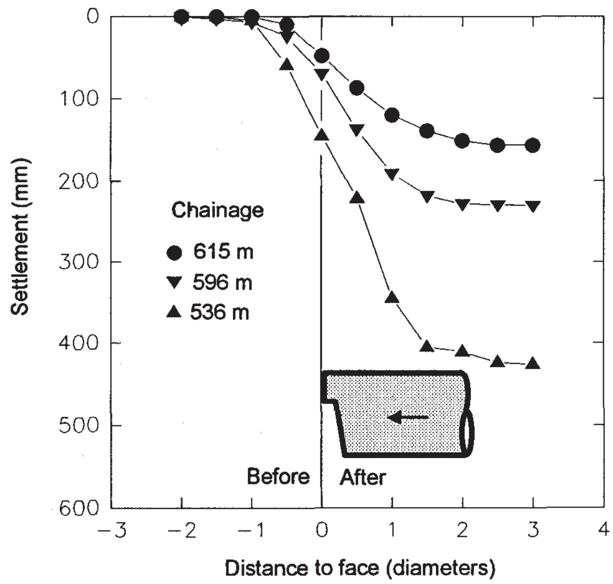


been constructed with reported success (Negro et al. 1992). It encompassed the four construction methods shown in Fig. 4 and described in Table 1.

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

The primary shotcreted lining was 21 cm thick and employed lattice girders spaced from 0.6 m up to 1 m, values that were selected depending on the excavation and deformation conditions. The secondary lining was 20 cm thick and employed two layers of steel meshes. The roof was shotcreted, but tunnel sides and the final invert was cast in place concrete for improved water tightness.

Fig. 8. Settlements as a function of the distance to the tunnel face.



Ground conditions at the tip of the south wing (shaft-S)

The observed behaviour of the tunnel will be described for a particular area at the tip of the south wing of Brasília, between chainages 650 and 550 m, where very poor ground conditions are encountered. This drive was named shaft-S because it started from a shaft heading southwards.

This area presents a high groundwater level reaching the midsection of the tunnel (Fig. 5) and 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. Dewatering was not effective in the silty clay and, assuming full saturation below the water level, an *undrained* behaviour prevailed in the silty clay during tunnelling, opposite to drained conditions in the porous clay. This has been particularly important when the silty clay layer occupied most of the tunnel section being excavated, as occurred close to chainage 550 m.

Instrumentation

A typical instrumented section is presented in Fig. 6. Instrumentation was installed at 50 m intervals along the tunnel and comprises the following external instruments: surface marks, vertical single point extensometers and an inclinometer. The internal instrumentation included convergence measurements with tape extensometers and settlement observations of pins embedded in the shotcrete.

Groundwater observations took place at the southern tip of the south wing of Brasília where the water level is observed at shallow depths interfering with tunnelling.

Fig. 9. Settlement trough at chainage 536 m.

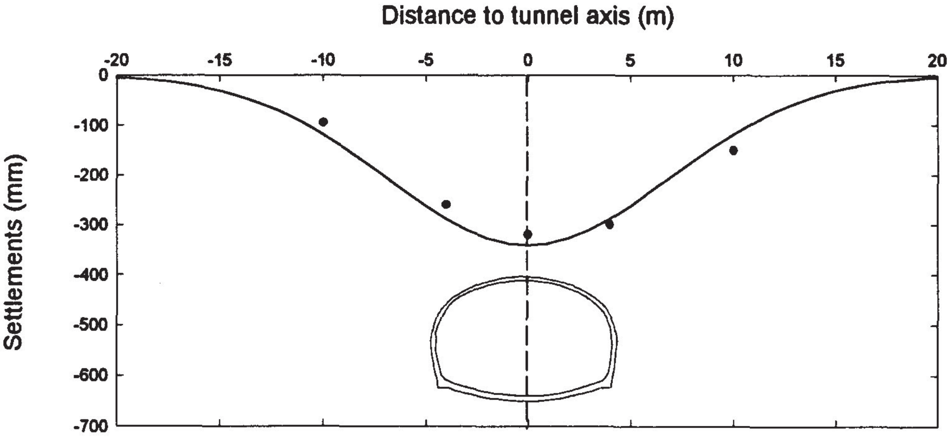


Fig. 10. Settlements along depth, chainage 536 m.

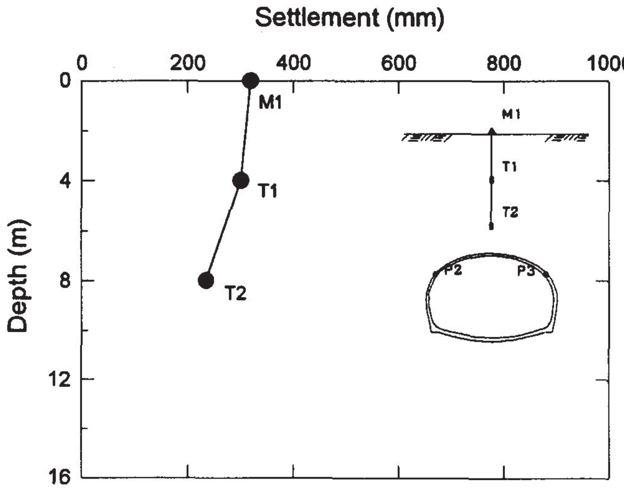
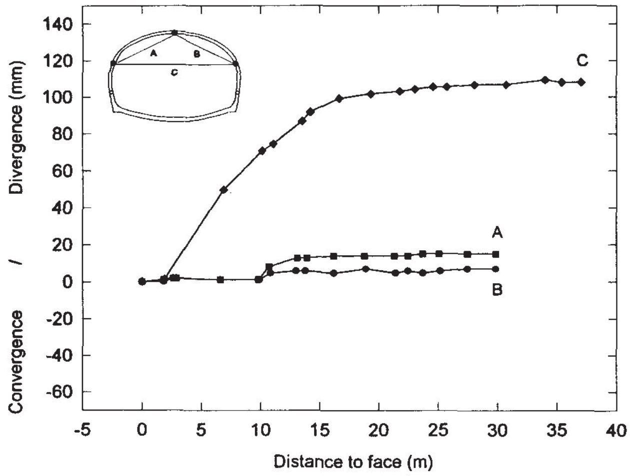


Fig. 11. Convergence measurements at chainage 536 m.



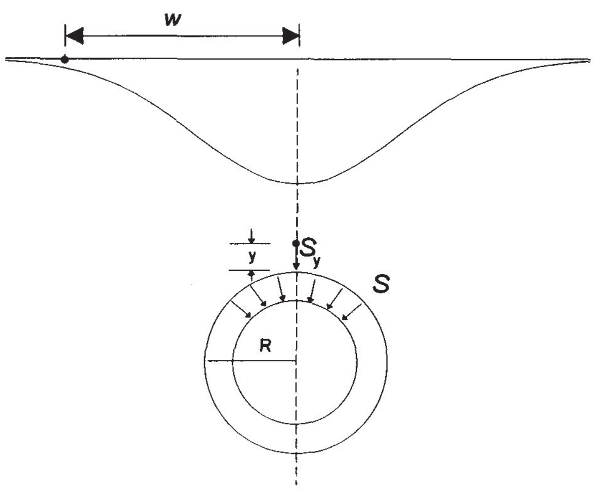
As a rule-of-thumb in tunnelling instrumentation, the radius of influence of displacements caused by excavation is 1.5D ahead of the tunnel face and 2D behind, where D is the tunnel diameter (Rowe and Lee 1992). In this case $D \cong$ 10 m. Therefore, secondary instrumented sections with surface marks 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, one being ahead and two being behind the tunnel face.

Observed behaviour at shaft-S drive

Construction of the shaft-S drive started from a vertical access shaft located at chainage 686 m and advancing southwards in the direction of chainage 550 m employing full-face excavation (method A) and dewatering by deep wells alongside the tunnel.

A longitudinal profile of settlements is presented in Fig. 7. They were obtained by monitoring surface marks located at the surface and along the axis of symmetry of the tunnel. The upper curve shows the maximum observed settlements after stabilization, i.e., after the excavation front has sufficiently advanced ahead of this chainage.

Fig. 12. Loss of ground (Cording and Hansmire 1975).



The lower curve shows observations when the tunnel face was passing just below the monitored section.

At the beginning of the construction settlements were in the range of 100–150 mm. Stability analyses and the fact

Fig. 13. Settlements in four tunnel drives.

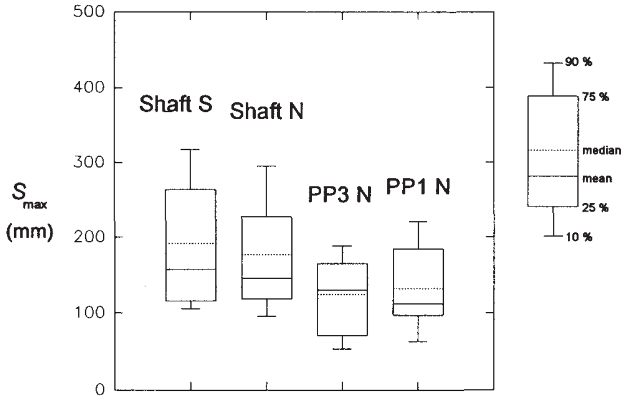
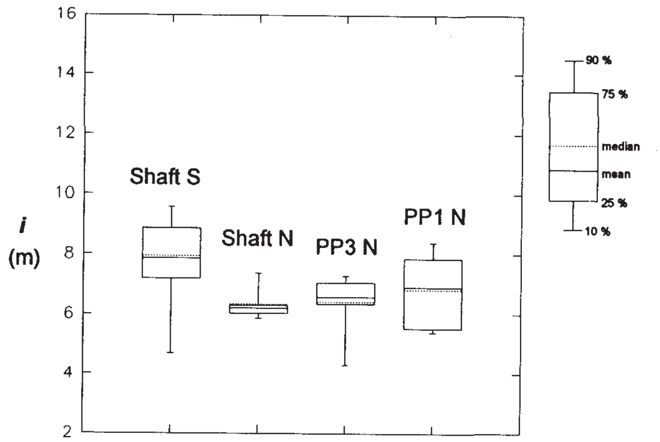


Fig. 14. *i* parameter.



that there is no sensitive structure nearby led to the conclusion that this large settlement was acceptable.

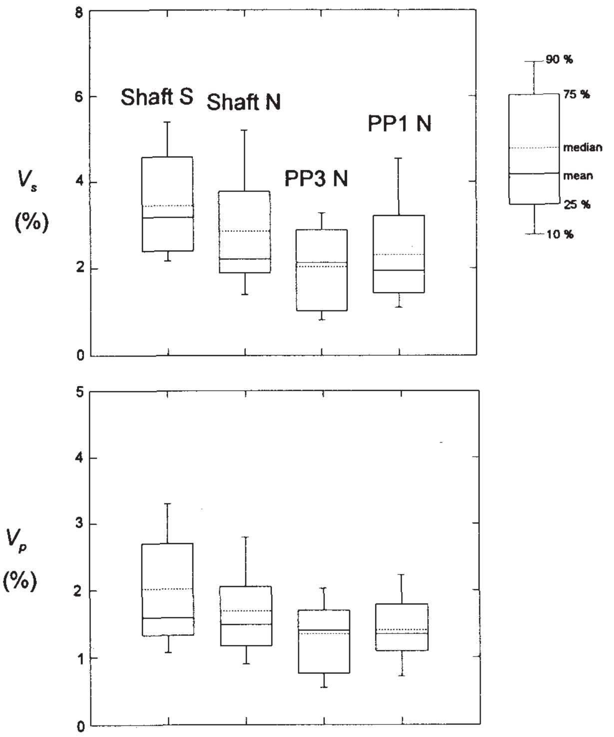
As tunnelling advanced, settlements increased, as shown in Fig. 7. At tunnel chainage 550 m settlements reached values of 400 mm. Safety considerations led to the decision to change the construction method from A (full face) to B (heading excavation and temporary invert). The results were immediately observed, and by chainage 520 m settlements had returned to the 100 mm level.

Figure 8 presents surface settlements against distance to the tunnel face for three different instrumented sections. Ground settlements start as the excavation approaches a distance of 1.5 tunnel diameters and ceases after 2 diameters after the face advances.

The fully instrumented section at chainage 536 m was selected for a detailed description of its behaviour. A cross section of settlements is presented in Fig. 9. An inverted Gauss function was fitted in the measurements and presented a maximum settlement close to 350 mm.

A vertical profile of settlements versus depth including data from surface marks, extensometers, and pins embedded in the lining is presented in Fig. 10. The striking feature of this plot is that settlement amplification from the tunnel lining to surface is observed, as a result of the collapsible nature of the ground.

Fig. 15. Settlement volumes: surface and loss of ground.



Observed divergence measurements are shown in Fig. 11. Maximum divergence was close to 110 mm.

Settlement analysis

Settlement observations at four different tunnel drives will be compared in terms of maximum settlement (S_{max}), the standard deviation i of the fitted Gauss curve, given by the following equation:

$$[1] \quad S = S_{max} \exp\left(\frac{-x^2}{2i^2}\right)$$

where S is the settlement; x is the horizontal distance to the axis of symmetry; S_{max} is the maximum settlement; and i is the standard deviation of the Gauss curve. The settlement volume V_s defined as the area of the Gauss curve times one metre of tunnel drive, is given by

$$[2] \quad V_s = 2.5\, i\, S_{max}$$

V_s is often expressed as a percentage of the tunnel area.

The loss of ground V_p was defined by Cording and Hansmire (1975) as the volume of settlements that cross the original tunnel section. It is calculated by the following simplified equation:

$$[3] \quad V_p = 2S_y(R + y)$$

where the terms are given in Fig. 12. The value of y was obtained from settlement gauges located 2 m above the tunnel crown.

Therefore, comparing V_s and V_p one should know if the ground was expanding ($V_s < V_p$) or collapsing ($V_s > V_p$).

Fig. 16. Settlement amplification ratio.

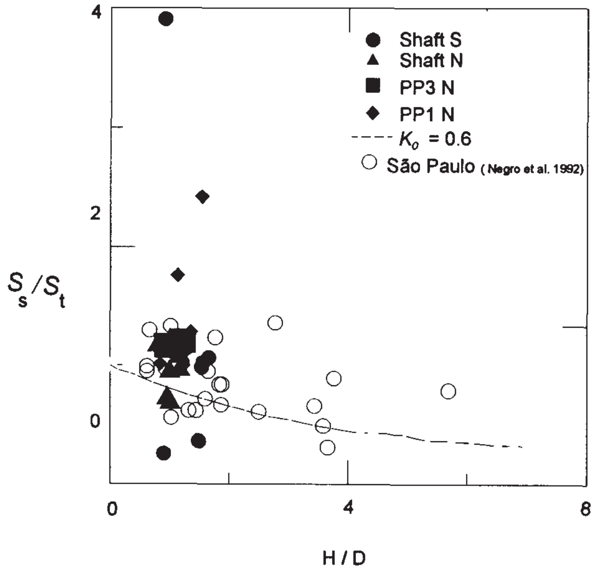
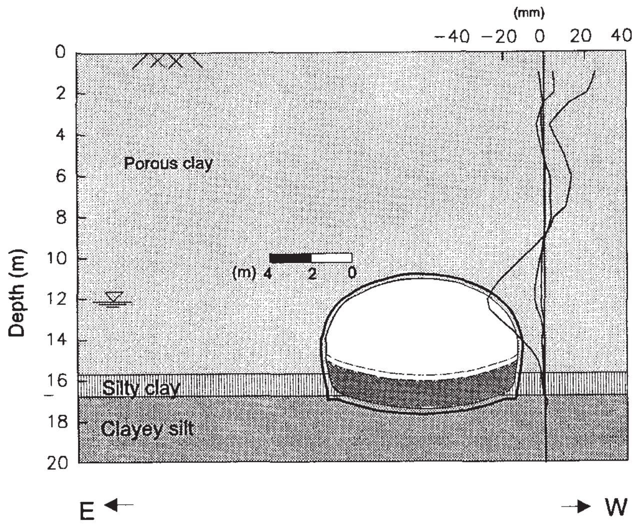


Fig. 17. Inclinometer data, transverse displacements, chainage 466 m.



Settlements of four tunnel drives are compared in Fig. 13. The drives are named after the underground station from which they depart followed by the direction, south or north. Settlements of shaft-south and north are greater than the other two drives shown in Fig. 13 because of the higher water level in these areas. Dewatering was not always effective enough to prevent some water inflow in the excavation area. This led to higher settlements and worse excavation conditions.

The corresponding i parameters are shown in Fig. 14. Drive shaft-S, where excavation conditions were poorer, presents the highest values, varying from 4 to nearly 10 m. Other drives averaged 6–7 m.

Settlement volumes, as a percent of tunnel area, are compared in Fig. 15. Surface settlements are greater than the loss of ground, i.e., $V_s > V_p$, which is an indication of soil collapse.

Comparison of observed settlements with tunnelling in São Paulo

The bulk of tunnelling experience in Brazil has taken place in São Paulo where more than 40 km of soft ground tunnelling has already been successfully constructed (Negro et al. 1992). The Brasília tunnelling experience was overwhelming because of the amount of settlement, and it is of interest to compare the tunnelling data from both locations.

Figure 16 presents the settlement amplification ratios observed at several tunnels in Brazil. This ratio is defined as the settlement at the surface (S_s) divided by the settlement at the tunnel crown (S_t). It is plotted against the ratio H/D, in which H is the thickness of soil above the tunnel crown and D is the tunnel diameter. A theoretical estimate by Negro et al. (1992) is plotted with a dashed line.

Data from Brasília plot in a narrow vertical band with H/D from 1 to 2. The band height is wide, varying from an amplification ratio of 0.5 to nearly 4. The majority of the data, however, lies between 1.2 and 1.3, agreeing with São Paulo tunnelling data.

Horizontal displacements analysis

Inclinometer results at chainage 466 m are shown in Fig. 17 (transverse horizontal displacements) within the cross section and in Fig. 18 (longitudinal horizontal displacements). Horizontal displacements in the transverse direction reach values of 30 mm, while in the longitudinal direction, they reach 120 mm.

Horizontal displacements were found to relate reasonably well with settlements occurring in a particular section, as presented in Figs. 19 and 20. Maximum transverse horizontal displacements (δ_{Tmax}) are approximately 12% of the maximum settlement in the same tunnel section. On the other hand, maximum longitudinal horizontal displacements (δ_{Lmax}) are estimated as 20% of the maximum settlement. Data like these can be very helpful in preliminary estimate of the damage that tunnelling can cause to nearby structures and their foundations.

Conclusions

Tunnelling in Brasília porous clay was a successful experience, proved by 6.5 km of shallow tunnels excavated in a soft material.

Tunnelling caused large settlements, up to 500 mm at one single section (without loss of stability), never observed before in similar works in Brazil. Settlement amplification from the tunnel crown to the surface caused by collapse was observed.

A laboratory testing programme is currently underway at the University of Brasília to evaluate the effect of the tunnelling-induced stress path in the volumetric strains in the porous clay.

Horizontal displacements were analysed and compared with the maximum displacement from the settlement trough at the same cross section. An empirical relationship was

Fig. 18. Inclinometer data, longitudinal profile, chainage 466 m.

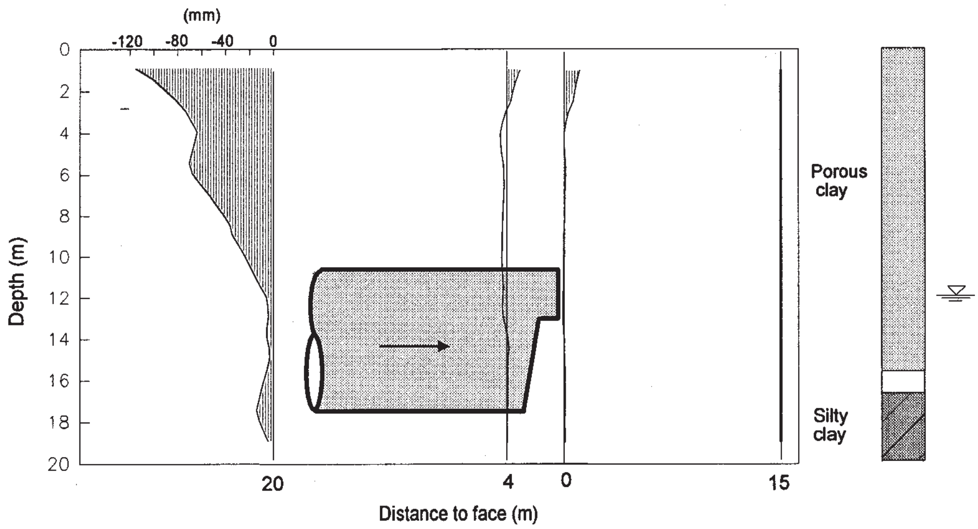


Fig. 19. Transverse horizontal displacements versus maximum settlement.

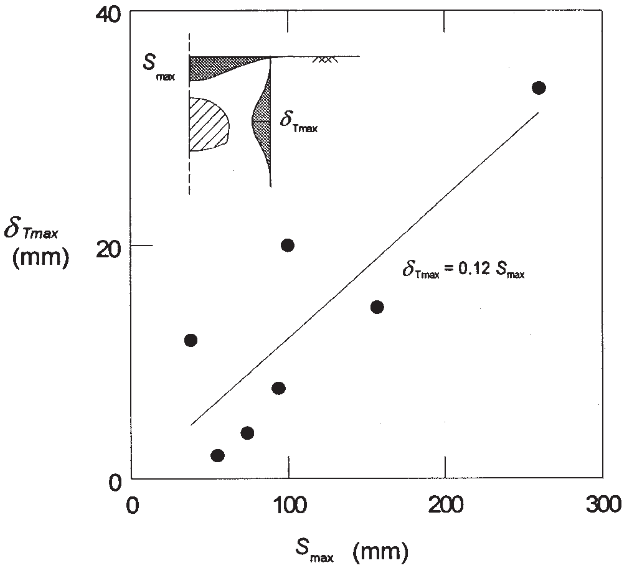
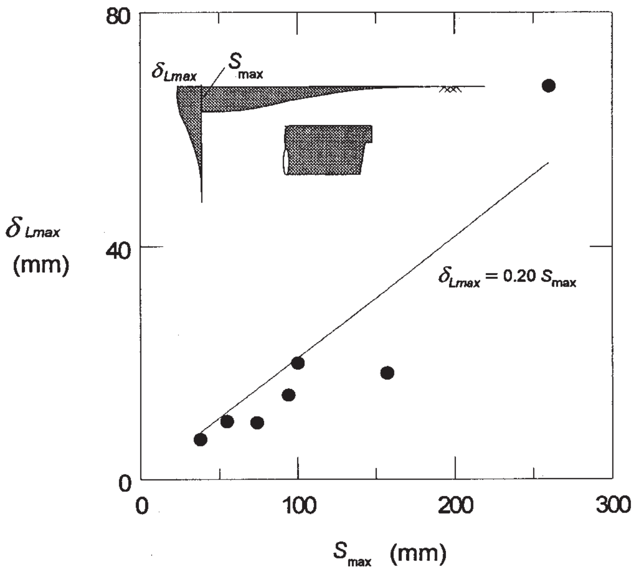


Fig. 20. Longitudinal horizontal displacements versus maximum settlement.



derived. This will enable rule-of-thumb predictions of damage to nearby structures due to tunnelling.

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